Proceedings of the 14th Dredging Seminar

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Texas A&M University **Sea Grant College Program**

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Compiled by

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BURIAL OF DREDGED SEDIMENT UNDER THE SEA FLOOR: CAN YOU DO IT?[1]

by Henry Bokuniewicz[2]

ABSTRACT

Yes, the submarine burial of dredged sediment in shallow water (30 to 80 feet) is technically feasible using conventional methods. It may be one way to isolate and contain contaminated dredged mud. The disposal site should be a pit that is naturally trapping fine-grained sediment and there should be no net long-term erosion on the ambient sea floor. Suitable pits may be created by offshore sand-mining operations. The pit should be at least several yards deep and over 250 yards in radius in order to contain the dredged material during discharge. Dredging should be done with a clamshell bucket or a similar dredge, and point-dumping should be used. About 3 to 5% of the dredged material will be dispersed during the discharge; the rest will form a compact deposit. If the mud has sufficient density and strength it can be covered (or capped) with sand to isolate the mud from the disturbance from waves

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1. Contribution number 302 of the Marine Sciences Research Center, State University of New York, Stony Brook, New York.

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and from the reach of burrowing organisms and to reclaim the sea floor. The strength of the deposit must be greater than twice the difference between the unit weights of the sand and mud multiplied by the height of the maximum irregularity in the sand-mud interface. A cap thickness of 24 inches should be sufficient except at high-energy, open-ocean sites where several feet would be necessary. The grain-size of the cap should be the same or slightly coarser than the ambient sea floor.

INTRODUCTION

Before I discuss the burial of dredged sediment under the sea floor, I should first discuss why anyone would want to do this. Regardless of the environmental criteria that are applied, some dredged sediment will be found unacceptable for unrestricted open-water disposal. This sediment is most likely to be mud because many of the most troublesome contaminants are found attached to fine-grained sediment particles. For this type of sediment, alternate disposal methods need to be developed. One alternative technique, called capping, is coming into use in northeastern waters. This technique involves covering, or capping, a deposit of contaminated sediment at a submarine disposal site with a blanket of clean or relatively uncontaminated sediment. The cap is intended to contain the contaminated sediment particles at the disposal site and to isolate the contaminated material from the water and beyond the reach of burrowing marine animals. Capping operations have been done with conventional equipment in Long Island Sound (Morton, 1980) and at the ocean disposal site off New York Harbor.

The burial of dredged sediment beneath the sea floor has the same advantages as capping but it also has some additional ones. The sea floor at the disposal site can be reclaimed by constructing the cap of a buried deposit to restore the ambient sediment type and topography. Furthermore, because the contaminated sediment is beneath the sea floor, the potential for erosion due to the presence of a mound on the disposal site is eliminated. As a result of these two additional advantages, submarine burial may allow safe deposition of dredged sediment in relatively shallow water near shore. Burial of dredged sediment in subaqueous pits is being considered in New York Harbor, in Duluth Harbor (Roy F. Weston, Inc., 1981) and in Halifax, Nova Scotia (S. MacKnight, personal communication, 1981).

Alternatives such as burial and capping present not only disposal problems but also engineering problems. The alternatives are engineering projects to construct a particular type of deposit on the sea floor using conventional equipment. Submarine burial is not now a standard technique; it has never been done as far as I am aware. It will be necessary, therefore, to convince both contractors and regulatory agencies that it can be done before it can be tried. Questions that have to be answered include:

1. What is a suitable disposal site? Is a suitable site available or can one be constructed?
2. What kind of dredged sediment is suitable for burial?
3. How should the dredging and disposal operation be conducted?
4. What type of cover is necessary?

Another important question is "how much will it cost?" I will discuss the technical questions in this article but I will not consider the cost of such operations in detail. If burial can be shown to be environmentally safe and technically feasible, perhaps the haulage costs could be reduced by opening disposal sites closer to the dredging area. New costs would be incurred, however. The clean capping material would have to be supplied and placed on the site. In addition, the discharge operation would have to be more closely controlled and it may be more costly than a usual open-water disposal. I expect that the costs will vary widely from case to case and I will not consider the economics of a burial operation any further here.

THE DISPOSAL SITE

You need a grave to bury dredged sediment at sea. Many pits, holes, or depressions already exist under the coastal waters of the United States. One hundred and twenty-five sites in estuaries, bays, rivers, and continental shelf areas of the Atlantic, Gulf, Pacific, and Great Lakes coasts of the United States have been catalogued (Broughton, 1977) and there are undoubtedly more. Some of these features are natural and some are man-made. Man-made pits include those that remain from submarine sand-mining operations where the sand has been removed to be used for beach nourishment, fill, or construction aggregate. Not all pits, however, will be suitable sites for the burial of dredged sediment. To be a suitable disposal area there should be no evidence of long-term net erosion on the sea floor around the pit so that the cap will not be subject to erosion after the operation is complete. Neither should there be any large bedforms on the ambient sea floor that would indicate that large amounts of sand are moved regularly by the currents. In addition, the pit, hole, or depression must be shown to be a containment site for fine-grained sediment and to be large enough.

Many artificial depressions on the sea floor are natural traps for fine-grained sediment and they are, therefore, likely to be good containment sites for dredged sediment. Pits that remain after submarine sand-mining operations are often found to accumulate mud at very rapid rates (Broughton, 1977; Polis, 1974). Sedimentation rates of several inches per year are not unusual in features like borrow pits or in dredged channels that are not in equilibrium with the environment. In channels in New York Harbor, Olsen(1979) measured sedimentation rates ranging in excess of 6 inches/year using geochemical techniques. Based on dredging records, the sedimentation rates in channels in New York Harbor have been calculated to be up to 39 inches/year with an average value for 27 projects of about 9 inches/year. Similar values were calculated for the channels of harbors in Connecticut (Bokuniewicz et al., 1979) and in channels in Chesapeake Bay (Ludwick, 1981). In borrow pits in New York Harbor, the sedimentation of mud has been shown to have occurred at an average rate of about 3 inches/year. While borrow pits or abandoned channels may be good containment sites for dredged mud, natural depressions or holes may or may not be. Natural depressions could be the result of active scouring by currents and they should be examined carefully before being chosen as a disposal site.

The minimum size of the pit is determined by the spread of the sediment during the discharge process. In shallow water almost all of the dredged sediment spreads over the sea floor in a thin, dense bottom surge of sediment and water. Over a flat bottom, the sediment is deposited within a few hundred yards of the point of release. This has been observed to occur under a wide range of conditions and in water depths up to 220 feet deep (Gordon, 1974; Custar and Wakeman, 1977; Bokuniewicz et al., 1978). In Lake Erie, muddy dredged sediment that was released from a hopper dredge into water 45 feet deep was found to spread no farther than about 220 yards from the discharge point (Bokuniewicz et al., 1978). In New York Harbor, dredged mud discharged from a scow into water 50 feet deep spread no farther than about 150 yards from the discharge point (Bokuniewicz, 1981). In similar situations, a pit with a radius of at least 250 yards should be sufficient to contain the spread of the bottom surge during the discharge.

The depth of the pit is also important. The walls of the pit should serve as a barrier to the spread of the bottom surge as the pit is filled. Little is known about the dynamics of the spreading bottom surge but we can get some idea of its ability to climb the pit walls from energy considerations. Based on the observations around a hopper dredge in Lake Erie

(Bokuniewicz et al., 1978), it appears that side slopes of even a few degrees will severely limit the spread of the bottom surge and that a pit wall 10 to 15 feet high will be an effective barrier (Bokuniewicz, 1981).

THE DREDGED SEDIMENT

Because of the association of many pollutants with fine-grained sediment particles, contaminated sediment is likely to be mud, but not all dredged muds would be suitable for submarine burial. The deposit of dredged mud must have sufficient strength to support the cap and this precludes the use of fluid muds.

The capping operation may result in a layer of denser sediment, perhaps sand, overlying a layer of less dense mud. Such a configuration is inherently unstable, although we know that under certain conditions it can persist for very long periods of time because sand-over-mud layers are preserved in the geologic record. According to Artyushkov (1963) two conditions must be met for internal instability to arise in a layered sediment deposit: (1) the upper layer must be more dense than the lower layer and (2) the shear stress along the interface between the layers must exceed the strength of the deposit. Mathematically the second condition is $\Delta\rho g h > (\alpha\tau)$ where $\Delta\rho g$ is the difference in the unit weights between the layers, h is the height of irregularities in the interface between the layers and $(\alpha\tau)$ is the creep limit stress of the deposit. The creep limit stress is some fraction, α, of the shear strength τ. Although few tests on marine sediments are available, the value of α may be about 0.5 (Sherif et al., 1980). By these criteria it is technically feasible to construct a stable deposit with mud that is typically dredged in a clam-shell dredging operation and such a deposit has probably been constructed in Long Island Sound (Bokuniewicz and Liu, 1981).

THE DREDGING AND DISPOSAL OPERATION

During the disposal operation from a scow or hopper dredge, we should expect less than 5% of the released sediment to remain in suspension and to be dispersed from the disposal site. This conclusion was first put forward by Gordon (1974). He made measurements during disposal operations in Long Island Sound and showed that less than 1% of the dredged silt released at the disposal site remained in suspension long enough to be dispersed by the tides. A similar conclusion was reached by Custar and Wakeman (1977) as a result of observations they made in San Francisco. They concluded that only between 1 and 5% of the mud that was discharged remained in suspension above 6.6 feet of the bottom. They also conducted laboratory experiments that reinforced their conclusion that the disposal operation causes very little disturbance in the upper part of the water column. The same conclusion was reached by Bokuniewicz et al. (1978) from observations that they made for the Dredged Material

Research Program during disposal operations in Puget Sound, Long Island Sound, Lake Erie, and Lake Ontario.

The MITRE Report (1977) also comes to the conclusion that almost all of the released sand and silt will be deposited quickly based on exploratory calculations for the New York Bight using a mathematical model. In the model calculations all the sand and silt were deposited within about 20 minutes and within 200 yards of the point of discharge. For a clay slurry the time may be considerably longer; some of the model runs showed that three hours will be needed to deposit 90% of the clay particles that were released as a slurry from the scow or hopper.

During a normal point-dumping operation with good navigational control, almost all of the sediment can be placed on the pit floor. The most effective form would be that of a truncated cone or pyramid. Its top surface should be relatively flat and below the elevation of the ambient bay floor. At its edges the surface of the deposit will slope downward toward the pit walls so that a shallow trough will be formed inside of the edge of the pit to hinder the escape of the bottom surges during the filling.

The slope of the surface will depend on the type of equipment used and the nature of the dredged sediment. If all of the dredged sediment is deposited from a bottom surge, as will probably be the case if a hopper dredge is used, the slope will be less than 0.05 (3°). This value has been calculated from energy considerations (Bokuniewicz and Gordon, 1980). Such low slopes were found on the flanks of a deposit of dredged sediment in Long Island Sound (Bokuniewicz and Gordon, 1980). A dredged sediment deposit in Chesapeake Bay was found to have a maximum surface slope of 0.01 and an average slope of 0.002 (Biggs, 1970). After a disposal operation using a hopper dredge in Lake Erie, the maximum slope of the deposit surface was found to be 0.005 (Alther and Wyeth, 1980). During laboratory tank tests to simulate the open-water disposal of dredged mud, mounds were formed with slopes on the order of 0.005 (Chase, undated). In all of these cases, it appeared that the sediment had been deposited from a slurry.

If, on the other hand, the sediment is cohesive and removed from a clam-shell dredge, there may be large blocks of material in the descending jet. Some will be broken on impact and add particles to the surge but some will remain intact and accumulate in a pile on the pit floor under the discharge point (Bokuniewicz and Gordon, 1980). In principle, the angle of repose on this pile could reach the angle of repose for coarse material, ~32°, but at a disposal site in Long Island Sound such a pile had side slopes of 0.1 (6°). The radius of this pile was 273 yards (Bokuniewicz and Gordon, 1980) and it contained about 1.5 million yards^3 of sediment. Recently, at the Central Long Island Sound disposal area, two other deposits have been created from dredged sediments from Stamford and

New Haven harbors, Connecticut (Morton, 1980). The larger of the two contains about 150,000 $yards^3$ of mud. This deposit has a radius of about 110 yards and side slopes as steep as 7°. The smaller deposit consists of a mound of mud, which contains 33,000 $yards^3$, and has a radius of 110 yards and side slopes as steep as 6°, covered with a layer of sand. The combined deposit contains about 76,300 $yards^3$ of sediment; the radius is about 220 yards and the side slopes reach angles as high as 8°. The two deposits at the Central Long Island Sound disposal area are both very compact. Their compactness is due in part to the use of point-dumping at a special taut-line disposal buoy. The disposal buoy was designed to maintain its position to within 3 feet and the barges were discharged as close to the buoy as possible.

In order to maintain the favorable qualities of the dredged mud, a clam-shell dredge should be used and, in order to produce a compact deposit, point-dumping from scows is recommended.

CAPPING

The capping sediment must be relatively uncontaminated and, as a result, it is likely to be sand. The technology is available to construct the cap (Johanson, 1978) and a sand cover has been placed over a dredged mud deposit in Long Island Sound by conventional dumping (Morton, 1980).

The cap must be thick enough to prevent resuspension of the underlying contaminated sediment. To resist erosion the grain size of the cap should be the same or slightly coarser than that on the ambient sea floor. Even if net erosion does not occur, however, the sediment may be disturbed to a considerable depth during severe storms. There are little data to suggest how deep a storm disturbance might extend, but on the submerged shoreface off the exposed coast of Long Island the depth of disturbance may be as deep as 3.3 feet (Sanders and Komar, 1975). In protected areas it should be much less. Direct measurements usually find that resuspension is limited to a fraction of an inch at the sediment surface (e.g. Olsen, 1979).

The cap should also be sufficiently thick so that burrowing animals will not reach the mud. Depth of burrowing in marine benthic communities has been a topic of controversy among benthic ecologists for some time. Numerous biological and chemical studies of these infaunal communities have been reported. If given enough information about a particular area, it is possible to make a conservative estimate of the depth of burrowing by infaunal species of those communities. (A. Mitchell, Marine Sciences Research Center, personal communication). It is reported in most studies that the majority (50-85%) of the macrofauna will be found in the upper 0.3 to 0.5 feet of the substrate with certain species expected to burrow to depths of 1 to 2 feet (Myers, 1977; Pratt and O'Connor, 1973; Guinasso and Schink, 1975, Arrhenuis, 1963; MacGintie, 1939; Molander, 1928). Pratt and O'Connor (1973), for example, found that in nearby Long Island Sound most benthic species occurred at depths of less than 0.3 feet but two species penetrated to depths of 1 foot. It would seem, therefore, that a cap thickness of about 24 inches would be adequate to put the buried mud out of the reach of almost all marine animals.

CONCLUSION

In this article, I have attempted to marshall the arguments to show that the burial of dredged sediment beneath the sea floor is a technically feasible disposal option. Such an operation is being designed in New York Harbor and, in addition to the technical questions that were addressed here, a wide range of biological and geochemical questions are being considered also. The technique does appear to be promising. It is clear, however, that this disposal option will require much more careful planning and stricter control than has traditionally been used in open-water disposals.

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BIOGRAPHICAL SKETCH

Dr. Bokuniewicz (Bo´ kun yev´ itch) is an assistant professor at the Marine Sciences Research Center of the State University of New York. He holds his baccalaureate in physics from the University of Illinois and a Ph.D. in geophysics from Yale University. His research interests include sediment transport and coastal processes and for the past 10 years he has applied his research to problems associated with open-water disposal of dredged sediment. He has studied disposal operations in Long Island Sound, Lake Erie, Lake Ontario, Puget Sound, Chesapeake Bay, and the New York Bight. His current research concerns the submarine burial of dredged sediment in New York Harbor.

DREDGING AND LOCKS: FOCUS ON THE NATIONAL WATERWAY STUDY RECOMMENDATION

by James R. Hanchey[1]

INTRODUCTION

Logo

The words "the nation needs and deserves a first class waterway system" have echoed throughout the waterways community ever since LTG Jack Morris', Chief of Engineers, 1975 speech to the Water Resources Council. His push for a national level study six years ago appears farsighted in 1981.

America in Ruins

A 1981 publication of the Council of State Planning Agencies America in Ruins recommended "Congress should direct the Executive Branch to undertake an inventory of public needs as they effect the economy". This sketchy assessment of all public works from water supply systems to transportation identified a national trend of wholesale deferred maintenance, postponed replacement of obsolete public works and cancelled new construction. The findings and conclusions and recommendations of the Congressionally-mandated National Waterways Study support this assessment for water transportation systems.

[1]Director, Institute for Water Resources, Ft. Belvoir, Virginia.

STUDY BACKGROUND

Section 158 Water Resources Act

The architects of the authorizing act--Section 158 of the Water Resources Development Act of 1976--recognized that a national study of the nation's waterways had not been conducted for 70 years. The existing waterway system reflects a project by project sequence of decision making. A national system is in place; however, it is being increasingly threatened by the combination of advancing age, delayed maintenance and replacement, and postponed construction. The National Waterways Study has had the opportunity over the past four years to review the existing system; evaluate its capability to meet national transportation needs as well as national defense and emergency requirements; appraise additional improvements needed to meet these needs, and propose recommendations.

A couple of crucial comments about scope limitations under which the study was performed: The National Waterways Study is a national water transportation study, but is

NOT - a national transportation study
- a national water resource study, or
- a series of project level studies.

Segment Map

Geographically the scope covers the entire U.S. This map depicts the 66 analytical segments and 21 reaches on which the analysis and recommendations are based. Regarding Ports, the assessment is for broad reaches and does not address port specific development. The study identifies reaches which due to the character and volume of traffic make these priority candidates for national level attention. The Base condition from which the plan grew is the system as it existed in 1978 plus navigation projects under construction.

THE EXISTING SYSTEM

The existing shallow draft system reflects Federal investment with a replacement cost estimated to be $24.4 billion in 1977 dollars. The deep draft elements of the existing system would have a replacement cost of $23 billion and the Great Lakes facilities a replacement cost estimated at approximately $5 billion. The existing system provides economical, safe and fuel efficient transportation for 2 billion tons of traffic annually.

Lock 8 on Monongahela

Deferred maintenance and replacement witnessed increasingly during the 1970's, carry the penalties of increased delay and linehaul costs. These in turn contribute to persistent relative price increases in construction and maintenance. The Monongahela River lock 8 as shown in this slide provides an example of rehabilitation deferral. Added inflation and large trade deficits from failure of the U.S. to compete due to costly transportation may well be the unfortunate consequence of do nothing policies.

FORECASTS

How did the National Waterways Study formulate its recommendations--by developing a range of forecasts which reflect several sets of assumptions about the future. These were then set against the physical existing system to establish needs.

Future Traffic Projections. Contractor Data Resources Inc. (DRI) initially developed four basic forecasts based on sets of economic and political assumptions. These forecasts display that the future share of bulk traffic future share moving by waterways is projected to decline due to: (1) increasing relative fuel costs; (2) imposition of user charges (the existing legislation was assumed in NWS analysis); and (3) shifting sources of energy. Rail traffic is projected to increase about 90 percent, while water traffic is projected to expand by less than 50 percent through the year 2003. The National Waterways Study projections show a net diversion away from water to other modes.

Iron and Steel

A defense scenario was added to portray a 5-year buildup for a two-front war. This scenario shifted the commodity mix on the waterway system to domestic energy and to Great Lakes ore sources, with a corresponding stress to the Great Lakes, Illinois, Lower and Middle Mississippi, and Ohio River systems. Conversely grain exports fall.

° The defense scenario places substantially additional traffic on the many systems:

- Greatly increased steel production (114 million tons greater than High Use)

- Sharply higher use of energy products (107 million tons over High Use)

- Reduced grain exports (drop of 35 million tons from high use). Given the balance of reduced exports and imports and the dramatic increase in defense industry commodities such as iron ore and energy, total forecasts are 9 percent higher. The Great Lakes are most dramatically affected by 120 tons inflex. The Ohio River jumps by 48 million tons over high use during the five year conflict.

Coal

A High Coal Export scenario was added to reflect the findings of the Interagency Coal Export Study. Substantial increases of coal exports were the result, with significant impacts on the Ohio system. The Ohio alone is forecasted to move over 280 million tons of coal in year 2003 under this future, 15 million tons over the High Use future. The coastal export reaches of the Mississippi south of Baton Rouge, the Middle Atlantic Coast and Mobile show substantial gains over High Use.

Finally, several independent forecasts of waterway traffic for specific segments and revisions to correct base year traffic statistics form an additional set of forecasts for a total of seven different sets of future conditions.

What do the Collective Forecasts Say? Basically, the forecasts come down to a finding of sharp declines in petroleum movements on the waterway system and a rapid buildup in coal shipments. Grain, iron and steel and chemical continue strong growth, although at lower than historic rates. Overall, the growth rate in waterway movements is expected to be smaller than recent history.

Regional All Commodities

What do the Forecasts Say for Regional Waterway Growth? The Great Lakes, Mississippi River south of Baton Rouge and Ohio River are leaders in absolute growth.

- Substantial growth in the iron and steel industry drives the increases on the Great Lakes.
- Contributing to the Lower Mississippi growth are the collective input from the Upper Mississippi and Illinois systems which will generate 45 percent of the U.S. waterborne grain traffic by the year 2003.
- The Ohio River shows coal growth in excess of 200 percent over the base year of 1977.
- Overall traffic in these prime growth areas is projected to increase:
  - from 30 million tons (1977) to 70 million tons by 2003 on the Upper Mississippi.
  - from 60.0 million (1977) to about 110 million tons by 2003 on the Illinois.
  - from 190 million (1977) to 410 million tons in 2003 on the Great Lakes.
  - on the Ohio from 170 million tons (1977) to 380 million tons in 2003.

EVALUATION OF CAPABILITY OF PRESENT SYSTEM

The forecasts were then applied to the existing physical waterway conditions to assess capability and the need for improvement. Primary measures of waterway capability were:

- Physical shortfalls in lock capacity.
- Obsolescence of structures and channels.
- Linehaul costs and utilization.
- Safety of navigation conditions.

The seven conclusions from the evaluation of the existing system were that:

1. Project traffic exceeded physical capacity on a small number of locks under all scenarios by year 2003.

2. The largestshortfall in lock capacity occurs at the new 1200 foot lock under construction at Lock and Dam 26. The second largest shortfall is at Gallipolis on the Ohio.

3. Under Defense Emergency, the single largest shortfall in lock capacity occurs on the St. Mary's River between Lakes Superior and Huron due to ore shipments. Second in terms of defense impact are the Illinois Waterways locks, particularly Marseilles, and Gallipolis on the Ohio.

4. Twenty-one locks reach 100 percent of physical capacity under one or more futures, and an additional 20 U.S. locks show high levels of congestion by year 2003. About 100 million tons of traffic by year 2003 would be diverted by lack of physical capacity. An additional nine locks require timely attention because of economic and physical obsolescence. Four of these are on the Monongahela and one, Winfield, is on the Kanawha.

5. Agriculture and coal industries are most directly affected by lack of lock capacity. Over two-thirds of unaccommodated traffic is in the four Mississippi Reaches, and the Illinois, Ohio and Great Lakes Reaches. The Ohio fails to serve up to 65 million tons if no actions are taken over the next generation.

Dredging

6. The cost and environmental impact problems for dredging and dredged material disposal are on the way toward resolution. The Corps research program and operating experience combine to reduce the uncertainties of environmental impacts, and show that dredged material can be disposed in acceptable ways.

7. Coastal port development to serve export coal is restricted by inland transportation systems and terminals, and by available channel depths. Coal and grain exports can be substantially expedited by developing several deep water ports capable of handling the modern world fleet.

ANALYSIS OF STRATEGIES

Alternative Strategies

To pose solutions to the evaluated capacity, linehaul cost, obsolesence and safety needs, the study contractor evaluated four major sets of assumptions about the level of funding which might be available for waterway investment, operation and maintenance and criteria for allocating available funding:

Strategy 1 - Continue present trends with fixed budget
Strategy 2 - Refocus present level of resources on present system
Strategy 3 - Refocus additional resources on present system
Strategy 4 - Improve waterways system

Conclusions from Strategies

The four strategies each offered useful management concepts for inclusion into the final program. Each had a failing too: Strategies I and II major failings were that most all needs were not met. Strategy II blindly eliminated existing projects based solely on cost/volume criteria. No consideration was given to economics in this strategy. Strategy III met most lock capacity needs but like I and II failed to consider the physically obsolete structures requiring replacement or gains to be obtained from inland channel modification and deep draft projects. Strategy IV gave more than could be reasonably obtained given economic constraints of the 1980's and environmental limitations. From assessment of all four it is concluded that the historical funding level is inadequate; the system needs to be managed based on national priorities. The system needs to be managed to reduce time and costs and to improve information to achieve this management.

PLAN FORMULATION

NWS Plan

The National Waterways Plan following the lessons learned from each strategy was formulated against all seven forecasts used in the study. It considered safety, increases of coal and grain exports, national defense requirements, obsolescent structures, waterway system linkages as well as environmental limitation. The principal distinction in criteria between the first four strategies and the framework plan was the <u>national goal</u> focus on <u>Economics</u>, <u>Exports</u>, <u>Defense</u>, and <u>Energy</u>. The draft <u>report assigned</u> priorities <u>to the actions in relation to these factors</u> and the number of forecasts supporting the need. Thus a project providing capacity where needed, providing significant reductions in delay costs and linehaul costs and which influence the performance of many other structures under a large number of projections, <u>and</u> contribute to the national goal was classified higher than one not meeting all the criteria.

National Waterways Plan

This slide depicts a summary of the NWS <u>Plan's</u> major identifiable <u>actions</u>. The costs including the existing system components under construction, is about $32 billion 1981 dollars. The 58 locks include a spectrum of lock needs ranging from replacement for obsolescence to additions to the Canadian Great Lakes System. Nearly all safety actions are Coast Guard and are not in their existing program. This in a nutshell, constitutes the framework national level plan, a plan to which an on-going process <u>must</u> be applied. It is only a framework and pieces have to be modified as alternative futures evolve.

Costs

A new construction and maintenance program to meet projected needs has lower costs than one might imagine. The National Waterways Study draft report showed a $22 billion program (in 1977 dollars). This figure includes the Lock and Dam 26, Tenn-Tom and Red River projects under construction. If these projects are removed from the investment requirement and operations-maintenance set aside for annual obligations, the remaining new investment is only $6.2 billion. This is an average of just $190 million a year from 1981 through 2013, - less than the 1969 construction budget.

The national waterways system provides an economical, fuel efficient component of the nation's transport system. Added investment is warranted to keep the system modern and safe. The waterways are prime energy movers, support the economic wealth of the majority of the U.S. metropolitan areas, enhance the nation's ability to compete for foreign grain and coal markets and offer much safer transit of hazardous materials than competing modes. The National Waterways Study lays out a reasonable plan for maintaining the existing system, adding capacity, increasing safety and meeting national defense buildup requirements. The First Class Waterway System as defined in 1981 is modest. No new extensions are shown. The included actions are only for the strongest parts of our existing system.

Cost recovery is a sharp issue for waterway interests. The National Waterways Study outlines the investment, operation and maintenance costs which could be expected in the next generation and shows the substantive returns which would accrue to added funding. Cost recovery should be evaluated in light of the study findings and the substantial equity and economic development issues.

CONCLUSIONS

Increased levels of funding for investment in the waterway system will be required to avoid further severe increases in delays and linehaul costs which will divert traffic away from waterways and increase costs to shippers. Whether the funds come from general tax revenues or user charges, the investment offers substantial economic, fuel, trade and environmental advantages over taking no action.

The U.S. Congress may wish to recast the 1909 authority for replacement of existing navigation projects, to allow consideration of projected future traffic and defense. This would allow the Chief of Engineers to accelerate the study - design - construction process and reduce the time required, by perhaps 4-6 years.

A 1981 first class waterway system is achievable and desirable. The National Waterways Study may serve a useful role in outlining a reasonable approach to that goal.

WHAT IS THE PAYOFF FROM A FIRST CLASS WATERWAY SYSTEM?

° Sharp reductions in linehaul costs up to $553 million in 2003, a real rate of return approximating 7.5 percent. The reduction on the Ohio River alone would be over $100 million annually.

° Safer transit of hazardous commodities. The waterway system is the safest mode of transportation and can handle hazardous commodities at much lower environmental and human risk. A modest investment in safety actions substantially reduces the remaining risk.

° The first class system prepared in the National Waterways Study draft can accommodate a substantive logistics buildup for a two-front war. The waterway system is an important component of national defense capability.

° A first class waterways system supports 87 percent of the nation's metropolitan areas, and consequently is central to economic development and economic recovery.

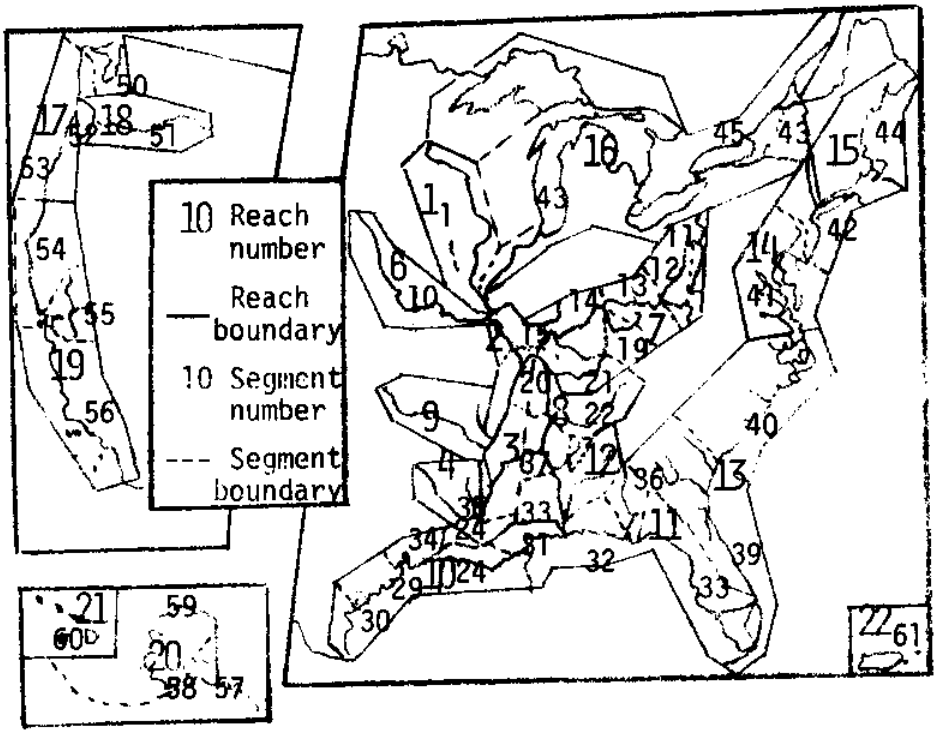
° The first class waterways system can substantially enhance coal and grain sales to foreign markets and thereby enhance the nation's trade balance.

SECTION 158 WATER RESOURCES ACT OF 1976

- Review Waterways
- Assess Capability
- Appraise Improvements
- Recommend Plan

NATIONAL WATERWAYS STUDY

WATERWAY SYSTEM REACHES AND ANALYTICAL SEGMENTS



Note: Boundary lines have no significance in themselves other than to outline the river and coastal areas designated by the reach and segment numbers.

FIGURE
NATIONAL WATERWAY STUDY
US WATERBORNE COMMERCE
1960 - 2003

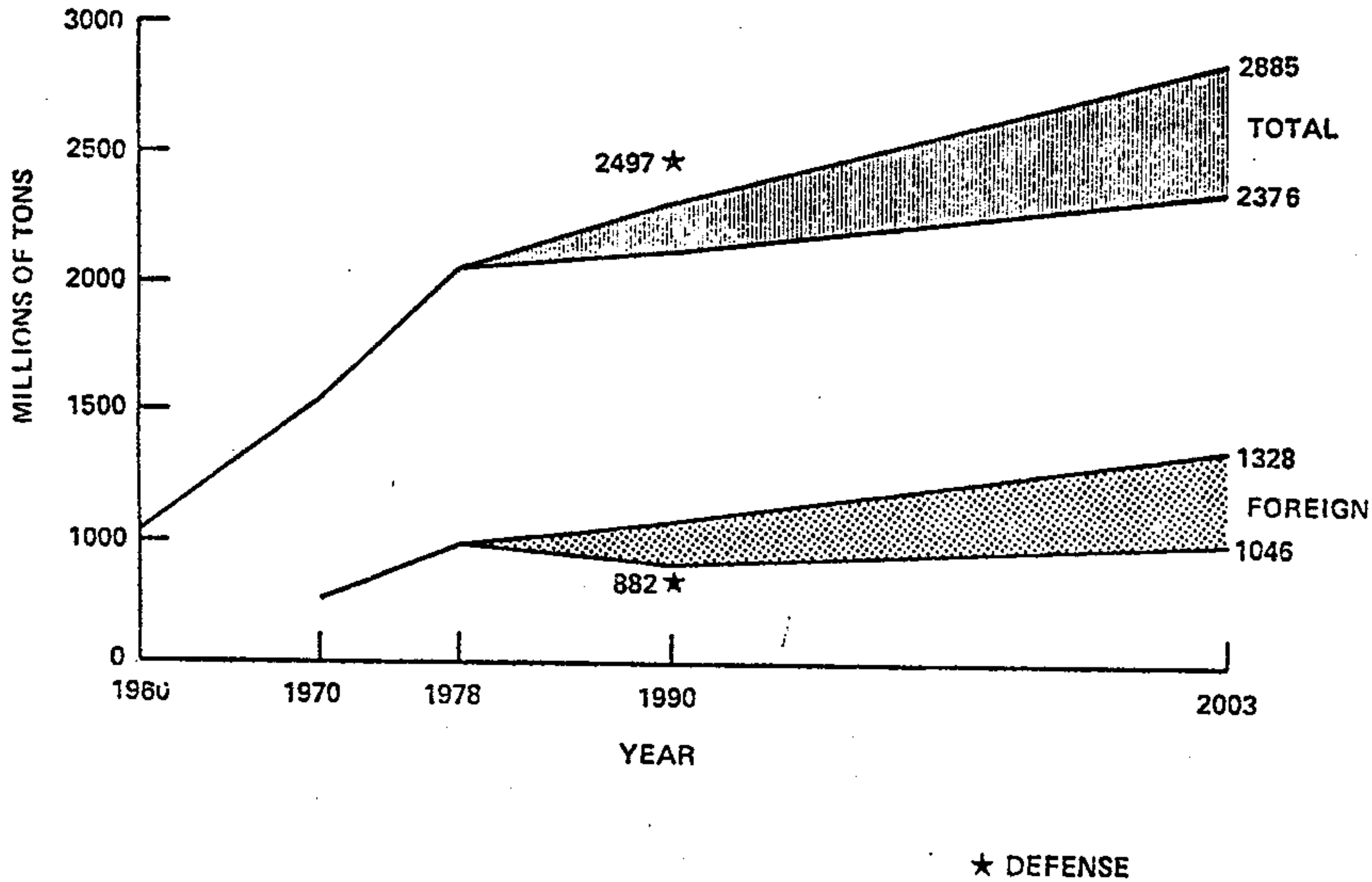


FIGURE
NATIONAL WATERWAY STUDY
US WATERBORNE IRON ORE, IRON & STEEL TRAFFIC
1977 -2003

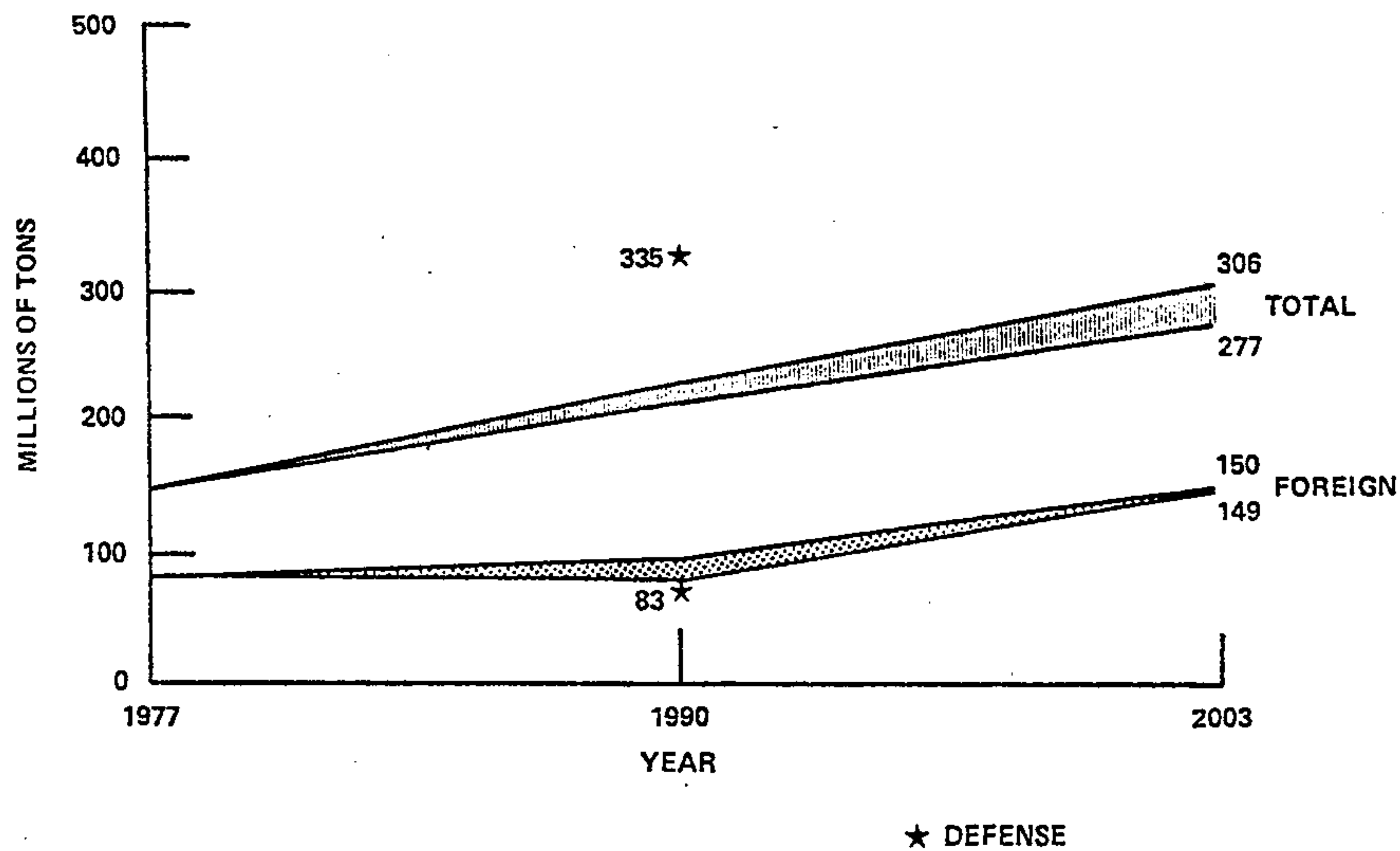
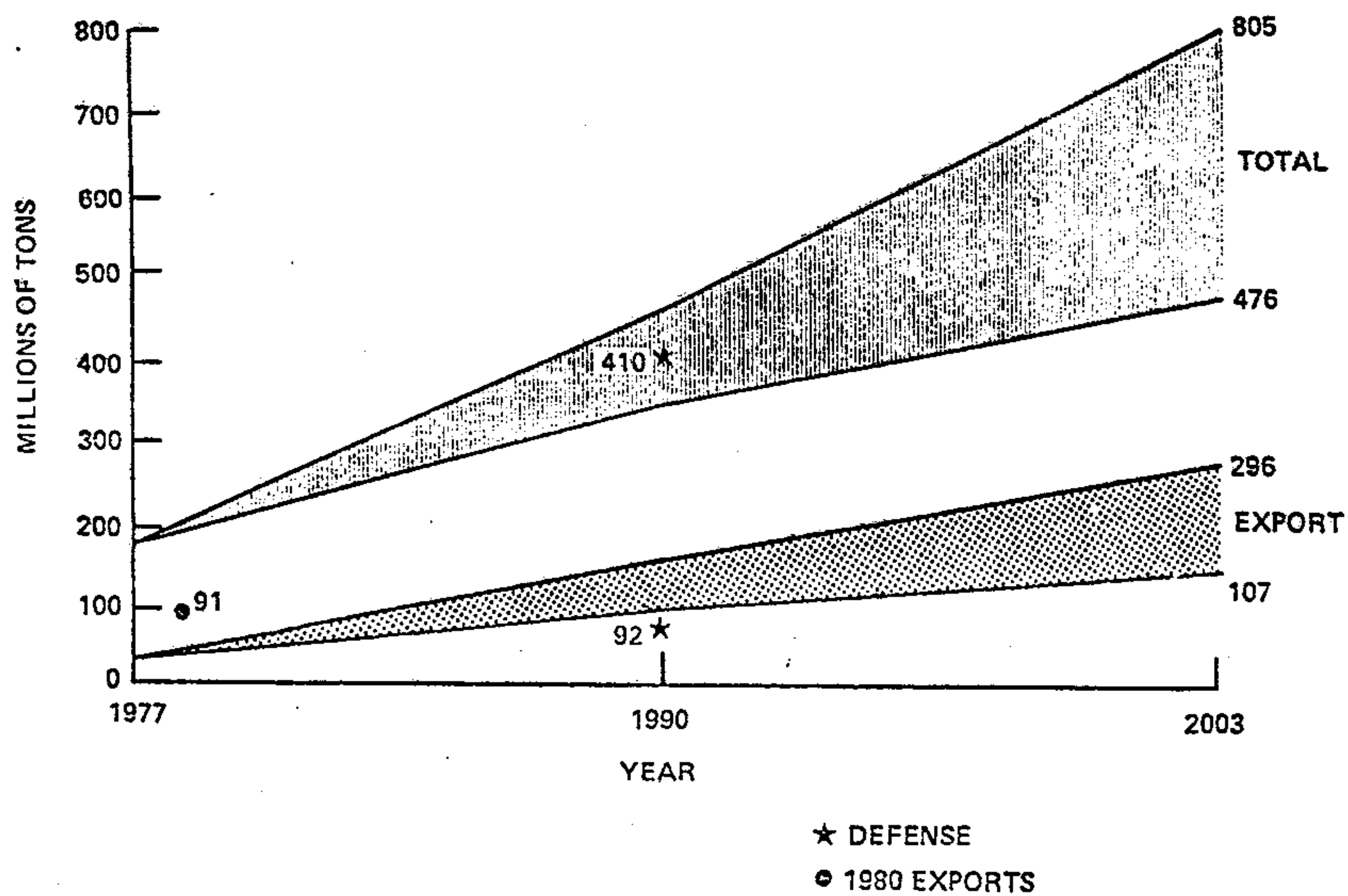
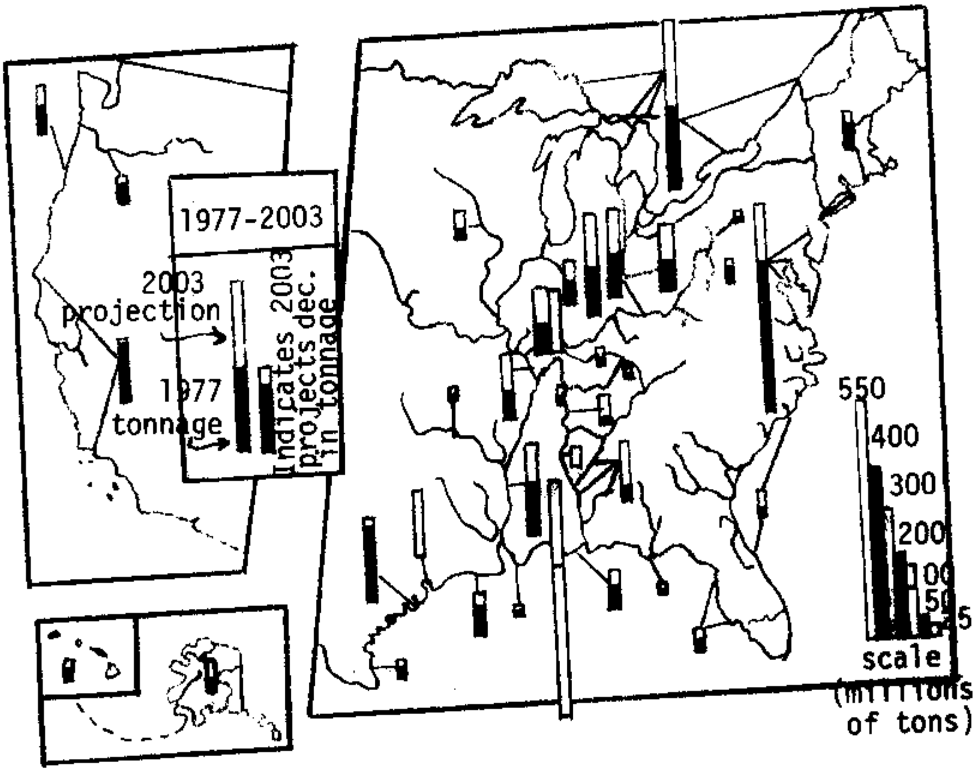


FIGURE .
NATIONAL WATERWAY STUDY
US WATERBORNE COAL TRAFFIC
1977-2003



TOTAL WATERBORNE COMMODITY FLOW



ALTERNATIVE STRATEGIES

- Continue Present Trends
  - Fails to Meet Needs
- Refocus Present System
  - Meet Minimum Requirements, but Lose Small Waterways and Ports
- Fully Fund Present System
  - Meet Minimum Requirements, Postpone Acting on Obsolete Structures
- Improve the Waterway System
  - Costs $24 Billion; Reduces Annual Costs by $460 Million

CONCLUSIONS FROM STRATEGIES

- Funds Fail to Meet Needs
- Manage as Navigation System
- Manage based on National Priorities
- Manage to Reduce Time and Costs
- Manage to Improve Cost and Operating Information Systems

ACTION SUMMARY

	Cumulative Costs (billion 1977 $)
43 U.S. Locks (+15 Canadian)	4.3
4 Shallow Draft Channels	2.0
3 Deep Ports and Others	1.2
Rehabilitation Program	1.2
206 Safety Actions	1.0
Maintenance Dredging	8.4
Other Operations and Maintenance	4.1
Total 1977 $ (1982-2013)	22.2
Total 1981 $ (1982-2013)	32.0

TABLE EVALUATION OF LOCK CONSTRAINTS AND CONGESTION THROUGH YEAR 2003

Reach	Maximum Potential Tonnage Shortfall (10^6 tons)	Primary System Constraint (earliest year[1])	Secondary System Constraint (earliest year[2])	Congested Lock By Year 2003[3]	No. of Futures Congested Constrained[4,5]	Lock Currentl Exceeds 40 Years of Age[6]
Upper Mississippi River[7]	14		Lock 22 (2003)		6	x
				Lock 18	1	x
				Lock 19	5	x
				Lock 20	1	x
				Lock 21	5	x
				Lock 24	5	x
				Lock 25	5	x
Lower Upper Mississippi River[7]	31	Lock 26-2nd Lock (1985)			7*	-
				Lock 27	5	-
Illinois Waterway[7]	18		LaGrange (1995)		7*	x
			Peoria (2000)[8]		7*	x
			Marseilles (2000)[8]		7**	x
				Starved Rock	7*	x
				Dresden Island	3*	x
				Brandon Road	7*	x
				Lockport	7*	x
Ohio River	65	Gallipolis (1995)			7**	x
		Uniontown (2000)			6	-
			Newburgh (2003)		6	-
			McAlpine (2000)		7*	-
			Lock 52 (1995)[8]		6	-
			Lock 53 (2003)[8]		6	-
			Montgomery (2003)		5*	x
				Dashields	1	x
				Emsworth	7*	x
				Smithland	1	-
				Canneltown	1	-
Tennessee River	18		Kentucky (2000)		6	x
Gulf Intracoastal Waterway West				Harvey	6*	x
				Algiers	6*	-
				Vermillion (New)	2*	-
Gulf Intracoastal Waterway East	10	Inner Harbor (1985)			7*	x

TABLE EVALUATION OF LOCK CONSTRAINTS AND CONGESTION THROUGH YEAR 2003 (Continued)

Reach	Maximum Potential Tonnage Shortfall (10^6 tons)	Primary System Constraint (earliest year[1])	Secondary System Constraint (earliest year[2])	Congested Lock By Year 2003[3]	No. of Futures Congested Constrained[4,5]	Lock Currently Exceeds 40 Years of Age[6]
Mobile River and Tributaries	16	Oliver (1990)			7*	x
			Warrior(1990)		6*	-
			Demopolis (1995)		6	-
			Coffeeville (2000)		3	-
			Holt (2000)		6	-
			Bankhead (2003)		1	-
Great Lakes/Saint Lawrence Seaway	17[9]	Welland-Canadian (1995)			7*	x
		Sault St. Marie (1990)			6**	-
Columbia Snake Waterway	6	Bonneville (1990)			1	x
TOTAL	106[10]	15 - 7 U.S. 8 Canadian	15	18		25

[1] Primary constraining lock restricts traffic to other locks in an interrelated locking system. 100% utilization is assumed.
[2] Secondary constraining lock restricts traffic growth to other system locks after additional capacity is provided for the primary constraining lock. 100% utilization is assumed.
[3] Congestion is assumed to begin at 80 percent utilization.
[4] Defense congestion in 1990 is designated by *. Defense constraint in 1990 is designated by **.
[5] Seven forecasts were used to evaluate congestion. This number means that the lock on that same line was congested or constrained under one or more forecasts of future traffic at 80% of its physical capacity.
[6] Age is used as in indicator of obsolesence.
[7] All closely interrelated--Constraining lock severely limits growth within these reaches.
[8] Congestion and/or constraining under low water conditions.
[9] Fails to pass 67 million tons in 1990 under Defense Scenario.
[10] Does not sum because of double counting across reaches.

TABLE

FRAMEWORK PLAN FOR INLAND AND GREAT LAKES LOCKS AND CHANNELS

Reach	Existing System Under Construction	Period I (1982-1995)	Period II (1996-2003)	Period III (2004-2013)
Mississippi	Lock 26* (First Lock) 12 foot Channel South of Cairo, IL	Lock 26* (Second Lock)	Lock 22**	Lock 21,20,19 Lock 24 Lock 25 Lock 27
Illinois	--	LaGrange**	Peoria** Starved Rock Marseilles**	Dresden Island Brandon Road Lockport
Ohio & Tributaries		Gallipolis* Lock 52** Monogahela 3,4,7,8 McAlpine** Uniontown Winfield Montgomery** Deepen Ohio	Newburgh** Emsworth Lock 53**	Kentucky** Dashields
Tennessee	Pickwick			
Oauchita-Black-Red	Locks, Calion and Felsenthal Channel Mod.			
Mobile River & Tributaries		Demopolis** Oliver* Warrior** Widen Channel	Holt** Coffeeville**	Bankhead
Tennessee Tombigbee	Waterway Project	--	--	--
Gulf Intra-coastal Waterway-East	--	Inner Harbor	--	--
Gulf Intra-coastal Waterway-West	Vermilion	Harvey	Algiers	--
Great Lakes	--	Sault St. Marie* Welland (8)*	--	
Columbia	--	Bonneville*	--	--

*Primary Constraints
**Secondary Constraints

NATIONAL WATERWAYS STUDY

Proposed Program (1982-2013)
(millions of 1977 dollars)

INVESTMENT		
EXISTING SYSTEM - UNDER CONSTRUCTION		$ 2,509
MODERNIZE SYSTEM		6,653
Engineers	$6,175	
U.S. Coast Guard	478	
TOTAL		$ 9,162
OPERATIONS & MAINTENANCE (O&M) - CUMULATIVE		
EXISTING SYSTEM (@ $334 year)		$10,688
MODERNIZE SYSTEM - INCREMENT		2,357
Engineers @ $57 year	1,814	
U.S. Coast Guard (VTS) @ $17 yr	543	
TOTAL		$13,054
TOTAL INVESTMENT PLUS CUMULATIVE O&M		$22,207

BIOGRAPHICAL SKETCH

Currently Mr. James R. Hanchey is Director of the Corps of Engineers Institute for Water Resources. He began his career with the Corps of Engineers in 1961 with the New Orleans District; was Chief of Levees Section of Design Branch until leaving the District in 1969 for two years of graduate study at Stanford University. He has been with the Institute for Water Resources at Fort Belvoir, Virginia since 1971. Assignments at IWR included Senior Water Resources Planner (1971-77), Study Manager National Hydropower Study (1977-78), Deputy Director for National Studies (1978-79) and Director since September 1979.

Education - B.S. in Civil Engineering, University of Southwestern Louisiana; M.S. in Civil Engineering, Tulane University; and 2 years graduate study in Engineering - Economic Planning at Stanford University.

As Deputy Director and later Director of IWR, Mr. Hanchey has been responsible for direction of two national water resources studies - the National Hydropower Study and the National Waterways Study.

CONSTRUCTION OF A HOPPER DREDGE

by C. B. Hakenjos[1]

Last Saturday, November 7, 1981 at 10:30 a.m. a momentous occasion happened to the dredging industry of the United States. The largest hopper dredge in this country slid down the ways into the Mississippi River at Avondale Shipyard.

The Dredge "STUYVESANT" will be chartered by the Stuyvesant Dredging Company which is a partnership between subsidiaries of Bos Kalis Westminster, Inc. and Zapata Corporation. This vessel will be crewed and operated by the Williams-McWilliams Company of New Orleans, Louisiana.

The keel for this vessel was laid on February 4, 1981 and nine (9) months later she was launched and will be operational about April 15, 1982, which is a real credit to Avondale Shipyard. Since February of this year I personally have been very close to the construction of this dredge and never ceased to be amazed at its growth every time I visited the shipyard. The complexity of its construction is hard to describe, much less explain to some individuals just what a hopper dredge is and especially how it operates.

Recently I had access to a film from Bos Kalis that showed the construction of one of their hopper dredges, the "PRINCE DE NETHERLAND", which is one of the largest hopper dredges in the world. The "STUYVESANT" was modeled after the "PRINCE DE NETHERLAND" and as I

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[1]Vice President of Marketing and Engineering, Williams-McWilliams Co., New Orleans, Louisiana.

watched the film I could now see the "STUYVESANT" being built. I would like to show you this film to give you an idea of how a hopper dredge is constructed, but before I do, I want to give you a few statistics to compare what you'll see in the film and relate it to our dredge the "STUYVESANT".

The PRINCE is 469' long, ours is 372 feet.

The beams of both vessels are the same at 72'.

The PRINCE has a depth of 40', ours is 34'.

The PRINCE's depth loaded is 33', ours is 29'.

Its displacement loaded is 18,000 tons, ours 16,500 tons and its capacity is 11,700 C.Y., ours is 8,800 C.Y., which is equal to 1,760-5 C.Y. concrete trucks.

The PRINCE can excavate to 110', we can go to 100'.

I might add that one prime reason for this deep capability is that a hopper dredge can be used to excavate offshore pipeline trenches and could also be used in Alaska for the construction of gravel islands in deep water in the Beaufort Sea.

Our pumping capacity, however, is larger than the PRINCE's, which we intend to use for pumping ashore. The PRINCE has two 1400 H.P. each motors on its pumps and we have two 1850 H.P. each motors. Its propulsion, however, has two 8400 H.P. motors while we have 6900 H.P. on each propeller. It has a speed of 18 knots compared to our 12 knot capacity, with a range of 6000 miles.

At the end of the film, I'll try to answer any questions.

And now -- "THE CONSTRUCTION OF A DREDGE."

BIOGRAPHICAL SKETCH

Carl Hakenjos has been with Williams-McWilliams for 32 years and is presently vice-president of marketing and engineering. He graduated from Tulane University in 1950 with a degree in mechanical engineering. His entire professional career has been dedicated to the dredging industry. He was largely responsible years ago for the conception of Williams-McWilliams' hopper dredge, which will be the largest in the United States and was very active with both industry and the Corps in their Industry Capability Program. He is past president of the Mississippi Valley Flood Control Branch of the Associated Contractors of America and is presently Vice-Chairman of the World Organization of Dredging Associations, "WODA"; president of the Western Dredging Association, "WEDA", president of the Gulf Coast Chapter of WEDA, and president of the Gulf Coast Dredging Association.

CONTAMINANT LEVELS IN DISPOSAL AREA EFFLUENTS - PROBLEM IDENTIFICATION AND PROPOSED RATIONALE

by

Michael Robert Palermo*

BACKGROUND

Confined dredged material disposal has increased in recent years, primarily because of constraints on open water disposal of sediments classified as polluted. All solids cannot be effectively retained during the disposal process, and associated contaminants are discharged with the particulates and the solution phase into receiving waters (see Figure 1). This is particularly important for highly contaminated sediments.

Guidance has been developed to predict contaminant release for proposed discharge of dredged material in open water, but no comparative guidance or procedures have yet been developed to predict contaminant levels in effluents from confined disposal operations.

Section 404 of PL 92-500 requires use of elutriate test results to determine if open water discharge is acceptable for a given situation. If the results indicate open water discharge is not acceptable, confined disposal of the dredged material in upland disposal areas is normally required. The effluent from confined disposal sites is also considered as a dredged material discharge under the regulations, and it has often been stated that the environmental impact of confined

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* Chief, Water Resources Engineering Group, Environmental Laboratory, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

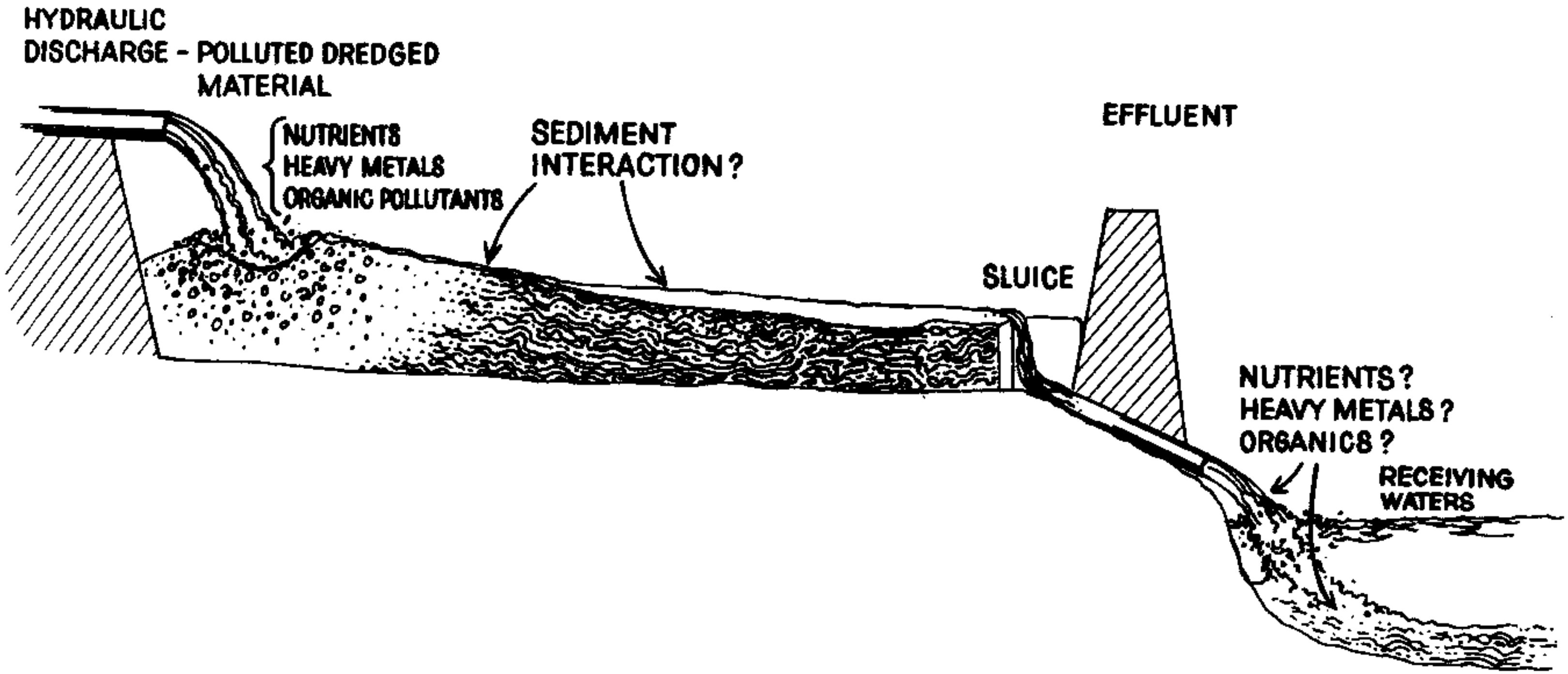

Figure 1. Simplified schematic of environmental interactions in an active confined disposal site (Modified from Hoeppel et al (1978)

disposal of dredged material classified as polluted may be more severe than open water discharge.

Presently, no method exists for prediction of contaminant levels in disposal area effluents during confined disposal operations. Such methods are required so that environmental impacts resulting from confined disposal operations may be properly evaluated.

PROPOSED RESEARCH

The Waterways Experiment Station (WES) will conduct a study under the Corps of Engineers Long-Term Effects of Dredging Operations (LEDO) program to develop a technique for prediction of contaminant levels in confined dredged material disposal area effluents. The work will include development of a laboratory test procedure for determining potential contaminant release, and formulation of a predictive technique for contaminant release considering disposal area characteristics and operational constraints. The laboratory phase will emphasize development of procedures for testing specific sediments to identify which contaminants may be relesed during a confined disposal operation and the relative degree of release. Procedures will be tailored to simulate actual field conditions related to such parameters as retention, type of sedimentation, and geochemical conditions. The predictive technique will involve an extension of the present knowledge of the behavior of disposal area supernatant waters, including flow behavior of particulate and dissolved fractions, and influence of disposal area geometry, withdrawal structure, and operating parameters.

LEGISLATIVE REQUIREMENTS

Engler (1980) contains an excellent summary of the historical evolution of regulatory criteria and guidelines regarding disposal of dredged material. Prior to the late 1960's, the regulation of dredging activities was limited to authority of the Corps of Engineers (CE) regarding obstruction of navigable waterways under the Rivers and Harbors Act of 1890. In the late 1960's, the CE expanded its review authority to include environmental considerations, and the passage of NEPA in 1969 set the stage for an expanded regulatory role.

Regulation of disposal activities has now developed into extensive and detailed procedures and criteria. Potential environmental impacts of ocean disposal must now be determined under Section 103 of the Marine Protection Research and Sanctuaries Act of 1972 (MPRSA). The permit program for inland waters is now governed by Section 404 of the Federal Water Pollution Control Act Amendments of 1972 (FWPCA). Confined dredged material disposal is regulated under Section 404.

Joint CE/EPA guidelines for evaluation of proposed disposal activities are updated by interim implementation and precedural manuals, using as a technical basis the results of current research. The implementation manual for ocean disposal (EPA/CE 1977) contains a thorough explanation of sampling and testing procedures, while the manual for Section 404 has yet to be published. These criteria are now overwhelmingly oriented toward evaluation of proposed open-water disposal. Short-term effects are estimated based on results of the well-known Elutriate Test and in some instances, bioassays. Sediments classified as polluted under the various criteria generally must be placed in diked disposal areas. No comparative development of criteria or guidelines for evaluation of pollution potential of diked disposal area effluents has yet been made. Gambrell (1978), in discussing alternatives for disposal of contaminated sediments, noted that while meaningful procedures have evolved for open water, criteria and testing procedures for confined disposal of contaminated sediments are "in their infancy." Gambrell further stated that little attention has been given to contaminant mobility in confined sites even though this disposal technique would potentially release more toxic materials than open-water disposal.

Recent guidelines (EPA 1980a) have been published to reflect the 1977 Amendments to Section 404 of PL 92-500. Proposed testing requirements (EPA 1980b) define dredged material according to four categories as shown in Figure 2. Category 3 includes contained or confined disposal with "potential for contamination of the water column only." These proposed testing requirements call for a "modified elutriate test" to evaluate short-term water column impacts of disposal area effluents. Specifically, the requirements are worded as follows:

> "Where the containment area is managed for maximum solids retention and, consequently, the liquid is retained for long periods, a modified elutriate test should be used, considering biological, chemical, and physical changes that may occur in the containment area. Settleability tests should be conducted to simulate the actual retention time" (EPA 1980b).

The development of such a modified elutriate test or "supernatant" test will be an integral part of any overall technique for predicting contaminant levels in disposal area effluents. Once contaminant levels in the effluent are predicted, a mixing zone may be considered in evaluation of the proposed discharge. Contaminant levels as reduced by mixing can then be compared with appropriate water quality criteria.

CONFINED DISPOSAL

The major elements of a confined disposal area are illustrated in Figure 3. Dredged material placed in a confined disposal area undergoes sedimentation resulting in a "thickened" deposit of material overlain by

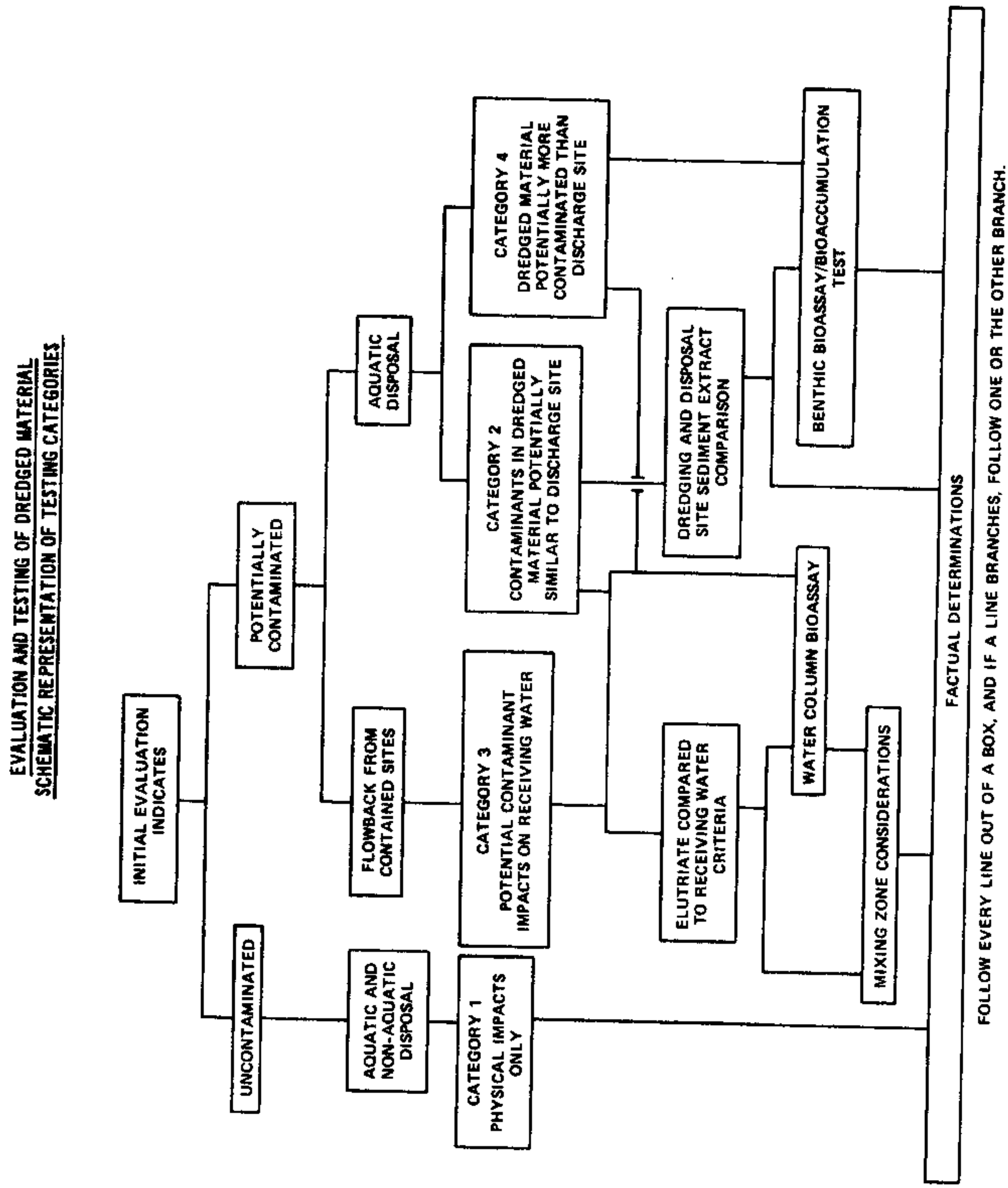

Figure 2. Testing flow chart (from EPA 1980 b.)

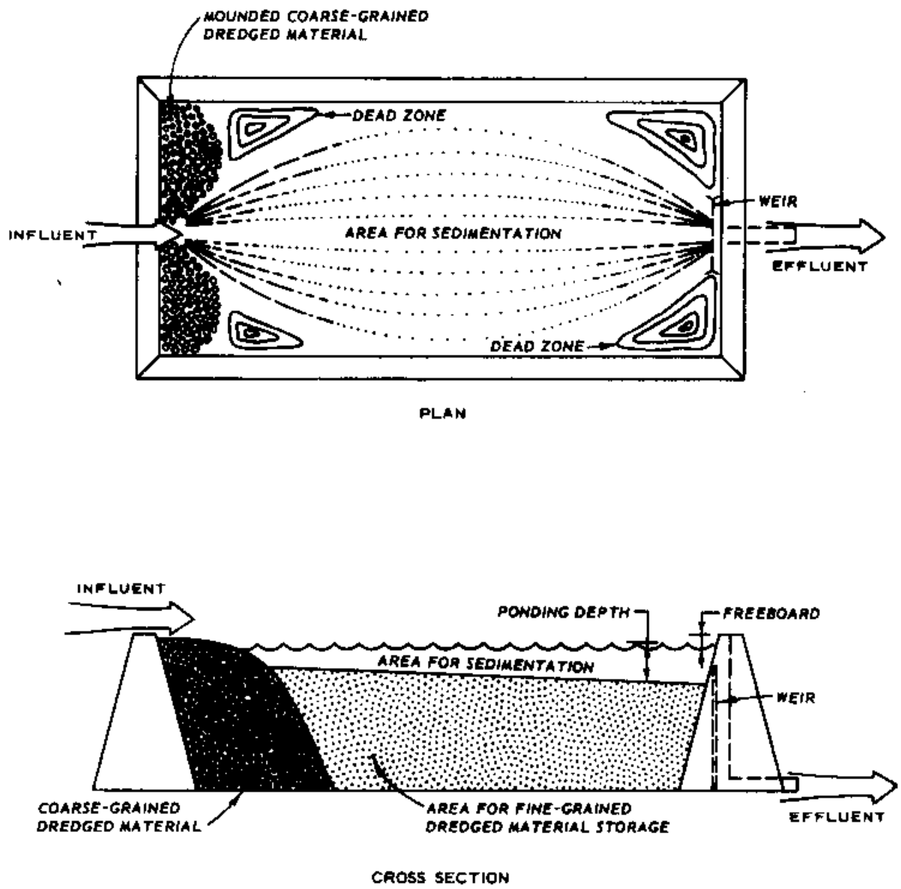

Figure 3. Elements of a confined disposal area (after Montgomery 1978)

a clarified supernatant. Release of these supernatant waters from confined disposal sites occurs after a lengthy detention time, sometimes up to several days. Furthermore, actual withdrawal of the supernatant is governed by hydraulics of the area and discharge weir. Procedures have been developed to predict gross concentrations of suspended solids in the disposal area effluents, taking into account settling behavior of the sediment in question. However, the physical makeup and movement of sediment particles within the supernatant waters is largely unknown. All solids cannot be retained during the disposal process, and associated contaminants are transported with particulates in the effluent to the receiving water.

Sedimentation behavior. Recent research has greatly advanced the state-of-the-art regarding confined disposal and much of this work is applicable to some degree to prediction of contaminent release. The most comprehensive work regarding dredged material sedimentation behavior in confined disposal areas has been conducted by Montgomery (1979). Montgomery found that flocculent or zone settling occurred in dredged material disposal areas. Design procedures were developed for sizing (required surface area and/or ponding depths) to achieve a desired effluent suspended solids concentration. The design procedures were limited to predicting gross concentrations of suspended solids in the effluent. No information regarding associated contaminants or physical characteristics of effluents (such as particle size distribution) was obtained.

Montgomery developed a column settling test to describe either flocculent or zone settling behavior of dredged material slurries. Field verification work conducted by Montgomery has shown that the existing column test procedure adequately simulates field settling behavior of fine-grained dredged material. However, the overall approach of this test, to include column size and materials, may be unsuitable for the analytical work required for prediction of contaminant release.

Hydraulic considerations. Design of confined disposal areas must consider their hydraulic efficiency, defined as the ratio of actual to theoretical detention times. Gallagher (1978) provides a detailed treatment of this topic to include a mathematical analysis of basin hydraulics and discussions of outlet structures, wind effects, and influence of interior or spur dikes. However, no numerical solution or computational procedure for determining hydraulic efficiency was proposed. Montgomery, in his overall development of containment area design, proposes an efficiency factor of approximately 40 percent based on observed field behavior.

Physicochemical environment of confined disposal sites. Gambrell et al. (1978) described conditions relating to contaminant mobility for subaqueous, intertidal, and confined disposal. He determined that short-term impacts (those associated with toxicity or stress during and shortly after disposal) will be due to elevated levels of contaminants associated with particulates in the effluent. In confined disposal sites, dredged material solids may become well oxidized and there may be substantial changes in pH. Removing sediment from an oxygen-free environment and placement in confined disposal sites may affect mobility of several contaminants, depending on the chemical composition of the material. Sediments containing chemically active iron or high levels of total sulfide may become acid under oxidized conditions, further affecting contaminant mobility.

WES EFFLUENT/LEACHATE FIELD STUDIES

Hoeppel et al. (1978) performed a monitoring study of effluents and leachates at nine confined disposal areas. Samples of influent, effluent, and surface receiving water were collected at each site and analyzed for a variety of chemical and physical parameters. The objectives of the study were generally to determine type and extent of contaminants mobilized during disposal and to identify factors controlling mobility.

Dissolved oxygen (DO), temperature, salinity, conductivity, and pH were measured at the time of sample collection. Chemical parameters included various metals, nutrients, and organics.

Hoeppel compared results of influent and effluent samples at eight sites. Results are summarized in Figure 4. Large net decreases were observed for most major elements and metals, generally correlating well with the retention of solids within the disposal area. However, as shown in Figure 4, considerable variability exists between specific chemical parameters. Furthermore, many contaminants now known to be associated with sediments (such as Kepone) were not studied. Hoeppel also developed some data regarding particle size fractionation and found that some constituents may have affinities for different particle sizes and specific gravities.

STANDARD ELUTRIATE TEST

The Standard Elutriate Test (Keeley and Engler 1974) is illustrated in Figure 5. The test procedure was specifially designed to simulate contaminant release for the open-water disposal condition. Some factors which influence test results include solid-liquid seperation and oxidizing conditions. These factors are vastly different under normal field conditions for open-water versus confined disposal.

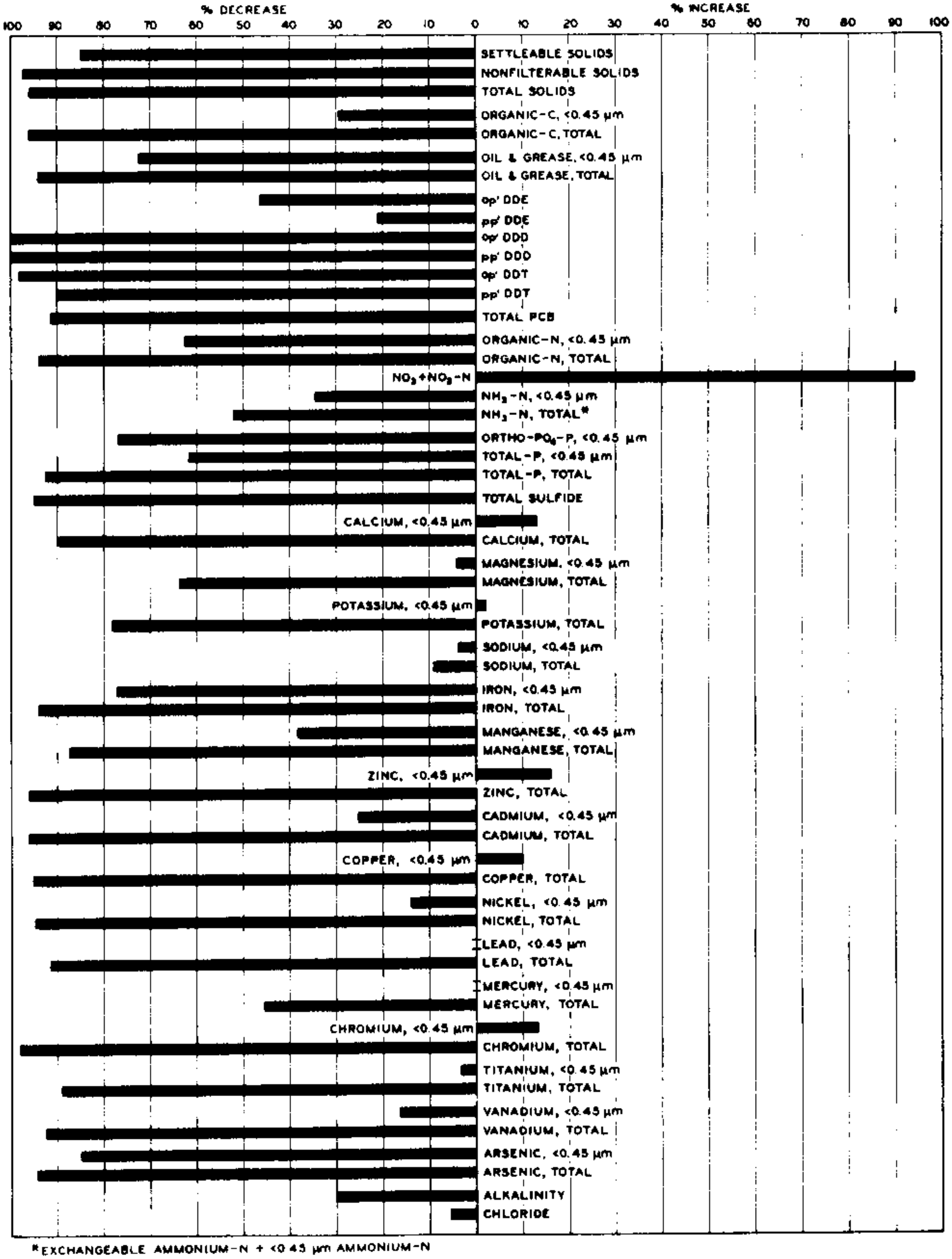

Figure 4. Percent increase or decrease of physical and chemical parameters in total and soluble phase effluent dredged material based on influent-effluent samples from eight land containment areas (after Hoeppel 1978)

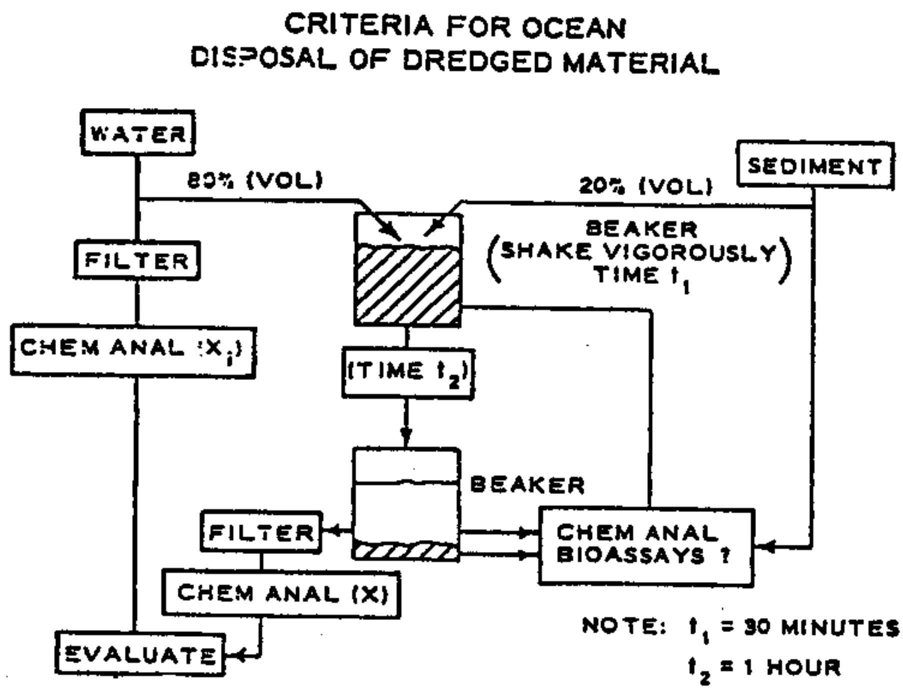

Figure 5. Elutriate test (Modified from Keeley and Engler 1974)

In absence of proven techniques for predicting effluent contaminant release, the standard elutriate test has been used as an indicator and has been evaluated by several studies. Blazevich (1977) for a study on the Duwamish waterway states that the Standard Elutriate Test appears to be valid for most metals, nutrients, and oil and grease. However, the results indicate that only half the pollutants tested were predicted within two times (2x) the observed level in field effluent. These results indicate the test may be valid in predicting <u>which</u> contaminants are released but clearly is not adequate for use in evaluation of proposed discharges from a regulatory standpoint.

Hoeppel et al. (1978) provides a more realistic evaluation of elutriate test applicability for predictive use. He states that the test is inadequate in its present form for predicting concentrations of contaminants and goes on to recommend modifications to the standard procedure. These include: (1) use of a variable settling time for the test to closely approximate field conditions, (2) use of a variable sediment-water mixing ratio, and (3) use of unfiltered supernatant water for analytical testing.

The elutriate test was also evaluated as a predictor for Upper Hudson River dredging operations. O'Brien found (1980) that predicted PCB concentrations from elutriate test results were generally lower than that measured in the effluent. This indicates the standard elutriate test was a nonconservative estimate for PCB concentrations.

SUMMARY OF PRESENT KNOWLEDGE

Although present regulatory policy calls for confined disposal of contaminated sediments, no method exists for prediction of contaminant levels in disposal area effluents. Such methods are needed so that environmental impacts resulting from these activities may be properly evaluated.

Past work related to effluent contaminant levels has been mainly limited to monitoring activities for ongoing disposal operations. A wide variety of contaminants have been studied, and it has been determined that retention of contaminants within the disposal area is generally proportional to retention of particulates. There is evidence that most contaminants, especially heavy metals, are closely associated with particulates; but contaminants may concentrate in smaller particle size fractions. Furthermore, these fractions are most difficult to retain during disposal.

Efforts to predict contaminant release have been generally limited to use of the standard elutriate test which was developed for evaluation of open-water disposal. This approach has proved unsatisfactory and developed predictive techniques must be tailored to simulate the confined disposal condition. Sufficient information exists in the literature to support development of a predictive technique based on laboratory tests and application of the principles of dredged material behavior in confined disposal sites.

PLAN OF STUDY

General approach. The proposed approach is to develop an appropriate laboratory testing rationale and couple this with a model accounting for behavior of supernatant waters within the disposal area. This approach is preferred over a purely mathematical/chemical "formula" for determining effluent release. Laboratory tests conducted on a specific sediment will yield data reflecting the physical/chemical nature of that sediment. The resulting laboratory test must be indicative of which contaminants are present in the supernatant and relative levels associated with the dissolved fraction and particulates of various grain sizes. Results of such an approach can be defended more easily from a regulatory standpoint. Refinement and extension of present knowledge regarding the behavior of supernatant waters will allow determination of which particulates are released with the dissolved fraction.

Preliminary Field Studies. A field site or sites will be identified which will be used for preliminary evaluation of the behavior of supernatant waters. Prior to dredging, samples of sediment will be taken and tests conducted for characterization and evaluation of sedimentation performance. Additional sediment samples will be retained for use in laboratory development of a supernatant test procedure. During subsequent dredging operations, samples will be taken at the influent and effluent points, and also within the disposal area supernatant waters. The purpose of this sampling will be to trace the movement of supernatant particulates and associated contaminants through the disposal area. Suspended solids determinations and grain size analysis will be taken on these samples. The results of the preliminary field study will be used to verify the validity of the supernatant test in simulating the characteristics of particulates in the supernatant waters.

Laboratory Test Development. A laboratory test procedure will be developed for simulation of contaminant release for the confined disposal case reflecting differences in field conditions. The test will be a modified elutriate or "supernatant" test. The supernatant test procedure must determine both grain size distribution within the supernatant waters and associated contaminants. The initial make-up and mixing of sediment and disposal site water, sedimentation procedure, and time allowed for sedimentation will be tailored to simulate the confined disposal field condition just as the elutriate test simulates the open water disposal field condition. New equipment and procedures will be developed specifically for this test.

Pertinent factors which will be considered in developing laboratory tests may be summarized as follows:

a) the test must reflect the retention time and sedimentation regime existing within the disposal area,

b) the test must be adaptable to sediments of varying characteristics,

c) the test must produce a supernatant from which suitable analytical data regarding contaminants may be extracted,

d) the final test must be such that it can be performed by non-research level laboratories normally employed by regulatory agencies.

Methods will be developed to extract representative samples for analytical work during the supernatant test. These samples will be analyzed for contaminants as appropriate. The test results must be indicative of contaminants released into the supernatant under quiescent settling conditions. Fractionation of the supernatant and associated contaminants into a liquid phase and several grain size ranges will be accomplished by stage filtration or other appropriate techniques. In this way contaminant levels associated with grain size ranges can be determined. All analytical work will be accomplished at the WES Environmental Laboratory, one of the best equipped such laboratory facilities in the nation.

Development of Predictive Technique. Once a suitable laboratory "supernatant test" has been developed, it can be included within an overall procedure for predicting contaminant release. The predictive technique must consider not only gradation of particles and associated contaminants in supernatant waters, but also the factors that dictate which of the particles will be released in the effluent. Hydraulic characteristics of the disposal area due to shape and topography, the selective withdrawal characteristics of outlet works, and operational considerations will have a major influence on particle distributions in the effluent. All these factors must be considered in developing an integrated procedure for prediction of contaminant release.

Information gained during the preliminary field study regarding behavior of particulates in disposal area supernatant waters will significantly contribute in development of the predictive technique. The laboratory supernatant test should prove a reliable indicator of both the dissolved and attached contaminants associated with various particle sizes. This information must be linked to the physical behavior of these particle sizes as they are transported through and released from the disposal area. A refinement and/or extension of present knowledge regarding hydraulic characteristics of both the flow through the area and the outlet structure will be required.

The predictive technique will be developed to allow a before-the-fact estimation of effluent contaminant levels using only the results of the laboratory supernatant test and known parameters of the disposal area and dredging operation. Projects involving highly contaminated sediments for which estimates of effluent contaminants are desired would normally be carefully designed and operated. It is likely that the existing design procedures (Montgomery 1979) would be carried out for such projects. Data which would normally be obtained during design will likely be required as part of the predictive methodology. Parameters such as disposal area geometry, surface area, anticipated ponding, dredge size, direction of dredging, etc., must be known prior to estimation of effluent contaminant levels.

INTENDED USE OF RESULTS

The technique for prediction of contaminant levels in disposal area effluents and the associated supernatant laboratory test procedure will become an integral part of the Corps of Engineers regulatory program under Section 404 of the FWPCA. An implementation manual for Section 404 permit evaluations is currently being prepared jointly by the EPA/CE Technical Committee on Criteria for Dredged and Fill Material. Representatives of the committee have stated that the procedures developed during this study will be included in subsequent revisions of the 404 implementation manual. This will be mandated when the proposed testing requirements (Federal Register 1980b) become law.

The procedures will then be used by Corps field offices and others in evaluation of all Category 3 (apparently contaminated material placed in confined disposal sites) disposal activities, including permitted activites. Considering the present policy regarding confinement of contaminated sediments, the use of the supernatant test procedure and predictive technique should become quite extensive.

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BIOGRAPHICAL SKETCH

Mr. Palermo is Chief of the Water Resources Engineering Group of the Environmental Laboratory at the Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi. As a Research Civil Engineer, he is responsible for planning, managing, and performing studies under the various research programs conducted by the Environmental Laboratory.

Mr. Palermo received his BS and MS degrees in Civil Engineering from Mississippi State University and has completed additional graduate studies at Vanderbilt University.

While at the Waterways Experiment Station, Mr. Palermo was an active participant in the Corps of Engineers' Dredged Material Research Program and was involved in studies concerning various aspects of dredging and dredged material disposal technology, including design, operation, and management of containment areas, fine-grained dredged material dewatering, and marshland development using dredged material. He is currently involved with research concerning dredging and disposal of contaminated sediments.

Mr. Palermo is a member of the Vicksburg Engineers Club, Chi Epsilon, Society of American Military Engineers, and American Society of Civil Engineers. He is a Registered Professional Engineer in the State of Mississippi.

EFFECTS OF SUSPENDED VOLCANIC ASH ON SUSCEPTIBILITY OF FISH TO DISEASE

T. M. Poston[1], T. L. Page[2], and R. W Hanf, Jr.[3]

ABSTRACT

Juvenile rainbow trout exposed to ash from the May 18, 1980, Mt. St. Helens eruption and subsequently exposed to a naturally occurring pathogenic bacterium (Flexibacter columnaris) had a greater mortality than fish exposed to ash or the pathogen alone. There was no difference in mortality to fish exposed to ash from this recent eruption compared to weathered ash from an eruption approximately 4000 years earlier. There was no difference in infection rates between weathered and "new" ash. It is likely that similar impacts may result from exposure of natural populations of fish to sediment plumes caused by dredging operations. Levels of suspended ash that enhanced the susceptibility of rainbow trout to infection by F. columnaris fall 1-2 orders of magnitude below reported lethal concentrations of suspended solids.

INTRODUCTION

On May 18, 1980, Mt. St. Helens, located in southwestern Washington state, erupted with a force estimated to be 500 times more powerful than the nuclear blast at Hiroshima. The immediate impact on both terrestrial and aquatic habitats was devastating. Approximately 200 square miles of land northwest and northeast of the mountain were completely devastated

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by the pyroclastic blast. A cubic kilometer of mountain was blown away in one cataclysmic moment, removing 1300 feet from the summit of Mt. St. Helens. Tremendous mudflows raced down the north and south forks of the Toutle River ultimately entering the Columbia River and clogging existing shipping channels to upstream ports.

The ash plume, which was blown eastward, blanketed the countryside with ash at depths ranging from 2 to 70 mm as far east as western Montana. This ash has found its way into many streams and lakes, either through direct deposition, wind-aided resuspension and deposition into bodies of water, or as surface runoff following episodes of rainfall. Because of the abrasive nature of the ash, it was hypothesized that gills of freshwater fish could be damaged by exposure to aqueous suspensions of ash and result in an enhanced susceptibility of exposed fish to a naturally occurring pathogenic bacterium, Flexibacter columnaris.

This hypothesis was tested under several sets of experimental conditions to determine the relationships between the concentrations of suspended ash, the age of the ash, the virulence of the pathogen, and the concentration of the pathogen. Although the experiments were designed to simulate an ashfall of 1-cm thickness over a shallow stream, the implications of the research are applicable to potential effects that dredge spoil plumes could have on resident populations of fish.

METHODS AND MATERIALS

Volcanic Ash

Two sources of ash were used in these studies. "Pristine ash" was collected at Vantage, Washington, 11 days following the May 18 eruption.

"Weathered ash" from an eruption ~4000 years ago was collected at Snoqualmie Pass, Washington. Both ash samples were separated into discrete fractions by sieving. Based on particle size distributions, the weathered ash fractions were reconstituted to match the pristine ash (Table 1) from the May 18th eruption.

Exposure Aquaria

Exposure aquaria measured ~70 cm by ~20 cm by ~30 cm and contained ~25 L. Each aquaria was placed on two Thermolyne® mag-stirrers with two-inch Teflon®-coated stir bars (Figure 1). A 3/8 in. (9.7 mm) mesh Vexar® floor was installed ~2.5 cm above the stir bars. Aeration was provided by a perforated air tube running the length of the aquaria and secured to the Vexar® floor. To minimize settling of ash in the corners, each aquaria had plexiglass half cylinders (30 cm by 10 cm radius) positioned vertically at the ends of the aquaria (not pictured). These half cylinders reduced the surface area of the aquaria to 1100 cm^2. Temperature was maintained at $20.0 \pm 1.0^{o}C$ by a heat exchanger manifold plumbed to a thermally regulated water source.

Pathogens

Two separate strains of F. columnaris were used in these experiments. A low-virulence strain was isolated from an infected rainbow trout at the Battelle Northwest facilities in February 1981. Using methods described by Strand (7), this strain resulted in a 48-hour 10% mortality rate in rainbow trout exposed for 30 min to a concentration of 9×10^6 cells/ml F. columnaris. A high-virulence strain, producing 80% mortality under identical conditions, was obtained from the collection of Dr.

TABLE 1. Particle Size Distribution of Pristine and Weathered Ash.

Seive Number	Seive Size, μm	Percent Composition	
		Weathered Ash wt %	Pristine Ash wt %
35	417	16.2	3.5
70	212	37.1	5.0
100	149	8.4	15.9
200	75	10.7	23.1
325	45	6.1	4.7
Pan	—	21.5	47.8

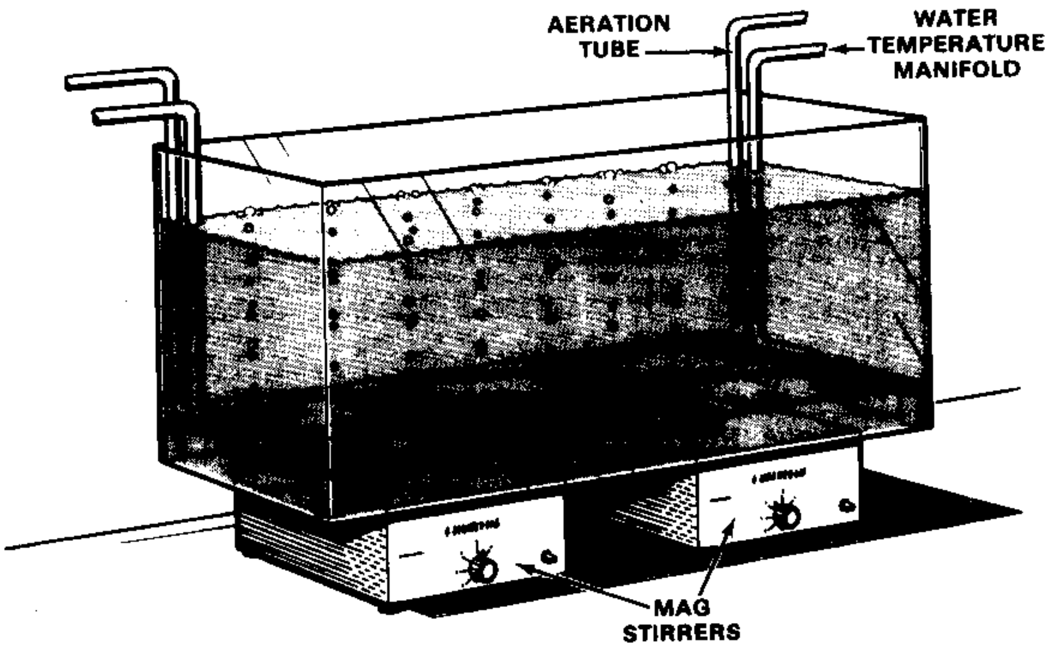

FIG. 1. - Ash Exposure Apparatus

Robert Pacha of Central Washington University, Ellensburg, Washington.

For testing purposes, pure cultures of F. columnaris were grown in culture broth containing 0.3% tryptone (weight to volume) and 3.0% yeast infusion (volume to volume) at a pH of 7.0. After 24-hours growth at 24°C with shaking, cell density was measured (Klett Colorimeter®, 535 μ) and adjusted with Columbia River water to a theoretical concentration of 1.8×10^8 cells/mL. The exposure solution was prepared by adding 100 mL of this solution to 1900 mL river water to yield exposure concentrations of 9×10^6 cells/mL.

Test Species

Juvenile rainbow trout were obtained at the Battelle Northwest facilities in Richland, Washington. The mean weight and lengths (± 1.0 standard deviation) of the experimental fish were 3.69 ± 1.21 g and 68.3 ± 6.8 mm, respectively. The experimental fish were acclimated to the test temperature of 20 ± 1°C several weeks prior to the start of the tests.

Experimental Protocol

Two separate tests were conducted. In the first test, 35 rainbow trout were added to each of three 25-L aquaria. Equal volumes (1100 mL), of pristine and weathered ash, each comparable to 1 cm of ash fallout, was sieved onto the water surface of separate aquaria to simulate fallout. The pristine ash, which weighed 1310 g, was easily dispersed, whereas the weathered ash, which weighed 818 g, formed clumps and tended to float. The third aquarium received no ash and served as a control exposure.

Ash exposures were terminated after approximately 10 hours by transferring the fish from the aquaria to polyethylene buckets supplied with airstones containing 1900 ml of Columbia River water. Thirty of the fish, taken 10 at a time, were segregated this way into separate buckets. The five remaining fish in each aquaria were used for gill examination.

For each exposure aquarium, there were three buckets containing ten fish each. Buckets received either 100 mL of high virulence F. columnaris culture, 1.8×10^6 cells/mL; 100 mL of low virulence F. columnaris culture, 1.8×10^6 cells/mL; or 100 ml of river water. The F. columnaris exposures lasted for 30 min, after which the fish from each bucket were removed, divided into two groups, and transferred to either static (with aeration) or flow-through 25-L aquaria. Mortality and signs of infection were monitored for the following 48 hours.

In the second experiment, pristine ash alone was tested at two levels, 0.3g/L and 11.5 g/L per aquaria. Because the heaviest particles settled out, the ash was added ~8 hrs prior to the addition of fish. This provided a relatively constant exposure of suspended solids for the duration of exposure.

Exposures to the pathogen were the same as previously described, except that the sample size was increased to 15 fish per bucket and all the fish were held after exposure in a flow-through system for 96 hours. After the ash exposure, fish were exposed to two concentrations of F. columnaris; 9×10^6 cells/mL, as in the first experiment, and 20 cells/mL. No ash-control fish were tested because it had been demonstrated in previous experiments that the 10-hour exposure was not lethal.

Measurements of dissolved oxygen, pH, temperature, turbidity and gravimetric analyses of total suspended solids were taken at the start and termination of the ash exposures (Table 2). All the experiments were conducted at 20 $\pm$ 1^{o}C with fish that had been acclimated to 20^{o}C in Columbia River water.

Histopathy

Following exposure to ash solutions, five fish were sampled from each aquaria for examination of gill damage. The branchial basket was removed in one piece and preserved in 10% neutral buffered formalin. The gills were sectioned transversely on the dorsal portion and frontally on the ventral portion of the gill arches. Samples from each fish were stained with hematoxylin and eosin and a separate periodic acid shift technique.

RESULTS

In the initial experiment, ash was added to each aquaria to simulate 1 cm of ash fallout. Immediately, and as the exposure progressed, the heavier particles settled into "dead" areas of the exposure aquaria which resulted in reductions of suspended solids (Table 2). The settling of ash resulted in reductions of suspended solids as high as 73% at the start of the ash exposure and over 99% at the end of the exposures. Consequently, the fish were being exposed to smaller, potentially less abrasive particles at concentrations that were considerably lower than the amount added to the aquaria.

TABLE 2. Water Quality and Ash Concentrations in Exposure Aquaria

	D.O. (mg/L)	pH Units	Temp (^{o}C)	Suspended Ash (g/L)	Turbidity (JTU)
Experiment 1 (pristine and weathered ash)					
Start					
Control	9.2	7.9	19.9	0	a
Pristine ash	9.2	7.9	19.7	17.2	a
Weathered ash	8.8	6.4	19.9	8.9	a
End					
Control	8.6	7.9	20.0	0	5
Pristine ash	8.6	7.9	19.7	9.5	2200
Weathered ash	8.3	6.6	20.2	0.6	320
Experiment 2 (pristine ash)					
Start					
Control	8.9	8.1	21.2	0	10
0.3 g/L	8.9	8.1	20.4	0.3	140
11.5 g/L	8.9	8.1	20.8	11.5	3100
End					
Control	10.3	7.8	20.0	0	15
0.3 g/L	10.3	7.8	20.6	0[b]	65
11.5 g/L	9.7	7.7	20.4	8.4	2300

(a) Not measured due to excessive settling of ash in measurement tube.
(b) Sample suspect-error possible error in tare weight.

Histopathological examination of gill tissue revealed no evidence of epithelial sloughing, separation or damage. Mild to moderate congestion of the gill lamellae was observed in all groups, including controls, but could not be attributed to ash exposure.

The first experiment compared pristine ash with weathered ash at equal volumes (1100 mL ash), but different weights. The weathered ash caused a reduction in pH (slight acidification) of the water. This did not appear to affect the response of the fish to cultures of high- or low-virulence _F. columnaris_ (Figure 2). Under static monitoring conditions, the rate of infection appeared to be slightly accelerated when compared to flow-through aquaria. This may have been caused by an accumulation of _F. columnaris_ from infected fish in the aquaria. Under flow-through conditions, these "fugitive" bacteria were flushed out. Ash exposure, on its own accord, was not lethal to rainbow trout.

The effects of ash on the enhanced susceptibility of fish to _F. columnaris_ of low virulence was more clearly defined than in the tests with the high-virulence strain (Figure 2). Mortality was minimal in the control fish, whereas almost 100% of the fish exposed to ash and pathogen died within 48 hours. Total mortality within 48 hours was observed in ash exposed and control fish exposed to high-virulence _F. columnaris_, however the mortality rate appeared to be slightly accelerated in the ash-exposed fish. Many, but not all fish, had gill lesions or dorsal saddles characteristic of columnaris disease in juvenile salmonids (1).

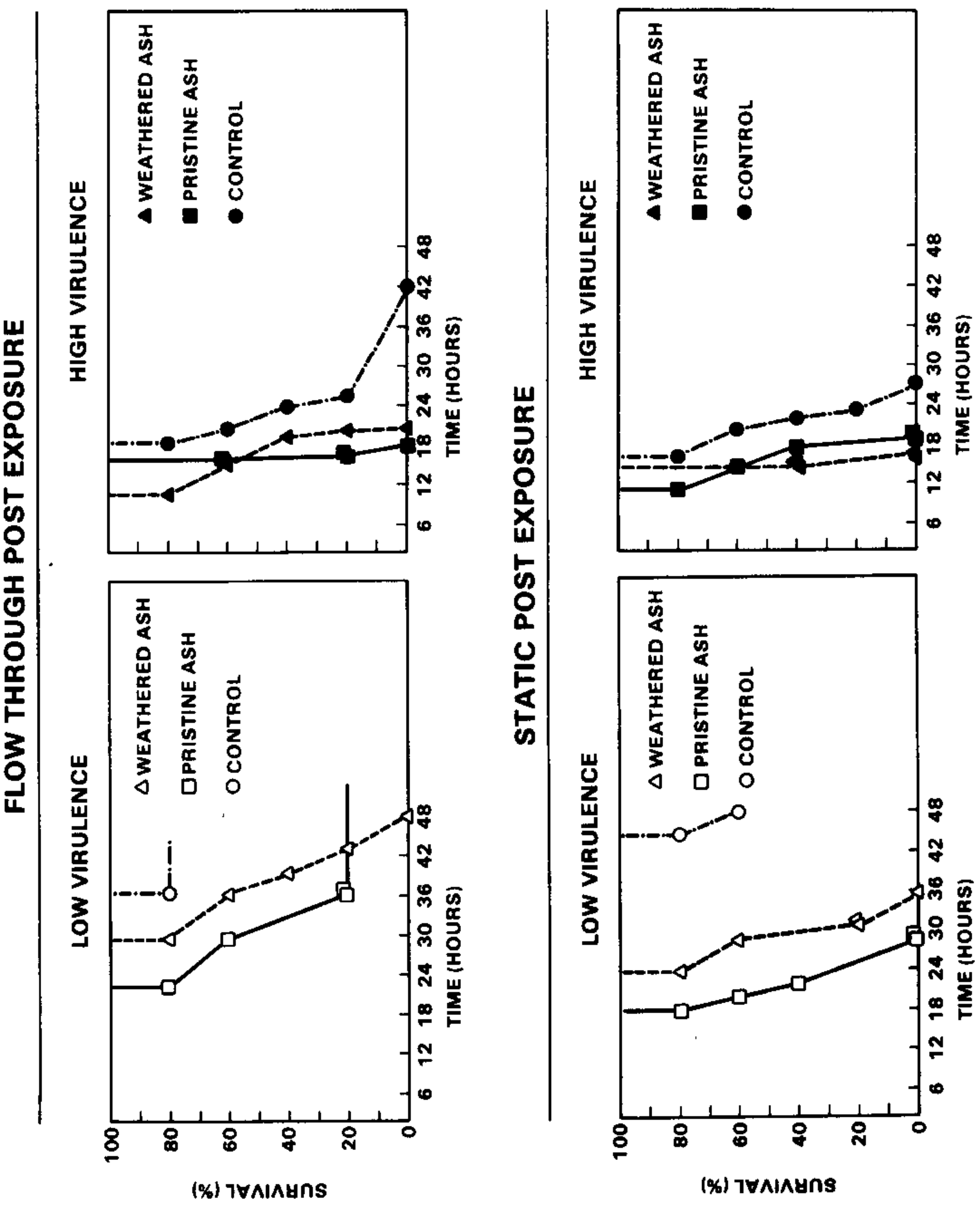

FIG. 2. - Survival of Rainbow Trout following Exposure to Volcanic Ash and F. columnaris; First Exposure.

Based on the response of control fish in the second experiment, there appeared to be a loss of virulence in both strains of columnaris between the first and second experiments. Enhanced disease susceptibility was evident with both strains of _F. columnaris_ at 11.5 g/L ash; however, there was no difference between the 0.3 g/L ash exposed fish and control fish with the low-virulence strain (Figure 3). A gradation of enhanced susceptibility as a function of ash concentration was apparent in the fish that were challenged with the high-virulence _F. columnaris_ (Figure 4). No mortalities could be attributed to exposure of either high- or low-virulence _F. columnaris_ at the hypothetical environmental concentration of 20 cells/mL. It is not known how the apparent loss of virulence in the _F. columnaris_ cultures affected this test.

DISCUSSION

Under the prescribed laboratory conditions, an enhancement of susceptibility to infection of laboratory cultured _F. columnaris_ was demonstrated in fish exposed to weathered or pristine volcanic ash. Efforts to demonstrate this effect at concentrations of _F. columnaris_ approaching environmental levels (20 cells/ml) were not successful; however if the cultured bacterium had not lost their virulence, and perhaps if larger groups of fish had been exposed, it is possible that enhanced susceptibility may have been demonstrated.

The ecological implications of this work are tentative. The experimental protocol did not allow for the simultaneous exposure of fish to ash and pathogen. It is possible that suspended ash may also have a

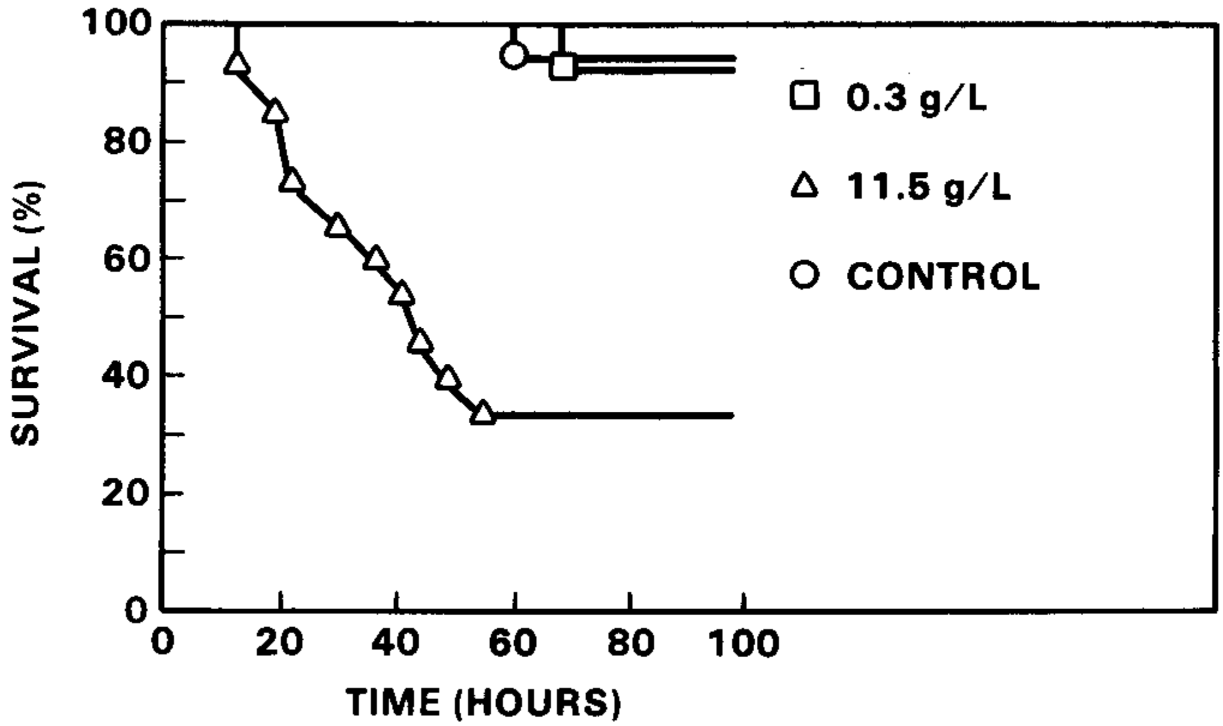

FIG. 3. - Survival of Rainbow Trout Exposed to Low Virulence F. Columnaris Following Exposure to Ash.

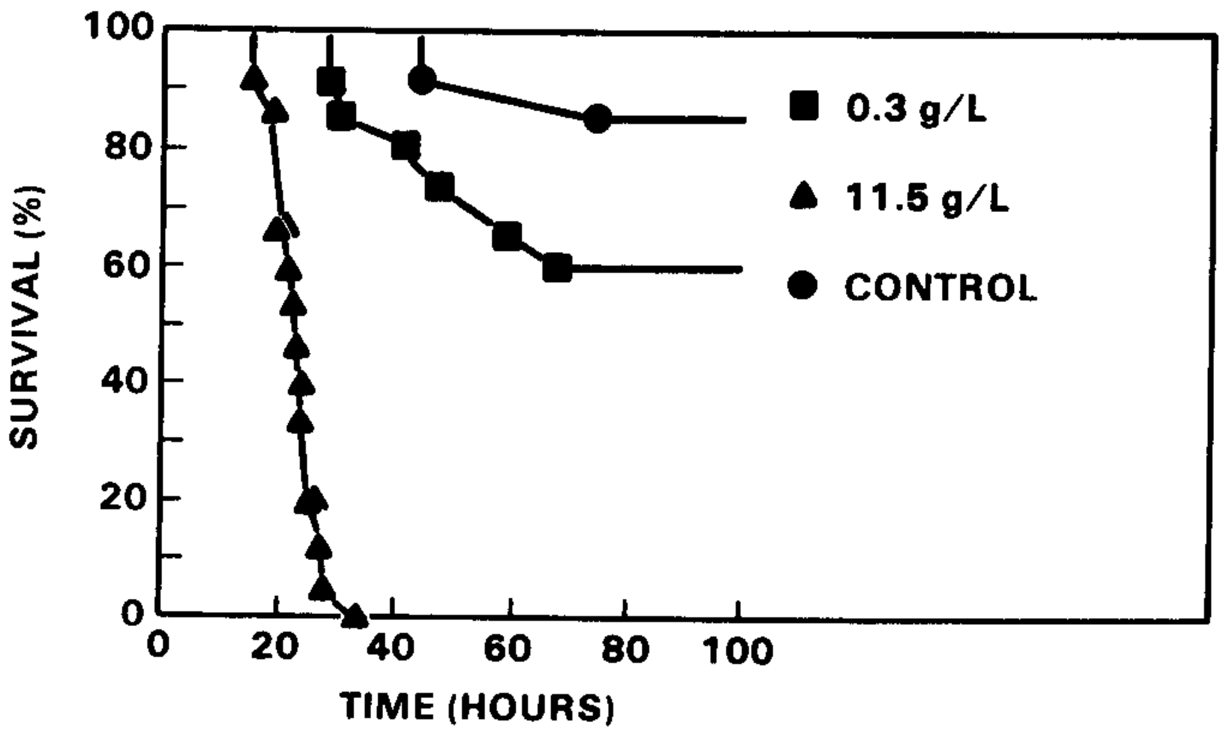

FIG. 4. - Survival of Rainbow Trout Exposed to High Virulence F. columnaris following Exposure to Ash.

deleterious impact on the pathogenic bacterium, or that it may alter the mechanism of infection. Apart from the interactions between pathogen and fish, many environmental factors such as D.O., pH, and temperature, (which were controlled in these experiments) vary significantly in the "real world" and greatly influence the incidence of disease in natural populations of fish. Epizootics of F. columnaris in natural populations of fish is usually correlated with high water temperature. Under hatchery conditions, crowding and reduced flow rates may also increase the incidence of F. columnaris outbreaks.

Although volcanic eruptions are rare and isolated events, research of this type should not be superficially viewed as novel and of little applied value. Erosion in the area devastated by the May 18th blast will continue to fill streams and rivers with the suspended ash and soil until the area is revegetated. Apart from the direct impacts (i.e., smothering of benthic organsims, abrasion, etc.) that these increased sediment burdens will have on stream ecosystems, the increased sediment loads may ultimately fill shipping channels in the Columbia River. Consequently, maintenance dredging in the Columbia River may increase over the short term. Resuspension of volcanic ash due to dredging is another vector of exposure that could impact resident populations of fish. Although volcanic ash may be regarded as an abrasive material, the smaller particles that remained in suspension probably would be less abrasive than the larger particles that settled out. This "finer" suspension may in fact be quite similar to resuspended solids resulting from the dredging of typical river or estuary sediments.

Reported effects of suspended solids on fish, when expressed in terms of an LC_x (i.e., the concentration that kills X% of a population), range from an LC_{10} of 1 g/L for the shiner perch (Cymatogaster aggregata) after 100-hours exposure (3) to an LC_{50} of 107 g/L for the mummichog (Fundulus heteroclitus) after 24-hours exposure (8). Direct effects of suspended solids in other aquatic organisms from other phyla occur at higher concentrations. Regarding duration of exposure and concentration of suspended solids, enhanced susceptibility of trout to F. columnaris was demonstrated under much less severe conditions.

The primary concerns of the environmental impacts of dredging are currently focused on the potential bioavailability of sediment contaminants that may occur when the sediments are resuspended (2). Recent research in this field suggests that the potential is low for biological uptake and transfer of the more prevalent contaminants in sediments (trace metals, pesticides); however, the physical effects of continued exposure of aquatic organisms to suspended sediments is exacerbated by the presence of industrial contaminants (5, 8). The specific interactions between the suspended sediments, contaminants and toxicity are not understood; nor is it understood how exposures to sediment bound contaminants may affect the immunity of fish to disease.

Several investigators have demonstrated a suppression of the immune response in fish exposed to trace metals. Levels of humoral antibody specific to MS2 bacteriophage were suppressed in brown trout (Salmo trutta) and mirror carp (Cyprinus carpio) continuously exposed to selected trace metals over 38 weeks (4). There was a significant suppression of the primary immune response in both species as a result of exposure to nickel (750 μg/L), copper (290 μg/L) and chromium (1.0 mg/L). The immune response of the blue gourami (Trichogaster trichopterus) was inhibited by exposure of fish to sublethal concentrations of copper and methyl mercury (6). It is possible that a synergistic relationship may exist between suspended sediments, pollutants and the disease susceptibility of fish.

It is suggested that the interaction between fish and disease may be applied as a biological indicator of potential stress resulting from exposure to resuspended sediments. It is possible that a previous history of exposure by fish to environmental contaminants, either in the water column or in the sediments, may predispose residence populations of fish to a greater susceptibility to endemic pathogens. Knowledge of these relationships could provide useful information for assessing the potential effects of dredging operations. Adverse effects may be determined at concentrations of suspended sediments lower than current methods are capable of detecting. Information concerning the resistance to disease of fish exposed to either contaminated or uncontaminated sediments may be useful for assessing the impacts of dredging operations.

ACKNOWLEDGEMENTS

Histopathology of gill tissue was conducted by Rodney A. Miller, veterinary pathologist from the Biology Department, Battelle. Ms. Carolyn Novich provided editorial assistance and Ms. Dawn Davis typed the manuscript.

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DEEPENING THE HAMPTON ROADS - THE EIS PROCESS

by William C. Muir[1], George D. Pence[2], and John R. Pomponio[3]

ABSTRACT

Since 1973 the foreign demand for coal has risen dramatically. The oil embargo of that year forced Europe and Japan to convert most of their energy demands from oil to coal. That demand for coal has stimulated growth in many U.S. ports, the greatest of which is the Hampton Roads complex. Last year over half of the nation's coal exports, approximately 50 million tons, came from Newport News, Norfolk, and Portsmouth which make up the Hampton Roads complex. Over the next five years, that volume is expected to double.

The existing channel depth in the Hampton Roads is 45 feet. This depth severely limits the ability of larger coal colliers, greater than 80,000 DWT, to load. The U.S. Army Corps of Engineers (COE) extensively studied the feasibility of deepening the channel to 55 feet and found it economically justifiable. In 1980 the COE prepared a Draft Environmental Impact Statement which proposed expansion of the channel to 55 feet but which presented a number of environmental concerns. These concerns centered on the impact of filling 6,000 acres of the Dismal Swamp, potential contamination of ground water supplies from spoil disposal, and changes in salinity in the tributaries to the Hampton Roads as a result of channel deepening.

Through the efforts of the U.S. Environmental Protection Agency and the Fish and Wildlife Service working with the COE, an extensively modified Final Environmental Impact Statement has been prepared. The final plan relies upon a mixture of dredged material disposal alternatives including beach nourishment of clean sand, containment of contaminated sediments and ocean disposal of the remaining, predominantly uncontaminated material. Channel deepening induced salinity changes should be more thoroughly understood as a result of extensive modeling planned to be accomplished prior to dredging. Monitoring programs will be established to gain further insight into chemical and physical changes associated with the project.

INTRODUCTION

The Hampton Roads, including the ports of Norfolk, Portsmouth, Chesapeake, Newport News, and Hampton, comprises the United States greatest coal export complex. Historically, the Hampton Roads has handled approximately 75 percent of all U.S. coal exports with Baltimore a distant second, followed by Mobile, New Orleans, and Philadelphia. Figure 1 shows the relative importance of each port by tons of coal shipped in 1980. Further, the Hampton Roads is a major exporter of grains and fertilizer and has the largest naval complex in the world.[1]

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The existing port complex has a controlling channel depth of 45 feet which limits the maximum vessel size using the port to 60,000 (DWT) of carrying capacity. This is roughly the median size for present coal shipments and the maximum size which fully loaded vessels can pass through the Panama and Suez Canals.[2] However, the demand for a deeper port has been dramatically shown. Long lines of foreign-flag colliers presently congregate in Chesapeake Bay, waiting up to two months before loading. While these lines are not due solely to inadequate channel depth, a deeper channel would allow use of deeper draft vessels and faster turn around.

In 1969, even before the increased interest in coal exports, Congress recognized the economic importance of deeper channels for the Hampton Roads. A study was authorized to determine the feasibility and optimal depth for the port and to develop a long-term plan for the disposal of all future maintenance dredged materials. The disposal of dredged materials is indeed one of the major blocks in port development throughout the country.

Within the Hampton Roads maintenance dredging requires the removal of over four million cubic meters of materials per year. At present most of this material is deposited at the Craney Island contained disposal site. Figure 2 shows the relationship between the disposal site and the main shipping channels.

In 1980 the U.S. Army Corps of Engineers (COE) completed the draft feasibility study for the Hampton Roads.[3] The report recommended that the shipping channels be deepened to 55 feet which is the maximum depth possible due to the Chesapeake Bay tunnel across the mouth of the bay. The report also recommended that the approximately 50 million cubic meters of dredged materials from the project be placed in a disposal site at the northern end of the Dismal Swamp. The disposal site would require 6,000 acres and was to provide both initial disposal capacity and a 50 year life for maintenance dredged materials.

Due to the extensive nature of the project, the COE was required to prepare an Environmental Impact Statement for the project and provide the public and concerned State and Federal agencies the opportunity to review and comment. The purpose of this paper is to review the regulatory process with emphasis on how the EIS was used as a decisionmaking tool for the Hampton Roads Channel project.

REGULATORY FRAMEWORK

Since 1968 several statutes have been passed which have direct impact on navigation interests. There are three primary laws governing dredging and disposal in navigable waters. These are the National Environmental Policy Act of 1969, the Clean Water Act of 1977, and the Marine Protection, Research, and Sanctuaries Act of 1972.

National Environmental Policy Act

The National Environmental Policy Act of 1969 (NEPA) is the nation's basic charter for protection of the environment. The primary purpose of NEPA is to ensure that sufficient and complete environmental and alternative project information is available to both the Federal agencies and the general public on issues that could have a significant impact upon the public.

Section 1500-1508 of Title 40 of the Code of Federal Regulations promulgated by the Council on Environmental Quality provides the regulatory framework for implementing NEPA.[4] Each Federal agency in carrying out its mandate must use NEPA as the decisionmaking process for all significant actions taken by that agency. This includes:

a. taking an interdisciplinary approach which will insure the integrated use of the natural and social sciences and the environmental arts in planning and in decisionmaking which may have an impact on man's environment;

b. identifying environmental effects and values in adequate detail so the economic and technical analysis for each alternative can be appropriately compared;

c. developing appropriate alternatives to courses of action which involve conflicts concerning alternative uses of resources; and,

d. providing for cases where actions are planned by private applicants or other non-Federal entities so that the Federal agencies can commence its NEPA process at the earliest possible time.

The Clean Water Act

In 1972 and again in 1977, amendments to the Federal Water Pollution Control Act of 1948 were enacted. The purpose of the 1977 amended act, known as the Clean Water Act (CWA), is to restore and maintain the chemical, physical, and biological integrity of the waters of the United States. Section 404 of the CWA established a set of criteria for regulating the discharge of dredged or fill material into waters of the United States.[5,6]

The Section 404(b)(1) guidelines require a thorough review of all alternatives to the dredging and disposal. Dredging considerations include analyzing channel locations, need for channel depths, and techniques for dredging. Included in the analysis of disposal site selection are the quality of materials to be disposed and the impacts on water quality, wetlands, and the benthic environment especially related to shellfisheries.

The guidelines specify conditions which must be met for any dredging project. These include:

a. compliance with State water quality standards;

b. compliance with EPA's toxic effluent standards;

c. no adverse effect through bioaccumulation of toxic substances;

d. no impacts to threatened or endangered species;

e. no impacts to marine sanctuaries; and

f. there must be no impacts which would cause or contribute to significant degradation of the waters of the United States.

Section B of the guidelines also outline areas of special concern. These areas include sanctuaries and refuges, wetlands, mud flats, vegetated shallows, coral reefs, and riffles and pools. The guidelines define these areas and provide insight as to why they should be protected.

The dredging of areas where contaminants are known or suspected to reside requires special care. Testing procedures under 404(b)(1), Subpart G provide for categories of dredged spoils from clean, with no potential for harm, to very polluted requiring extensive bioassays to assess impacts.

Since wetlands had been commonly used for dredged materials disposal, special consideration is given to this valuable resource. The function of wetlands are multifold. First, they provide the primary food link for most estuarine organisms and are vital to such commercial fisheries as shrimp, salmon, oyster, menhaden, crabs, flounders, and clams.[7] The organic production alone from a salt marsh is two and one-half times greater than from a fertile hayfield.[8] As an outstanding example of wetlands values and potential losses, San Francisco Bay once had extensive wetlands and provided seven major commercial fisheries with multi-million dollar estimated values. Unfortunately, today, due to diking and filling operations, only a fifth of the wetlands remain and none of the commercial fisheries.[9]

Wetlands also act as a primary recharge for much of the nation's ground water. They provide excellent erosion control, and they act as a pollution filtration system. Last, and of major importance, wetlands act as flood prevention buffers both by decreasing sheet flow and by water storage.[10]

The Marine Protection, Research, and Sanctuaries Act

As a result of the concern for dumping of dredged spoils, sewage sludge, and industrial wastes into ocean waters, the Congress in 1972 enacted the Marine Protection, Research, and Sanctuaries Act of 1972 (MPRSA). Similar to the Clean Water Act, the MPRSA provides criteria for at sea disposal of dredged materials.

In the review of ocean dumping of dredged materials, final criteria were promulgated on January 11, 1977.[11] These criteria are similar to those under the CWA but contain additional provisions under the Convention on the Prevention of Marine Pollution by Dumping of Wastes and Other Matter.[12] A summary of the criteria is as follows:

a. justification of need and the lack of alternatives;

b. there are no unacceptable adverse effects on aesthetics, recreational or economic values;

c. there are no unacceptable adverse effects on other uses of the ocean;

d. the material does not contain

   (i) high-level radioactive wastes,

   (ii) radiological, chemical, or biological warfare agents,

   (iii) persistent inert synthetic or natural materials that float; and,

e. no greater than trace concentrations of such pollutants as organohalogens, mercury, cadmium, oil, and petroleum products; and known carcinogens, mutagens, or teratogens.

Ocean Dumping criteria place emphasis on the assessment of impacts using bioassay techniques similar to those required by the CWA and upon the location of the disposal site.

THE HAMPTON ROADS PROJECT

The deepening of the Hampton Roads ship channels to 55 feet entails the removal of approximately 50 million cubic meters of materials. There are two areas of concern for environmental impacts to the project. First, the dredging of huge quantities of potentially polluted materials may directly impact the water column and the benthic organisms within the ship channel; and second, the disposal could have impacts ranging from loss of wetlands to ground water quantity and quality impacts.

<u>Impacts of Channel Deepening</u>

The impacts from the dredging to 55 feet can be categorized as either short-term (e.g. increased turbidity, decreased dissolved oxygen, resuspension of contaminants) or long-term (e.g. changes in salinity, reduction or modification of estuarine circulation, disruption of fish migration).

To limit the short-term impacts, use of a hopper dredge is recommended in the EIS. The hopper dredge is a self-propelled vessel, equipped with a hydraulic suction dredge system and with hopper bins to contain and carry the dredged material to a place of disposal. This would minimize the resuspension of sediments which will reduce turbidity, minimize the potential release of pollutants to the water column, and reduce the potential release of oxygen demanding sediments.

Other short-term impacts include burial or removal of organisms within the channel. Excessive burial or burial during critical life cycles of important estuarine organisms may result in death or inhibition of growth or reproduction.[13] To insure minimal aquatic impacts, project construction schedules may require time of year restrictions chosen to protect finfish and shellfish. As the lower Chesapeake Bay is a critical overwintering area for the commercially valuable blue crab, impacts to this species would be especially critical.[14] Therefore, dredging schedules should be established to insure minimal impact to the area.

To further reduce dredging related turbidity, economic loading will not be performed during project construction. Economic loading involves increasing the concentration of solids by the spillage of highly turbid water from the hopper dredge which increases environmental impacts.

In general, the short-term impacts can be mitigated to lessen their severity, however, the long-term impacts of dredging and disposal, must be carefully evaluated because of the difficulty in both mitigation and in the actual prediction of impacts.

The DEIS was deficient in that the COE was not able to predict long-term impacts. To overcome the deficiencies, the EPA recommended the use of the Chesapeake Bay Physical model which presently exists at Kent Island, Maryland. EPA recommended that the model address changes in estuarine circulation and current patterns. These results should provide the necessary data to predict impacts to finfish migration patterns, the blue crab overwintering grounds, and impacts to submergent and emergent vegetation. The model would also be used to predict changes in salinity gradients in the lower James River as this area is a productive seed bed for the commercially valuable oyster.

Finally, the resource agencies have recommended that in conjunction with the project, a monitoring program be established both to gain insight into the chemical, physical, and biological changes associated with the project, and to develop mitigation where necessary.

Recognizing the limitations of the physical modeling program, the U.S. Army COE has agreed to monitor the post project salinity wedge, dissolved oxygen concentrations, migratory fish patterns, benthic habitat, and submergent and emergent aquatic vegetation. Should monitoring data reveal unforeseen significant degradation of bay habitat or water quality, the resource agencies and the COE will work together to recover Chesapeake Bay quality.

Impacts of Disposal

In the selection of a disposal site, the DEIS reviewed 12 possible alternatives and their potential impacts. These included (Figure 3):

a. Raising existing levees at the present disposal site--this would not provide sufficient capacity for the channel deepening.

b. Western extension of the present disposal site--this would require the filling of 2,380 acres of submerged bottom land.

c. Willoughby Bay--this would require filling 1,280 acres of shallow water habitat.

d. Ocean View Area--this would require filling 2,700 acres of marine bottom land.

e. Hampton Flats--this would require filling 1,800 acres of shallow water and mud flat habitat.

f. Ragged Island--this would entail the filling of 2,320 acres of salt marsh wetlands.

g. Buckroe Beach--this would require the filling of 3,500 acres of marine bottom land.

h. Chesapeake Bay--this would entail filling 2,450 acres of the Chesapeake Bay near Thimble Shoal.

i. Suffolk--this would require the destruction of 6,000 acres of wooded wetlands.

j. Ocean disposal--this would cause temporary destruction to 3,000 acres of benthic habitat.

k. Upland disposal--this would entail transporting materials to upland borrow pits, strip mines or other upland areas.

l. No action

All of the alternatives had drawbacks. It was jointly agreed that all of the sites requiring major aquatic fills were unacceptable. This included the western extension of Craney Island, Willoughby Bay, Ocean View, Hampton Flats, Buckroe Beach, and the Chesapeake Bay (Thimble Shoal) site. Further, Ragged Island was a prime salt marsh wetland and its use would have violated the Section 404(b)(1) guidelines. Of the remaining alternatives, upland disposal was considered cost prohibitive; raising existing structures provided limited capacity; and ocean disposal had environmental unknowns. Through the process of elimination Suffolk became the chosen site, with the hope that detailed site evaluation would reveal a predominantly upland habitat with little of the well known wetland values.

The proposed use of the Suffolk site involved diking of 4,000 acres for long-term maintenance disposal and 2,000 acres for the channel deepening. Review of the site by the resource agencies revealed significant environmental problems. Of greatest concern was the direct loss of up to 6,000 acres of wooded wetlands. However, initially, both the COE and the resource agencies believed that the site would be predominantly upland. The Suffolk site is at the northern end of the Dismal Swamp and is cut off from the main swamp by U.S. Route 460 (Figure 4). As a result, the site was not initially considered an integral part of the swamp. Further, the site had been timbered and trenched which led agency representatives to believe wetlands and their values would be diminished.

However, in-depth review of the Suffolk site revealed that it is a fully vegetated area consisting primarily of mixed scrub-shrub and forested wetland. Therefore, from the perspective of wetlands impacts, the site became unacceptable.

In addition to the general loss of habitat, the EPA was particularly concerned with the impacts to the ground water, which is the City of Suffolk's drinking water supply, and the hydrologic connection between the site and the Dismal Swamp. There is evidence that the site is a major recharge area for the Dismal Swamp.[15,16]

The COE proposed the construction of a trench which was to isolate the Suffolk site. Upon review of the proposal, we believe the trench would have been totally inadequate to prevent contamination of the ground water and would have exacerbated the hydrologic problem with the Dismal Swamp.

As an alternative solution, the EPA and F&WS proposed the use of ocean dumping of the clean materials, beach nourishment with clean sand, elevation of the dike at Craney Island for contaminated material and the long range development of a rehandling program.

Ocean Dumping

Independent of the Hampton Roads Deepening project, the COE and EPA had been working since 1977 to designate a permanent disposal site off the mouth of the Chesapeake Bay. The identification and approval of the site was carried out in conjunction with the Section 228 of the Final Regulations as described earlier in this paper. The final site designation is expected early in 1982 as the DEIS has just recently been issued.[17]

In conjunction with compliance with the regulations, the EPA and the COE utilized the Section 103 Implementation Manual[18] which prescribes testing procedures with which the ecological effects of dredged material disposal are evaluated before an open ocean site may be designated. The COE and the National Oceanic and Atmospheric Administration (NOAA) conducted an extensive base line survey in support of the ocean disposal site. In addition, a separate study was undertaken over the entire lower bay, including the Hampton Roads, to determine which sediments could be acceptable for ocean disposal.

Ocean dumping of dredged material elicits two basic concerns. First, the effects on water column perturbations must be identified. Second, the effects of placement of esturine sediments on the benthic biota at the disposal area must be addressed. The analysis of potential water column effects enlists bioassays to determine limiting permissible concentrations of toxic substances in liquid, suspended, dissolved and particulate phases of dredged materials. This study is followed by an analysis of mixing zone characteristics to apply those results to the dump sites. Analysis of effects of dredged materials accumulating on the bottom sediments emphasizes physiological effects on disposal site biota, and involves use of solid phase bioassays. Because of the rapid dilution and dispersion of dissolved and suspended materials upon their release in a disposal site, water column perturbations are normally of short duration. Bottom sediments have the greatest overall potential for causing long-term undesirable effects.

Based upon test organisms that were indigenous to the area and sensitive to pollutants, and procedures agreed upon by the EPA, the COE has found the following concerning the Hampton Roads:

a. Liquid and suspended solid phase bioassays conducted on materials from various dredge sites indicated that sediments from most stations exhibited low levels of acute toxicity for the grass shrimp, Palaemonetes pugio. However, a few of the stations from the most highly industrialized region of the Southern Branch of the Elizabeth River, along its middle reaches, contained materials which produced significant lethal effects (mortalities of 50 to 90 percent) during the 96-hour bioassays. The lethal effects which were observed for these stations occurred quite rapidly, usually within the first 4 hours, and appeared mainly to be associated with the finely suspended solid materials.

b. Mortalities observed during the solid phase bioassays were considerably lower than those seen for suspended solid tests which demonstrated significant lethal effects. The mud snail, Nassarius obsoletus, was observed to have 100 percent survival during all experiments. In experiments with P. pugio, the sediments taken from the industrialized area produced mortalities which were significantly greater than those observed for the controls, but the differences were quite small (10 to 20 percent) and highest mean mortalities were less than 50 percent. These results tend to support the belief that the lethal effects observed in these tests are mainly associated with fine materials which at the disposal site would be diluted and dissipated in the water column, rather than with the heavier fraction to which the benthic organisms may be exposed for longer periods. In fact, the experimental elevation of mortalities observed in test populations during the solid phase bioassay is only slightly above the level proposed by the Implementation Manual as being the minimum difference (10 percent over controls) required to predict any adverse impact.

c. Significant depression in respiration rates was noted for grass shrimp exposed to elutriates from certain of the potential dredge sites. The most dramatic effects were noted for sediments from the same stations producing significant lethal effects. Liquid phase elutriates from these stations produced a progressive depression in respiration, with the final rates being less than 40 percent of control values. Sediments from sites on the extreme upstream and downstream boundaries of industrial activity produced no significant sublethal effects on respiration. Suspended solid phase tests appeared to produce interactive effects on grass shrimp respiration. The suspended solid load at all stations caused an elevation of respiration rates, while the progressive depression in respiration observed in the liquid phase experiments became apparent over the course of the experiment when sediments from the middle reaches of the Southern Branch were used. Therefore, there appear to be some sublethal toxic effects associated with dissolved materials in the liquid fraction from the highly industrialized region, which may become lethal when combined with the additional stress of the heavy suspended solid load (and any associated adsorbed or absorbed contaminants) noted for these sites.

d. The osmoregulation experiments which followed the solid phase bioassays indicated that N. obsoletus, is an osmoconformer throughout most of the salinity range tested (15 to 30 parts per thousand, (ppt.)). However, there were indications of an active or passive capacity to maintain a hypo-osmotic condition when exposed to the highest salinity (34 ppt.). At this salinity, snails, previously exposed to sediments from one station in the highly industrialized region, were significantly closer to isotonicity than were controls, while snails from the experiments with sediments from adjacent sites were quite variable, exhibiting an "all or none" response in terms of osmoregulation capacity. These results indicate that, although the snails suffered no lethal effects and the sublethal effects were far from dramatic, the same trend observed in the other sublethal and lethal experiments was apparent.

e. As reported from preliminary investigations, only a fraction (22 percent) of the metal-sediment combinations tested during the bioaccumulation study exhibited statistically significant bioaccumulation patterns. Iron, manganese, and zinc were the only metals observed to significantly accumulate in either P. pugio or N. obsoletus, with highest accumulations of these metals being associated with sediments taken from the main branch of the Elizabeth River. The levels of iron and manganese are probably of no ecological concern, while the concentration of zinc in the grass shrimp may be the basis for some concern, although the magnitude of accumulation is far from alarming.[19] Lead levels in some of the tissue samples were also seen to be elevated over sediment concentrations, although no significant bioaccumulation pattern could be detected for this metal.

f. Simplistic mixing zone models were calculated based upon data presented in the Implementation Manual as average values to predict the magnitude of dilution which might be expected at the disposal site during the 4-hour initial mixing period. These models indicated that all stations evaluated for lethal and sub-lethal effects were technically within the guidelines established by the Implementation Manual for open ocean disposal since the concentrations that produced significant effects were not within two orders of magnitude of the diluted levels predicted after 4 hours of mixing. However, there are a number of environmental concerns not addressed by these models which should be noted: (a) these models are not site specific, and therefore conditions unique to the Norfolk disposal site (e.g., pycnocline position, current patterns, sediment transport processes, etc) or the disposal operations (e.g., site and type of barge, frequency of discharge, type of sediments, etc.) may alter the predicted dilution rates; (b) the models do not address the ecological impact upon the organisms of the water column (particularly the meroplankton) during the initial mixing period; (c) the assumption is made that fauna indigenous to the disposal site are roughly as tolerant as the test species; and (d) subtle long-term effects on indigenous communities cannot be assessed. Nonetheless, the mixing zone models appear to represent the best (and probably conservative) predictions of the relative concentrations of the sediment fractions at the site following initial mixing which can be calculated with data currently available. Therefore, a decision based on the assessment described in the Implementation Manual may be made with some caution expressed because of unforeseen impact of the dredged materials from the highly industrialized region of the Southern Branch of the Elizabeth River.

The preliminary results of this testing indicate that over 200 million out of a total project volume of 275 million cubic yards are suitable for ocean disposal.

SUMMARY

The Corps of Engineers and the other Federal resource agencies as well as the State and local agencies recognize that the Hampton Roads Channel Deepening project is important to the future of the regional port and the national economy. Through concerted efforts to insure expeditious but full project review, an environmentally sound project has been proposed.

In March the COE published the Final Environmental Impact Statement for the channel deepening of the Hampton Roads. It has been amended to include ocean disposal as the prime site for 200 million cubic yards of clean materials which meet criteria. The Corps will also establish a management program at Craney Island which is to receive the remaining 75 million yards of dredged material. Post-project monitoring for both the channel project and for the ocean disposal site is to be performed in accordance with guidance provided by EPA and NOAA. Finally, prior to any construction, a model of the project will be thoroughly evaluated.

We believe that this project has shown that the regulatory program can work by interagency dialogue and cooperation. The scoping process by which the channel deepening was conducted has proved vital to providing both the environmental safeguards and the expedient processing of all necessary reviews.

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FIGURES

BIOGRAPHICAL SKETCH

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Mr. Muir has been with the U.S. Environmental Protection Agency since 1972. He is presently the Regional Oceanographer and has the responsibility for all marine-related projects with the region including ocean dumping, dredging, and offshore drilling.

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PREDICTION OF SHOALING RATES FOR DEEPENING ESTUARINE NAVIGATION CHANNELS

by Michael J. Trawle[1]

Abstract

Whenever deepening of a dredged channel is under investigation, a prediction must be made as to the effect of the deepening on the existing dredging requirements. If the deepening is related to advance maintenance dredging rather than to an increase in authorized depth, the prediction becomes even more difficult because the project is allowed to shoal over a wide range of depth. Currently, a variety of arbitrary, rule-of-thumb procedures are used for predicting the effect of increased depth on dredging requirements. This paper presents an empirical method of shoaling analysis based on historical dredging and shoaling records that results in reliable predictions of future shoaling for deepened channel conditions resulting from either an increase in authorized channel depth or advance maintenance. The method presented was designed to be general enough so that it can be applied to most navigation projects without difficulty. The procedure is described step by step using the Texas City Channel as an example and the results are discussed.

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INTRODUCTION

Objective

The objective of this paper is to present an empirical method of shoaling analysis based on historical dredging and shoaling records that results in reliable predictions of future shoaling for deepened channel conditions. Deepened conditions can result either from an increase in the authorized channel depth or from advance maintenance dredging.

Background

A typical dredged channel with no provision for advance maintenance dredging is illustrated in Figure 1. Basic specifications for the dredged dimensions are authorized depth, authorized bottom width, and authorized side slopes which describe the authorized channel prism. Where advance maintenance dredging is not utilized, the authorized channel is the same as the required channel prism. The inclusion of allowable dredging tolerances for the bottom and side slopes of the channel to compensate for dredging inaccuracies provides for adjusted channel dimensions which define the allowable pay prism of the channel.

Allowable dredging tolerance should not be confused with advance maintenance dredging (Figure 2). Allowable dredging tolerance, usually 1 to 3 ft, is simply a margin of error that allows the contractor

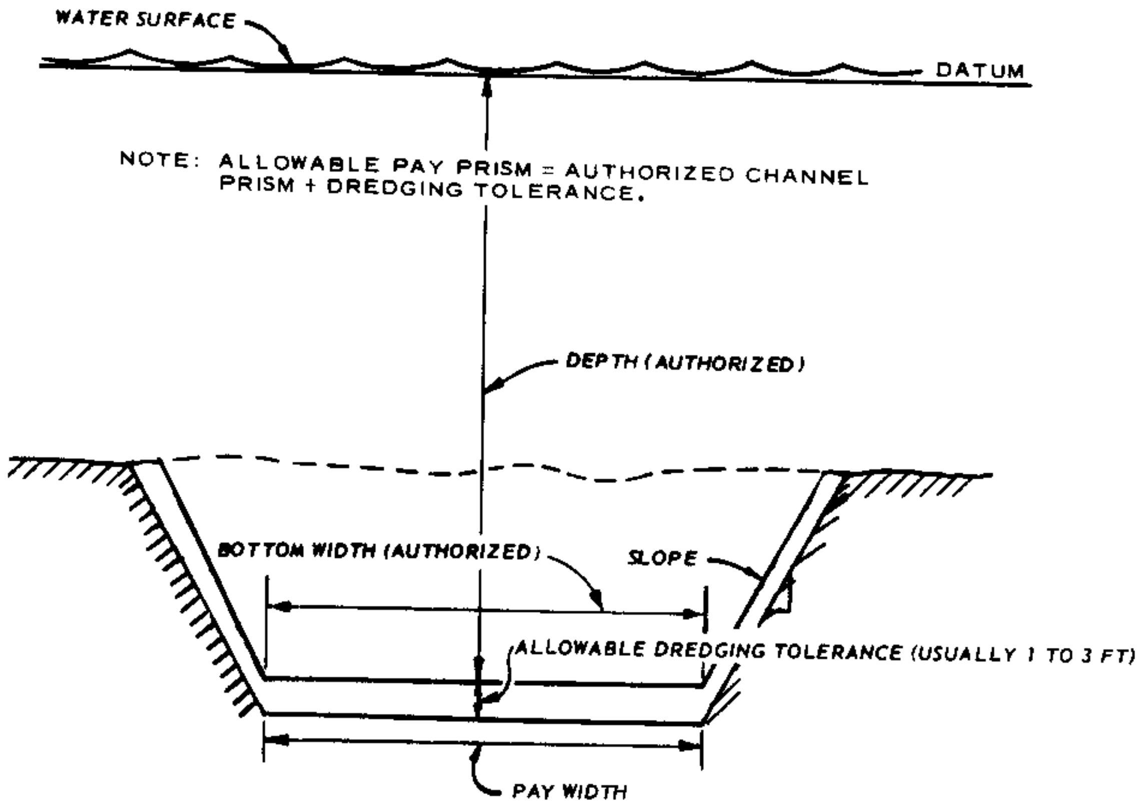


Figure 1. Typical dredged channel cross section

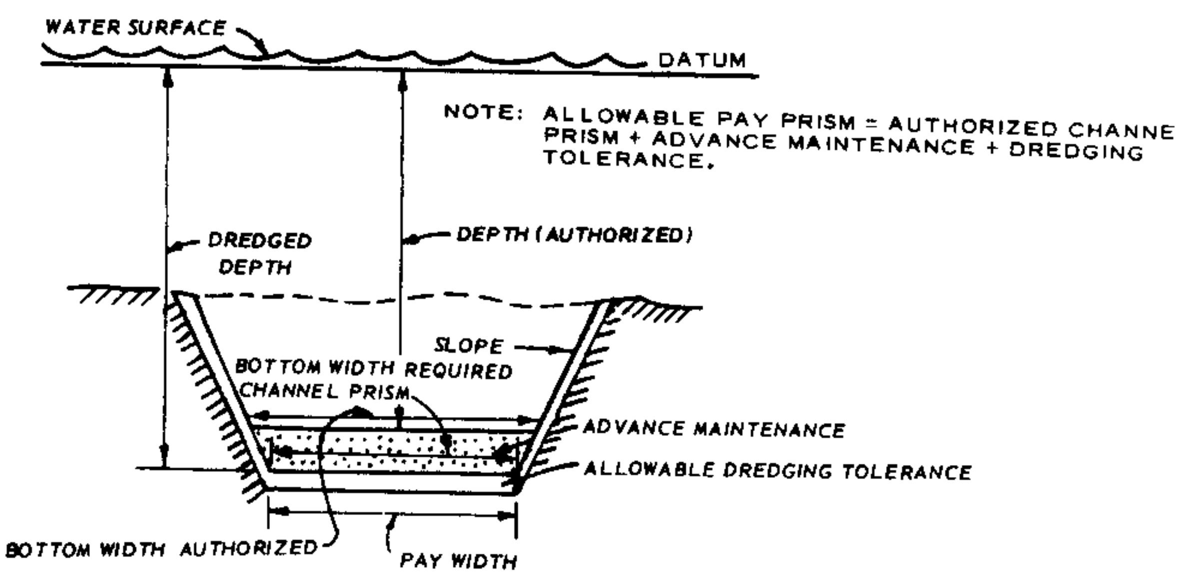


Figure 2. Dredged channel cross section with advance maintenance included

to be paid for material dredged within a specified depth (usually 1 to 3 ft) below the authorized depth. Allowable dredging tolerance is necessary to allow for dredging inaccuracies. Factors that contribute to the need for a tolerance for industry and for Corps dredges include wave action, tidal forecast variances, and equipment conditions and controls.

Whenever deepening of a dredged channel is being investigated, a prediction must be made as to the effect of the deepening on the existing dredging requirements. If the deepening is related to advance maintenance dredging rather than to an increase in the authorized depth, the prediction becomes even more difficult because the project is allowed to shoal over a wide range of depth. As a result of the environmental regulations created within the last decade, dredging has become a much more expensive operation than in the past, and costs will be felt even more heavily in the future. For this reason, predictions of shoaling for deepened conditions, whether advance maintenance dredging or increase in authorized depth, should be reliable. Currently a variety of procedures are followed by Corps districts for predicting the effect of depth on dredging requirements. Four of the most used procedures in the past are presented in the following subparagraphs:

a. Increase in cross-sectional area. The basic premise in this procedure is that for any dredged navigation channel, the percent increase in the shoaling rate caused by deepening is proportional to the percent increase in cross-sectional area of the channel below natural depth (Figure 3) or, presented in equational form,

$$S_d = S\left(\frac{A_d}{A}\right)$$

where

S = existing channel shoaling rate

S_d = deepened channel shoaling rate

A = existing channel cross-sectional area

A_d = deepened channel cross-sectional area

b. Increase in wetted perimeter. The basic premise in this procedure is that for any dredged navigation channel, the percent increase in the shoaling rate caused by deepening is proportional to the percent increase in the wetted perimeter of the channel below natural depth (Figure 3) or, presented in equational form,

$$S_d = S\,\frac{x_d + y_d + z_d}{x + y + z}$$

where

S = existing channel shoaling rate

S_d = deepened channel shoaling rate

x = length of existing channel side slope (left)

y = length of existing channel bottom

z = length of existing channel side slope (right)

x_d = length of deepened channel side slope (left)

y_d = length of deepened channel bottom

z_d = length of deepened channel side slope (right)

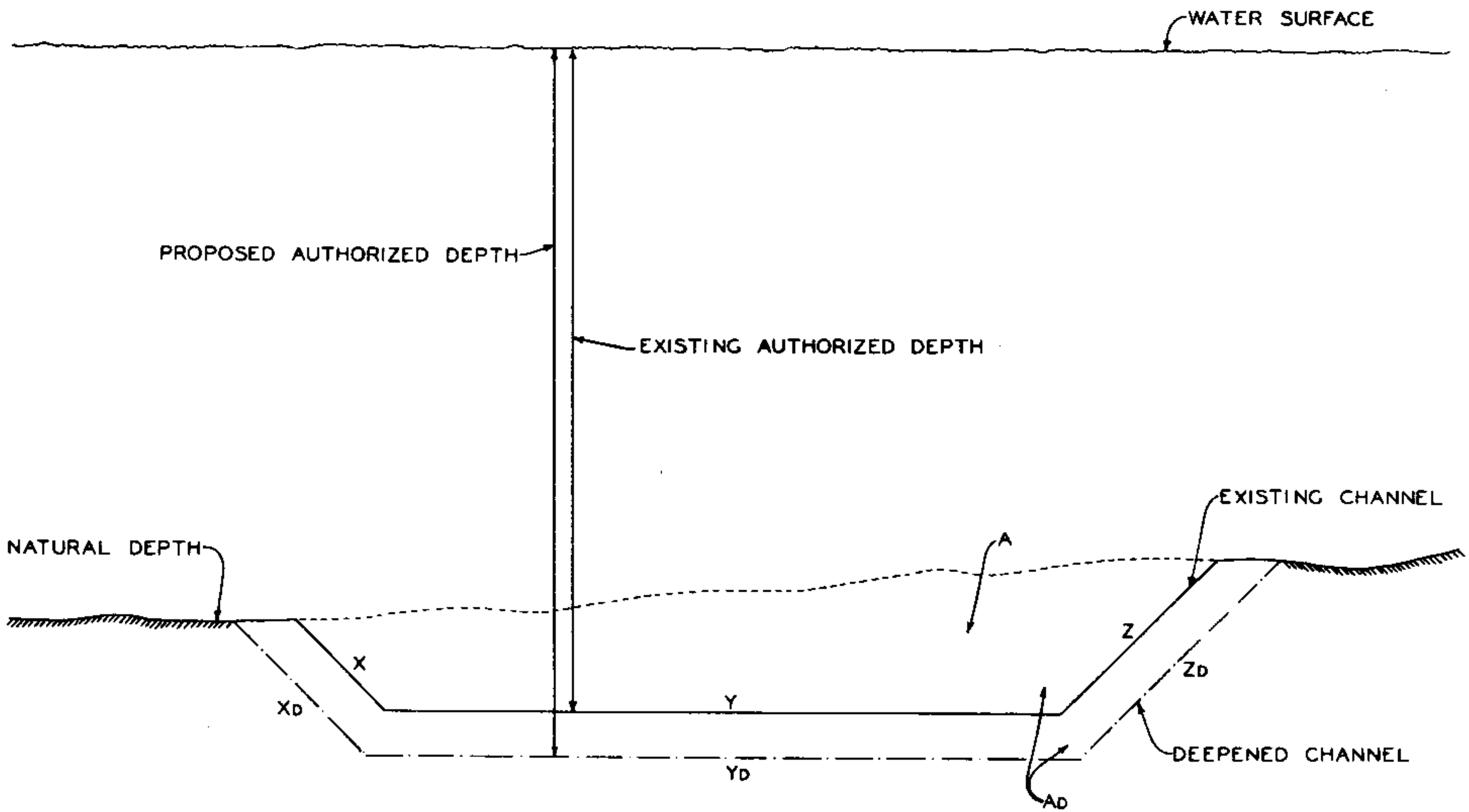


Figure 3. Channel cross section for existing and enlarged dimensions

c. Experience in nearby areas. When navigation channels in nearby areas have already been deepened to the depth being considered for the channel under investigation, results of the deepening in the nearby channel may be used to predict the future dredging requirements for the channel in question.

d. Limited historical dredging or shoaling data. Often the prediction of shoaling for a deepened channel is made on the basis of limited historical dredging or shoaling data. For example, based on hydrographic surveys of a navigation channel, suppose it's determined that for a period between dredging (say 3 years) for the existing channel depth, the shoaling rate was X cu yd per year. For a period between dredging for the previous channel depth (say 4 ft less than that existing), the shoaling rate was Y cu yd per year. The percent increase in shoaling from the previous to existing depth is simply

$$\text{Percent increase in shoaling} = \frac{100(X - Y)}{Y}$$

The rate of increase can then be extrapolated to the greater depths being considered. If a few more data points are available, a predictive equation can even be generated that allows for other than a linear extrapolation.

Of the four procedures above, the first two--increase in cross-sectional area and increase in wetted perimeter--are based on the assumption that all navigation channel shoaling increases with depth can be related, at least approximately, to some function of channel geometry such as cross-sectional area or wetted perimeter. The problem with this approach is that the assumption is often not valid. Each navigation channel should be treated as unique, since shoaling depends on a multitude of factors, including such factors as sources and types of shoaling material, wind and wave action, ship traffic, past dredging practices, floods, droughts, storms, and changes in density currents, as well as geometry. To assume that shoaling can be predicted on the basis of channel geometry alone is a gross oversimplification and should not be considered reliable.

The third method, presented in item c, can be a valid method of prediction if the channel in the nearby area is indeed similar to the channel being evaluated. However, one cannot assume that a channel will behave the same as a nearby channel based on proximity alone. Again this is an oversimplification resulting in a prediction that should not be considered reliable.

The fourth method of shoaling analysis, presented in item d, differs from the others presented in that the prediction is based on historical dredging or shoaling data. The problem with the approach usually lies in the fact that the amount of data used in the evaluation is insufficient to determine representative shoaling rates. The nature of shoaling phenomena requires that long periods of time be evaluated because the variance in short-term shoaling for most projects is tremendous. In many cases, a few short time periods with one set of channel dimensions are compared with a few short time periods at another set of dimensions; and a prediction for deepened condition is made based on the limited historical data evaluated. The approach can result in (a) shoaling rates not representative of the corresponding channel dimensions and (b) a poor predictive model. As a general rule, the more historical dredging and shoaling data used in the evaluation of a project, the more likely a predictive model that extrapolates the data will be valid.

Approach

The method of shoaling analysis presented will be described using an example project. The example project selected is the Texas City Channel in Galveston Bay, Texas, which is typical of many estuarine dredged navigation projects maintained by the Corps of Engineers.

For any dredged navigation project, shoaling rates are considered to vary in both time and space. For example, the shoaling rate at one location and depth is not constant over time. The shoaling rates at the same location and time but at different depths are not necessarily the same. The shoaling rates at the same depth and time but at different locations within the project are not the same. The variation of shoaling with time may be cyclical in nature as in the case of seasonal changes; there may be a long-term, man-induced, or natural change which gradually affects the shoaling rates within the project; or there may be abrupt changes caused by shocks to the system such as storms or man-made modifications in nearby areas such as dams, locks, flow diversions, and so on. All of these factors should be considered in the analysis of shoaling for any dredged navigation project.

Assumptions

Ideally, a predictive scheme for deepening and subsequent shoaling analysis would include all factors that affect the shoaling rates within a dredged navigation channel, a goal which is generally not achievable. In the method of shoaling analysis presented in this paper, the following simplifying assumptions were made:

a. The variation in shoaling rates within a project can be discretized to form a reasonable number of sections. For example, a 6000-ft-long channel could be divided into six 1000-ft sections and an average shoaling rate for each section for a given depth used in subsequent computations, as long as the shoaling within each section was relatively evenly distributed. This discretization procedure could tie in quite nicely with the frequently used dredging clause which indicates that the project is divided into "acceptance sections" with lengths ranging from 1000 to 3000 ft or so.

b. The variation in shoaling rate with depth for a given location can be discretized to form a reasonable number of depth intervals. For example, if shoaling was being considered at depths from 40 to 50 ft, the variation in shoaling with depth for a given location could be discretized to form 1- to 4-ft intervals (40 to 44 ft, 44 to 46 ft, 46 to 49 ft, and 49 to 50 ft) for computational purposes.

c. Short-term variation in shoaling, such as seasonal variation, is not considered, since most shoaling intervals (periods between dredging activity) to be investigated in estuarine navigation projects are at least 1 year in length. However, if the shoaling is highly seasonal and sufficient data are available to develop shoaling rates with respect to depth for each appropriate period (for example, April through October and November through March), the method in this report could be applied to each period and results coupled.

d. Channel depth changes within the range considered do not significantly affect the distribution of shoaling material within the project. The validity of this assumption for a particular project can be addressed by inspection of the shoaling distribution patterns during previous project depth increases.

ANALYSIS OF THE TEXAS CITY CHANNEL

In order to determine the effectiveness of advance maintenance for any maintenance dredging project, the relation between shoaling characteristics of that project and project dimensions must first be determined. The shoaling characteristics of a project can usually be investigated as follows:

a. Analysis of maintenance dredging records from the Corps of Engineers Annual Reports (Phase 1).

b. Analysis of shoaling rates as determined from dredging records from the Corps of Engineers Annual Reports (Phase 1-Modified).

c. Analysis of shoaling rates as determined from periodic hydrographic surveys (Phase 2).

The analysis of maintenance dredging records from Corps Annual Reports (Phase 1) is the easiest to apply, but also the least accurate. The analysis of shoaling rates as determined from dredging records from the Corps Annual Reports (Phase 1-Modified) is an extension of Phase 1, but requires additional information. Analysis of shoaling rates as determined from periodic hydrographic surveys computes shoaling rates directly and is therefore the most accurate; but the surveys required are usually not available for the entire history of the project. The three approaches will be demonstrated using the Texas City Channel. The use of Phase 1 or Phase 1-Modified combined with Phase 2 analysis is required for predictive purposes.

Bay Description

Galveston Bay, located in the southeastern part of Texas on the Gulf of Mexico (Figure 4), is approximately 60 miles west of Port Arthur, Texas, and 50 miles south of Houston, Texas. With the exception of the area between Galveston Island and Bolivar Peninsula, known as Bolivar Roads, the bay is relatively shallow and varies generally from 7 to 9 ft in depth, except for the deepened channels that are maintained by dredging. Bolivar Roads is connected to the various ports in or near Galveston Bay by Galveston, Houston Ship, and Texas City Channels and is connected to the Gulf of Mexico by the Galveston Harbor entrance or jetty channel. The improvements to the natural pass between Galveston Bay and the Gulf of Mexico include a jettied entrance channel from deep water in the gulf to Bolivar Roads, a distance of about 7 miles, and north and south rock jetties, about 5 and 7 miles long, respectively.

Currents in the channels and bays are largely the result of Gulf of Mexico tides. The mean diurnal range is about 2 ft in the Gulf of Mexico at Galveston Bay and about 0.5 ft in the San Jacinto River and

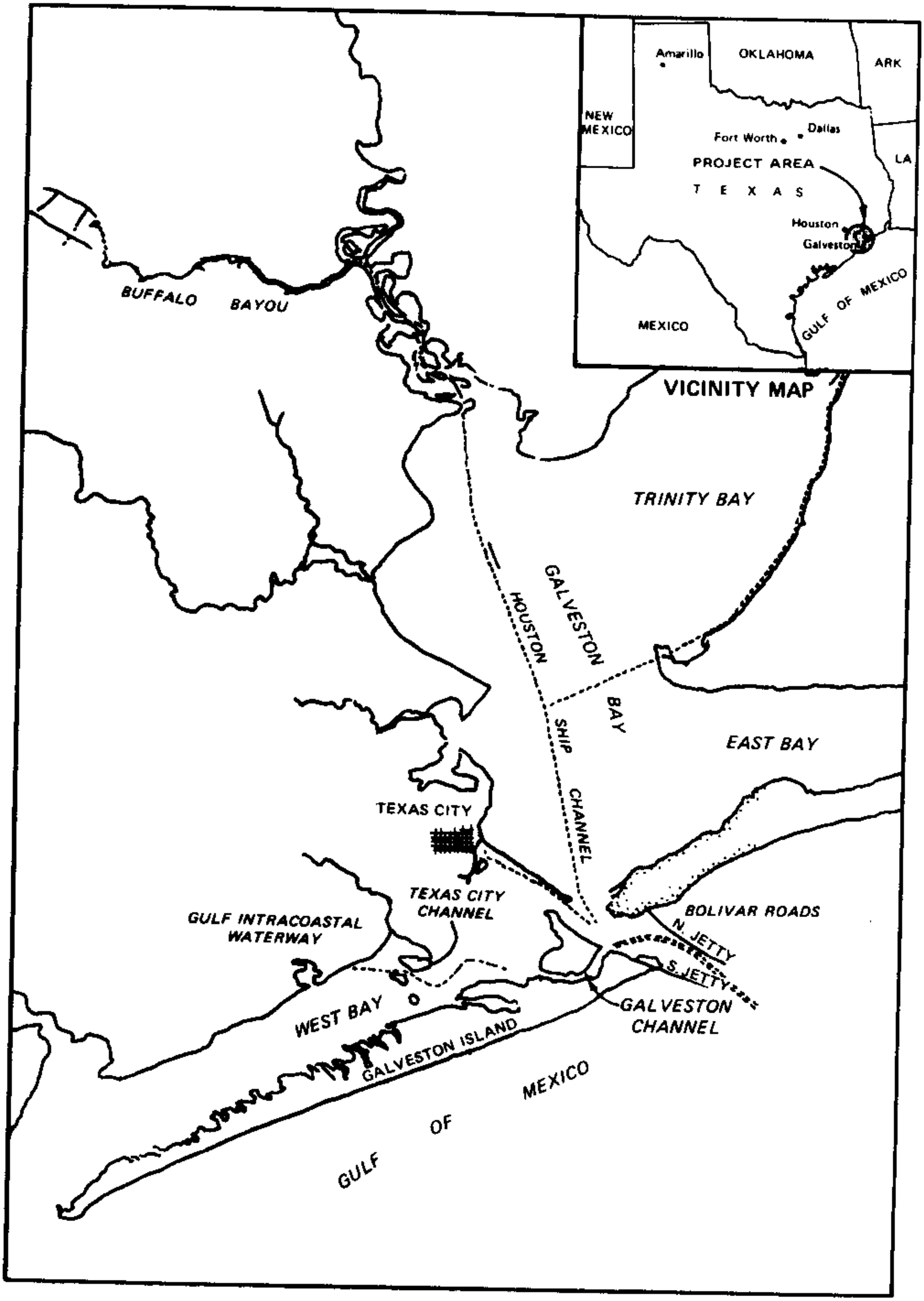


Figure 4. Location map

Buffalo Bayou. The normal water-surface elevation at the entrance to Galveston Bay has been lowered by amounts up to 4.3 ft below mean low tide by strong north winds in the winter season, and has been raised by amounts up to 15 ft above mean low tide by tropical hurricanes which approach from the south, usually in late summer or early fall.

Texas City Channel

The Texas City Channel, located in the western part of lower Galveston Bay (Figure 4), was originally authorized by Congress in 1899 at dimensions of 25 ft × 100 ft. Natural depths varied from 4 to 8 ft. In 1915 and 1916, the channel was deepened and widened to 30 ft × 300 ft. In 1931 the turning basin was enlarged from a 600- to an 800-ft width. The change in turning basin dimensions was not considered significant enough to prohibit comparison of pre-1931 project dredging data and post-1931 project dredging data. In 1937, the channel was deepened so that the dimensions were 34 ft × 300 ft. During 1959 and 1960, the channel was again deepened and widened to 36 ft × 400 ft. In 1966 and 1967, the channel was again deepened, resulting in the existing dimensions of 40 ft × 400 ft over a distance of 6.75 miles. The pre- and postdredge survey sheets for the Texas City Channel from 1962 to 1975, obtained from the Galveston District, indicated that from 1961 to 1965 the contract dredging depths were 36 ft plus 2 ft advance maintenance plus 2 ft allowable dredging tolerance; from 1965 to 1966 the depths were 36 ft plus 3 ft advance maintenance plus 2 ft allowable dredging tolerance; and from 1966 through 1975, the depths were 40 ft plus 3 ft advance maintenance plus 2 ft allowable dredging tolerance. These design depths and the actual average postdredge depths for the periods from 1962 to 1975 are shown in Plate 1.

Phase 1

The annual report dredging data for the Texas City Channel are tabulated in Table 1 and graphically displayed in Plate 2 as maintenance dredging and new work volume versus fiscal year. The second data plot, annual maintenance dredging versus accumulated new work, is presented in Plate 3. The regression curve indicates required dredging volumes as follows:

Project	Total Depth* ft	Maintenance Dredging Volumes Millions of cu yd/year	Percent Change**
25 ft × 100 ft	27	0.40	--
30 ft × 300 ft	32	0.95	+138
30 ft × 300 ft†	32	1.05	+11
34 ft × 300 ft	36	1.14	+7
36 ft × 400 ft	40	1.35	+18
40 ft × 400 ft	45	1.47	+9

* Includes allowable dredging tolerance and advance maintenance.
** Compared with immediately previous project.
† Enlarged turning basin.

Phase 1-Modified

The preceding analysis was based only on fiscal year dredging volumes from the Corps Annual Reports. Since the dates of dredging activity for the Texas City Channel are also provided in the Annual Reports, the analysis can be refined to increase its predictive capability by computing shoaling rates based on dredging volumes and actual time intervals rather than intervals restricted to whole years as was the case for Phase 1. The average shoaling rates based on dredging volumes for each shoaling interval are shown in Table 2. These shoaling data result in the regression curve shown in Plate 4, which indicates overall shoaling rates as follows:

Project	Total Depth* ft	Shoaling Rate Millions of cu yd/year	Percent Change**
25 × 100	27	0.42	
30 × 300	32	0.99	+136
30 × 300†	32	1.10	+11
34 × 300	36	1.20	+9
36 × 400	40	1.44	+20
40 × 400	45	1.60	+11

* Includes allowable dredging tolerance and advance maintenance.
** Compared with immediately previous project.
† Enlarged turning basin.

Phase 2

Hydrographic survey data were available from the Galveston District from 1960 through 1975. Phase 2 results will be compared with the results obtained by Phases 1 and 1-Modified and will be used to determine shoaling rates along the Texas City Channel.

The survey data associated with the dredging activity occurring during November 1961-February 1962; May-June 1963; January-May 1965; May-August 1966; March-May 1968; February-April 1970; May-July 1972; and August 1974-January 1975 were available from the Galveston District. Using these survey data, the project was segmented into nine sections (Figure 5), and the section shoaling rate for each of the shoaling periods was computed as shown in Table 3. Project shoaling for the three periods with the 36-ft authorized depth with 2 to 3 ft advance maintenance and 2-ft allowable dredging tolerance averaged 1.53 million cu yd/year. Project shoaling for the four shoaling

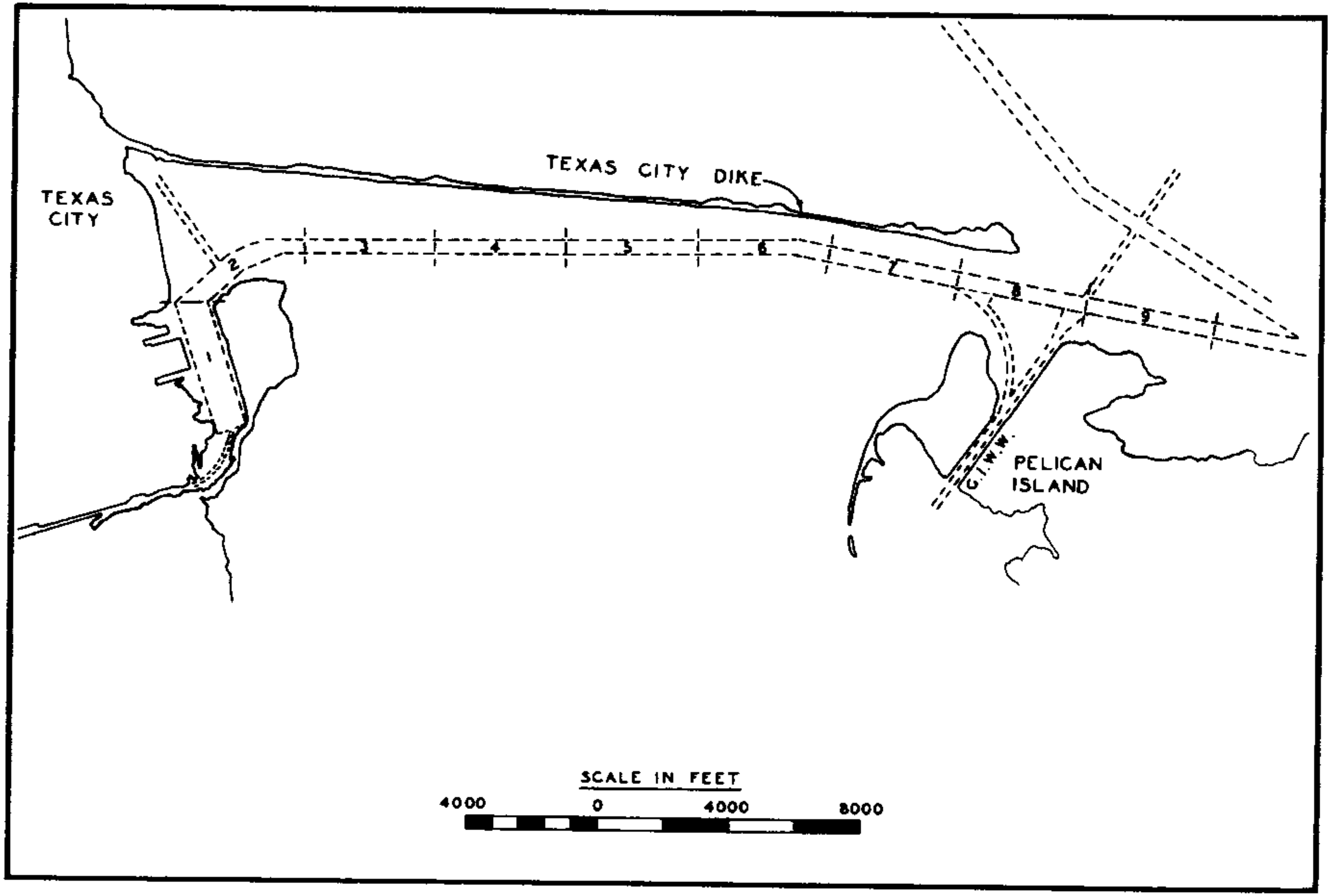


Figure 5. Texas City Channel location map

periods with an authorized depth of 40 ft with 3-ft advance maintenance and 2-ft allowable overdepth averaged 1.65 million cu yd/year.

The average dredging interval for the existing channel, determined from the data in Table 2 (see "Shoaling Interval" column), was 1.89 years. Based on the shoaling rates from Table 3, the existing shoaling pattern at 1.89 years after dredging is shown in Figure 6, indicating a controlling depth of 36.7 ft (in section 8).

Shoaling predictions

The controlling depth, which occurs in section 8, is assumed to be 36.7 ft to agree with the shoaling pattern in Figure 6. Increased advance maintenance can be applied either to reduce the dredging frequency and maintain the 36.7-ft controlling depth in section 8 or to increase the controlling depth to 40.0 ft (authorized) while maintaining or reducing the dredging frequency. A summary of Phases 1, 1-Modified, and 2 results is as follows:

	Shoaling in Thousands of cu yd/year					
Phase	25 × 100 ft	30 × 300 ft	30 × 300 ft (enlarged TB)	34 × 300 ft	36 × 400 ft	40 × 400 ft
1	400	950	1,050	1,140	1,350	1,470
1-Mod	420	990	1,100	1,200	1,440	1,600
2	--	--	--	--	1,533	1,640

Comparison of Phase 2 with Phase 1-Modified indicates that Phase 1-Modified is a reasonable estimation of the historical shoaling rates in the Texas City Channel.

The increase in the shoaling rate for the 4-ft increment from 36 to 40 ft (from 20,340 to 26,510 thousand cu yd accumulated new work) indicated by the Phase 1-Modified curve is 11 percent. The curve also indicates a decreasing rate of increase with depth. For

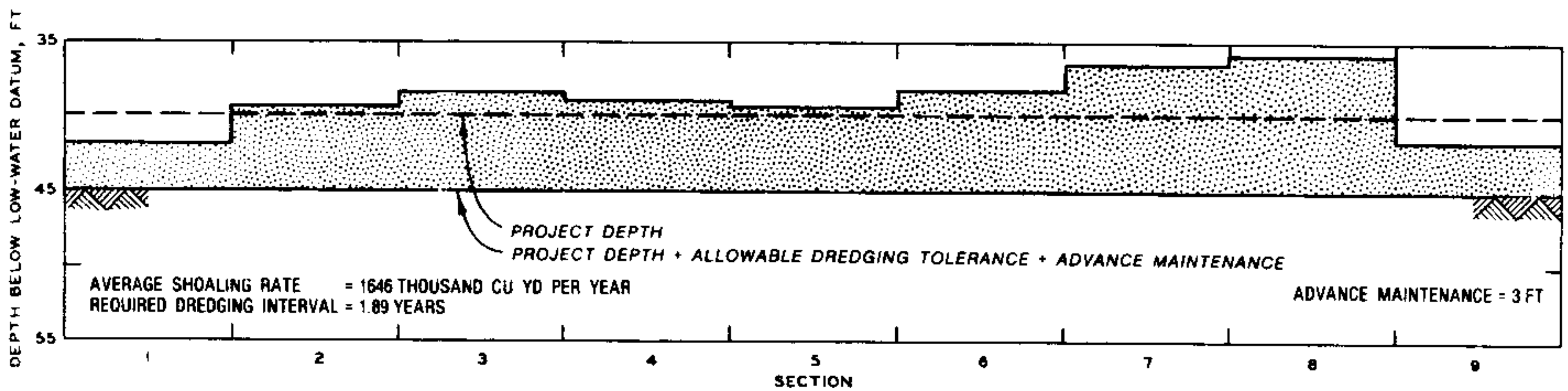


Figure 6. Texas City Channel existing shoaling pattern

the following evaluation of dredging tolerances, the increase in shoaling will be held constant at 5 percent (per 2-ft increment) rather than decreased.

Using the 5 percent rate of increase from 3 to 9 ft of advance maintenance, the shoaling rates for the nine sections (rounded to the nearest tenth of a foot per year) are:

Depth, ft	Predicted Shoaling Rates, ft/year, for Sections								
	1	2	3	4	5	6	7	8	9
Less than 45*	1.5	2.7	3.2	2.9	2.7	3.2	4.1	4.4	1.6
45 to 47	1.6	2.8	3.4	3.0	2.8	3.4	4.3	4.6	1.7
47 to 49	1.7	2.9	3.6	3.2	2.9	3.6	4.5	4.8	1.8
49 to 51	1.8	3.0	3.8	3.4	3.0	3.8	4.7	5.0	1.9

* These shoaling rates are taken directly from Table 3.

The above incremental shoaling rates will be applied to all advance maintenance dredging predictions for the Texas City Channel.

Reduction in dredging frequency while maintaining controlling depth

Increased advance maintenance of 5, 7, and 9 ft was evaluated along with varied advance maintenance. Varied advance maintenance is the practice of fitting the greatest depth of advance maintenance to the section of channel with the heaviest shoaling. Resulting shoaling patterns are shown in Plates 5-7. For varied advance maintenance, the high shoaling rate sections were considered to be sections 7 and 8; and the low shoaling rate sections were considered to be sections 1, 2, 3, 4, 5, 6, and 9. The average shoaling rates and required dredging intervals for each of the advance maintenance schemes investigated are summarized as follows:

Advance Maintenance ft	Average Shoaling Rate Thousands of cu yd/year	Required Dredging Interval, year
3 (existing)	1,646	1.89
5	1,677	2.32
3 and 5	1,655	2.32
7	1,712	2.74
5 and 7	1,681	2.74
3 and 7	1,663	2.74
9	1,756	3.14
7 and 9	1,715	3.14
5 and 9	1,667	3.14

As can be seen, for a given dredging interval the application of varied advance maintenance can result in a slight reduction in the dredging volume compared with the same depth of advance maintenance applied uniformly to the channel. Figure 7 presents the shoaling rate-dredging interval curve using the most efficient of the combinations of advance maintenance evaluated. This curve would be used for any subsequent economic analysis to determine the applicability of advance maintenance.

Conclusion

Before a prediction of future dredging requirements for a proposed deepened or an advance-maintained channel can be attempted, a determination of the effect of depth on shoaling must be made. This paper demonstrated an empirical method, based on historical dredging and shoaling data, which provides a rational approach to the problem.

The approach included several simplifying assumptions, listed in the Introduction. Before any project is evaluated as described in this paper, it should be first determined that the assumptions made will not severely affect the results. If it is determined that an assumption is not valid for the project to be investigated, the procedure

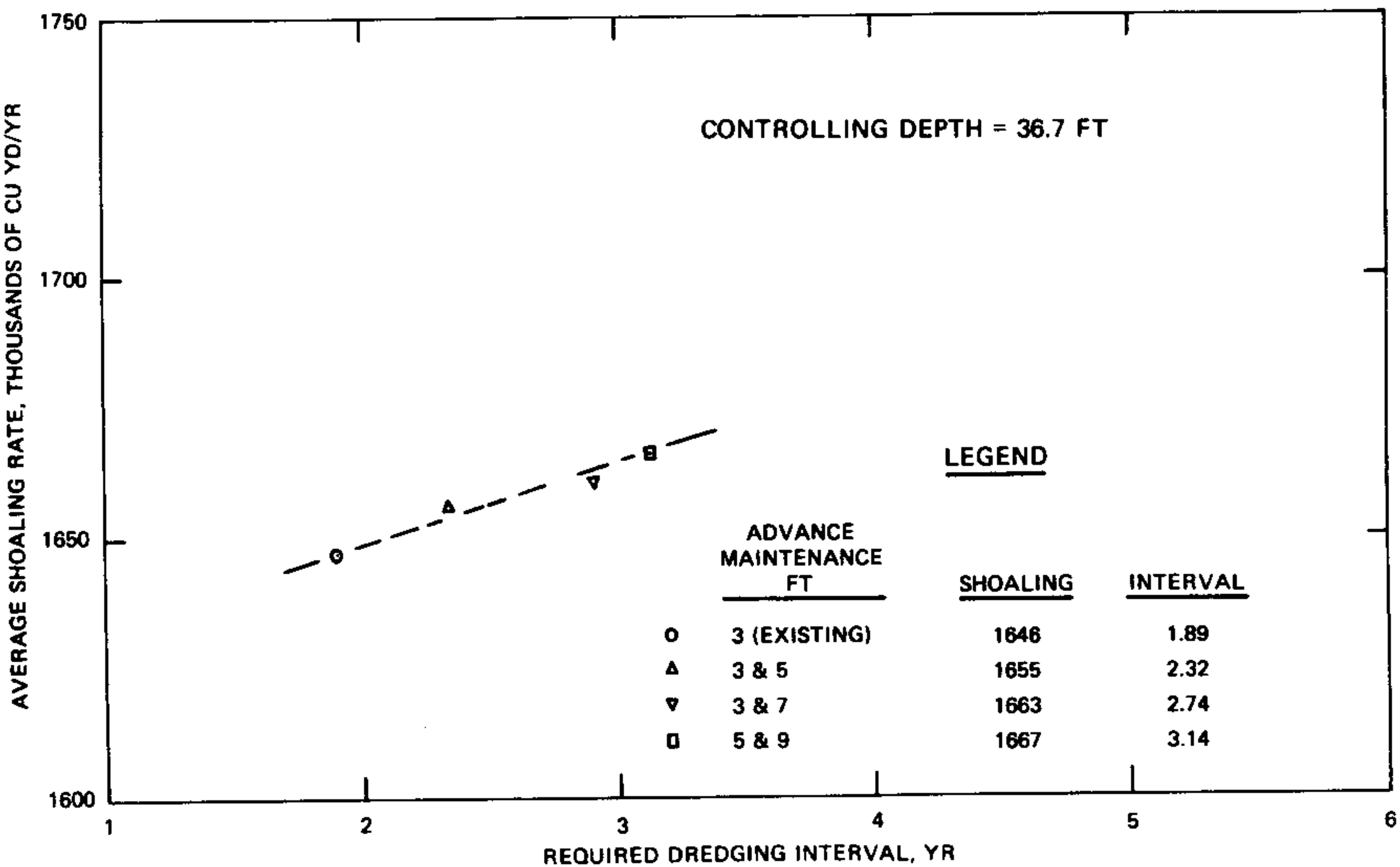


Figure 7. Texas City Channel dredging interval-shoaling relation for various schemes of advance maintenance with a controlling depth of 36.7 ft

should be modified to avoid the offending assumption.

The method of shoaling analysis was demonstrated using the Texas City Channel as an example. The procedure basically involved two steps. The first step was to determine the effect of past changes in depth on shoaling. The second step was to extrapolate the shoaling-depth relation to the proposed advance maintenance or deepened condition to determine required dredging frequencies and volumes. Problems can occur when dealing with a project in which reported dredging volumes are not in reasonable agreement with the observed shoaling volumes for the period investigated. Additional research is then required to determine the adequacy of available data before a predictive technique to define shoaling as a function of depth could be considered reliable.

In summary, the approach presented in this paper requires considerably more effort than the arbitrary, rule-of-thumb procedures predictors described in the Introduction; but the result should be a much more reliable prediction of the effect of advance maintenance dredging or channel deepening on a dredged navigation project.

Acknowledgment

The work described herein was conducted by personnel of the Hydraulics Laboratory, U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, under the Civil Works Research and Development Program, Office, Chief of Engineers, U. S. Army.

Table 1

Dredging History of Texas City Channel, Millions of Cubic Yards

Fiscal Year	Annual New Work	Accumulated New Work	Annual Maintenance
1901	0.815	0.	0.
1902	0.967	0.815	0.
1903	1.105	1.782	0.
1904	0.684	2.887	0.
1905	1.010	3.571	0.
1906	0.	4.581	0.698
1907	0.	4.581	0.
1908	0.	4.581	1.001
1909	0.	4.581	0.273
1910	0.	4.581	0.700
1911	1.153	4.581	0.100
1912	0.	5.734	0.225
1913	0.072	5.734	1.155
1914	0.465	5.806	0.657
1915	3.639	6.271	0.353
1916	2.524	9.910	0.430
1917	0.	12.434	0.666
1918	0.	12.434	1.177
1919	0.	12.434	0.458
1920	0.	12.434	0.585
1921	0.	12.434	0.626
1922	0.	12.434	0.796
1923	0.	12.434	0.
1924	0.	12.434	0.287
1925	0.	12.434	2.226
1926	0.	12.434	4.240
1927	0.	12.434	0.056
1928	0.	12.434	2.396
1929	0.	12.434	1.940
1930	0.	12.434	2.451
1931	1.840	12.434	1.288
1932	0.	14.274	1.002
1933	0.	14.274	2.932
1934	0.	14.274	0.
1935	0.	14.274	1.921
1936	0.	14.274	0.
1937	1.846	14.274	0.980
1938	0.	16.120	0.
1939	0.	16.120	1.083
1940	0.	16.120	1.425

(Continued)

Note: Data taken from CE Annual Reports.

Table 1 (Concluded)

Fiscal Year	Annual New Work	Accumulated New Work	Annual Maintenance
1941	0.	16.120	0.385
1942	0.	16.120	1.973
1943	0.	16.120	0.173
1944	0.	16.120	0.659
1945	0.	16.120	1.879
1946	0.	16.120	0.
1947	0.	16.120	0.
1948	0.	16.120	3.348
1949	0.	16.120	0.
1950	0.	16.120	1.946
1951	0.	16.120	0.
1952	0.	16.120	1.329
1953	0.	16.120	0.
1954	0.	16.120	1.503
1955	0.	16.120	0.
1956	0.	16.120	1.250
1957	0.	16.120	0.
1958	0.	16.120	1.718
1959	2.145	16.120	0.
1960	3.884	18.265	0.818
1961	0.	22.149	0.
1962	0.	22.149	3.502
1963	0.	22.149	1.245
1964	0.	22.149	1.286
1965	0.	22.149	3.639
1966	1.143	22.149	0.058
1967	5.027	23.292	1.368
1968	1.143	28.319	2.923
1969	0.	29.462	0.
1970	0.	29.462	3.048
1971	0.	29.462	0.
1972	0.	29.462	1.700
1973	0.	29.462	1.045
1974	0.	29.462	0.
1975	0.	29.462	3.258

Table 2

Texas City Channel Shoaling Rates Computed from Dredging Volumes

Channel Dimensions ft	Accumulated New Work*	Maintenance Dredging				Shoaling Interval			Computed Shoaling Rate*	Computed Average Shoaling Rate*
		FY	From	To	Volume*	From	To	Years		
25 × 100	4,581	1906	1 Jul 05	7 Jun 06	698	1 Jul 05	7 Jun 06	0.91	767	
		1906-08	8 Jul 07	19 Sep 07	1,001	8 Jun 06	19 Sep 07	1.31	763	
		1908-09	1 Nov 08	15 Nov 08	273	20 Sep 07	15 Nov 08	1.16	236	
		1909-10	22 Sep 09	18 Apr 10	700	16 Nov 08	18 Apr 10	1.42	492	
		1910-11	22 May 11	29 Jun 11	100	19 Apr 10	29 Jun 11	1.20	84	462
Transition	5,734	1911-12	2 Oct 11	24 Oct 11	225	30 Jun 11	24 Oct 11	0.32	702	
		1912-13	8 Sep 12	17 Dec 12	1,039	25 Oct 11	17 Dec 12	1.15	905	
		1913	8 Sep 12	15 May 13	116	18 Dec 12	15 May 13	0.41	284	735
Transition	5,806	1913-14	11 Aug 13	16 Jan 14	657	16 May 13	16 Jan 14	0.67	975	975
Transition	5,882	1914-15	12 Aug 14	9 Nov 14	353	17 Jan 14	9 Nov 14	0.81	437	437
Transition	9,910	1915-16	1 Jul 15	30 Jun 16	430	10 Nov 14	30 Jun 16	1.64	262	262
30 × 300 (plus 2 ft allowable dredging tolerance)	12,434	1917	28 Aug 16	4 Oct 16	666	1 Jul 16	4 Oct 16	0.26	2,531	
		1917-19	1 Feb 18	23 Aug 18	1,635	5 Oct 16	23 Aug 18	1.88	867	
		1919-20	27 Sep 19	18 Nov 19	409	24 Aug 18	18 Nov 19	1.24	330	
		1920	29 Dec 19	31 Jan 20	176	19 Nov 19	31 Jan 20	0.20	868	
		1920-21	19 Nov 20	9 Mar 21	626	1 Feb 20	9 Mar 21	1.10	568	
		1921-22	1 Dec 21	30 Jun 22	796	10 Mar 21	30 Jun 22	1.31	608	
		1923-24	24 Dec 23	16 Mar 24	287	1 Jul 22	16 Mar 24	1.71	168	
		1924-27	1 Jul 24	6 Jul 26	6,522	17 Mar 24	6 Jul 26	2.31	2,867	
		1927-29	1 Jul 28	11 Oct 28	3,061	7 Jul 26	11 Oct 28	2.27	1,351	
		1929	1 Nov 28	21 Dec 28	1,177	12 Oct 28	21 Dec 28	0.19	6,052	
		1929	21 May 29	7 Jun 29	98	22 Dec 28	7 Jun 29	0.46	213	
		1929-30	12 Jul 29	30 Jan 30	2,451	8 Jun 29	30 Jan 30	0.65	3,774	
		1930-31	23 Sep 30	15 May 31	1,288	31 Jan 30	15 May 31	1.29	1,000	1,290

(Continued)

Note: Dredging volumes are similar to data available in CE Annual Reports; dates of surveys may be in CE Annual Reports or in district files.

* In thousands of cubic yards.

(Sheet 1 of 3)

Table 2 (Continued)

Channel Dimensions ft	Accumulated New Work	Maintenance Dredging FY	Maintenance Dredging From	Maintenance Dredging To	Maintenance Dredging Volume	Shoaling Interval From	Shoaling Interval To	Shoaling Interval Years	Computed Shoaling Rate	Computed Average Shoaling Rate
30 × 300 (plus 2 ft allowable dredging tol- erance) harbor extended	14,274	1931-32	4 Mar 32	25 Mar 32	1,002	16 May 31	25 Mar 32	0.86	1,165	
		1932-33	9 Nov 32	25 Jan 33	2,932	26 Mar 32	25 Jan 33	0.84	3,498	
		1933-35	19 Nov 34	23 Jan 35	1,921	26 Jan 33	23 Jan 35	1.99	963	
		1935-37	23 Oct 36	4 Feb 37	980	24 Jan 35	4 Feb 37	2.03	482	1,194
34 × 300 (plus 2 ft allowable dredging tolerance)	16,120	1937-39	1 Feb 39	15 Apr 39	763	5 Feb 37	15 Apr 39	2.19	348	
		1939	1 May 39	30 Jun 39	300	16 Apr 39	30 Jun 39	0.21	1,442	
		1939-40	14 Feb 40	30 Jun 40	1,425	1 Jul 39	30 Jun 40	1.00	1,425	
		1940-41	7 Mar 41	4 May 41	385	1 Jul 40	4 May 41	0.84	456	
		1941-43	27 Mar 42	5 Jul 42	2,146	5 May 41	5 Jul 42	1.17	1,834	
		1943-45	17 May 44	23 Aug 44	2,538	6 Jul 42	23 Aug 44	2.13	1,189	
		1945-48	23 Aug 47	5 Oct 47	2,925	24 Aug 44	5 Oct 47	3.12	938	
		1948	10 Oct 47	28 Oct 47	423	6 Oct 47	28 Oct 47	0.06	6,712	
		1948-50	31 Jan 50	28 Mar 50	1,946	29 Oct 47	28 Mar 50	2.41	806	
		1950-52	27 Feb 52	12 Mar 52	1,329	29 Mar 50	12 Mar 52	1.96	679	
		1952-54	13 Jan 54	25 Feb 54	1,503	13 Mar 52	25 Feb 54	1.96	767	
		1954-56	20 Oct 55	20 Nov 55	1,250	26 Feb 54	20 Nov 55	1.73	721	
		1956-58	9 Nov 57	18 Jan 58	1,718	21 Nov 55	18 Jan 58	2.16	795	
		1958-60	1 Jul 59	10 Apr 60	818	19 Jan 58	10 Apr 60	2.22	368	840
36 × 400 (plus 2 ft allowable dredging tol- erance and 2 ft advance maintenance)	22,149	1960-62	18 Nov 61	14 Feb 62	3,502	11 Apr 60	14 Feb 62	1.85	1,894	
		1962-64	26 May 63	30 Jun 64	2,531	15 Feb 62	30 Jun 64	2.37	1,067	
		1965	19 Jan 65	7 May 65	3,639	1 Jul 64	7 May 65	0.85	4,270	
		1965-67	16 May 66	13 Jun 67	1,368	8 May 65	13 Jun 67	2.10	679	1,547
Transition	28,319	1967-68	1 Jul 67	30 Jun 68	2,923	14 Jun 67	30 Jun 68	1.05	2,793	2,793

(Continued)

(Sheet 2 of 3)

Table 2 (Concluded)

Channel Dimensions ft	Accumulated New Work	Maintenance Dredging				Shoaling Interval			Computed Shoaling Rate	Computed Average Shoaling Rate
		FY	From	To	Volume	From	To	Years		
40 × 400 (plus 2 ft allowable dredging tolerance and 3 ft advance maintenance)	29,462	1969-70	4 Feb 70	17 Apr 70	3,048	1 Jul 68	17 Apr 70	1.80	1,696	
		1970-73	13 May 72	7 Jul 72	2,745	18 Apr 70	7 Jul 72	2.22	1,235	
		1973-75	24 Aug 74	2 Jan 75	3,258	8 Jul 72	2 Jan 75	2.49	1,308	1,390

(Sheet 3 of 3)

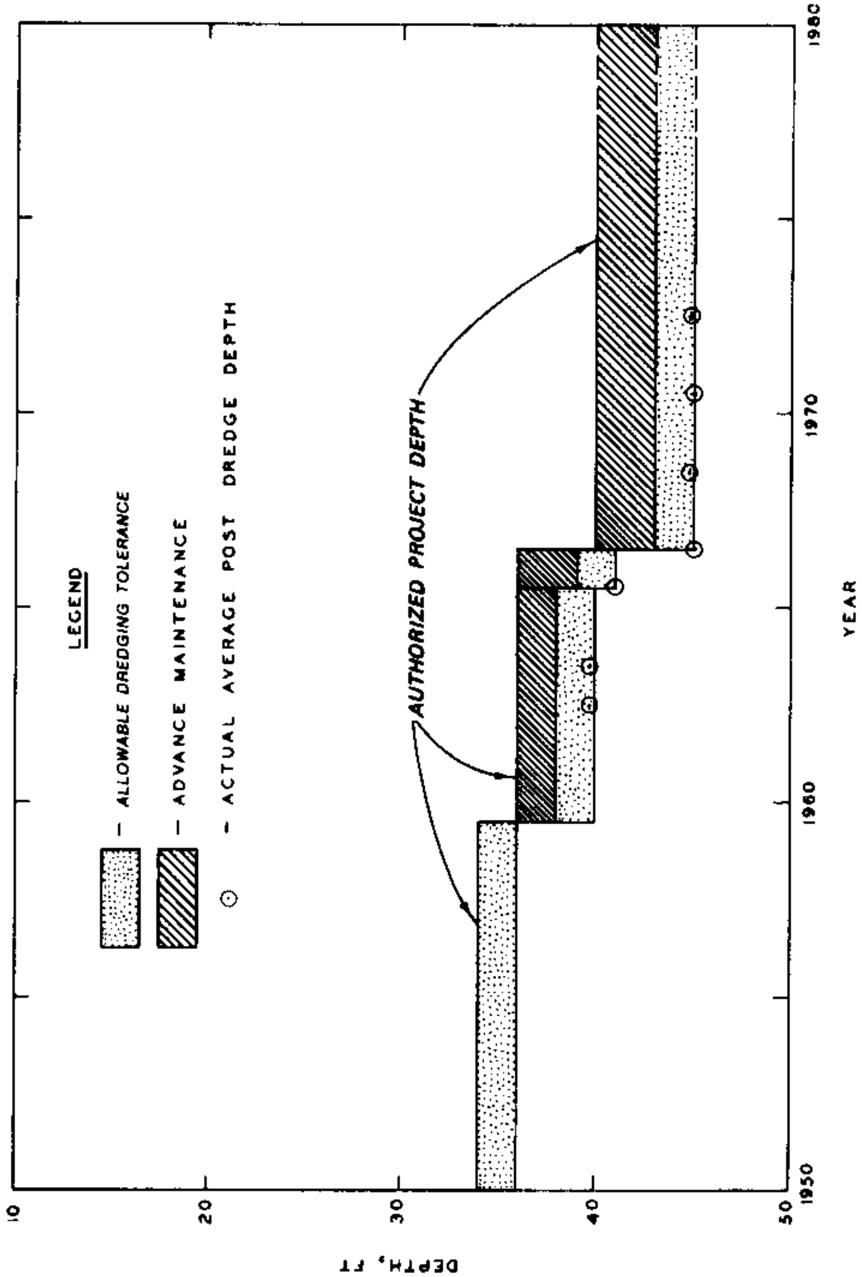
Table 3

Texas City Channel Shoaling

Thousands of Cubic Yards per Year and in Feet per Year

Channel Section	36-ft Channel with 2 to 3 ft Advance Maintenance								40-ft Channel with 3 ft Advance Maintenance									
	Jan 62 to Jun 63		Jul 63 to Sep 64		Apr 65 to Aug 66		Avg		Aug 66 to Mar 68		Apr 68 to Mar 70		Apr 70 to Jan 72		May 72 to Oct 74		Avg	
	thou cu yd	ft/yr	thou cu yd	ft/yr	thou cu yd	ft/yr	thou cu yd	ft/yr	thou cu yd	ft/yr	thou cu yd	ft/yr	thou cu yd	ft/yr	thou cu yd	ft/yr	thou cu yd	ft/yr
1	239	2.0	261	2.2	368	3.1	291	2.5	--	--	178	1.5	--	--	--	--	178	1.5
2	143	2.4	220	3.7	80	1.4	143	2.4	397	6.7	127	2.1	103	1.7	87	1.5	162	2.7
3	212	3.6	163	2.8	156	2.6	177	3.0	370	6.2	133	2.2	182	3.1	129	2.2	190	3.2
4	178	3.0	120	2.0	90	1.5	129	2.2	240	4.1	144	2.4	190	3.2	136	2.3	171	2.9
5	173	2.9	159	2.7	111	1.9	147	2.5	215	3.6	125	2.1	179	3.0	133	2.2	158	2.7
6	189	3.2	147	2.5	100	1.7	145	2.4	237	4.0	167	2.8	220	3.7	158	2.7	190	3.2
7	230	3.9	180	3.0	167	2.8	193	3.3	269	4.5	240	4.1	273	4.6	201	3.4	241	4.1
8	307	5.2	254	4.3	162	2.7	239	4.0	305	5.1	291	4.9	320	5.4	174	2.9	263	4.4
9	92	1.6	42	0.7	69	1.2	69	1.2	204	3.4	106	1.8	49	0.8	44	0.7	93	1.6
Total							1,533										1,646	

Note: -- surveys not available (no data).



TEXAS CITY CHANNEL
CONTRACT DREDGING DEPTHS

PLATE I

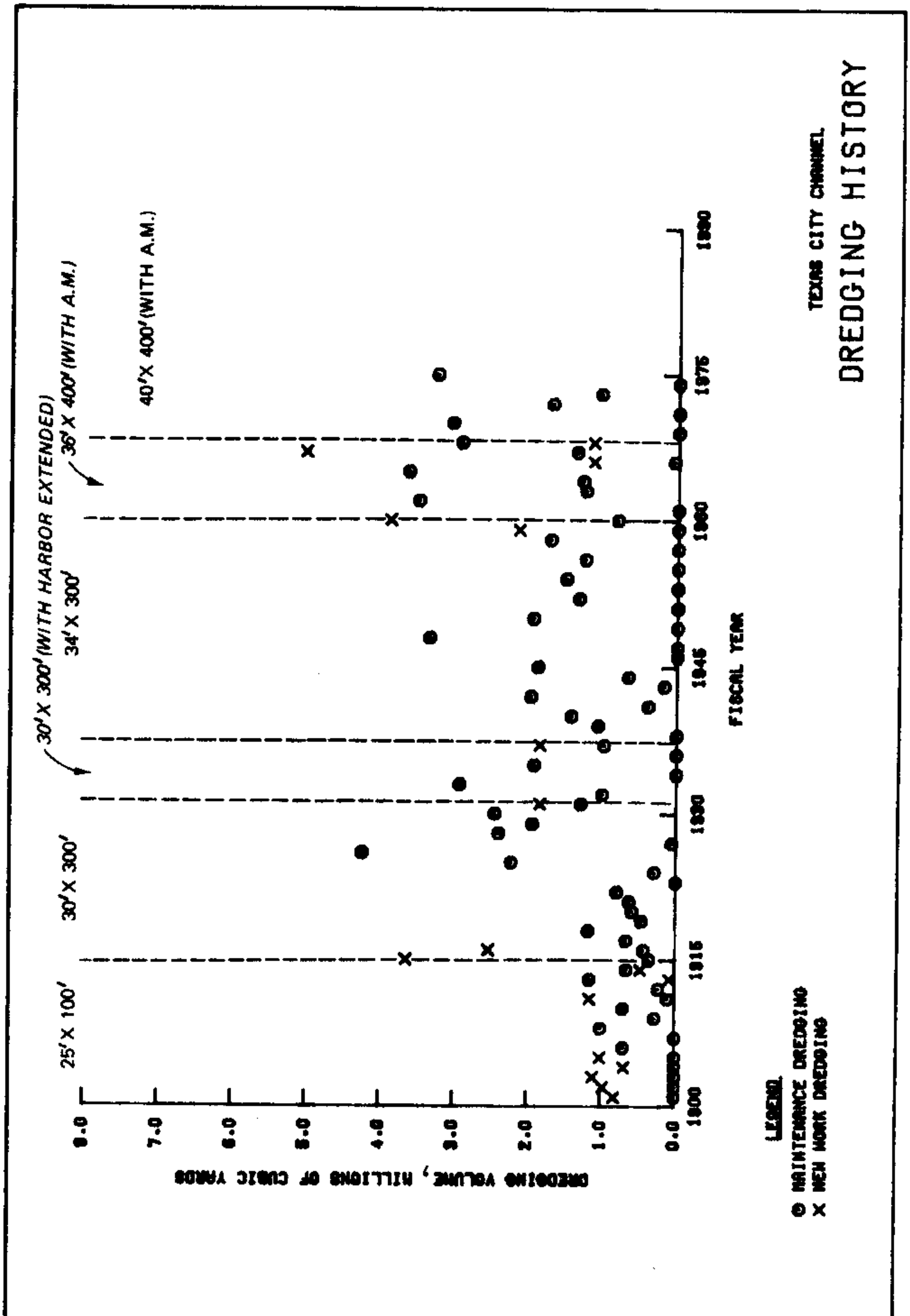

TEXAS CITY CHANNEL
DREDGING HISTORY
DREDGING VOLUME, MILLIONS OF CUBIC YARDS
FISCAL YEAR
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
1900
1915
1930
1945
1960
1975
1990
25'X 100'
30'X 300'
30'X 300'(WITH HARBOR EXTENDED)
34'X 300'
36'X 400'(WITH A.M.)
40'X 400'(WITH A.M.)
LEGEND
MAINTENANCE DREDGING
NEW WORK DREDGING

PLATE 2

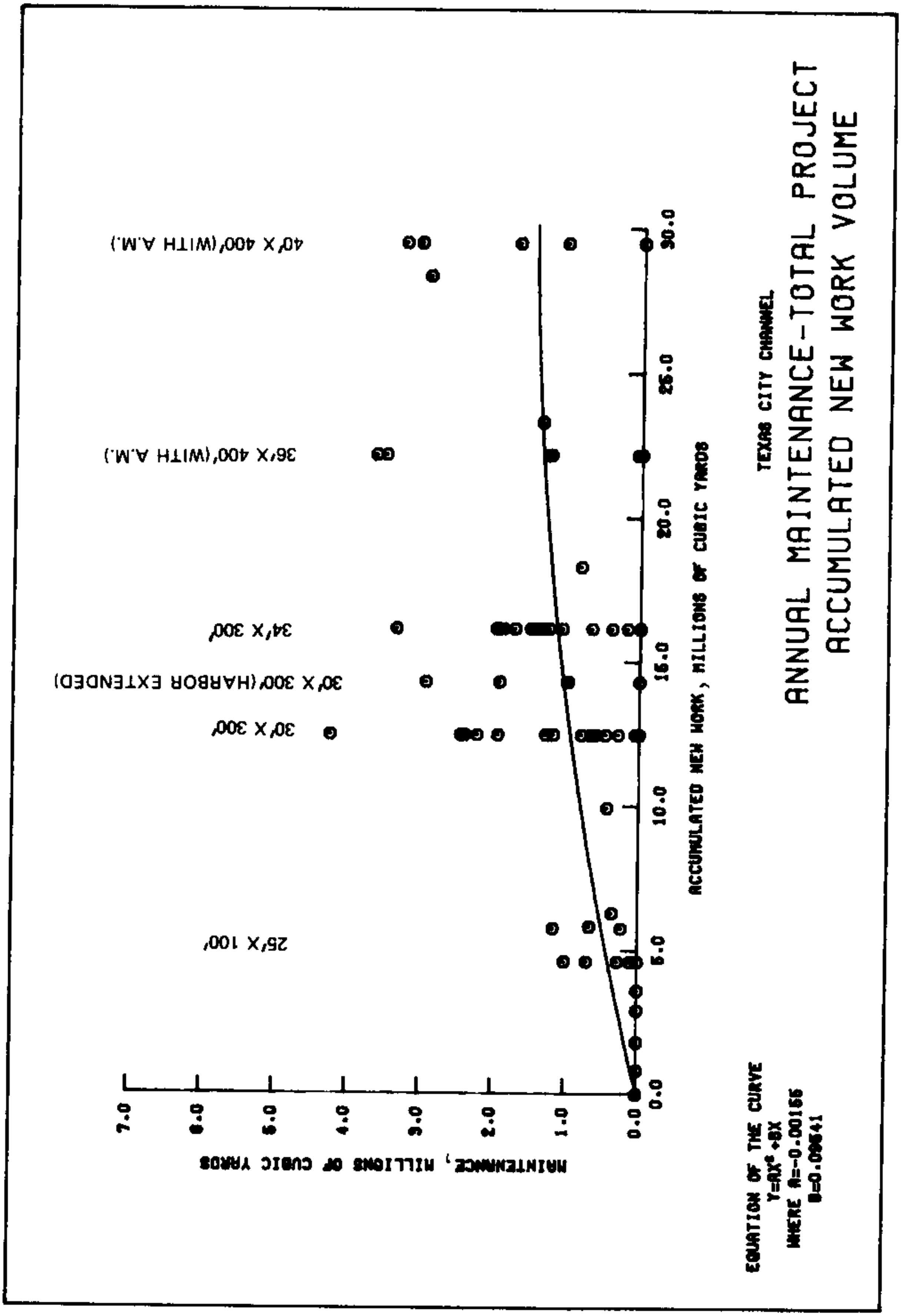

TEXAS CITY CHANNEL
ANNUAL MAINTENANCE-TOTAL PROJECT
ACCUMULATED NEW WORK VOLUME
MAINTENANCE, MILLIONS OF CUBIC YARDS
ACCUMULATED NEW WORK, MILLIONS OF CUBIC YARDS
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
5.0
10.0
15.0
20.0
25.0
30.0
25'X 100'
30'X 300'
30'X 300'(HARBOR EXTENDED)
34'X 300'
36'X 400'(WITH A.M.)
40'X 400'(WITH A.M.)
EQUATION OF THE CURVE
Y=AX²+BX
WHERE A=-0.00156
B=0.09641

PLATE 3

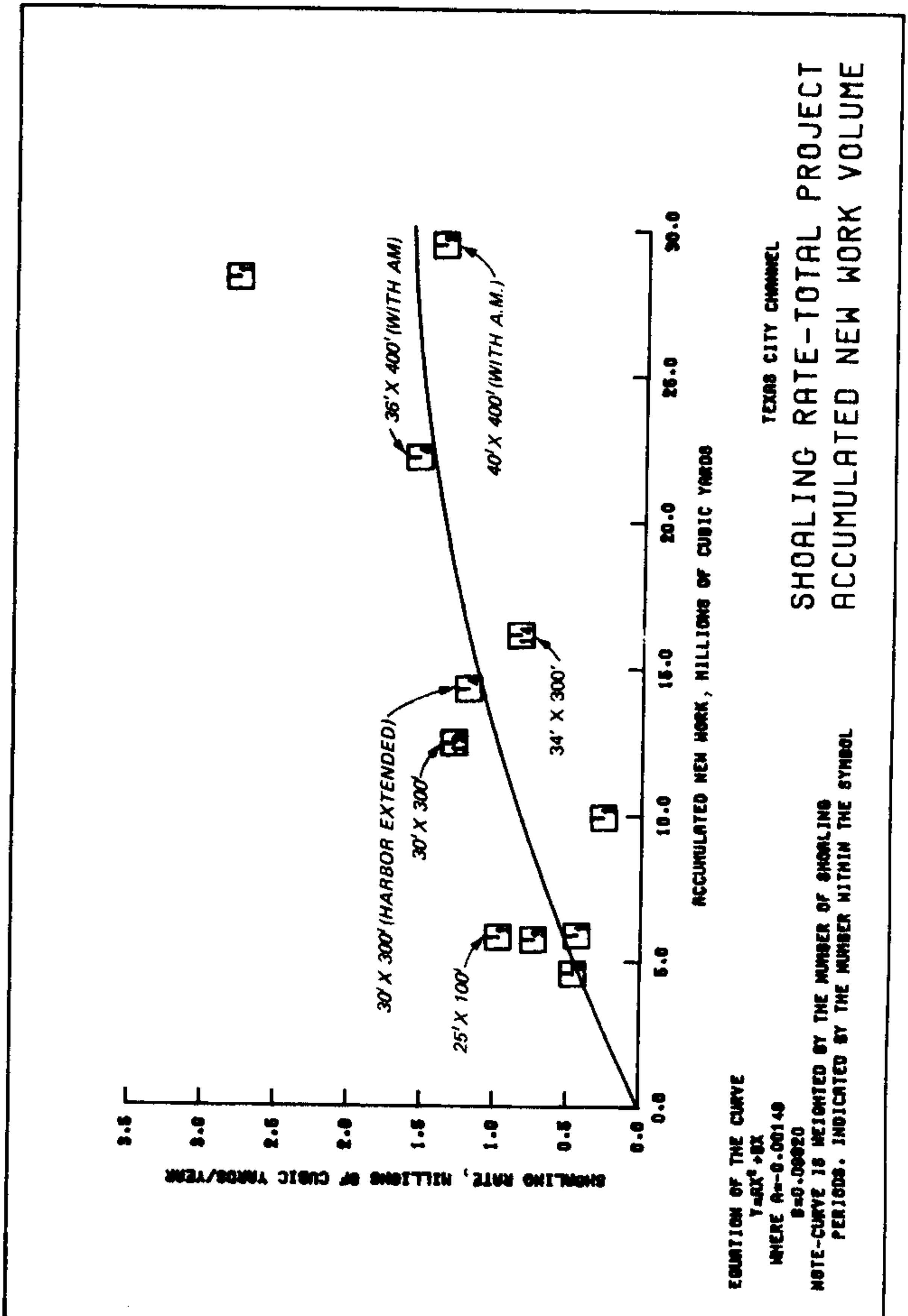

TEXAS CITY CHANNEL
SHOALING RATE-TOTAL PROJECT
ACCUMULATED NEW WORK VOLUME
SHOALING RATE, MILLIONS OF CUBIC YARDS/YEAR
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
ACCUMULATED NEW WORK, MILLIONS OF CUBIC YARDS
0.0
5.0
10.0
15.0
20.0
25.0
30.0
25' X 100'
30' X 300' (HARBOR EXTENDED)
30' X 300'
34' X 300'
36' X 400' (WITH AM)
40' X 400' (WITH A.M.)
EQUATION OF THE CURVE
WHERE A=-0.00140
NOTE-CURVE IS WEIGHTED BY THE NUMBER OF SHOALING PERIODS. INDICATED BY THE NUMBER WITHIN THE SYMBOL

PLATE 4

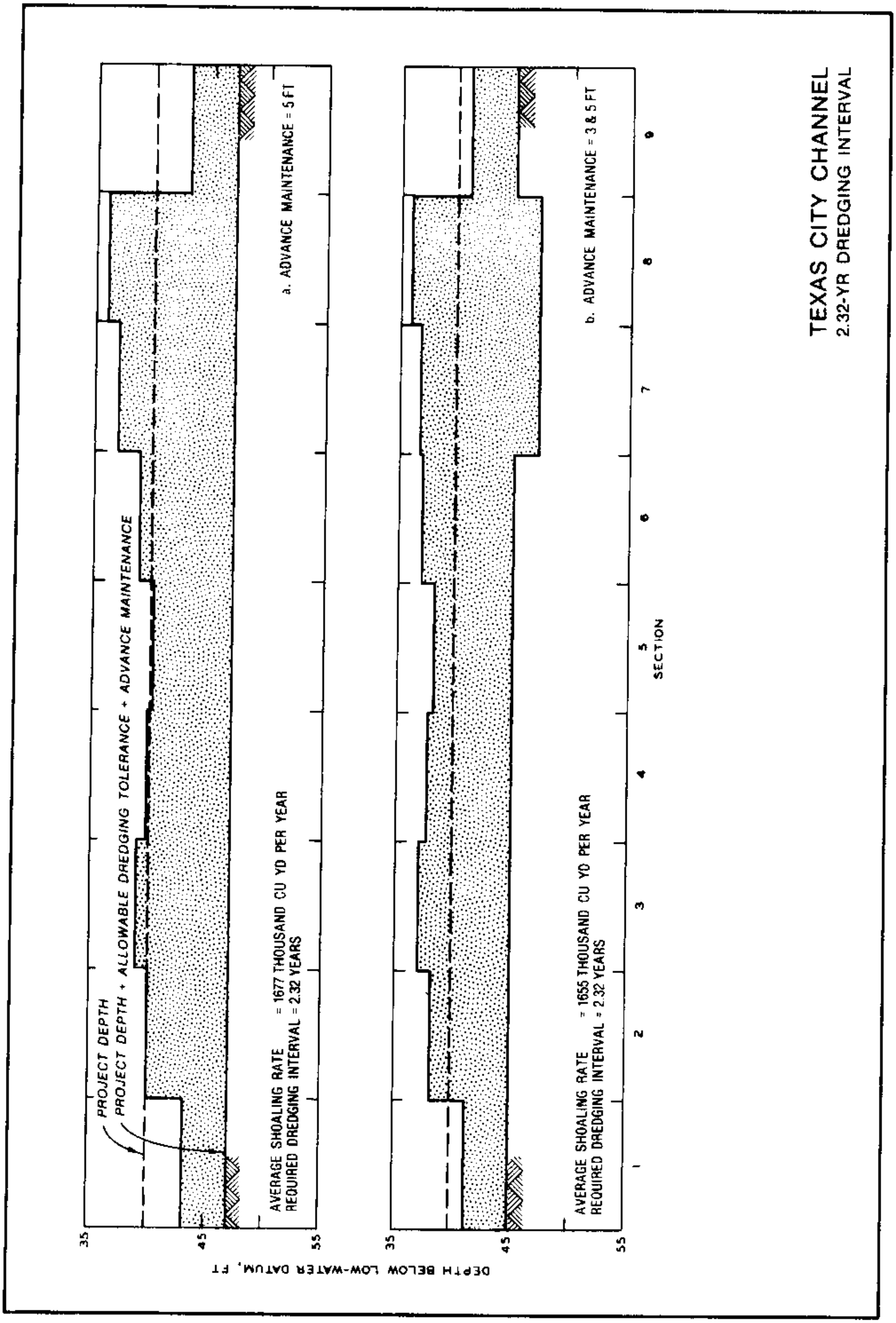

TEXAS CITY CHANNEL
2.32-YR DREDGING INTERVAL
PROJECT DEPTH
PROJECT DEPTH + ALLOWABLE DREDGING TOLERANCE + ADVANCE MAINTENANCE
AVERAGE SHOALING RATE = 1677 THOUSAND CU YD PER YEAR
REQUIRED DREDGING INTERVAL = 2.32 YEARS
a. ADVANCE MAINTENANCE = 5 FT
AVERAGE SHOALING RATE = 1655 THOUSAND CU YD PER YEAR
REQUIRED DREDGING INTERVAL = 2.32 YEARS
b. ADVANCE MAINTENANCE = 3 & 5 FT
DEPTH BELOW LOW-WATER DATUM, FT
35
45
55
SECTION
1
2
3
4
5
6
7
8
9

PLATE 5

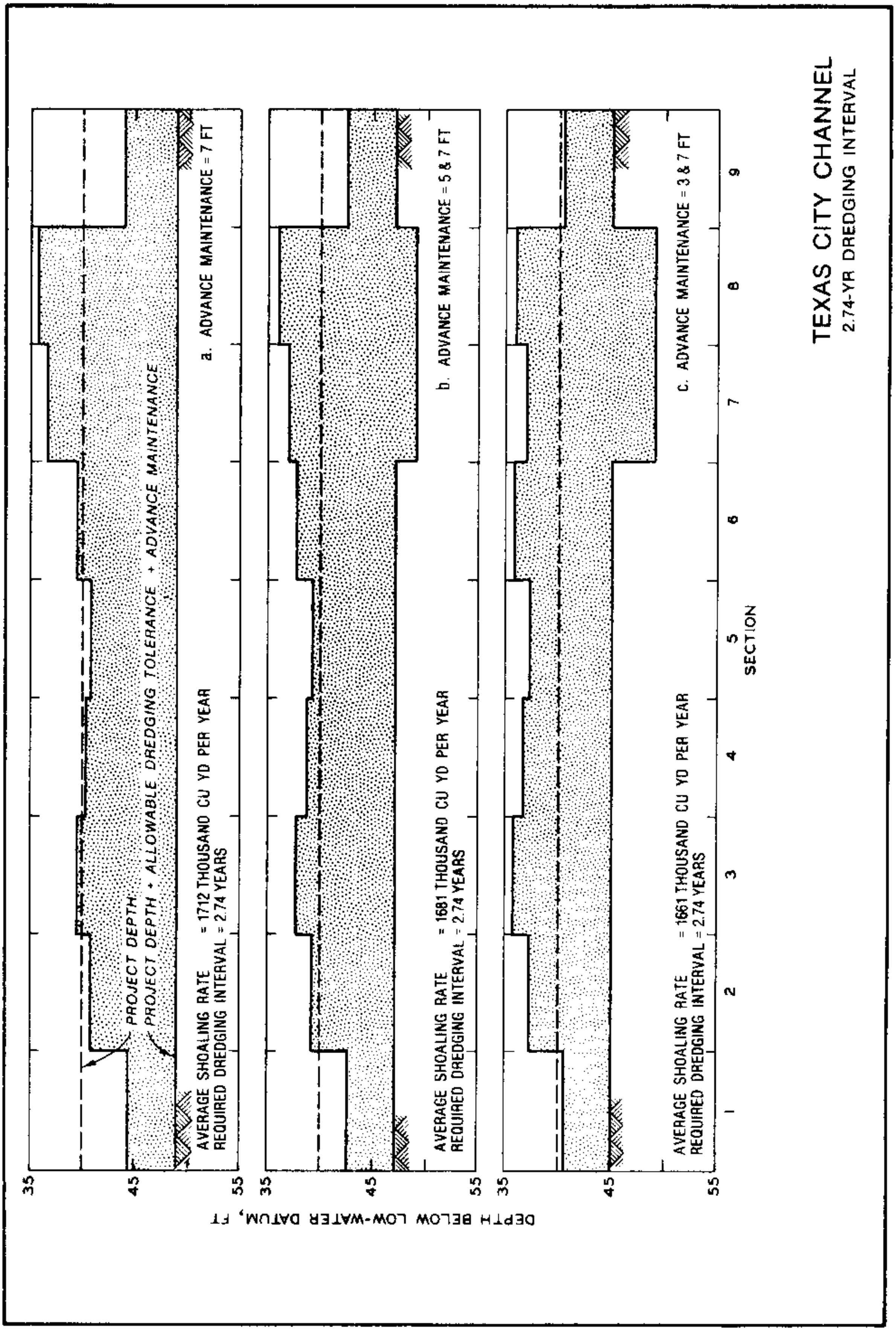

TEXAS CITY CHANNEL
2.74-YR DREDGING INTERVAL
PROJECT DEPTH
PROJECT DEPTH + ALLOWABLE DREDGING TOLERANCE + ADVANCE MAINTENANCE
a. ADVANCE MAINTENANCE = 7 FT
AVERAGE SHOALING RATE = 1712 THOUSAND CU YD PER YEAR
REQUIRED DREDGING INTERVAL = 2.74 YEARS
b. ADVANCE MAINTENANCE = 5 & 7 FT
AVERAGE SHOALING RATE = 1681 THOUSAND CU YD PER YEAR
REQUIRED DREDGING INTERVAL = 2.74 YEARS
c. ADVANCE MAINTENANCE = 3 & 7 FT
AVERAGE SHOALING RATE = 1661 THOUSAND CU YD PER YEAR
REQUIRED DREDGING INTERVAL = 2.74 YEARS
DEPTH BELOW LOW-WATER DATUM, FT
35
45
55
SECTION
1
2
3
4
5
6
7
8
9

PLATE 6

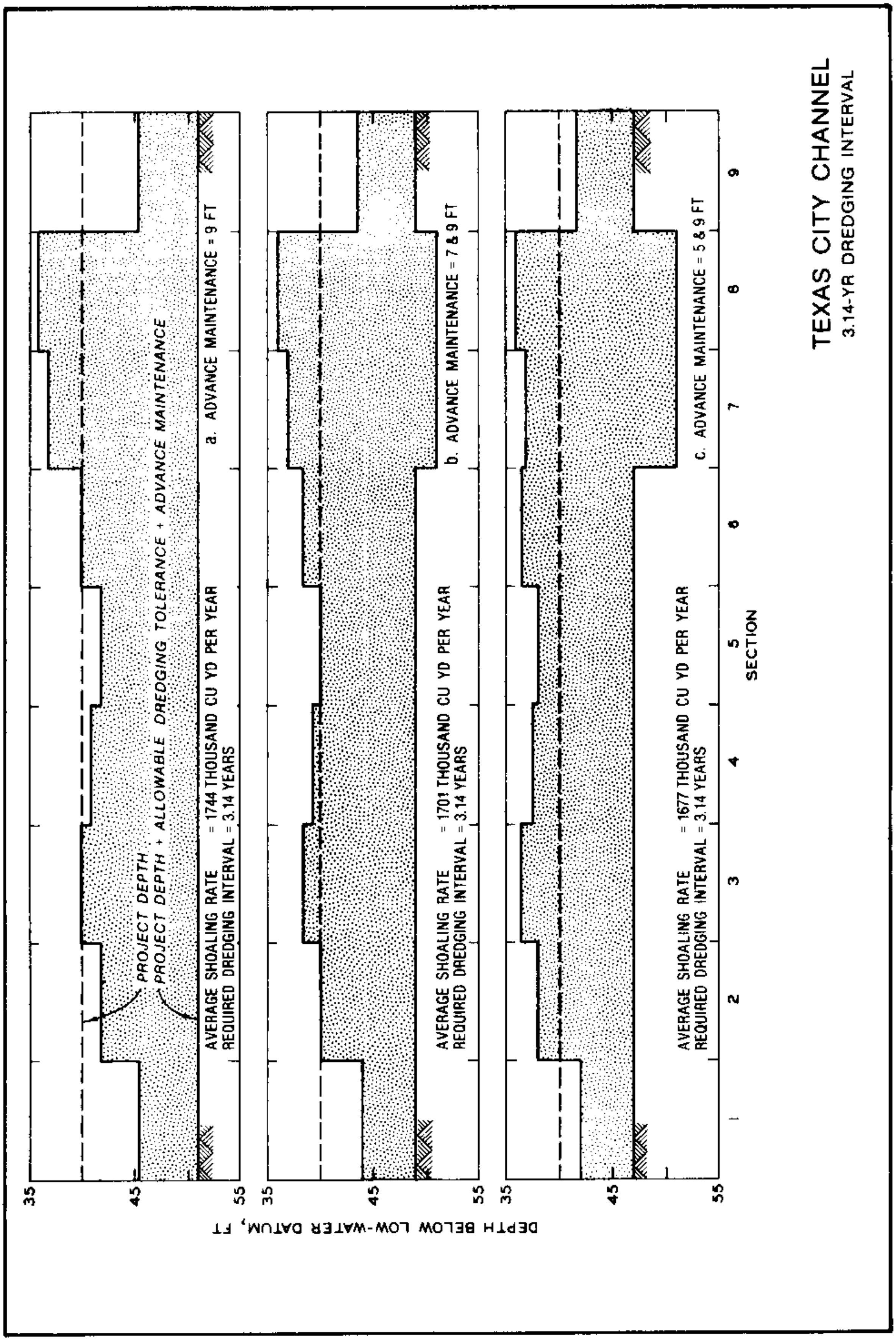

TEXAS CITY CHANNEL
3.14-YR DREDGING INTERVAL
PROJECT DEPTH
PROJECT DEPTH + ALLOWABLE DREDGING TOLERANCE + ADVANCE MAINTENANCE
AVERAGE SHOALING RATE = 1744 THOUSAND CU YD PER YEAR
REQUIRED DREDGING INTERVAL = 3.14 YEARS
a. ADVANCE MAINTENANCE = 9 FT
AVERAGE SHOALING RATE = 1701 THOUSAND CU YD PER YEAR
REQUIRED DREDGING INTERVAL = 3.14 YEARS
b. ADVANCE MAINTENANCE = 7 & 9 FT
AVERAGE SHOALING RATE = 1677 THOUSAND CU YD PER YEAR
REQUIRED DREDGING INTERVAL = 3.14 YEARS
c. ADVANCE MAINTENANCE = 5 & 9 FT
DEPTH BELOW LOW-WATER DATUM, FT
SECTION

PLATE 7

Michael J. Trawle

Biographical Sketch

Education

B.S., Civil Engineering - University of Texas at El Paso - 1967

M.S., Civil Engineering (Hydraulics Option) - Colorado State University - 1969

One Year Post Masters Work in Ocean Engineering - Texas A&M University - 1977

Experience

Civil Engineer - Geotechnical Laboratory - USAE Waterways Experiment Station, 1969-70

Civil Engineer - Shell Oil Company, Los Angeles, California, 1971

Research Hydraulic Engineer - Hydraulics Laboratory, Waterways Experiment Station, 1972 to present. Involved in estuarine water quality and sedimentation research, using both physical and mathematical models.

Organizations

Registered Professional Engineer in Mississippi

Member Tau Beta Pi

Member of American Society of Civil Engineers (ASCE)

Corresponding member of Task Committee of the Hydraulics of Tidal Inlets (ASCE)

RESOLUTION OF INTERNATIONAL DREDGING DISPUTES BY ARBITRATION

by John Huston, P.E.[1]

PROLOGUE

In 1978 the Central Dredging Company got into a disagreement with the owner of the dredging project on which it was working. Unable to come to an agreement, Central filed a lawsuit in the owner's country. From the date of filing the suit, it took 28 months for the case to come to trial. Then, principally because of the delays common to litigation, it took another 10 months to try the case. Central's claim was for $1,800,000. It lost the suit. In addition, it incurred expenses of $112,000.

At about the same time, but in another part of the world, the B&F Corporation got into a dispute with its project's owner - - a dredging dispute incidentally, quite similar to that of Central's. Like Central, B&F and the owner were unable to come to an agreement. However, instead of filing a lawsuit, B&F went to arbitration, having previously provided for it when the contract was written. From the time the claim

[1]Consulting Engineer, John Huston, Inc., Corpus Christi, TX

was filed until the time of the award, the arbitration proceedings took only 124 days. B&F's claim was for $1,300,000. They were awarded $975,000 by the arbitrators. Their total expenses for the arbitration were only $33,000.

Now the names of these two companies, and the amounts of their claims and awards, are not the actual ones. I've changed them - - as they say - - to protect the innocent. However, the times involved in both cases are. Central waited over three years to not only lose its case, but to run up some pretty high expenses. On the other hand, B&F received a substantial award in four months, and their expenses were nominal compared to those of Central.

Examples like these are common. As the first example points out, court litigation can be expensive and time consuming, and decisions are often inequitable. Rulings are usually made by judges who, in most cases, have little or no knowledge of the construction industry - - let alone dredging, and, although it may not be true, always seem to favor thier own countrymen.

On the other hand, as the second example indicates, the delays and inequities of court decisions can be overcome by arbitration. Proceedings move rapidly, and awards are made by arbitrators who are familiar with the practices, customs and costs of the industry.

IMPORTANCE OF ARBITRATION

International arbitration is becoming increasingly important to the world-wide dredging industry. Because of the nature of the dredging process, the great size of projects, and the many changing conditions, dredging creates a situation that is productive of a whole variety of disputes. These, plus the increasing complexity of equipment and rapidly increasing costs, have caused a proliferation of disputes. Arbitration has proved to be the most advantageous means for the settlement of the great majority of them.

ADVANTAGES OF ARBITRATION

The advantages of arbitration over litigation are many. Following are just a few of them.

Expertise. One of the chief advantages of arbitration is that arbitrators are chosen who are familiar with the practices and customs of the industry of the case being arbitrated, and understand the technical issues. Judges in litigation cases seldom do.

Informality. Arbitration operates in an atmosphere of informality. It is far less hostile than court proceedings. Arbitrators tend to compromise far more than do the courts, and they are not required to follow legal rules of evidence. They often accept evidence that would not be permitted in a court of law. Each party has the right to be represented by legal counsel but the hearing is conducted in a less rigid and more informal atmosphere.

Economy. Because of the informality in the proceedings, less time is required to prepare and present a case for arbitration than for a court of law. Although a party's attorney may charge a regular per diem rate, the fewer days involved in arbitration usually result in a much lower total bill.

Convenience. Another advantage of arbitration is that a claimant's attorney can present his case wherever it is most convenient or practical to hold the hearings. In the United States for instance, he does not have to retain associate counsel if the hearing is in a state where he is not licensed to practice, as he would in a court of law. This can often provide a savings in legal fees for parties who may engage attorneys from states other than that where the arbitration proceedings are held.

Neutral-Country Setting. International arbitration also has the advantage of providing for the possibility of a truly international forum. One party does not have to go into the national courts of the other.

Privacy. Disputes are resolved in an atomosphere of privacy and are not subjected to press coverage or public record as are most litigation cases. Results of the arbitration proceedings and awards are known only to the arbitrators and the parties to the proceedings.

ACCEPTANCE OF ARBITRATION

For the reasons mentioned, and many others as well, international arbitration is being increasingly used, not only in connection with disputes between parties from countries which have a long history of trade between them, but also for disputes between parties from countries where trading has been only recent - - such as between capitalistic and socialistic countries, or between the industrialized and developing countries. Additionally, international arbitration is one of the best means available for settling contract disputes between those with different cultural backgrounds.

INTER-AMERICAN ARBITRATION COMMISSION

As in the United States, there is strong support for arbitration in Latin America. The Inter-American Commercial Arbitration Commission (IACAC) was established in 1934 as a result of the seventh International

Conference of American States in Montevideo, Uruguay, in 1933, It maintains and administers a system throughout the western hemisphere for the settlement by arbitration of international disputes. It includes almost all countries of the western hemisphere.

This Commission provides services to parties who request arbitration, in accordance with the Rules of the Commission. Arbitration is conducted by arbitrators who are selected by the parties, or by the Commission, in accordance with its Rules, utilizing existing panels of highly qualified individuals who render awards on the merits of the dispute. These Rules of the IACAC have the substantive provisions of the United Nations' Commission on International Trade Law (UNCITRAL), and were recommended by the United Nations General Assembly on December 15, 1976.

HISTORY OF ARBITRATION

Early arbitration cases were usually arbitrated quite informally. Each party selected its own arbitrator, and these two then selected a neutral one. This system resulted in many complaints however, for there were no established rules or administrative procedures. Cases dragged on, procedures were unregulated and numerous problems arose. Even today in some maritime arbitrations is the time-honored practice of an arbitration proceeding presided over by an arbitrator who conducts a "hotel-room" hearing.

Most arbitration procedures are now more organized, and specific Rules of procedure are defined. Additionally, numerous organizations are available throughout the world to administer the proceedings.

For those of us in the western hemisphere, the most important one is the American Arbitration Association (AAA).

PROVIDING FOR ARBITRATION

Arbitration should be provided for before a dispute arises. To do this a provision should be included in the construction contract. The provision is expressed in a Future Dispute clause. When this clause naming an administering institution and Rules of procedure, such as the AAA, is included in the contract, that institution will carry out the administration of any dispute that arises with speed and efficiency. However, this institution should be advised immediately whenever the clause is included in a contract, not later, after a dispute has arisen. Should the Future Dispute clause not have been placed in the contract, and a dispute arises, a signed statement, a Submission Agreement, by both parties, can bring the dispute to arbitration. Such statements require a brief description of the issue in contest, and the parties' agreement to arbitrate under the institution's Rules. Also included in the Future Dispute clause and the Submission Agreement should be the parties' desires as to the number of arbitrators, the place of arbitration and the language to be used.

Each institution providing arbitration services has its own model clause for the purpose of contract inclusion. It should be noted however, that in preparing these clauses for international arbitration, a considerable number of factors come to bear - - as will be pointed out later in this article. Consequently, clauses should be prepared only after a thorough study of all the factors having to do with the proceedings of arbitration. It would additionally be wise to consult with the institution chosen to administer the proceedings prior to preparing the clause, thereby assuring proper wording.

CONCILIATION - A PRE-ARBITRATION PROCEDURE

Conciliation sometimes can eliminate the need for arbitration. This involves the mutual agreement between the parties that one independent, neutral and qualified expert will be engaged to prepare a detailed report covering the dispute, giving his opinion as to the responsibilities of both parties. A number of such cases have occurred in international disputes, and differences have been resolved accordingly. In other instances, where conciliation was not accomplished by this procedure, the expert consultant was requested to act as a single arbitrator to settle the issue. Several such cases as these were noted during the IACAC meeting in Mexico in March of 1978.

WHERE TO HOLD THE ARBITRATION PROCEEDINGS

The answer to the question of where to hold the arbitration proceedings is difficult to answer. It is however, becoming evident through experience, that international arbitration cases should be conducted in a third country - - one different from the countries of the parties. Persons who come from different social and economic backgrounds often feel uncomfortable in each others' national courts or arbitration tribunals. The solution has proven to be for them to choose to arbitrate in a third or neutral country.

In choosing the neutral country however, a considerable number of factors have to be considered. For one thing, arbitration proceedings are typically governed by the law of the country in which the arbitration takes place. Consequently, it is important to know whether the law in a specific country will facilitate or hinder the arbitration. Additionally, since it may be necessary to appeal to the courts if a party refuses to honor the arbitration agreement or award, the law of the country where inforcement will be sought becomes exceptionally important.

It is necessary also in choosing the place to arbitrate, to determine if the law of the country being considered will permit the arbitration to be conducted in accordance with the arbitration Rules selected. Legalities preventing use of the Rules would be detrimental, because procedural rules not only facilitate the conduct of the arbitration, but they also help to insure fairness and uniformity.

Parties considering the location of the arbitration proceedings should also realize that not everything can be arbitrated everywhere. What can be arbitrated is determined by the law governing arbitration in the country where the arbitration is to take place. Different countries' laws vary widely in this respect.

The consequence of not being able to arbitrate in some locations is in direct opposition to the policy in the United States. There anything can be arbitrated that has been provided for. Arbitrators in the United States have the power to decide questions of law as well as fact. However, in England and the Commonwealth, for instance, the courts control questions of law that may arise during the arbitration proceedings.

Before a decision is made concerning where to hold the arbitration proceedings, a thorough study of that country's arbitration laws should be made. As was previously pointed out, such precautions are not necessary when arbitration proceedings are conducted in the United States.

APPOINTMENT OF ARBITRATORS

Experience has demonstrated that arbitrators serve best when they are not of the parties' own countries. Third-country arbitrators are usually more effective and successful in reaching satisfactory agreements.

Where parties agree to arbitrate under the Rules of an institution such as the AAA, the Rules provide for delegating authority to name the arbitrators, if the parties themselves fail to do so. Under the Rules, the arbitrators named will usually be persons who are of different nationality than that of the parties to the dispute. However, if the parties fail to specify the Rules to be used, or some other method of appointing arbitrators, the court of the country where the arbitration is held will make the selection. It is interesting to note that judges in such instances, or so it seems, almost invariably appoint nationals of their own country.

ADMINISTRATIVE SERVICES

Most arbitration institutions provide for administrative services. UNICITRAL, however, does not render such services, but permits parties or the arbitrators to arrange for such services by some other administrating organization.

Only a few arbitration institutions, the AAA being most notable, are prepared to render administrative services throughout the world.

Most Chambers of Commerce and most other organizations will only provide administrative services within the border of their own country, and sometimes only in their headquarters city.

Experience has proven that for western hemisphere arbitration proceedings, designating a major city in the United States is preferable, not only because of the administration services available in the AAA's national network of 24 regional offices, but also because of the availability of business services, adequate and rapid transportation and communication facilities, and so on. Furthermore, the reputation and prestige of the AAA is internationally recognized, and western hemisphere parties more often than not agree upon a site in the United States for their arbitration.

THE AMERICAN ARBITRATION ASSOCIATION

The AAA is a non-profit organization with over 50 years of arbitrational experience. It has grown to be the largest arbitrational organization in the world. With more than 400 employees, its experience is based upon a combined caseload of hundreds of international cases annually, many of them involving substantial sums. The AAA itself does not arbitrate disputes. It provides panels of arbitrators and the facilities for conducting arbitrations - - administration services, hearing rooms, and orderly procedure according to its rules and standards.

Its fees for arbitration services are set to cover only the actual costs of administering the proceedings. Upon filing of a case, the claimant pays an initial filing fee of $150. The balance of the fee is based upon a percentage of the amount of the claim as disclosed at the time of filing. Fees are nominal. For instance, the fee for a claim less than

$10,000 would be three percent of the claim. For a claim between $200,000 and $5,000,000, the fee would be $1850 plus 0.25 percent of the amount over $200,000.

ARBITRATION PROCEEDINGS UNDER THE AAA

Upon receiving a request for arbitration, the AAA sends each party a list of names of proposed arbitrators, technically and administratively qualified to resolve the controversy. These names are taken from the AAA's extensive file of trained and qualified arbitrators in the fields of construction and engineering. The arbitrators will include contractors, engineers, and other experts who are familiar with the claimant's industry. In small-sum cases - - less than $50,000 - - one arbitrator is usually all that is appointed. In larger-sum cases three arbitrators are usually selected.

Parties to the dispute are given a specified number of days to study the list of proffered arbitrators, cross off names of persons to whom they object, and to number the remaining names in order of preference. Where parties need more time, or want more information about a proposed arbitrator, the AAA provides it upon request. When both parties' lists are returned to the AAA, it compares them and appoints arbitrators whom both parties have approved. Where mutual choices are not possible, additional lists are submitted. If the parties still cannot agree upon arbitrators, the AAA makes administrative appointments. However, in no case will an arbitrator be appointed whose name has been crossed out by one of the parties.

After the arbitrators are appointed, the AAA consults with the parties to determine a mutually-convenient time and place for the hearings. The attempt in all instances is to make the location and the time convenient to all concerned.

All arangements for the arbitration proceedings are made by the AAA. This relieves the arbitrators of routine duties, and eliminates the danger that in the course of conversations outside the hearing room, one party might offer arguments on the merits of his case that the other party would not have the opportunity to rebut.

At the hearing the arbitrators listen to all the evidence they feel is relevant and material to the case. Usually everyone meets around a single conference table. Each party is given the opportunity to present their case by offering evidence, expert witnesses, and any other form of information that they feel may convince the arbitrators of the correctness of their position.

Usually the complaining party presents their case first. Following this the other party is given the opportunity to cross-examine witnesses presented by the first party, and then to present their case. The first party is then allowed to cross-examine witnesses of the second party. This order may be varied however, depending upon what the arbitrators think advisable. Each party, nevertheless, is allowed their chance to present what they think is the correctness of their position, and the proceedings continue until each party has had full opportunity to do so.

At any time during the hearing the arbitrators may question witnesses, or ask either or both of the parties for supporting information to assist them in their deliberations. It is noteworthy to point out here the marked contrast between the United States' procedure and that of many other countries - - the United States employs the time-honored practice of utilizing witnesses and cross-examination in determining the facts, while other countries place great emphasis on documentation alone.

Following both parties' presentation of their cases and the submission of all evidence, the arbitrators adjourn the hearing to determine their findings. They are required to report their decision within a specified number of days following the closing of the hearings. Upon announcing their decision, in writing, to both parties, their power as arbitrators end.

The decision of the arbitrators cannot be changed unless both parties agree to reopen the case, or unless applicable law provides for a reopening. The awards made by the arbitrators may be enforced under common law in all states.

SOURCE OF INFORMATION ON ARBITRATION

The AAA is the best known source for obtaining information on all phases of arbitration. They can be contacted at their central office in New York - - 140 West 51st Street, New York, N.Y. 10020; Telephone Number: 212-484-4000 - - or through any of their 24 regional offices throughout the United States.

Anyone contemplating the contracting of dredging projects anywhere would be well advised to consult with the AAA prior to completing the contract documents.

REFERENCES

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BIOGRAPHICAL SKETCH

John Huston, P.E., is a marine civil consulting engineer specializing in dredging. He has conceived, investigated, planned, designed, guided, and trouble-shot dredging projects along the coastlines, and in the waterways of Canada, the continental United States, Mexico, the Caribbean, Central and South America, and the Near and Far East.

As a recognized authority on hydraulic dredging, he is constantly called upon by private companies, industrial firms, marine interests, universities, municipalities, states, and governments throughout the world to provide consultation, engineering services, expert witnessing, lectures, and study programs in the many aspects of dredging.

He is a widely-published author. Hydraulic Dredging is used by dredgers throughout the world, and his papers and articles have appeared in many technical and professional journals and publications.

He received his original engineering degree from Iowa State College in Ames, Iowa, and has since completed several specialized courses of study. He was honored in 1974 by his alma mater with a Professional Achievement Citation. He was appointed by the American Arbitration Association to their international panel as an Arbitrator for dredging disputes.

He has been associated with dredging all of his professional life, beginning in dredging as a civil engineer just out of college, advancing to chief engineer and general manager of dredging, and then establishing his dredging consulting practice in 1954.

DEEP DRAFT ACCESS TO THE PORTS OF NEW ORLEANS AND BATON ROUGE, LOUISIANA AND THE CAPABILITY OF THE U.S. PRIVATE DREDGING FLEET TO ACCOMPLISH SIMULTANEOUSLY NEW U.S. PORT DEEPENING PROJECTS IN THE YEARS AHEAD

A LUNCHEON TALK

to the

GULF COAST CHAPTER OF WEDA
and the
GULF COAST DREDGING ASSOCIATION

November 13, 1981

Monteleone Hotel

by

Herbert R. Haar, Jr.
Assistant Executive Port Director
Port of New Orleans

NEED

The impetus behind the 55-foot channel and associated Corps studies is in two factors--progressive increase in ocean-going commerce through the Ports of New Orleans and Baton Rouge and a worldwide trend toward larger ocean-going vessels having correspondingly deeper drafts and, thus, achieving greater economies in operating costs. The restrictions imposed by the existing deep-draft approaches to these two major ports prevent their efficient utilization by an increasing percentage of ships that must move through these approaches only partially loaded.

STUDY ALTERNATIVES

The U.S. Army Corps of Engineers considered a number of alternatives for providing a deep-draft link between the River and the Gulf of Mexico, including locks and open water channels. Project depths of 45, 50, 55 and 60 feet were analyzed. In fact, during the preliminary planning stages, the Corps of Engineers addressed 18 different plans. Several concepts were rejected outright, and the submitted draft main report and draft EIS studied in detail six plans--all having a 55-foot project depth. Incidentally, the final report and revised draft EIS have been completed and are now in Washington for review and approval.

GENERAL DESCRIPTION OF PROJECT

The tentatively recommended plan provides for the modification of the Mississippi River, Baton Rouge to the Gulf of Mexico, Louisiana project to channel dimension of 55 feet deep by 750 feet wide between Baton Rouge and the Gulf via Southwest Pass. The plan also provides for a 55-foot deep by 1,600-foot wide by 4,000-foot long turning basin at the upstream end of the enlarged channel in Baton Rouge, immediately below the U.S. Highway 190 bridge. Training works would be provided in South Pass and Pass A Loutre to redistribute flows to Southwest Pass to reduce its maintenance.

CONSTRUCTION AND MAINTENANCE

The construction of the enlarged channel in the Southwest Pass would require the removal of approximately 62.5 million cubic yards of material. Maintenance of the enlarged channel in this sector would increase from an average of 14 million cubic yards to 29 million cubic yards; thus doubling the maintenance effort. The construction of the enlarged channel from the Head of Passes to Baton Rouge and the turning basin in the Mississippi River will require the removal of 70.5 million cubic yards whereas the average annual maintenance requirements for this sector would increase from about 8 million to about 59 million cubic yards of material.

All the material dredged for the construction and maintenance of the turning basin and the fifty-five foot channel between Venice and Baton Rouge would be deposited in the Mississippi River.

The material excavated from the river between Venice and Head of Passes would be used to build marsh and other habitat types. Marsh is being lost in this area as a result of erosion and subsidence. The material excavated for the construction and maintenance of the enlarged channel in Southwest Pass would be used to build marsh areas, agitated into suspension or deposited on Gulf bottoms by hopper dredges.

With the existing channel depth there are 9 crossings, all north of New Orleans, requiring maintenance (Baton Rouge Crossing, Red Eye, Medora, Granola, Bayou Goula, Alhambra, Philadelphia, Belmont, and Fairview). Red Eye Crossing just south of Baton Rouge requires constant observation and the most maintenance effort. By enlarging the navigation channel, the Corps of Engineers estimates that maintenance requirements will increase to 13 crossings north of New Orleans, and, quite possibly, 2 crossings below New Orleans may require minimum dredging.

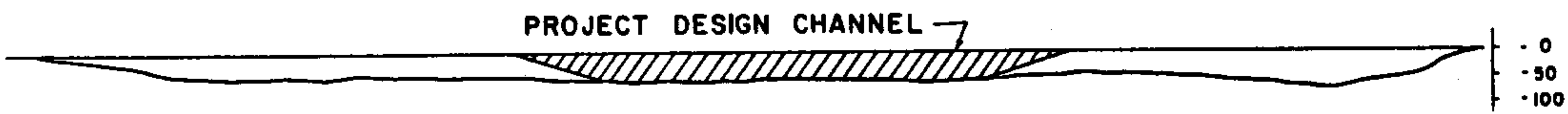
The total estimated first cost of the tentatively selected plan is now $435 million and the estimated annual cost is $145 million. The average annual benefits, or savings in transportation costs, is estimated at $892 million. The ratio of average annual benefits to average annual costs is 9.1.

TRAINING WORKS

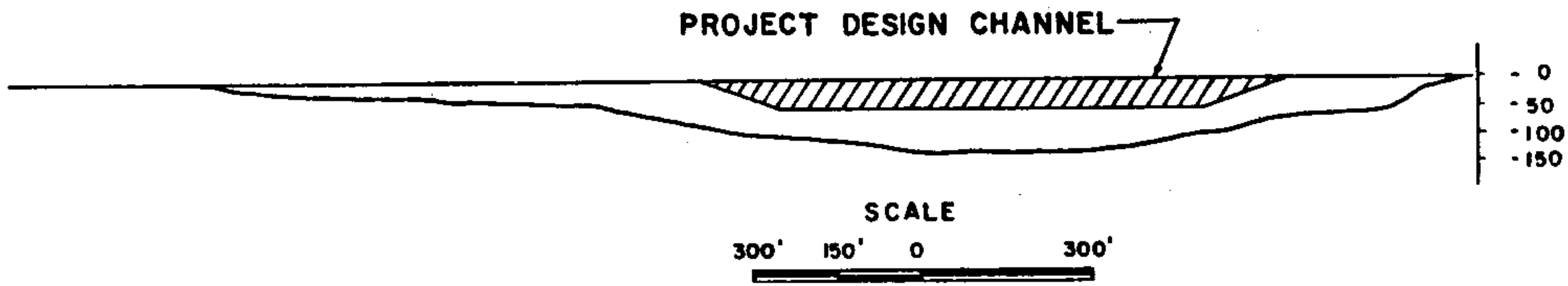
Training works would be required in several outlets of the Mississippi River to divert more flows to Southwest Pass in order to make the maintenance of the enlarged channel in the pass practicable. The enlargement of Southwest Pass to the 55-foot by 750-foot design without measures to increase its flows would effect a significant increase in shoaling rates due to the decrease in flow velocities and, therefore, its sediment-carrying capacity.

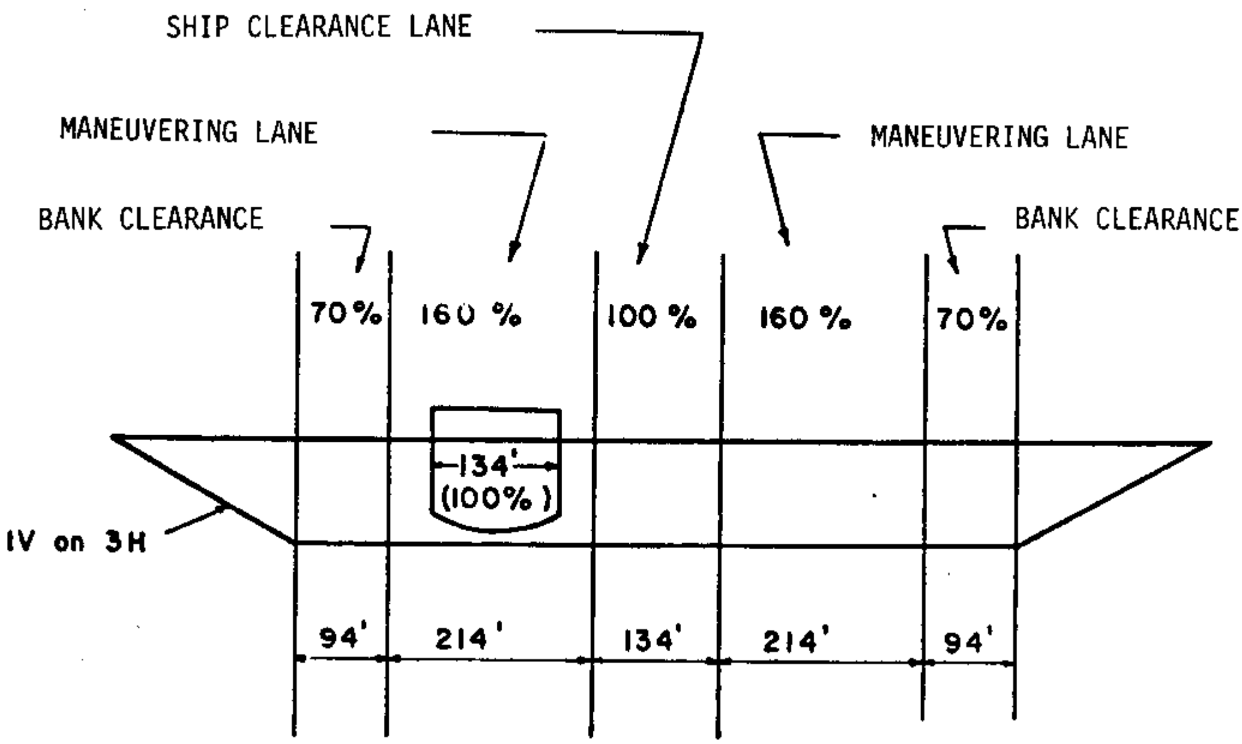
The most feasible method for increasing flows in Southwest Pass would be to construct spur dikes in other Mississippi River outlets such as South Pass and Pass a Loutre in order to divert more flow to Southwest Pass. As South Pass is maintained for navigation, the most feasible method for the diversion of flows to Southwest Pass, from an engineering standpoint, is the constriction of Pass a Loutre by a system of spur dikes.

CROSS SECTION OF TYPICAL CROSSING



CROSS SECTION OF TYPICAL BENDWAY





DESIGN CHANNEL

However, the environmental consequences of reducing flows in Pass a Loutre are much greater than those associated with South Pass. It was determined analytically that the diversion of approximately 50 percent of South Pass's flow to Southwest Pass would effect a significant decrease in shoaling in the enlarged channel in Southwest Pass while causing a relatively minor increase in shoaling in South Pass. The placement of the dike system downstream in the South Pass (between mile 6 and mile 9 below Head of Passes) would minimize the adverse effects to the South Pass subdelta associated with the reduced flows.

To determine what would be required to accomplish a 50 percent reduction in flow in South Pass, an analytical computation was made in South Pass with and without spur dikes. First, the existing conditions profile was computed for its average annual discharge of 78,000 cubic feet per second. Then, by trial and error, the length and spacing of spur dikes along the channel were varied until the same stage was achieved at Head of Passes for only 50 percent of the average discharge through the South Pass. It was determined that treated timber spur dikes providing for a 450-foot wide opening would be needed along 3 miles of South Pass at a spacing of 300 feet in order to duplicate the stage. The dikes in Pass a Loutre were added to stabilize its channel and prevent its capture of flows diverted from South Pass. These dikes are also of treated timber construction at a spacing of 300 feet along 3 miles on each side of the Pass--in this case beginning at the mouth of the Pass a Loutre. The dikes consist of a series of 4 pile clusters forming the anchorage for a continuous vertical curtain wall of piles perpendicular to the channel banks. It is interesting to note that the estimated maintenance requirements for the Southwest

Pass for a 55-foot by 750-foot channel without Training Works is 40.5 million cubic yards and with Training Works is 29.3 million cubic yards--a 28% reduction in yardage--thus, providing an estimated net annual benefit of approximately $8.9 million when comparing cost of the Training Works with the average annual savings in maintenance dredging cost.

RELOCATIONS

No wharves or barge fleeting operations, or other shore facilities would be relocated. However, construction will require relocation of about sixty-four submarine pipelines and eighteen submarine cables, the majority of these being upstream from New Orleans, with seven pipelines being located in the Southwest Pass.

Although the project depth for the proposed channel enlargement is 55 feet, the initial dredging will probably include 3 feet of advance maintenance and 2 feet of allowable overdepth. Current permit policy for the Mississippi River requires a pipeline to be installed 25 feet below the project depth or 25 feet below the river bottom, whichever provides the greater clearance. Coverage along the sides of the channels may be less than the 25-foot requirement when the stability of the levee system would be affected. In areas that require maintenance dredging pipelines and cables that are currently 70 feet or deeper will not require relocation. Pipelines less than 70 feet would have to be relocated and would be required to be buried to 80 feet. Pipelines existing unburied in water deeper than 60 feet would not have to be relocated and buried. In the interim prior to the possible authorization of a 55-foot project depth channel, all new pipelines and cables crossing the river will be required to be placed 80 feet deep or 25 feet below the river bottom, whichever provides the greatest clearance. As an alternative to this interim requirement, they can be permitted to install their pipelines and cables 65 feet deep (25 feet below the existing 40-foot channel) or 25 feet below the river bottom, whichever provides the greatest clearance, provided that they sign an agreement that their facilities will be relocated at the owners expense, when and if a 55-foot project depth is authorized.

SALT WATER INTRUSION

The implementation of the tentative plan for channel enlargement would cause an increase in salt water intrusion in the Mississippi River. It would result in marsh creation near the mouth of the Mississippi River, and it would impact on the river's fisheries and water quality.

The enlargement of the navigation channel in Southwest Pass would cause an increase in the frequency and duration of salt water intrusion up the Mississippi River during extended low water periods. The denser salt water from the Gulf moves upstream along the bottom of the river, while fresh water flows downstream at the surface. Deepening the channel would allow more flows to move upstream along the bottom during any low water period.

This increase would not significantly affect municipal water supplies in the New Orleans area as the duration of salt water intrusion exceeding the E.P.A. standard for drinking water would be less than one day. Downstream of New Orleans, the frequency and duration of salt water intrusion would be slightly increased over present conditions. Mississippi River water in these areas already exceeds E.P.A. standards for salinity during low water periods and water must be stored for use during these low water periods. Included in the recommended plan are additional municipal water storage reservoirs at East and West Pointe a la Hache in lower Plaquemine Parish, Louisiana. A 2 million gallon reservoir would be added at the East Pointe a la Hache water treatment plant (mile 49.2) and a 30 million gallon reservoir would be added at the West Pointe a la Hache water treatment plant (mile 49.0 AHP).

Water from the reservoirs would be mixed with Mississippi River water to prevent chloride concentrations from exceeding U.S.E.P.A. standards. The average dilution would be 50 percent. The storage capacity was designed to accommodate the average increased duration of saltwater intrusion of 5 days over the 50-year project life assuming a 100 percent increase in usage, based on projected population growth in Plaquemine Parish.

The east bank facility would serve only the East Pointe a la Hache water treatment plant while the West Pointe a la Hache reservoir would serve the West Pointe a la Hache and the Boothville-Venice water treatment plants. The facilities are connected by pipeline.

I would now like to touch briefly on the impact on the U.S. dredging industry due to the construction simultaneously on a number of deep draft ports starting next year. I am very much indebted to the Great Lakes International, Inc. and C. F. Bean Corporation for the data that I am about to present. They developed the information and presented it to a meeting of my AAPA Special Dredging Task Force in July of this year. First, I will highlight the findings of the report and then I will discuss in somewhat more detail some of the specifics from the study.

IMPACT ON UNITED STATES DREDGING INDUSTRY DUE TO CONSTRUCTION OF DEEP DRAFT PORTS

1. The industry is well able to undertake the workload envisioned in the construction of 5 deep draft ports simultaneously over a 3-year period in addition to the normal workload of Corps of Engineers' projects and private work.
2. Based on actual 1978-1980 experience, the industry's fleet of 191 large dredges can handle an annual workload of at least 711,000,000 cubic yards.
3. During the 1978-1980 period, the industry operated at 53% of capacity.
4. Projected annual Corps maintenance workload is 201,800,000 cubic yards. This is based on an 11-year study of Corps records.
5. Estimated non-Corps normal workload is 167,000,000 cubic yards.
6. Simultaneous construction of Hampton Roads, Mobile, New Orleans, Baltimore and Galveston in a 3 year span will require an annual workload of 170,000,000 cubic yards.
7. Additional maintenance dredging resulting from the above deep ports is estimated to be 59,900,000 cubic yards annually.
8. Summation of the above is a total annual peak workload of 598,700,000 cubic yards.
9. The peak workload is 84.2% of the industry capability.
10. The industry has spent a quarter of a billion dollars in the last 5 years in construction of 15 major dredges - 4 cutterheads, 10 self-propelled hoppers and 1 dustpan.

11. Revenues derived from dredging during the 3 year period envisioned for development of the 5 deep ports plus normal dredging will be over $1.2 billion annually.

12. The dredging industry will employ over 7,000 employees having annual wages over $200 million. Employee income taxes will be over $40 million per year.

13. Assuming normal profitability, the industry will pay over $75 million in federal and local taxes.

14. Maintenance of equipment, petroleum products, spare parts, etc. will provide over $400 million in revenues to suppliers annually.

I will now cover a few of the details from the report that you may find of interest (see enclosed).

It has been a pleasure to be here with you today and I will be happy to answer any questions that you may have.

LOUISIANA PORTS - WATERBORNE COMMERCE

May 1981

Total Navigable Waterways in Louisiana - 6,905 miles

Total Waterborne Commerce Tonnage (Foreign & Domestic) as reported by Corps of Engineers, U.S. Army - 1978

Total U.S.	2,021,350,000	
Baton Rouge to Gulf	572,181,000	
Gulf Intracoastal Waterway	84,364,000	
Rivers (other than Miss.)	13,431,000	
Bayous	10,119,000	
Other Waterways	7,334,000	
Total Louisiana Waterways	687,429,000	(includes through traffic)

Louisiana total waterborne commerce tonnage, including through traffic, was 34% of total U.S. for 1978. Louisiana ports handled approximately 404,000,000 tons or 20% of the U.S. total.

Total Waterborne Commerce Tonnage - 1979
Corps of Engineers, U.S. Army

New Orleans	167,135,000	1st in U.S.
Baton Rouge	76,703,422	4th in U.S.
Lake Charles	35,951,037	

Total Foreign Waterborne Trade Tonnage as reported by the U.S. Department of Commerce - 1979

Total U.S.	962,658,000
Louisiana Ports	194,696,000
Louisiana percent of U.S.	20%

Total Foreign Waterborne Trade Value - 1979

Total U.S.	$244,982,000,000
Louisiana Ports	34,277,000,000
Louisiana percent of U.S.	14%

Total Grain shipments in bushels as reported by the U.S. Department of Agriculture - 1980

Total U.S.	4,951,687
Louisiana Ports	1,947,061
Louisiana percent of U.S.	39%

Economic Impact of Foregin Trade generated by Louisiana Ports. (Taken from a preliminary report of the U.S. Economy and port industry as constructed by the Port Authority of N.Y. and N.J. Some estimates from the computer for present impact are: Each 600 tons of foreign trade [ex. petroleum] equals one job. The direct impact of each ton [ex. petroleum] is $44. The economic impact, direct and indirect, is $70 per ton.)

Total Louisiana Foreign Trade except petroleum: 136,000,000 tons
136,000,000 tons divided by 600 equals - 226,667 jobs
136,000,000 tons times $44 equals - $5,984,000,000
136,000,000 tons times $70 equals - $9,520,000,000

PORT OF NEW ORLEANS - 1980

TOTAL WATERBORNE TONNAGE: 167 million tons (1979) 1st in U.S.A.

VESSEL ARRIVALS: 5100

VALUE OF FOREIGN TRADE: 15 billion dollars

U.S. CUSTOM COLLECTIONS: 171 million dollars

CONTAINERS: 280,000 (20' equivalents)

PRINCIPAL EXPORTS(in millions of dollars):
Grain $1,809; Machinery $1,476; Oilseeds $488; Animal Feeds $427; Nonferrous Metals $405; Organic Chemicals $374; Oils and Fats $329; Metal ores and scrap $300; Iron and Steel Products $258; Coal $177.

PRINCIPAL IMPORTS (in millions of dollars):
Crude Petroleum $2,370; Coffee $992; Iron and Steel Products $809; Machinery $460; Metalliferous Ores and Scrap $315; Nonferrous Metals $293; Sugar $255; Crude Rubber $137; Meat and Products $128; Manufactures of Metal $102.

PRINCIPAL TRADING COUNTRIES (in millions of dollars):
Japan $993; United Kingdom $797; Fed. Rep. Germany $781; Netherlands $687; Nigeria $686; Brazil $615; Mexico $473; Italy $474; Algeria $424; Venezuela $388.

DIRECT ECONOMIC BENEFITS TO THE STATE OF LOUISIANA AND ITS CITIZENS:
$2,100,000,000.
(Tax Revenues $142 million; Employment 110,000)

PUBLIC FACILITIES

101 TOTAL BERTHS. 86 cargo (includes 4 full container; 12 RO/RO; 3 Lash/Seabee); 4 grain; 7 repair; 3 barge; 1 fleeting. Total wharf area 16,630,598 sq. ft.; Covered area 6,718,921 sq. ft.; Marshalling area 5,142,254 sq. ft.; 4 Paceco cranes @ 40 tons; heavy lift derricks to 600 tons; 12 ship repair yards.

2 GRAIN ELEVATORS: 240 million bushels - corn, wheat, soybean, etc.

PUBLIC BULK TERMINAL: Being converted to handle coal.
(4 million tons per year)

EXISTING AND PLANNED NEW COAL TERMINALS FOR THE MISSISSIPPI RIVER-BATON ROUGE TO THE GULF AND THE MISSISSIPPI RIVER-GULF OUTLET

NAME & LOCATION	1984 CAPACITY	1990 CAPACITY
Electro-Coal Transfer Terminal, Mile 55 Above Head of Passes (AHP) Existing	12 million tons(a)	30 million tons (b)
International Marine Terminals, Mile 57 AHP Existing	15 million tons(a)	25 million tons
Ryan-Walsh Stevedoring Bulk Terminal (MG-GO) Existing	4 million tons	4 million tons
Freeport Coal Terminal Co. Port Sulphur, Mile 39.2 AHP Under Design	4 million tons	8 million tons
International-Matex Tank Terminals Mile 46.6 AHP (West Bank) Under Design	12 million tons	15 million tons
NOLA Coal Loading Facility, Inc., Mile 47 AHP (East Bank) Under Design	2 million tons	3 million tons
Citrus Lands, Inc. Mile 56 AHP Under Design	6 million tons	6 million tons
Hunt Energy Corp. Mile 205 AHP (East Bank) Under Design	15 million tons	15 million tons
Gateway Terminals, Inc.** Mile 162 AHP (East Bank) Under Design	10 million tons	20 million tons
Miller Coal Systems, Inc. Terminal, Mile 174 AHP Under Design	10 million tons	20 million tons
River and Gulf Transportation* Co., St. Gabriel, Iberville, Mile 213 AHP, Note #4 Under Design	12 million tons	15 million tons
SUB TOTALS	102 million tons	161 million tons

	1984 CAPACITY	1990 CAPACITY
Mid-Stream Capability	24 million tons(c)	24 million tons
TOTALS	126 million tons	185 million tons

(a) by 1982
(b) by 1983
(c) by 1984
**Joint venture by Peabody Coal Co. and French Government being discussed.
*Subsidiary of U.S. Steel Corp.

November 5, 1981

U.S. COAL EXPORTS

1978 ACTUAL 40 MILLION TONS

1979 ACTUAL 66 MILLION TONS

1980 ACTUAL 90 MILLION TONS

2000 POTENTIAL *400 MILLION TONS

*300 MILLION TONS OF STEAM COAL AND 100 MILLION TONS OF METALLURGICAL COAL.

CURRENT STATUS OF CORPS OF ENGINEERS STUDY OF 55-FOOT CHANNEL FOR MISSISSIPPI RIVER - BATON ROUGE TO GULF

CURRENT SCHEDULE

DATES	STEPS
October 1980	Preliminary submission to Lower Mississippi Valley Division (LMVD) of draft report and environmental impact statement (EIS)
January 1981	Public meeting on draft report
June 1981	Submission of final district report to EIS to LMVD
June 1982	Final planning and completion of EIS to final public hearings and congressional authorization
December 1982	Completion of AE&D for channels and authorization for construction
June 1984	Preliminary 50-foot channel completed. Completion of AE&D for training works
December 1985	Completion of 55-foot channel (see notes 1 & 2)
June 1987	Completion of training works

Note #1: This is an ideal schedule -- all deadlines met in a timely manner with funding provided by the Congress -- without fast tracking, a more realistic schedule would probably be not sooner than December 1988.

Note #2: If the above schedule is to be achieved, Congress will need to authorize "fast track" legislation on a timely basis during the 1981 session.

March 30, 1981

RECOMMENDED FEATURES OF
DEEP DRAFT FAST-TRACK LEGISLATION

and

ECONOMIC ASSESSMENT OF 55' CHANNEL

RECOMMENDED FEATURES OF
DEEP DRAFT FAST-TRACK LEGISLATION

In connection with the deep draft legislation being considered by the committee at this hearing, I would like to briefly summarize certain features which the Port of New Orleans believes should be included in any legislation adopted. A more detailed discussion of these recommendations is included in this statement as Appendix "A".

1. The project documents submitted to Congress (including Environmental Impact Statements) should only have to demonstrate "consideration" of -- not "compliance" with -- the guidelines established under 404(b) of the Clean Water Act and 102 and 103 of the Marine Protection Research and Sanctuaries Act of 1972. If "compliance" is to be required, the fast-track legislation should mandate, or Sec. 404(b), Sec. 102, and Sec. 103 should be amended to require, the consideration of all relevant factors (economic, social, and political as well as environmental) in making the overall determination of acceptability under the guidelines.
2. A 30-60 day period should be provided for congressional disapproval of maintenance projects and navigation improvements.
3. The consequential effects of project authorization (arising from the absence of congressional disapproval and/or the execution of a cost-sharing agreement) should include "deemed compliance" with all procedural and substantive requirements applicable to the project under: (a) the fast-track legislation itself; (b) Sections 301, 402, and 404 of the Clean Water Act (including state-administered 402 and 404 programs); (c) Section 103 of the Marine Protection Research and Sanctuaries Act of 1972; (d) The Rivers and Harbors

Act; (e) the Fish and Wildlife Coordination Act; (f) the National Environmental Policy Act; (g) the Coastal Zone Management Act of 1972; and (h) applicable executive orders.

4. Memoranda of agreement among federal agencies to implement the act should provide for interagency cooperation only insofar as consistent with the act, with fixed periods for interagency review which cannot be extended.

5. The cost reimbursement obligation of non-federal interests should be: (a) limited to 40 percent for navigation improvements and 25 percent for operation and maintenance; (b) limited to a net federal cost (actual cost less benefits accruing on a national or regional basis, or in furtherance of a national interest); (c) limited so as to exclude work made necessary by natural disasters (e.g., force majeure); and (d) binding and payable only out of revenues collected by non-federal interests through a duty of tonnage on users of the funded navigation improvement or O&M (or other financial arrangements acceptable to the non-federal interest and the secretary), which duty shall not be subject to a "rate ceiling" in the act but shall be determined by relevant economic considerations subject to review by the secretary.

6. The sole and exclusive means of judicial review under the act should be in the United States Court of Appeals for the Circuit where the navigation improvement is located, with the review procedures to follow those set forth in "the Trans-Alaska Pipeline Act," 43 USCA Section 1652 (g).

7. Provision should be made for expedited handling of applications for federal permits for construction and operation of related marine cargo facilities at deep draft commercial ports.

ECONOMIC ASSESSMENT OF 55' CHANNEL

The Port of New Orleans had a study made in July, 1981, of the major bulk commodities, coal, grain, and petroleum products which would benefit from a 55-foot channel to the Gulf of Mexico as proposed in the Corps of Engineers' "Deep Draft Access to the Ports of New Orleans and Baton Rouge, Louisiana, Feasibility Report," dated December 1980.

The currently authorized channel depth is 40', but this is frequently less at the southwest pass because the training works have not been completely restored since Hurricane Betsy in 1965. Despite this obstacle, 40% of the grain exported by the U.S. uses the Mississippi River export terminals.

Investigation of the receiving ports for grain shows that 35% of the export grain would benefit from a deeper channel. The present export of grain from the Lower Mississippi Terminals is 70,000,000 tons in 1980, and is forecast to increase to 110,000,000 tons by the year 2000.

Ninety percent (90%) of the petroleum imports can now benefit from a deeper channel, but these imports are expected to decrease from 78,000,000 tons in 1980 to between 45,000,000 to 62,000,000 tons in the year 2000.

Over 50% of coal exports, a new commodity for this area, will benefit from a deeper channel. This projection is basedon the construction of deep water receiving terminals in importing nations. Based on 60% utilization of existing and under-construction coal terminals having 55-foot draft capability located on the Lower Mississippi River between Baton Rouge and the Gulf, the exports could exceed 100,000,000 tons per year by the early 1990's.

The composition of the world bulk fleet is rapidly changing. In 1964, almost 100% of the vessels were 60,000 DWT or less. These vessels, which require a 42-foot draft, now represent only 60% of the world fleet, and the percentage is expected to decline to only 30% by the year 2000. Over 3,000 vessels now serving the ports from Baton Rouge to the Gulf exceeded 60,000 DWT in 1981.

The daily operating cost of bulk vessels per ton is about 40¢ per 60,000 DWT vessel and increases rapidly per vessel of lesser tonnage. For vessels of 120,000 DWT, which can utilize a 55-foot channel, the daily operating cost is only 23¢ per ton. Based on the use of larger vessels, the savings in transportation cost could yearly exceed the initial cost of constructing a deeper channel by the year 1990.

A very significant point is that the export of grains is governed by the crop yield in the importing nations; the demand for coal is predictable and not subject to yearly fluctuations. For this reason, importers of coal will negotiate long-term contracts with the coal producers, the vessel charterers, and the export terminals. If the U.S. cannot provide ports which can accommodate efficient vessels, then this export market will be severely curtailed.

In assessing the impact of proposed user fees, one salient point was uncovered, and that was that although the cost savings could support the deeper channel project, the actual fee could not be limited to a maximum of one dollar per ton as is proposed in some bills.

SUMMARY OF PRESENTATION, AAPA AD HOC DREDGING TASK FORCE

NEW ORLEANS, LA, July 10, 1981

The following points were set forth:

1. The industry operates 191 major dredges - 101 cutterheads; 78 bucket; 11 hoppers and 1 dustpan. In addition, over 150 smaller units are operated.

2. Four large cutterheads, 10 hoppers and 1 dustpan have been built in the 1977-82 period in response to the opportunities afforded the industry under PL 95-269 and the Industry Capability Program. Cost of these units was in excess of over a quarter of a billion 1981 dollars.

3. Hopper bin capacity of the 1982 industry fleet will be 25% greater than that of the 1977 Corps figure. Due to technological improvements, capability is more than 25% greater.

4. In addition to handling the national requirements in the late 1970's the industry did considerable work in the Middle East. Operating in both areas, the industry operated at only slightly above 50% of capacity.

5. During the 1963-66 period, the industry did new work Corps projects having a 1981 dollar value over one billion dollars.

6. In response to a National Association of Dredging Contractors' questionnaire, the U.S. dredging industry has an annual capability as follows:

Cutterhead	453,000,000 c.y.
Bucket	164,000,000 c.y.
Hopper	83,000,000 c.y.
Dustpan	11,000,000 c.y.
Total	711,000,000 c.y.

7. Total average workload of industry dredges (including private and overseas work) was 350,400,000 c.y. for the 1978-80 period.

8. Based on Bryant Engineering Study, Corps O&M annual workload is:

Non-hopper	153,900,000 c.y.
Hopper	47,900,000 c.y.
Total	201,800,000 c.y.

9. NADC questionnaire results and Bryant Study showed average annual workload of non-Corps dredging is 167,000,000 cubic yards.

10. Dredging industry projects maximum concurrent deep port construction will be 1 - Galveston; 2 - Hampton Roads; 3 - Mobile; 4 - New Orleans; 5 - Baltimore. Based on three year duration of each, new work annual workload is 170,000,000 cubic yards.

11. Project additional annual maintenance generated from the deep ports is 59,900,000 cubic yards.

12. Total workload resulting is:

Normal Corps maintenance	201,800,000 c.y.
Non-Corps workload	167,000,000 c.y.
Deep Port Construction	170,000,000 c.y.
Deep Port Additional Maintenance	59,900,000 c.y.
Total	598,700,000 c.y.

13. Workload from (12) is 84.2% of industry capability of 711,000,000.

14. Additional capability of over 110,000,000 cubic yards could more than cover additional ports should situation occur.

15. Three new Corps hoppers will have annual capability of 20,000,000 cubic yards. Actual determination of Corps "minimum fleet" is yet to be decided.

16. American dredging industry is easily capable of handling a workload well in excess of the most optimistic projects of deep port construction.

DATA SHEET - United States Dredging Industry's Ability to Undertake Construction of Deep Draft Ports - Prepared for presentation to the American Association of Port Authorities - New Orleans, Louisiana on July 10, 1981

1. United States Industry Dredges:

Type		Size	Number
Cutterhead		42"	1
Cutterhead		36"	1
Cutterhead		30"	7
Cutterhead		25" - 27"	22
Cutterhead		22" - 24"	23
Cutterhead		18" - 21"	47
	Total		101
Bucket		20 cy - 22 cy	4
Bucket		15 cy - 18 cy	6
Bucket		12 cy - 14 cy	8
Bucket		8 cy - 10 cy	25
Bucket		5 cy - 7 cy	35
	Total		78
Hopper		12,000 cy	1
Hopper		6,400 cy - 8,800 cy	2
Hopper		3,600 cy - 4,000 cy	5
Hopper		1,200 cy - 1,500 cy	3
	Total		11
Dustpan		38"	1
	Total		1
TOTAL UNITS			191

2. Annual Capability of United States Industry Dredges:

Type	Annual Capability - Millions CY
Cutterhead	453
Bucket	164
Hopper	83
Dustpan	11
Total	711

3. Actual Workload of United States Industry Dredges

Year	Total Dug - Millions CY	Utilization
1978	369.1	57.8%
1979	344.7	51.4%
1980	336.9	50.3%
Average	350.4	53.1%

4. Total Corps of Engineers Maintenance Annual Workload:

Non-Hopper	153.9 million cy
Hopper	47.9 million cy
Total	201.8 million cy

5. Non-Corps Annual Workload:

Year	Quantity - Million CY
1978	163.5
1979	185.4
1980	151.5
Average	167.0

6. Galveston Deep Port Dredging - Millions CY:

	Total	Annual Workload (3 years)
Hopper	9.3	3.1
Non-Hopper	38.2	12.7
Total	47.5	15.8

7. Deep Draft Coal Ports - Millions CY

Port	Total			Annual Workload per Corps of Engineers Schedule			
	Hopper	Non-Hopper	Total	Years	Hopper	Non-Hopper	Total
Hampton Roads	48.9	51.2	100.1	7	7.0	7.3	14.3
Mobile	19.1	122.1	141.2	7	2.7	17.4	20.1
New Orleans	39.0	94.0	133.0	5	7.8	18.8	26.6
Baltimore	36.6	51.6	88.2	3	12.2	17.2	29.4
TOTAL	143.6	318.9	462.5		29.7	60.7	90.4
Annual Workload if all done in 3 years					47.9	106.3	154.2

8. Additional Annual Maintenance Workload due to Deep Ports

Port	Quantity - Millions CY
Galveston	0.5
New Orleans	57.1
Norfolk	1.2
Mobile	0.7
Baltimore	0.4
TOTAL	59.9

9. Total Dredging Workload during Deep Port Construction based on Corps of Engineers Time Schedule:

Type	Quantity - Millions CY
Normal Maintenance - Corps	201.8
Non-Corps Workload	167.0
Galveston	15.8
Coal Ports	90.4
Additional Maintenance - Deep Ports	59.9
TOTAL	534.9
Industry Capability	711.0
% Utilization	75.2%

10. Same as (9) Except All Deep Ports Done in 3 Years:

Total	598.7
Industry Capability	711.0
% Utilization	84.2%

ALTERNATIVE ON SLOPE PROTECTION FOR ARCTIC ISLANDS

by

Ing. Mauricio Porraz J.L.[1], Ing. Miguel Angel Bazua[2],

and Ing. Alejandro Solar S.[3]

1. INTRODUCTION

In recent years the search for oil and gas has moved into more hostile and remote environments. Artificial islands, as platforms for drilling are one such adaptation pioneered by Esso Resources Canada Ltd. for use on the coastal shelf of the Beaufort Sea in the Western Canadian Arctic.

The offshore environment of the Beaufort Sea is one of the harshest in the world. Ice which grows to seven feet thick covers the surface of the ocean for nine months of the year and in the Winter it is cold and dark. On the other hand, the long day light in the Summer helps construction of the islands, but interruption from Summer storms and delays due to late clearing of the ice are common.

Since 1972, more than twenty islands have been built in water ranging from 5 to 60 feet. Island design is influenced by materials and techniques available for construction as a function of location and season. In addition, the surface area required for drilling, and the freeboard required to provide stability against ice forces will control the island size and fill requirements. Beach slopes, which also

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influence the volume, are decided partly by construction techniques and foundation conditions; and partly by the need to protect the islands against erosion. Freeboard also has to be sufficient to prevent serious wave over-topping during the life of the island.

Soil conditions of the seabed in the wall location must be known to determine:

a) The bearing capacity and the settlement of the island,

b) The most suitable borrow area for silt or sand,

c) The stability of the shore protection and,

d) The resistance of the islands against ice forces.

Islands can be built either during the Winter by trucking gravel over the ice or in the short Arctic Summer (75 to 90 days) using hydraulic dredges.

Three factors influence the technical feasibility of the construction of artificial islands.

1. Sufficient quantity of suitable borrow material must be available at the site. This material may or may not be supplemented by hauling from a remote location.
2. Enough dredging power must be available on site to produce the material within the time allowed for construction.
3. There must be a period of relatively calm weather at the end of the construction season during which the island can be topped off.

A working area of 320 ft in diameter is needed to accommodate an Arctic rig, camp, drilling supplies and fuel tanks.

The type of ice action on artificial islands depends on the degree of ice movement. We can establish that there are three main interactions of ice with the structure: ice ride-up/pileup/overtopping. At shallow locations, the ice becomes stable soon after freeze-up and subsequent movements are small and slow. This results in limited ice action with few observable cracks and ridges.

At deeper locations large ice movements in the Fall generate extensive ice rubble around the islands. The rubble refreezes to form a solid ice annulus around the island. Ice action due to small cyclic ice movements occur on the outside of the refrozen rubble. Although the rubble is often grounded and provides additional sliding resistance, the total ice force on the island is generally increased because of the larger width of the active zone. When the ice approaches maximum thickness, its failure mode in the active zone is in crushing or flaking.

If necessary, ice defense slots can be cut in the relatively stable ice around the islands to reduce ice pressure. To date, no islands have suffered major damage from ice force.

To date no problems of ice ride-up islands in the Beaufort Sea have been experienced. Observations have suggested that when large ice movements occur in the Fall the ice is too thin to ride-up and forms the mentioned extensive rubble piles. On the other hand, during Summer break-up when the ice is thicker, it appears to be so deteriorated that it disintergrates on the beach without ride-up.

Despite these favorable experiences, the potential for an ice ride-up exists at more exposed locations, where, for instance, polar ice may invade.

Wave conditions in the Beaufort Sea can be considered as quite mild when compared with other ocean areas.

The wave height at any site is dependent on: the magnitude and duration of the winds, the distance over which the wind travels to generate waves or fetch, the water depth at the site, and any nearby coastal features which may change the wave before it impinges on the structure.

One of the nature's methods of dissipating wave energy is by inducing the waves to break over gradually-sloped beaches. On a gradually-sloped beach structure, the wave will reform and break again and may do this several times before finally running up the beach foreshore. Thus, the wave energy is dissipated gradually as it approaches the structure. Even though, consideration must be taken onto the fact that a too flat slope will increase ice ride-up. For that reason some islands have been constructed with sacrificial beach and other islands have been constructed using sand-bagged retaining berms, economizing on the fill material required.

Design practice, as exercised by the oil companies, is to design these temporary islands for the 1 in 10 years recurrence interval. Short duration occurrences of waves higher than the design wave will not destroy an island but will only dislodge some of the slope protection; which, if necessary, can be replaced immediately after storm.

Erosion mechanisms which have been identified to date are occasional ice scour of the slope protection and wave erosion of the island fill after ice scour has damaged sections of the slope protection. Wave erosion of the beaches can be controlled with different slope protection devices or with the loss of material of sacrificial beaches. This last solution is only valid in shallow waters.

To date, the performance of artificial islands under wave action during the short Summer months has been totally acceptable, and methods of slope protection have proven themselves excellent for their intended purpose of temporary protection.

Successful defense against wave erosion has been achieved by using large sand bags or by the placement of sacrificial beaches.

II. HYPOTHETICAL DESIGN OF AN ARTIFICIAL ISLAND

Basically, man-made islands consist of two main parts:

(a) The body of the islands, which must provide a sound base (with a minimum radius at the surface of 160 feet) for the drilling operations and obviously must have adequate bearing capacity.

(b) The slope protection construction, which must protect the slopes of the islands against itself, suffer minimum damage as a result of ice movement in the Winter and wave forces in the Summer season.

Basic Considerations

Diameter of working area	:	400 feet (122 m)
Depth of water in the site	:	40 feet (12 m)
Significant wave height	:	13 feet
Freeboard	:	15 feet
Slope protection	:	A defense and retaining system consisting of large sand textile containers (5'3" diameter by 60' length) of 66 ton, assembled in set of six sandtainers (Mexandtainers) weighing 397 ton each assembly.

Slope of ring retaining from sea bed to water level	:	0.6 in 1
Slope of protection over water level	:	1 in 1
Volume to fill	:	The volume to fill will be roughly of 312,000 yds^3
Material dredged	:	Considering the material lost due to waves and current (30%) to compaction and settlement (10%) the volume pumped by the dredge must be of 520,000 yds^3
Possible building time in Summer season	:	80 days
Material pumping rate (27" dredge)	:	20,000 yds^3/day

III. CONSTRUCTION PROCEDURE

1. Locate and Concentrate the Construction Equipment

Considering that the building season of the island is about 80 days and that there is no available construction equipment in the area, it will be necessary to plan a first stage in order to locate and concentrate the necessary equipment in the nearest port to the site, before final mobilization to Prudhoe Bay.

Necessary Main Equipment

Quantity	Description
1	Hydraulic dredge 27" equipped with 2,000 ft floating pipeline
2	100,000 U.S. Gallons fuel barge
4	2,500 ton deck cargo barge
1	2,000 H.P. ocean going tug

2	1.5 H.P. tug
4	600 H.P. tug (surveying)
1	330 H.P. dredge tender

2. Mobilization from Anchorage or Seattle to Prudhoe Bay

The mobilization time of the construction equipment from Anchorage to Prudhoe Bay can be estimated as follows:

Distance from Anchorage to Prudhoe Bay	2,000 nautical miles
Towing velocity of the convoy	6 knots
Estimated navigation time	14 days
Unforeseen	6 days
Total time	20 days

Considering that the landfast ice in Prudhoe Bay will be cleared on July 1st we must plan the mobilization of the equipment on June 10, in order to arrive at Prudhoe Bay on June 30.

3. Preparing the Dredge, the Floating Line, etc.

From July 1st to July 9 the crew of the dredge will work checking the machinery and the auxiliary equipment, preparing the dredge for operation, filling up the fuel tanks and assembling the floating pipeline.

4. Surveying Work

At the same time, July 1st to July 9, the surveying team will work in locating the island's center, collecting depth and position information from echosounder and position system, locating anchors and buoys at the different working circular paths, etc.

5. Mobilization to the Construction Site

The mobilization of the equipment to the construction site will take a few hours on July 10.

6. Construction of the Island

The philosophy of the solution presented in this paper for the construction of an artificial island in Beaufort Sea, is based on the construction of a spoil containing dike with a new and fast method, which permits reducing the material dredge to fill the island, thus saving in this way time and money. Besides, in this new and fast method of construction of dikes, the authors are proposing the utilization of interconnected Bolsacreto to protect the slope of the island against the wave action, using a new method of placing and filling up the bags without aid of divers.

IV. ALTERNATIVE PROPOSED

The alternative proposed considers the construction of a spoil containing dike in an alternate way with the filling of the island.

The dike consists of rows of large sand Textile Containers named Sandtainers*, assembled in a triangular pyramidal shape named Mexandtainers*. The Mexandtainers, formed by six sandtainers, are placed on seabed one after another all around the perimeter of the island, forming a retaining ring.

1. Construction of the First Ring of Mexandtainers Placed on the Seabed

The fill up (with sand) of the sandtainers and their assembly in order to make the mexandtainers will be done on deck of the 2,500 ton barges, over hydraulic dumping or inclining platforms. The fill up will be done by means of the dredge, whose discharge line will be equipped with a "Y" connection and with a butterfly valve, to deviate part of the flow to a manifold located over the deck of the barges. The mixture of

*All these conceptual designs for the construction methods and all new constructive modules have been protected by several patent pendings in Mexico and several other countries around the world.

sand and water flowing through the manifold is injected to the sandtainers by means of flexible hoses, filling 18 sandtainers, at the same time.

The sandtainers are placed on deck of the barges in pre-assembled way, with the three lower bags mounted over halves of corrugated sheet pipe, (like those used in culverts), with cable and bolt attachments like straps.

The filling of sandtainers start with the three lower bags of each mexandtainers. Once the lower bags are full, the crew disconnects the hoses and connects them with the three upper bags of each mexandtainer; placing halves of corrugated pipe between the bags and the pre-assembled strapping cables, making sure that the halves of the corrugated pipe will remain well placed around the bags during the fill up.

The philosophy of the utilization of halves of corrugated pipe around the sandtainers is to reinforce them and to diminish the friction between the fabric of the sandtainers and the metallic surface of the platform, and to eliminate the possibility of rupture of the fabric at the sliding of the mexandtainers.

Once the six mexandtainers are full, the barge is towed to the construction site and it is located over the circular path of the first row by means of electrical winches, which were connected to the pre-fixed anchors.

The first mexandtainer is dumped by one side of the barge and descends to the seabed; afterwards, the barge is moved with the winches and relocated to dump the second mexandtainer beside the first one, and, in this way, dump the six mexandtainers, watching not to unbalance the barge.

At the same time that the dredge is filling sandtainers, it is pumping material to the island center, raising the seabed level.

The first row of the spoil containing dike will be composed of 24 mexandtainers and the total time to fill up and place them will be estimated as follows:

a) Fill up time

Volume of sandtainer	:	36.90 m^3
Unit weight of sand	:	1.8 tons/m^3
Weight of each sandtainer	:	66.8 tons
Volume of mexandtainer	:	220 m^3
Weight of mexandtainer	:	396 tons

Considering a production of the dredge of 30 m^3/hr, deviated to fill up sandtainers, and a settlement rate of sand inside the bags of 75%, the total time for filling the first 18 sandtainers will be the following:

$$ts, \qquad ts = \frac{18 \times 36.8}{300 \times 0.75} = 2.944 \text{ hours}$$

$$ts_2 \quad = \quad ts_1 = 2.944 \text{ hours}$$

The time lost in making connections and maneuvers will be estimated in 2 hours. Then, the local time to fill up 6 mexandtainers will be estimated in:

Fill up time = 3 + 3 + 2 = 8 hours/barge

b) Movement and location time

The time utilized in the movement and location of the barges between the dredge and the construction site will be estimated in 4 hours.

c) Dumping and placing time

The time utilized in dumping and placing the mexandtainers will be estimated in 24 hours (4 hours per mexandtainer). Then, the time utilized in fill up, transport and placement of 6 mexandtainers will be estimated in 36 hours.

Therefore, the total time utilized in building the first row of mexandtainers will be of 3 days, from July 11 to July 13.

2. Filling the Island from Seabed to the Upper Level of the Mexandtainers in the First Row. (First stage)

The height of mexandtainer will be of 13' (approx. 4 m) and the diameter of this first row of mexandtainers will be of 141.2 m (463 ft); then, the volume to fill is as follows:

$$V_1 = \frac{3.1416 \times 141.2^2 \times 4}{4} = 62,636 \text{ m}^3$$

Considering the material lost due to waves and current (30%) and to compaction and settlement (10%) the volume to pump by the dredge will be the following:

$$Vd_1 = \frac{62.636}{0.6} = 104,393 \text{ m}^3$$

Pumping rate = 1000 m^3/hr

Operation time of dredge = 17 hours/day

Material pumped to the island during the fill up the mexandtainers = 700 m^3/hour = 11,900 m^3/day

= 11,900 x 3 = 35,700 m^3

The time to pump the rest of the material to fill the first stage of the island is as follows:

$$\text{Filling time} = \frac{104,000 - 35,700}{17,000} = 4 \text{ days}$$

Then, the total time to fill the first stage of the island will be of 7 days, from July 14 to July 20.

3. Construction of the Second Row of Mexandtainers*(23).

The time estimated for the construction of the second and third rows of mexandtainers, under the water level, will be considered equal to the time utilized for the first row.

Then, the construction of the second row of mexandtainers will be in 3 days, from July 21 to July 23.

4. Fill the Island

From the upper level of the first row of mexandtainers (4 m over seabed) to the upper level of the second row of mexandtainers (8 m over seabed) (second stage).

The filling time of the second stage of the island can be estimated as the time utilized for the first stage of seven days from July 24 to July 31.

5. The Construction of the Third Row of Mexandtainers (22).

Will be made in 3 days, from August 1st to August 3.

6. The Filling of the Third Stage of the Island will take a little less time than the filling of the stages 1 and 2, due to less losses by waves and currents, because the upper level of the dike will be over the water level.

Volume pumped during the filling of sandtainers = 11,900 x 3 x 0.6 = 21,420 m^3

Volume required to fill the third stage $= 50{,}511\ m^3$

Additional time to pump the volume remained $= \frac{50{,}511 - 21{,}420}{17{,}000} = 1.71$ days

Then the total time to fill the third stage of the island will be of 5 days, from August 4 to August 8.

7. The filling and placing of the last (4th) row of mexandtainers over the water level (12 m) will be easier than the anterior rows, the time will be estimated in 3 days, from August 9 to August 11.

8. The filling of the last stage of the island from the water level to the working level (freeboard) will be faster, due to no losses of material by waves and currents, only there are losses by compaction and settlement (10%).

Volume required to fill the fourth stage $= 53{,}423\ m^3$

Production of dredge $= 17{,}000\ m^3/day$

Time to fill the fourth stage $= \frac{43{,}423}{17{,}000 \times 0.9} = 3.5$ days

Then the filling of the fourth stage of the island (freeboard) will be made in 4 days, from August 12 to August 15.

V. SLOPE PROTECTION

1st Alternative

1. Philosophy for In-situ Filling Bolsacreto Elements

The slope protection against the wave and ice action will be achieved using Bolsacreto BC-containers* and it will be placed by a new method without aid of divers.

The armor will be built from 4 m (12') under water level to the upper level of the freeboard slope and the total area to cover will be of 3,964 m^2.

First example: the dimensions of Bolsacreto are 1 m width, 3 m length and 0.5 m height, containing 1.5 m^3 of concrete and covering 1.5 m^2 of armor each one. Therefore, it will be necessary to place 2,642 Bolsacretos in the armor.

These Bolsacretos will be interconnected and anchored with nylon cables in order to give them more stability and more resistance against the wave and ice forces.

The berm over the freeboard of the island will be protected with standard interconnected Bolsacreto* and it will be filled in place. The Bolsacreto on berm will be in the amount of 1,150 making a dike of 1.5 m high to eliminate the possibility of ice ride-up onto the island.

The Bolsacreto* on armor will be placed using guidelines anchored from the third row of mexandtainers at 4 m (12') under water level. These guidelines will be bolted to the halves of steel corrugated pipe around the mexandtainers.

2. Construction Procedure

Necessary main equipment

QTY	DESCRIPTION
1	Concrete mixing plant of 20 m^3/hr capacity
3	Concrete pump of 10 m^3/hour capacity
1	Lot of interconnection and distribution pipe, from the mixing plant to the pumps and from the pumps to the job sites (Manifolds).

2 25 ton hydraulic cranes

2 Tilting ramps

The construction of the armor starts with the installation of the concrete mixing plant and concrete pumps. At the same time, the tilting ramps and cranes are placed on the site.

The duration of these activities will be estimated in seven days, from August 15 to August 21.

Before the cranes are fixed, the crew inserts 19 empty bags in the guidelines and then, the crane operator tenses the guidelines.

These special bags will be filled with concrete over a tilting platform using a flexible hose connected to a manifold. Once the bag is full, the ramp is unlocked and tilting will dump the Bolsa-creto* sliding and descending it through the guidelines to the bottom and finally resting over a berm on the second level of filling; (thereafter) the crew will place another empty bag and proceed to fill it with concrete from the manifold. Previously all bags (19) per line were inserted in the guidelines and were hung from the crane's boom.

The construction time of the armor will be estimated as follows:

Time to place the empty bags, tense the guidelines, locate the tilting platform, etc...	= 4 hours/line
Time to fill up the 19 bags per line, considering a 10 minutes filling time, 10 minutes to place the empty bag and make the hose connection and 35% for unforeseen, per bag	= 8 hours/line
	12 hours/line

The total lines of Bolsacreto*to build will be in the amount of 139 and considering two teams starting in the same point and working in different ways around the island with 76 Bolsacretos filled per day (114 m^3 of concrete/day), the construction of the armor will be made in 35 days from August 22 to September 25.

At the same time, another team will build the dike over the berm; this activity is considered easier than the construction of the armor, building will be necessary to start 2 days after the armor, because there is not enough room to work in both activities. After two days, there will be 24 m of armor finished and 24 m of berm ready to work.

The filling rate of Bolsacreto*over the berm will be of 33 pieces per day (50 m^3/day), ending this activity on September 27.

The dismantling of mixing plant and pumps will take 3 days and the surface of the island will be clean and ready to rig up the drilling rig on September 30, after 82 days of construction activities.

2nd Alternative

1. Philosophy of In-Situ Filled and Placed Alambroca Mattresses with Polymer Georid Redlon-Tensar.

A second alternative for slope protection against wave and ice action is the use of gravel and red clay filled-in Geogrid Polymer Mattresses Alambroca.

These Alambroca Mattresses would be designed in a trapezoid form in such a way to cover the slope completely and with the minimum overlapping, due to the difference of ratios from the bottom base of the island and the surface base.

They would have an approximate 0.5 m thickness controlled by intermittent tensors placed within the two flat sheets of the mattress, and can be constructed of various dimensions ranging from 6 m to 18 m at the major base.

2. Construction Procedure

Before the construction of the armor is started, it will be necessary to build the dike over the berm. This will be conformed by three sandtainers placed in a single layer constituting a 4.8 m wide, 1.6 m high barrier. The filling of the sandtainers by means of the dredge will take around 3 days, from August 15 to August 17.

The main equipment, additional to that in hand, necessary to build the Alambroca Redlon armor is the following:

QTY	DESCRIPTION
1	1000 ton cargo barge with high structure and belt conveyor to fill the Alambroca Redlon mattress.
2	1000 ton cargo barge to transport the filling material from the coast to the island, furnished with a 2 cy clamshell machine.
1	300 ton floating crane with 120' boom.
1	2 cy dragline crawler machine
5	8 cy dump trucks

The cargo barges will be loaded at the coast with 500 m^3 of gravel and red clay (available in great quantities at the neighborhood of the working place) using the dragline and the dump trucks; or a portable belt conveyor. The loading time is estimated in 12 hours per barge.

Once the barge is loaded, a tug boat will drag the barge to the island, spending 6 hours in the journey.

The loaded barge is located beside the filling barge, and the material is unloaded with the clamshell into the hopper of the conveyor; which will carry the material up to the mouth of the Alambroca Redlon* mattress, hanging vertical from the 15 m structure over the deck of the barge.

The manufacturing products of the Alambroca Redlon* mattress will be: GEOTEXTILE FIJASOL 325/GEOGRID REDLON CE-131, reinforced with 3/8" galvanized steel cables in the shape of a bag; strong enough to hold a 150 ton weight and close enough not to allow the loss of filling material.

These bags will be hung from a special steel structure located at the end of the barge, the filling material (gravel and red clay) is dumped inside the bags while cement is added; in order to obtain a soil-cement composure, giving the mattress extra resistance.

Once the Alambroca Redlon* mattress is full (with 28 m long, 6 m wide, and 5 m thick dimensions, and weighing 150 tons) a 300 ton floating crane will lift it from the steel structure and gently rest it over the slope of the island: from the bottom to the edge of the berm, covering all the sandtainers and forming a strip of the armor.

In order to eliminate the possible sliding of the Alambroca Redlon* mattress, by the wave and ice forces action, all of them will be anchored to the top of the island with screw type anchors.

To build the complete armor around the islands' perimeter, 81 mattresses will be needed. If we take into consideration that 6 Alambroca Redlon mattresses can be filled and placed in one day; the time required to build the armor will be of 14 days. Adding 7 days of unforeseen delays, the total time of 21 days will be from August 18 to September 7.

Ending the armor's construction, will leave the island ready to rig up the drilling rig on September 8, after 60 days of construction activities.

Figure I shows a profile of the filling barge. The clamshell device aboard the loading barge drops the gravel and clay into the hopper (1) and the conveyor (2) transports the material to the mattress' mouth (3).

Figure II depicts the maneuvering of the barges. Barge (A) has lifted the mattress from the structure at barge (B) by means of the crane (CR) and lowers it gently besides the rest of the mattresses previously filled. Note loading barges (C) and (D) one of them still on position and the other one being tugged ashore for reloading.

This system is quite attractive and allows the possibility of using several barges at the same time and leave free the dry area in the artificial island to other useful and necessary works such as building the freeboard and deflecting structures, also the working time necessary to protect the slope will be minimized as required by the Arctic conditions.

In conclusion, the in-situ filling techniques to build the retaining dike to reduce the necessary dredge material and the alternative solutions for the slope protection with diverless techniques are feasible and can be very economic, practical and simple to execute Beaufort Sea marine operations.

VI. ESTIMATED COST OF CONSTRUCTION OF AN ARTIFICIAL ISLAND IN THE BEAUFORT SEA: Analysis of Costs - Alternative I

1. Rental Equipment
2. Materials
3. Operating Costs
4. Labor
5. Overhead
6. Unforeseen

Estimation of Costs

1. Rental Equipment

a) 27" hydraulic dredge with 2000' floating pipeline fuel barge and dredge tender without crew.

We will consider the rental rate in the amount of 6% monthly of the price of a new dredge. Considering the price of a new 27" hydraulic dredge of $4,000,000, the rental rates will be the following:

Rental rate = 4,000,000 x 0.06 = $240,000/month

The total time of utilization of the dredge was estimated in 5 months, because it will be available from May 1st to September 30th.

Dredge rental cost: $1,200,000

b) Tugs and barges.

The rental rates of tugs were estimated including labor costs, maintenance and supplies; only the fuel and lubricants shall be supplied by the oil company.

QTY	DESCRIPTION	RENTAL RATES	TOTAL
1	2000 HP oceangoing tug	$7,000/day	$ 7,000/day
2	1500 HP tug	$5,000/day	$10,000/day

4	600 HP tug	$2,000/day	$ 8,000/day
4	2500 T cargo deck barge	$ 500/day	$ 2,000/day
		TOTAL	$27,000/day

Rental rate of tugs and barges = $27,000

Rental time for small tugs and barges was estimated in 153 days, from May 1st to September 30th and for the 2000 HP oceangoing tug was estimated in 110 days, from June 1st to September 18th.

Tugs and barges rental cost = $20,000 x 153 + 7,000 x 110

Tugs and barges rental cost = $3,060,000 + 770,000

Tugs and barges rental cost = $3,830,000

c) Construction equipment.

We did not consider that the construction equipment was rented, because this equipment will be utilized from August 21 to September 27, and for this ultimate date, all barges and tugs will be in Anchorage or Seattle ports. Then, this equipment must spend the winter in the island, and therefore, we will charge the total investment to the construction of two islands.

QTY	DESCRIPTION	UNIT PRICE	INVESTMENT
2	20 T Mobile crane	$200,000	$400,000
1	20 m^3/hour concrete mixing plant	150,000	150,000
2	10 m^3/hour concrete pump	60,000	120,000
1 lot	miscellaneous equipment	200,000	200,000
		TOTAL	$870,000

Construction equipment cost = $435,000/island

Summarizing:

RENTAL EQUIPMENT COSTS	
27" hydraulic dredge	$1,200,000
2000 HP oceangoing tug	770,000
Tugs and barges	3,060,000
Construction equipment	435,000
Rental Equipment Costs:	$5,465,000

2. Materials

a) Installation of hydraulic dumping ramps on deck cargo barges.

The weight of each ramp to lift a Mexandtainer of 400 T was estimated in 50 T. Considering a price for structural steel assembled of $1000/ton. The cost of the structure will be of $50,000 adding the cost of the six 100 T hydraulic cylinders and the power unit installed, the total cost of each ramp is estimated at $75,000. If each barge will have six ramps the total cost of adaptation of each barge will be of $450,000. We had considered three cargo barges. To fill and place the Mexandtainers, then the total cost of adaptation of the deck cargo barges will be of $1,350,000. In order to fill the sandtainers we had considered the installation of a pipe manifold with an estimated cost of $50,000 in each barge.

Adaptation cost of deck cargo barges = $1,500,000

b) Containers, cables, sliding plates, etc.

The Mexandtainers placed under water level will be filled and assembled over the dumping ramps on the cargo barges, they will be tied with steel cables and the sandtainers at the bottom will be filled over curved steel plates in order to diminish the friction between the fabric to the sandtainers and the ramp when slipping.

QTY	DESCRIPTION	UNIT PRICE	TOTAL
540	BR Sandtainers textile bags	$285	$153,900
14,000 ft	1/8" steel cable	$2.33/ft	31,220
1,400	1/8" cable clamps	$6.28/pc	8.792
621	3/16" x 66" x 20' curved	$382.50/pc	237,532
1 lot	miscellaneous materials	$15,000	15,000
		TOTAL	$446,444

Mexandtainers materials cost + = $446,444

c) Bolsacreto BC-Concretainers, cables, clamps, anchors, etc.

QTY	DESCRIPTION	UNIT PRICE	TOTAL
3,800 pc	Bolsacreto BC-Concretainers	$29.00	$110,200
23,000 ft	1" steel cable	2.50	57,500
1,700 pc	1" cable clamps	6.50	11,050
280 pc	screw type anchor	200.00	56,000
		TOTAL	$234,750

Slope protection materials = $234,750

d) Concrete.

Considering that the construction site in the Beaufort Sea is an isolated area, we will use for the cost estimation of the concrete the double quality of materials as cement, gravel and sand.

The concrete necessary to fill 3792 Bolsacretos will be in the amount of 5688 m^3 (7484 yds^3) but we must consider the double quantity, therefore, the materials to move will be the following:

CONCRETE	CEMENT	GRAVEL	SAND
11,376 m^3	64,000 sacks (100#)	10,000 m^3	6,000 m^3
15,000 yd^3	64,000 sacks (100#)	13,500 yd^3	8,684 yd^3

MATERIALS COST

QTY	DESCRIPTION	UNIT PRICE	TOTAL
64,000 sacks	cement sack	$4.00/sack	$256,000
13,500 yd^3	gravel	$10.00/$yd^3$	135,000
8,684 yd^3	sand	$10.00/$yd^3$	86,840
		TOTAL	$477,840

Concrete Materials Cost = $477,840

NOTE: The gravel and sand shall be drawn out of the area.

Summarizing:

1) Adaptation of cargo barges		$1,500,000
2) Mexandtainers materials		446,444
3) Slope protection materials		234,750
4) Concrete		477,840
	TOTAL	$2,659,034

Materials Costs = $2,659,034

3. Operating Costs

a. Maintenance (dredge)

b. Fuel and lubricants (dredge, tugs and equipment)

c. Supplies (dredge)

a) Maintenance.

The maintenance cost per year of the dredge will be estimated at 7.5% of the price of a new dredge.

Considering the price of a new 27" hydraulic dredge is $4,000,000 the maintenance cost will be as follows:

Maintenance cost = $300,000/year

Maintenance cost = $25,000/month

Operating time = $125,000/island

b) Fuel and lubricants.

1) Dredge. The average consumption of fuel for the dredge will be considered 0.4 pounds/HP/hour.

That total horsepower installed in a 27" hydraulic dredge will be estimated in 8,000 HP, then the consumption of fuel will be the following:

fuel consumption	=	0.4 x Hp x Fu
fuel consumption	=	0.4 x 8000 x 0.7 = 2240 pounds/hour
fuel consumption	=	1130 lts/hour = 300 gals/hour
fuel consumption	=	7,200 gals/day
operating time	=	35 days
consumption of fuel in the 27" hydraulic dredge	=	252,000 U.S. Gals.

2) Tugs. The horsepower installed in tugs will be estimated in 9400 HP and the utilization factor in 0.5.

fuel consumption	=	0.4 x 9,400 x 0.5 = 1880 pound/hour
fuel consumption	=	252 gals/hour = 6048 gals/day
operating time including mobilization and demobilization	=	86 days

fuel consumption of tugs = 520,128 gals

3) Construction equipment. The fuel consumption in the construction equipment was estimated in the amount of 200 gals/day.

operating time = 35 days

fuel consumption of construction equipment = 7,000 gals

Summarizing:

1)	fuel consumption in dredge	=	252,000 gallons
2)	fuel consumption in tugs	=	520,128 gallons
3)	fuel consumption in construction equipment	=	7,000 gallons
	TOTAL		779,128 gallons

Considering a unit price for fuel of $0.83/U.S. gallons, the cost of the fuel will be as follows:

Fuel cost = $646,676

The lubricants cost will be considered in the amount of 12% of the fuel cost.

Lubricants cost = $77,600

c) Supplies.

The supplies to the dredge, as cables, greases, etc., will be considered at 10% of the fuel cost.

Supplies cost = $64,668

Summarizing:

a)	Maintenance cost	=	$125,000
b)	Fuel and lubricants cost	=	724,276
c)	Supplies cost	=	64,668
	TOTAL		$913,944

Operating costs = $913,944

4. Labor.

In this concept of cost included the labor utilized in surveying, in dredging, filling and placing the mexandtainers and in filling, and placing the Bolsacretos.

a) Labor in Surveying.

The surveying will be conducted or performed with electronic equipment mounted onboard the 600 HP tugs.

The electronic equipment will be operated by 8 men different than the crews of the tugs.

QTY	POSITION	MONTHLY SALARIES	TOTAL
8 men	Surveying operation	$4,500	$36,000/mo

Payment time = 5 months

Surveying salaries cost = $180,000

Workman compensation 13% =	$23,400
Fringe benefits 8% =	14,400
Social security 5%	9,000
Unemployment payment 2.7%	4,860
Retirement and hospitalization 2.3%	4,140
	$55,800

Surveying labor cost = $235,800

b) Dredging Labor.

The crew of the dredge will be integrated by 3 shifts, working 12 hours per shift.

QTY	POSITION	MONTHLY SALARIES	TOTAL
1	Superintendent	$8,000	$ 8,000
1	Chief Engineer	7,500	7,500
1	Civil Engineer	6,000	6,000
1	Deck Captain	4,500	4,500
3	Lever man	7,500	22,500
3	Mate	4,000	12,000
9	Deckhands	3,600	32,400
3	Assistant Engineers	4,500	13,500
9	Oilers and Greasers	3,600	32,400
3	Welders	4,000	12,000
3	Boat Operator	3,600	10,800
6	Cooks and helpers	3,600	21,600
44 men			$178,700/mo

Workman compensation	13%	
Fringe Benefits	8%	
Social Security	5%	
Unemployment payments	2.7%	
Retirement and hospitalization	2.3%	
	31%	$ 55,397/mo
	TOTAL	$234,097/mo

Payment time = 5 months

Dredging labor cost = $1,170,485

c) Filling and Placing Mexandtainers labor.

The crews that will work on barges filling and placing mexandtainers will be the following:

QTY	POSITION	MONTHLY SALARIES	TOTAL
12	Dump foreman	$4,000	$ 48,000
20	Deck man	3,600	72,000
			$120,000

Workman compensation, fringe benefits, Social Security, etc. 31%	37,200
	$157,200

Payment time = 3 months

Filling and placing Mexandtainers labor cost = $471,600

d) Filling and Placing Bolsacretos.

The same men that filled and placed the Mexandtainers, will fill and place the Bolsacretos.

QTY	POSITION	MONTHLY SALARIES	TOTAL
2	Concrete plant operator	$4,000	$ 8,000
2	Loader operator	4,000	8,000
4	Concrete pump operator	4,000	16,000
8	Operator helper	3,600	28,800
4	Filling foreman	4,000	16,000
4	Crane operator	4,000	16,000
8	Hose keeper	3,600	28,800
			$120,000/mo

Workman compensation, fringe benefits, Social Security, etc. 31%		37,200/mo
	TOTAL	$157,200/mo

Payment time = 2 months

Filling and placing Bolsacreto labor cost = $314,400

Summarizing:

1)	Surveying labor cost	$ 235,800
2)	Dredging labor cost	1,170,485
3)	Mexandtainers labor cost	471,600
4)	Bolsacreto labor cost	314,400
	TOTAL	$2,192,285

e) Living Quarters and Meals.

All men will live aboard the dredge and tugs and we will consider that there will be enough berths for all people.

The meals and cleaners of the rooms will be estimated at $50/day/man except the crews of the tugs, since their salaries and meals are included in the rental rates.

Meals cost = 84 x 50 = $4,200/day

Average working time = 82 days

Meals cost = $344,400

Then, the total labor cost will be as follows:

Labor cost = $2,536,685

5. Overhead

The overhead cost will be considered as a percentage of the preceding costs and in the amount of 25%.

SUMMARY:

1.	Rent equipment cost	=	$ 5,465,000
2.	Materials costs	=	2,659,030
3.	Operating costs	=	913,944
4.	Labor cost	=	2,536,685
		TOTAL	$11,574,659

Overhead cost = $ 2,893,366.47

TOTAL = $14,468,323

6. Unforeseen

The unforeseen expenses will be estimated in the amount of 10% of the preceding costs.

In this concept are included the following costs:

a. Concrete mixing plant installation

b. Tilting platform fabrication

c. Concrete additives for cold weather

Unforeseen expenses = $1,446,832

Then, the estimated cost of construction of an artificial island in Beaufort Sea will be the following:

Estimated cost of construction of an artificial island = $15,915,155

VII. ALTERNATIVE II

Estimated cost of construction of 5 artificial islands in the Beaufort Sea.

Costs.

1. Fixed costs
   a) Depreciation
   b) Investment cost
   c) Maintenance
2. Materials
3. Operating cost
4. Labor
5. Overhead
6. Unforeseen

Estimation of Costs.

1. Fixed Costs
   a) Depreciation

In order to calculate the depreciation of the equipment used to build the 5 islands, we will estimate the total investment in dredge, tugs, barges, construction equipment and others.

A. Floating Equipment

QTY	DESCRIPTION	UNIT PRICE	TOTAL
1	27" new hydraulic dredge with floating pipeline, fuel barge and dredge tender	$4,000,000	$4,000,000
2	2250 Hp used ocean tug	1,600,000	1,600,000
3	1500 Hp used utility boat with 28 berths	400,000	800,000

4	600 Hp used tug	250,000	1,000,000
3	2500 T used deck cargo barge	750,000	2,250,000
1	15,000 bbls used fuel barge	550,000	550,000
3	Adaptation kit of deck barge with dumping ramp and manifold from 2 (a) Alternative I		1,500,000
			$11,700,000

B. Construction Equipment

QTY	DESCRIPTION	UNIT PRICE	TOTAL
2	20 T mobile crane	$ 200,000	$ 400,000
1	20 m^3/hour concrete plant	150,000	150,000
2	10 m^3/hour concrete pump	60,000	120,000
2	yd^3 front loader	50,000	100,000
2	tilting ramp for Bolsacreto	25,000	50,000
1 lot	miscellaneous equipment	50,000	50,000
		TOTAL	$ 870,000

Total investment in machinery and equipment = $12,570,000

Considering the construction of 5 islands and a rescue value of 20%, the charge per depreciation of the equipment will be the following:

$$\text{Depreciation cost} = \frac{\$12{,}570{,}000 \times 0.8}{5} = \$2{,}011{,}200$$

b) Investment cost.

In this concept will be included some interests, insurances and storage costs and will be considered in 20% of the average investment in 5 years.

$$\text{Average investment} = \frac{h + 1}{2h} = \frac{5 + 1}{10} = 0.6$$

Investment cost = 12,570,000 x 0.8 x 0.2 x 0.6

Investment cost = $1,206,720/island

c) Maintenance.

The maintenance cost per year of the equipment will be estimated in 7.5% of the price of new equipment.

Maintenance cost = 15,000,000 x 0.075 = $1,125,000/year

Operating time = 5 months/year

Maintenance cost = $93,750/month

Maintenance cost = $468,750/island

Summarizing:

a) Depreciation	$2,011,200
b) Investment cost	$1,206,720
c) Maintenance cost	$ 468,750
TOTAL	$3,686,670

FIXED COSTS = $3,686,670/island

2. Materials

a) Containers, cables, sliding plates, etc.

The mexandtainers placed under water level will be filled and assembled over the dumping ramps on the cargo barges. They will be tied with steel cables, and the sandtainers at the bottom of the mexandtainers will be filled over curved steel plates in order to diminish the friction between the fabric of the sandtainers and the ramp during the slipping.

QTY	DESCRIPTION	UNIT PRICE	TOTAL
540	sandtainer bags	$285	$153,900
14,000 ft	7/8" steel cable	$2.23/ft	31,220
1,400	7/8" cable clamps	$6.28/pc	8,795
621	3/16" x 66" x 20' curved steel plate	$382.50/pc	237,532
1 lot	miscellaneous materials	$15,000	15,000
		TOTAL	$446,444

Mexandtainers Materials Cost = $446,444/island

b) Bolsacreto bags, cables, clamps, anchors, etc.

QTY	DESCRIPTION	UNIT PRICE	TOTAL
3,800 pc	Bolsacreto bag	$29.00	$110,200
23,000 ft	1" steel cable	$2.50/ft	57,500
1,700 pc	1" cable clamps	$6.50	11,050
280 pc	screw type anchor	$200	56,000
		TOTAL	$234,750

Bolsacreto Materials Cost = $234,750/island

c) Concrete

Considering that the construction site in the Beaufort Sea is an isolated area, we will use for the estimation of cost of the concrete, the double quantity of materials as cement, gravel and sand.

The concrete necessary to fill 3792 Bolsacretos will be in the amount of 5688 m^3 (7484 yds^3) but we must consider the double quantity, therefore, the materials to move will be the following:

Concrete 2,485 PSI	Cement	Gravel	Sand
11,376 m^3	64,000 sacks (100 #)	10,000 m^3	6,600 m^3
15,000 yd^3	64,000 sacks (100 #)	13,500 yd^3	8,684 yd^3

MATERIAL COSTS

QTY	DESCRIPTION	UNIT PRICE	TOTAL
64,000 sacks	cement sack	$4.00/sack	$256,000
13,500 yd^3	gravel	$10.00/$yd^3$	135,000
8,684 yd^3	sand	$10.00/$yd^3$	86,840
1 lot	cold weather additives		22,160
		TOTAL	$500,000

Concrete Materials Cost = $500,000

NOTE: The gravel and sand shall be drawn out of the area.

Summarizing:

a) Mexandtainers materials	$ 446,444
b) Bolsacreto materials	234,750
c) Concrete	500,000
TOTAL	$1,181,194

MATERIALS COST = $1,181,194

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3. Operating Costs

   a. Fuel

   b. Lubricants

c. Supplies

a) Fuel.

1) Dredge. The average consumption of fuel in the dredge will be considered of 0.4 pound/Hp/hour.

The total horsepower installed in a 27" hydraulic dredge will be estimated in 8,000 Hp, then the consumption of fuel will be the following:

Fuel consumption	=	0.4 x Hp x Fu
Fu = utilization factor of the power	=	0.7
Fuel consumption	=	0.4 x 8000 x 0.7 = 2240 pounds/hour
Fuel consumption	=	1130 lts/hour = 300 gals/hour
Fuel consumption	=	7200 gallons/day
Operating time	=	35 days/island
Consumption of fuel in the 27" hydraulic dredge	=	252,000 gallons/island

2) Tugs. The horsepower installed in tugs will be estimated in 9400 Hp and the utilization factor of power in 0.5.

Fuel consumption	≐	0.4 x 9400 x 0.5 = 1880 pounds/hour
Fuel consumption	=	252 gallons/hour = 6048 gals/day
Operating time including mobilization and demobilization of the 2250 Hp tug	=	86 days/year
Fuel consumption of tugs	=	520,128 gallons/island

3) Construction equipment. The fuel consumption of the construction equipment was estimated in the amount of 200 gallons/day.

Operating time	=	35 days
Fuel consumption of construction equipment	=	7000 gallons/island

Summarizing:

1)	Fuel consumption in dredge	252,000 gallons
2)	Fuel consumption in tugs	520,128 gallons
3)	Fuel consumption in construction equipment	7,000 gallons
	TOTAL	779,128 gallons

Considering a unit price of fuel of $0.83/U.S. Gallons. The cost of the fuel will be as follows:

Fuel cost = $646,676/island

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b) Lubricants.

The supplies to the dredge and tugs, as cables, greases, etc., will be considered at 10% of the fuel cost.

Supplies cost = $64,668/island

Summarizing:

a) Fuel cost	=	$646,676
b) Lubricants cost	=	77,600
c) Supplies cost	=	64,668
TOTAL		$788,944

Operating costs = $788,944/island

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4. Labor

In this concept of cost will be included the salaries, fringe benefits, social security, and the meals for the crews of tugs, dredge, barges and equipment construction personnel.

a) Labor in Towing Service.

1) Salaries. The crews of the tugs will be integrated as follows:

(1) 2250 Hp Tug

QTY	POSITION	MONTHLY SALARY	TOTAL
1	Captain	$4,500	$ 4,500
1	Chief Engineer	4,000	4,000
1	Mate (pilot)	4,000	4,000
3	Deck man	3,600	10,800
2	Engineer	3,600	7,200
1	Cook	3,600	3,600
9 men			$34,100/mo

(2) 1500 Hp Utility Boats

QTY	POSITION	MONTHLY SALARY	TOTAL
2	Captain	$4,500	$ 9,000
2	Chief Engineer	4,000	8,000
2	Mate	4,000	8,000
6	Deck man	3,600	21,600
4	Engineers	3,600	14,400
2	Cook	3,600	7,200
18 men			$68,200/mo

(4) 600 Hp Tugs

QTY	DESCRIPTION	MONTHLY SALARY	TOTAL
4	Captain	$4,000	$16,000
4	Chief Engineer	4,000	16,000
8	Deck man	3,600	28,800
8	Engineer	3,600	28,800
24 men			$89,600/mo

Towing salaries	=	$191,900/mo
Workman compensation		13%
Fringe benefits		8%
Social Security		5%
Unemployment payment		2.7%
Retirement and hospitalization		2.3%
TOTAL	=	31%
Additional benefits	=	$ 59,489/mo
Towing service labor cost	=	$251,389/mo
Payment time	=	5 months (from May to September)
Towing service labor cost	=	$1,256,945/island

b. Labor in Surveying

The surveying will be performed with electronic equipment mounted onboard the 600 Hp tugs.

The electronic equipment will be operated by 8 men different from the crews of the tugs.

QTY	POSITION	MONTHLY SALARIES	TOTAL
8 men	Surveying operator	$ 4,500	$36,000/mo
	Surveying salaries =	$36,000	

Workman compensation	13%
Fringe benefits	8%
Social security	5%
Unemployment payment	2.7%
Retirement and hospitalization	2.3%
TOTAL	31% = $11,160/mo
Additional benefits	$11,160/mo
Surveying labor cost	47,160/mo
Payment time	5 months
Surveying labor cost	$235,800/island

c) Dredging Labor.

The crew of the dredge will be integrated by 3 shifts, working 12 hours per shift.

QTY	POSITION	MONTHLY SALARIES	TOTAL
1	Superintendent	$8,000	$ 8,000
1	Chief Engineer	7,500	7,500
1	Civil Engineer	6,000	6,000
1	Deck Captain	4,500	4,500
3	Lever man	7,500	22,500
3	Mate	4,000	12,000
9	Deck hands	3,600	32,400
3	Assistant Engineers	4,500	13,500
9	Oilers and greasers	3,600	32,400

QTY	POSITION	MONTHLY SALARIES	TOTAL
3	Welders	$4,000	12,000
3	Boat operators	3,600	10,800
6	Cook and helpers	3,600	21,600
44 men			$178,700/mo
	Workman compensation, fringe benefits, social security, etc.	31%	55,397/mo
		TOTAL	$234,097/mo

Payment time = 5 months

Dredging labor cost = $1,170,485/island

d) Filling and Placing Mexandtainers labor.

The crews that will work on barges filling and placing Mexandtainers will be the following:

QTY	POSITION	MONTHLY SALARIES	TOTAL
12	Dum Foreman	$4,000	$ 48,000
20	Deck man	3,600	72,000
32 men		TOTAL	$120,000/mo
	Workman compensation, fringe benefits, social security, etc.	31%	37,200/mo
		TOTAL	$157,200/mo

Payment time = 3 months

Filling and placing mexandtainers labor cost = $471,600/island

e) Filling and Placing Bolsacretos.

The same men that filled and placed the Mexandtainers, will fill and place the Bolsacretos.

QTY	POSITION	MONTHLY SALARIES	TOTAL
2	Concrete plant operator	$4,000	$ 8,000
2	Frontend Loader Operator	4,000	8,000
4	Concrete Pump Operator	3,600	14,400
4	Crane Operator	4,000	16,000
4	Filling Foreman	4,000	16,000
8	Operator Helper	3,600	28,800
8	Hose Keeper	3,600	28,800
32 men		Sub-Total	$120,000/mo
	Workman compensation, fringe benefits, social security, etc.	31%	37,200/mo
		TOTAL	$157,200

Payment time = 2 months

Filling and placing Bolsacreto labor cost = $314,400/island

f) Living Quarters and Meals.

All men will live aboard the dredge and tugs and we will consider that there will be enough berths for all people.

The meals and cleaners of the rooms will be estimated at $50/man/day.

Total men working in the construction of the island	=	135 men
Meals cost	=	135 x $50 = $6,570/day
Average working time	=	82 days
Meals cost	=	$553,500/island

Summarizing:

a)	Labor in towing service	$1,256,945
b)	Surveying labor	235,800
c)	Dredging labor	1,170,485
d)	Filling and placing mexandtainers labor	471,600
e)	Filling and placing Bolsacreto labor	314,400
f)	Living quarters and meals	553,500
	TOTAL	$4,002,730/island

Labor cost = $4,002,730/island

---

5. Overhead

The overhead cost will be considered as a percentage of the preceding costs and in the amount of 25%.

Summary:

1.	Fixed costs		$ 3,686,670
2.	Materials costs		1,181,194
3.	Operating costs		788,944
4.	Labor cost		4,002,730
		TOTAL	$ 9,659,545
	Overhead cost =		2,414,886
		TOTAL	$12,074,431

6. Unforeseen

The unforeseen expenses will be estimated in the amount of 10% of the preceding costs.

Unforeseen expenses = $1,207,434

Then the estimated cost of construction of 5 artificial islands in the Beaufort Sea will be the following:

Estimated cost of artificial island in the Beaufort Sea	=	$13,281,874

VIII. ALTERNATIVE III

Estimated cost of construction of 10 artificial islands in the Beaufort Sea.

Costs.

1. Fixed costs
   a. Depreciation
   b. Investment cost
   c. Maintenance
2. Materials
3. Operating cost
4. Labor
5. Overhead
6. Unforeseen

Estimation of Costs.

a. Depreciation

Considering the construction of 10 islands and a rescue value of 10%, the charge per depreciation of the equipment will be the following: (from 1(a) Alternative II)

$$\text{Depreciation cost} = \frac{\$12,570,000 \times 0.90}{10}$$

Depreciation cost = $1,131,300/island

b. Investment cost

Average investment = $\frac{10 \times 1}{20}$ = 0.55

Investment cost = 12,570,000 x 0.55 x 0.90 x 0.20

Investment cost = $1,244,430/island

c. Maintenance

The maintenance cost per year of the equipment will be estimated in 7.5% of the price of new equipment.

Maintenance cost = 15,000,000 x 0.075

Maintenance cost = 1,125,000/year

Operating time = 5 months/year

Maintenance cost = $93,750/month

Maintenance cost = $468,750/island

Summarizing:

a. Depreciation	=	$1,131,300
b. Investment cost	=	1,244,430
c. Maintenance cost	=	468,750
TOTAL		$2,844,480

Fixed costs = $2,844,480

2. Materials cost = $1,107,757/island

3. Operating cost = 788,944/island

4. Labor cost = 4,002,730/island

5. Overhead Cost

The unforeseen expenses will be estimated in the amount of 10% of the preceding costs.

Unforeseen cost	=	$1,102,169/island

1.	Fixed cost	=	$2,844,480
2.	Material cost	=	1,181,194
3.	Operating cost	=	788,944
4.	Labor cost	=	4,002,730
			$8,817,348
	Overhead cost		2,204,337
	Unforeseen cost		1,102,169
	TOTAL		$12,123,854

Estimated cost of construction of 10 artificial islands in the Beaufort Sea = $12,123,856/island

IX. ANALYSIS OF COSTS ASPECTS

10. Alternative I

Renting the main equipment as the program requires and keeping only the construction and concrete equipment that may be used in other islands or sold next Summer.

a)	Rental equipment	$5.5 million U.S.
b)	Materials cost	$2.6 million U.S.
c)	Operating costs	$0.9 million U.S.
d)	Labor costs	$2.5 million U.S.
		$11.5 million U.S.
	Plus Overhead 25%	14.4
	Plus Unforeseen 10%	15.8

11. ALTERNATIVE II (Buying the equipment to build 5 islands)

a)	Fixed costs	3.7
b)	Materials costs	1.1
c)	Operating costs	0.8
d)	Labor costs	4.0
		9.6
	Plus Overhead 25%	12.0
	Plus Unforeseen 10%	13.1

12. ALTERNATIVE III (Buying equipment to build 10 islands)

a)	Fixed costs	2.8
b)	Materials costs	1.1
c)	Operating costs	0.8
d)	Labor costs	4.0
		8.7
	Plus Overhead 25%	10.9
	Plus Unforeseen 10%	12.0

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Chari, T.R. and Peters, G.R., "Engineering challenges in the cold oceans," 1980.

BIOGRAPHICAL DATA

NAME: Mauricio Porraz J.L.

EDUCATION:

Civil Engineer, F.I. UNAM, 55-59
Post Graduate Studies in France, Holland, 63-64 & 68
10 courses related with Offshore Technology, Welding, Explosives and Marine Engineering, 65-67 & 74

PRESENT POSITION:

President of Control de Erosion, S.A.
President of Contratista Nacional de Control de Erosion, S.A.
President of Equipos y Tecnicas, S.A.
President of Organizacion Submarina Mexicana, S.A.
President of Estructuras Marinas de Concreto, S.A. de C.V.

OTHER EXPERIENCES:

Professor: F.I. UNAM
Professor: (CEC) UNAM
Professor: (CEP) IPN
Professor: IMCYC
Professor: CICM
Participated in 62 National and International Congresses.

TECHNICAL PAPERS:

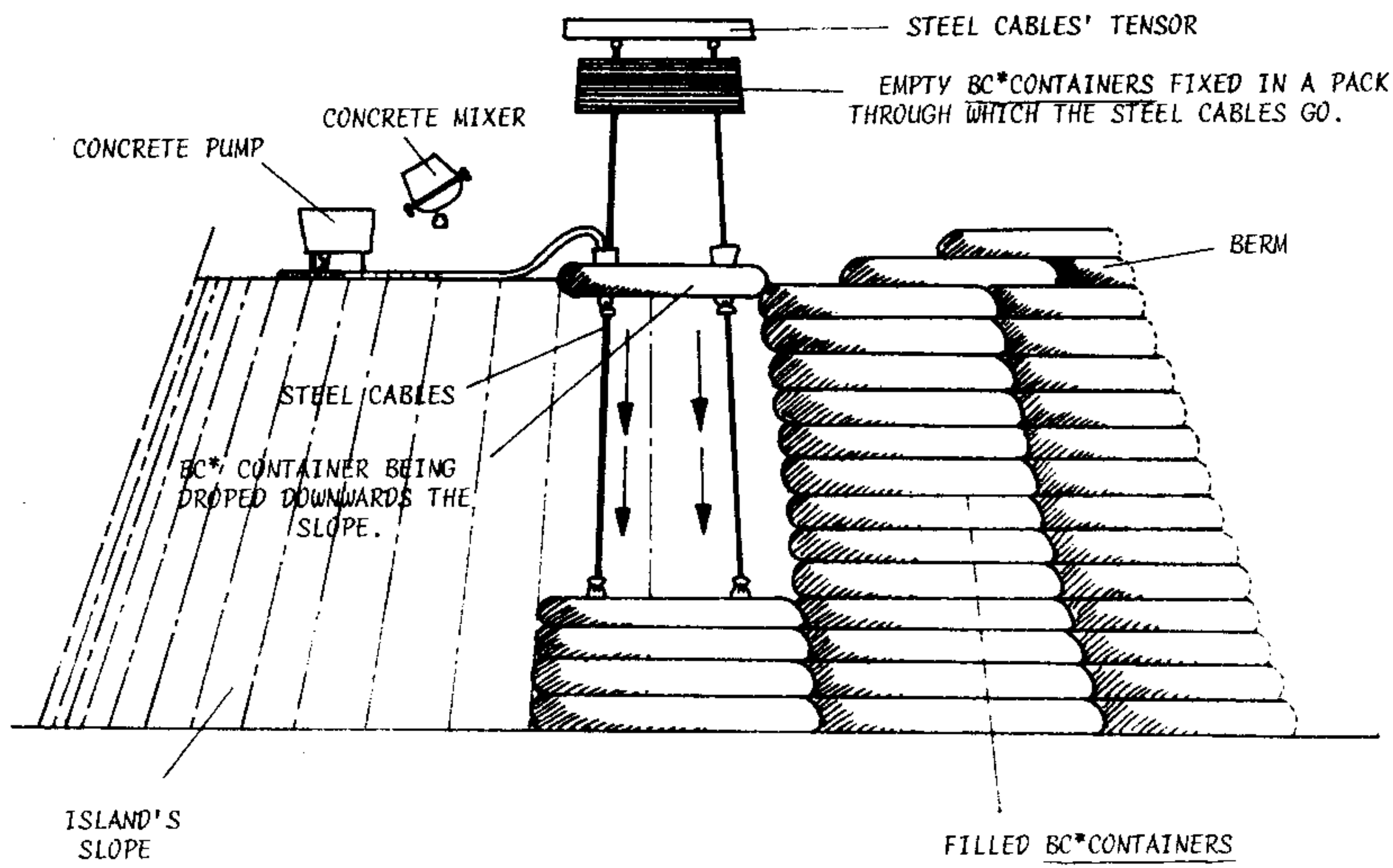
Presented 44 technical papers; one book on Engineering and Diving and Hold 51 Patents around the world on devices for Erosion Control.

NOTEWORTHY ACHIEVEMENTS:

Graduated with honors (Honorific Mention F.I. UNAM)
The Best Thesis of the Year 1963 CICM-FIAC
Chairman of the Board of the Mexican Committee on Engineering Ocean Resources.
Member of the Board Mexican Federation of Engineering Associations.
President of the Ocean Engineering Commission at the Panamerican Federation of Engineering Associations.
President of the Permanent Commission for Panamerican Ocean Engineering Congress.
President of the Marine Commission from the Mexican Society of Engineers.
Vice President of CEDAM.
Vice President of International Erosion Control Association.
Vice President of the Mexican Institute of Marine and Fisheries Studies Harbor.
Member of 33 Professional Societies, Mexican and International, including Honorary Member from the Royal Oceanographical Society.

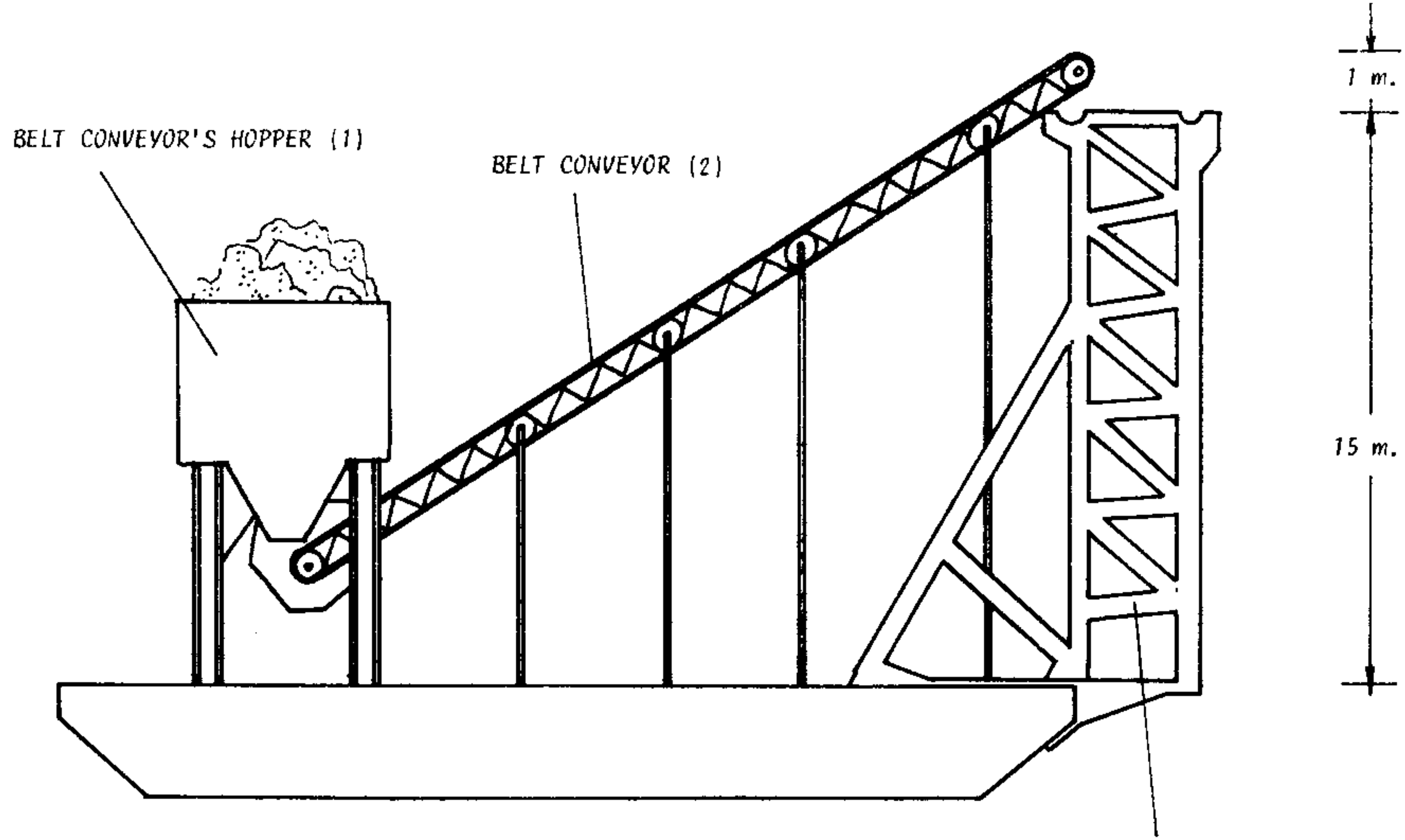
DISTINCTIONS:

Medal "Ordre du Merit" de la Recherche and Invention, Paris, France.
International Coastal Engineering Award, ASCE's Board approved its creation in October 1977.
Member of the National Academy of Engineering.



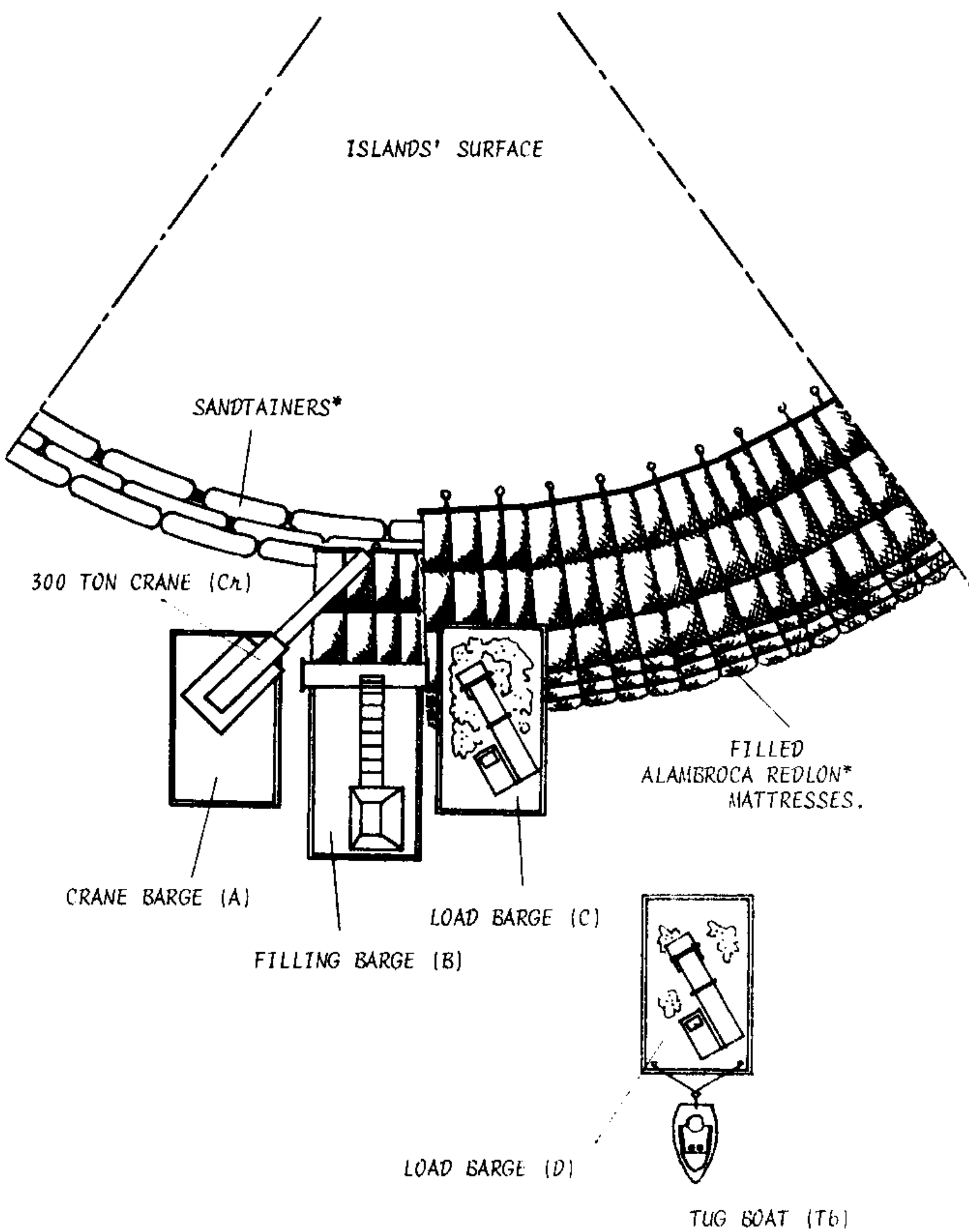
IN-SITU FILLED IN BOLSACRETO* CONTAINERS' FILLING AND PLACING METHOD.

FIGURE I



PROFILE OF THE 1,000 TON. FILLING BARGE FURNISHED WITH A 24" BELT CONVEYOR AND HOPPER.

FIGURE II



FILLING MANOUVERING PROCEDURE

GRAVEL ISLANDS FOR EXPLORATION AND PRODUCTION

by

Jerry Machemehl[1]

ABSTRACT*

As a result of the hydrocarbon potential in the Beaufort Sea a major exploration program is being conducted by U.S. oil companies. Since 1972 artificial islands have been constructed in the shallow waters (3 ft. to 40 ft.) of the Beaufort Sea for exploratory drilling. This seminar lecture describes the elements considered in the design of offshore islands.

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*Full manuscript was not received prior to publication.

WATER QUALITY ASPECTS OF DREDGED MATERIAL DISPOSAL IN THE GULF OF MEXICO NEAR GALVESTON, TEXAS

by

G. Fred Lee[1] and R. Anne Jones[1]

ABSTRACT

A study was conducted in the mid 1970s on the water quality impact of the disposal of sediments dredged from the Galveston Bay Entrance Channel, and Texas City Channel (Texas), in the open waters of the Gulf of Mexico near Galveston, Texas. The study focused on the heavy metals, chlorinated hydrocarbon pesticides and PCBs, nitrogen and phosphorus compounds, and other chemical constituents of the sediments, and their release from the sediments during laboratory elutriate tests and during dredged sediment disposal. These shipping channel sediments contained elevated concentrations of many contaminants, however, except for manganese and ammonia, little or no release to the watercolumn occurred upon open water disposal of the dredged sediments. The ammonia and manganese release was such that it would not

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adversely affect water quality because of the rapid dilution of the turbid plume arising from the disposal of the dredged sediment. It was found that the elutriate test developed by the Corps of Engineers and the US EPA for the purpose of predicting the release of contaminants upon open water disposal of hydraulically dredged sediment, reliably predicted the direction and approximate magnitude of contaminant release. It was concluded from these studies that the open water disposal of Galveston Bay Entrance Channel and Texas City Channel sediments would not cause a significant adverse impact on water quality upon open water dredged sediment disposal in the Gulf of Mexico.

INTRODUCTION

In the late 1960s and early 1970s considerable concern was voiced in various parts of the U.S. about the water quality significance of chemical contaminants in U.S. waterway sediments dredged for maintenance of navigation channel depth. In the early 1970s the U.S. Congress authorized the U.S. Army Corps of Engineers (CE) to conduct a $30 million, five-year program designed to evaluate various aspects of the environmental impact of dredged sediment disposal. A significant part of the total study program conducted as part of the Dredged Material Research Program (DMRP), was devoted to evaluating the significance of contaminants in waterway sediments that might be released to the water-column at an open water dredged material disposal site.

Early attempts by the US EPA and its predecessor organization to regulate dredged sediment disposal had been based on bulk sediment criteria, in which the total concentrations of selected contaminants in the sediment were used to judge the potential for the sediments to cause adverse impacts on the beneficial uses of the waters at the dredged material disposal site. Because of the weakness of the technical basis for this evaluation approach, as part of the DMRP the authors conducted several studies around the U.S. in fresh, estuarine, and marine waters to evaluate the hazard that dredged sediment-associated contaminants represented to aquatic life and other beneficial uses of water. These studies also included evaluation of the suitability of the elutriate test as a basis for assessing potential water quality problems associated with dredged sediment disposal.

Several sites on the Texas Gulf Coast (Figure 1) were selected by the CE to be included in the DMRP studies; most of them were concentrated in the Houston-Galveston area (Figure 2), which is one of the most heavily industrialized regions of the U.S. The U.S. Army Corps of Engineers maintains a dredged channel that extends from approximately 8 km out in the Gulf of Mexico to the Houston Ship Channel Turning Basin located near the center of the city of Houston. In addition, the CE maintains a number of side channels, one of the most important of which is the Texas City Channel which leads to an industrial complex in the Texas City, TX area. As a result of dredging in this area, deep-draft

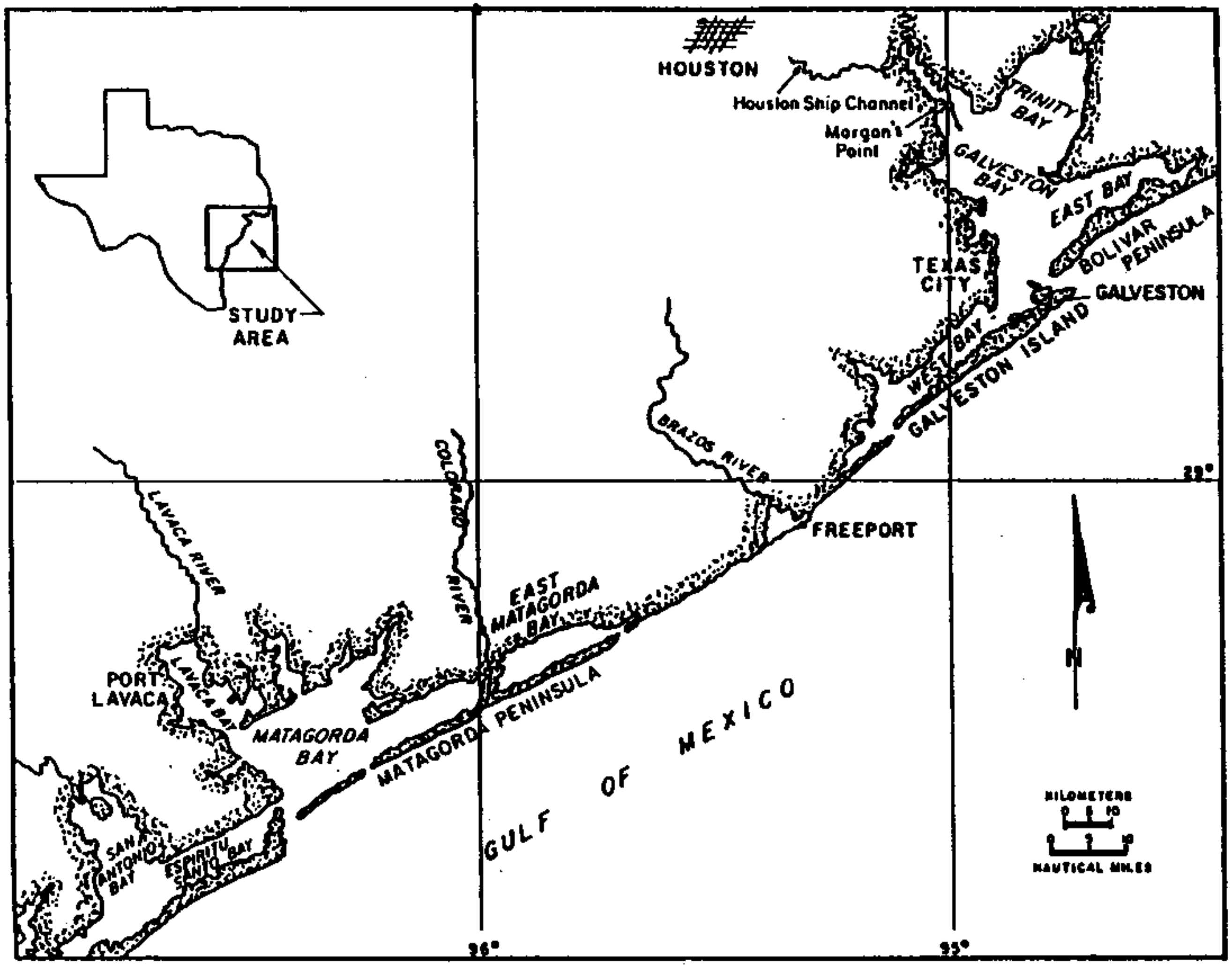

Figure 1. Texas Gulf Coast Study Areas

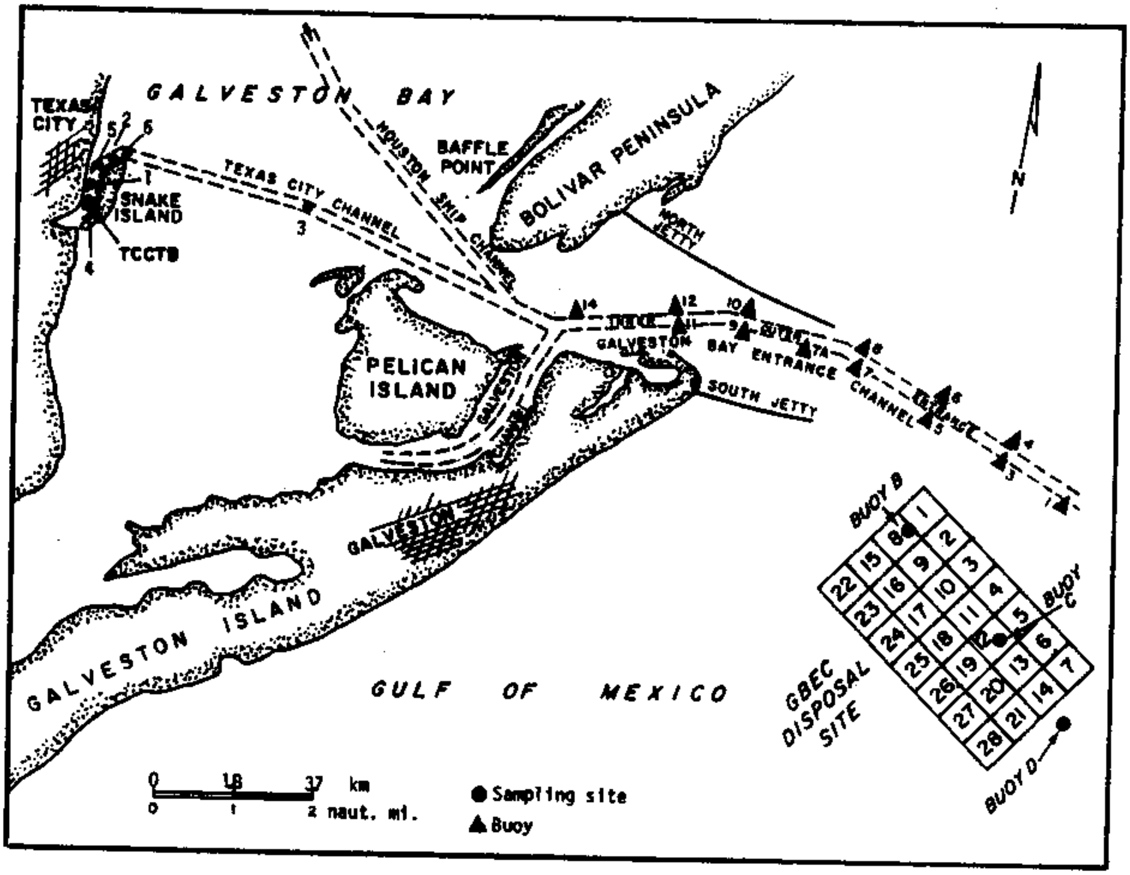

Figure 2. Galveston and Texas City Channel, Texas Study Areas

ships are able to load and unload cargo many kilometers inland.

Three major study areas in this region were included in the authors' investigation of the significance of dredged sediment-associated contaminants to disposal site water quality. One study area was the Houston Ship Channel (HSC), including several locations in the channel proper, near Morgan's Point which is where the HSC enters Galveston Bay, and within the HSC about 2 km into Galveston Bay beyond Morgan's Point. The second study area was the Texas City Channel (TCC) and the Texas City Channel Turning Basin (TCCTB) shown in Figure 2. The third and most intensive study was conducted in the Galveston Bay Entrance Channel (GBEC) and associated disposal area (GBEC Disposal Site). As shown in Figure 2, this channel leads all deep-draft ships through the nearshore waters of the Gulf of Mexico to the Houston-Galveston area ports.

Several other sites were also examined as part of the authors' Texas Gulf Coast DMRP studies. These included Cox Bay near Port Lavaca, TX (Figure 1), and Galveston Channel from which sediments were hydraulically dredged and disposed of on Pelican Island (Figure 2).

The results of the authors' Texas Gulf Coast DMRP studies as well as the details of the experimental and analytical procedures used, have been presented and discussed by Lee et al. (1977, 1978) and Jones and Lee (1978). The latter two reports also present the results of the authors' entire DMRP investigation. This paper will present a summary of the studies conducted on the Texas City Channel and the Galveston Bay Entrance Channel sediments.

METHODS

There were basically four phases of work associated with both of these two sites. The sediments and water being considered were evaluated for their bulk characteristics, including dry weight, oxygen demand, and content of selected contaminants. In the second phase, elutriate tests were run on the sediments. This test procedure was developed by the CE and US EPA as a tool to help estimate the release of chemical contaminants from hydraulically dredged sediment during open water disposal. The test is conducted by vigorously mixing one volume of sediment with four volumes of water (a 20% V/V test) for 30 min with compressed air (oxic test) or oxygen-free nitrogen gas (anoxic test). The mixture is allowed one hour of quiescent settling after which the supernatant is filtered through a 0.45 μm pore-size membrane filter, or for organic compound evaluation, is centrifuged. Other sediment : water ratios were also used during this study, usually 5% sediment by volume. The testing conditions are identified for each test in the tables and text as, for example "20% oxic" meaning that 20% sediment by volume was mixed with the site water under oxic conditions. Total elutriate (sediment plus water) volume used was 2 liters.

The third phase of work was bioassay in which 96 hr static bioassays with Palemonetes pugio (grass shrimp) were performed on the settled sediment/water mixture.

The fourth phase of the study involved monitoring dredged sediment disposal operations. The sampling vessel was positioned and anchored generally 30 to 300 m downcurrent from the established point at which the dredged sediment was to be dumped, such that the turbid plume would pass directly beneath the sampling vessel. Water samples were collected from the surface (1 m depth), mid-depth, and about 1 m above the sediment-water interface periodically during the 30 min or so prior to disposal to characterize background characteristics. Samples were collected in rapid succession during the disposal event as the plume of turbidity created by the disposal (the turbid plume) passed beneath the sampling vessel. Sampling generally continued for an hour or so after the dump event or until characteristics returned to their background conditions. The presence of the turbid plume was indicated by a decrease in percent light transmission of the water. The close proximity of the sampling vessel to the dumping point enabled a comparison to be made between the increase in turbidity and the release of contaminants. Parameters such as percent light transmission, dissolved oxygen, pH, and salinity were measured with submersible transducers. Water samples were collected using submersible pumps; sediment samples were collected with a Ponar grab sampler. All samples were stored at 4°C in the dark until used in elutriate tests.

Included in this phase of the study was limited sampling of the dredged material in the hopper bins of the hopper dredge during its travel from the dredging site to the disposal site.

GALVESTON BAY ENTRANCE CHANNEL STUDY

GBEC Sediment Elutriate Tests

Sample Sites. Sediment and water samples for elutriate tests were collected from Buoy 1 in the GBEC on June 11, 1975, and from GBEC Buoys 9 and 11 on April 12, 1975 (Figure 2). Elutriate tests were performed using sediment and water from the same location. They were also run on Buoy 9 and 11 sediments with disposal site water collected from a 2 m depth at GBEC Disposal Site Grid Square (GS) 8 on April 17, 1975.

General Sediment Characteristics and Oxygen Demand. One of the GBEC sediment samples was analyzed for oxidation-reduction (redox) potential (Eh) sulfide; percent dry weight and oxygen uptake were measured on all three (Table 1). The redox potential of the GBEC Buoy 1 sediment indicated that it was not strongly reducing, although it had a fairly substantial amount of sulfide present. Percent dry weight values were about 41% at GBEC Buoys 1 and 9 and 60% at GBEC Buoy 11.

The oxygen demand of the sediments was determined by

TABLE 1. GENERAL SEDIMENT CHARACTERISTICS AND OXYGEN UPTAKE: GALVESTON BAY ENTRANCE CHANNEL BUOYS 1, 9 AND 11 SEDIMENT SAMPLES

Sample Designation	Eh (mv)	Sulfide (mg/kg)		Percent Dry Wt		Oxygen Uptake First Hour	
		$\bar{X}$	SD*	$\bar{X}$	SD*	Per Cubic Meter ($g\ O_2$)	Per Gram Dry Wt ($mg\ O_2$)
GBEC Buoy 1	+101	446	57	40.7	2.7	3.6×10^2	0.67
GBEC Buoy 9	-	-	-	41.2	0.9	1.5×10^2	0.16
GBEC Buoy 11	-	-	-	60.0	0.1	80	0.14

* Mean and standard deviation calculated from triplicate analyses.

Dash (-) indicates not applicable.

placing a known volume of the sediment in a BOD bottle which contained water from the area and stoppering the bottle with a BOD bottle dissolved oxygen membrane electrode probe. Dissolved oxygen (DO) concentrations were measured over time while the sample was continuously being stirred by a magnetic stirrer inside the bottle. Figure 3 shows the DO data plotted as a function of time for the GBEC Buoy 1 sediment and water. The standard deviations between duplicate runs ranged from 0 to 0.2 g O_2/m^3 indicating good reproducibility. The fast component of the demand was found to be -0.023 mg O_2/l/min., while the slow component was -0.006 mg O_2/l/min. The overall first hour oxygen demand (Table 1) for this sediment was 3.6×10^2 g O_2/m^3, and 0.67 mg O_2/g dry wt.

The oxygen demand test was also run on samples from GBEC Buoys 9 and 11. The overall first hour oxygen uptake rates for Buoy 9 and 11 sediments were 1.5×10^2 g O_2/m^3 and 80 g O_2/m^3, respectively. While there was more than an order of magnitude difference in the per volume uptake rate, the uptake rates per gram dry weight were similar, 0.16 and 0.14 mg O_2/g dry wt for Buoys 9 and 11, respectively.

Heavy Metals. Table 2 lists the concentrations of the selected heavy metals measured in the sediments collected from Buoys 1, 9, and 11 in the GBEC. Manganese concentrations in all three sediments were high, ranging from 1170 mg/kg at Buoy 9 to 1430 mg/kg at Buoy 1. The zinc concentration was 300 mg/kg in the Buoy 1 sediment but decreased with

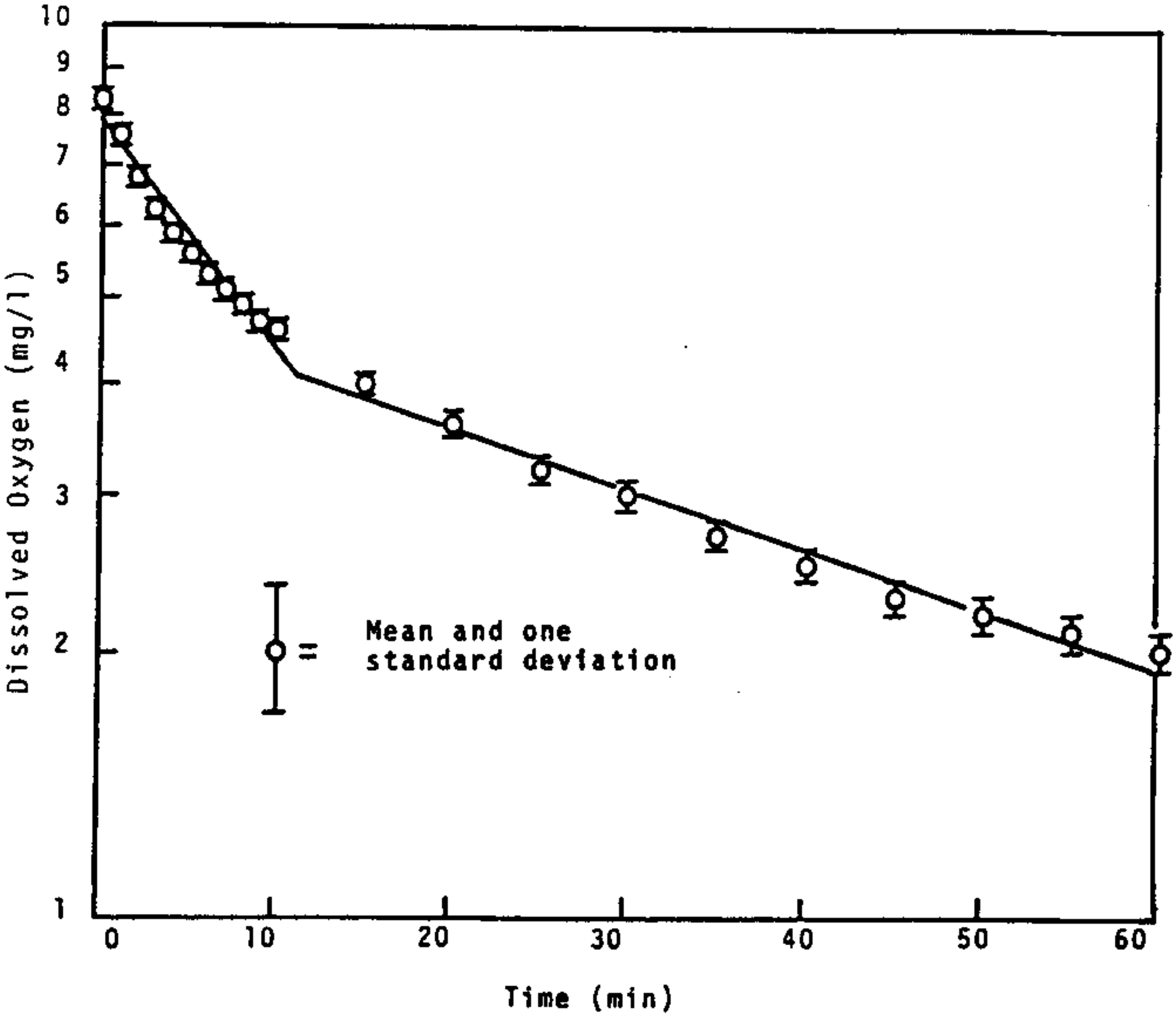

Figure 3. Oxygen Demand Test: Galveston Bay Entrance Channel Buoy 1 (Sample Size - 5 cc)

TABLE 2. TOTAL HEAVY METAL CONCENTRATIONS: GALVESTON BAY ENTRANCE CHANNEL SEDIMENT

(mg/kg dry wt)*

Sample Designation	Mn		Cd		Cr		Zn		Ni		Pb		Cu		Fe*	
	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD
Buoy 1	1430	20	<5	≈0	40	25	300	0	40	20	15	5	7.3	2.5	40	1.3
Buoy 9	1170	130	41	3	20	10	38	10	40	10	10	0	2.7	1.2	13	2.1
Buoy 11	1290	50	38	4	30	14	12	1	17	16	28	11	2.5	2.1	94	6.9

*Iron content given in g/kg.

Mean and standard deviation calculated from duplicate analyses.

decreasing distance from the industrialized areas of the channel to 38 and 12 mg/kg at Buoys 9 and 11, respectively (Figure 2). Cadmium concentrations were below the detection limit (5 mg/kg) at Buoy 1, and 41 and 38 mg/kg at Buoys 9 and 11, respectively. These last two concentrations are among the highest cadmium concentrations found in the many sediments analyzed in the Lee _et al_. (1978) studies. Iron concentrations were also high, ranging from 13 to 94 g/kg. Concentrations of nickel, chromium, lead, and copper were below levels generally considered indicative of heavy metal contamination of sediments.

Tables 3 and 4 present the soluble heavy metal concentrations in the elutriates of GBEC Buoy 1, 9, and 11 sediments and in the site waters used. Table 3 shows that Buoy 1 sediment released manganese in large amounts, 1165 μg/l and 370 μg/l, in the 5 and 20% tests, respectively. What appeared to be small releases of chromium, zinc, nickel, copper, iron, and mercury were not statistically significant at the 95% confidence level. There were no detectable changes in the lead or cadmium concentrations as a result of elutriation.

Elutriation of Buoy 9 and 11 sediments resulted in the release of manganese generally to 1000 to 5000 μg/l. There were indications of slight releases of chromium with both of these sediments. Small and probably insignificant uptake of copper, zinc, and mercury was also noted to result from elutriation. No significant changes in concentrations of

TABLE 3. SOLUBLE HEAVY METAL CONCENTRATIONS: GALVESTON BAY ENTRANCE CHANNEL BUOY 1 ELUTRIATE TESTS

(μg/l)

Sample	Mn		Cd		Cr		Zn		Ni		Pb		Cu		Fe		Hg	
Designation	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD
Site Water	2.6	0	1.2	0.1	1.6	0.0	3.5	1.1	2.9	0.4	<1.0	0	1.9	0.1	5	0.6	<0.006	0
5% Oxic	1165	11	1.2	0.2	2.3	0.3	9.2	4.4	4.6	1.1	<1.0	0	2.5	0.6	7	0.3	0.0018	0.06
20% Oxic	370	8	1.2	0.6	2.6	0.3	7.3	2.9	5.8	1.8	<1.0	0	2.9	0.4	103	4.6	0.026	0.02

Mean and standard deviation calculated from duplicate analyses.

TABLE 4. SOLUBLE HEAVY METAL CONCENTRATIONS: GALVESTON BAY ENTRANCE CHANNEL BUOYS 9 AND 11 ELUTRIATE TESTS

(μg/l)

Sample	Mn		Cd		Cr		Zn		Pb		Cu		Fe		Hg	
Designation	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD
GBEC Disposal Site Water	40.3	-	< 0.3	-	8.0	-	12.4	-	4.0	-	3.7	-	< 1	-	0.041	-
Buoy 9:																
Buoy Site Water	17.2	-	< 0.3	-	<2.0	-	12.4	-	2.1	-	2.5	-	< 1	-	0.069	-
5% Oxic*	1010	8	< 0.3	-	10.0	-	5.0	5.0	2.0	0	< 1.0	-	< 1	-	0.034	-
5% Oxic**	467	39	< 0.3	-	5.1	0	13.7	0.2	3.1	1.4	< 1.0	-	< 1	-	0.034	-
20% Oxic**	1780	101	< 0.3	-	5.9	1.1	15.4	1.1	1.1	1.3	< 1.0	-	< 1	-	0.028	-
Buoy 11:																
Buoy Site Water	48	-	< 0.3	-	<2.0	-	16.0	-	4.0	-	1.4	-	< 1	-	0.021	-
5% Oxic*	1730	3	< 0.3	-	6.0	3.0	8.3	1.0	5.0	1.4	< 1.0	-	< 1	-	0.020	-
5% Oxic**	1840	40	< 0.3	-	4.0	0	12.8	1.4	4.0	0	1.0	0.3	< 1	-	0.021	-
20% Oxic**	5050	58	< 0.3	-	4.5	0.7	13.7	1.2	5.0	1.4	< 1.0	-	< 1	-	0.014	-

Dash (-) indicates single analysis.
Mean and standard deviation calculated from duplicate analyses.
* Elutriated with buoy site water.
**Elutriated with disposal site water.

cadmium, lead, or iron were observed. No consistent measurable difference in heavy metal release occurred as a result of using disposal site as opposed to Buoy site water or as a result of 5% versus 20% sediment in the elutriate tests.

Nitrogen Compounds. Table 5 presents the total kjeldahl nitrogen (TKN) concentrations in the GBEC sediments; the concentrations were 152 mg N/kg in GBEC Buoy 9 sediment, and 6 and 9 times higher in Buoy 11 and 1 sediments, respectively.

Tables 6 and 7 show that organic N, nitrate, and ammonium* concentrations in GBEC Buoy 1 water were lower than concentrations in other GBEC or disposal site water samples. In general, elutriation resulted in no discernible change in the organic N or nitrate concentrations whereas ammonium was released in all tests. Ammonium concentrations in the elutriates, which ranged from 0.41 to 1.7 mg N/l, were 5 to 24 times the site water values; more ammonium was released in the 20% sediment tests than in the 5% tests. Percentagewise, elutriations of Buoy 1 sediments resulted in the greatest increase in ammonium concentrations, while elutriates of Buoy 11 sediment contained the highest concentrations.

Phosphorus Compounds. Total phosphorus concentrations

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*The word "ammonium" is used herein to mean the sum of the ionized and un-ionized ammonia concentrations.

TABLE 5. NITROGEN COMPOUND CONCENTRATIONS: GALVESTON BAY ENTRANCE CHANNEL SEDIMENTS

(mg N/kg dry wt)

Sample Designation	Total Kjeldahl Nitrogen	
	$\bar{X}$	SD
Buoy 1	1325	211
Buoy 9	152	1
Buoy 11	1013	92

Mean and standard deviation calculated from duplicate analyses.

TABLE 6. NITROGEN COMPOUND CONCENTRATIONS: GALVESTON BAY ENTRANCE CHANNEL BUOY 1 ELUTRIATE TESTS

(mg N/l)

Sample Designation	Organic N		Ammonium		Nitrate	
	$\bar{X}$	SD*	$\bar{X}$	SD*	$\bar{X}$	SD**
Site Water	0.17	0.04	<0.05	∿0	<0.04	∿0
5% Oxic	<0.05	∿0	0.41	0	<0.04	∿0
20% Oxic	<0.05	∿0	1.2	0.1	0.05	0

*Mean and standard deviation calculated from duplicate analyses.

**Mean and standard deviation calculated from triplicate analyses.

TABLE 7. NITROGEN COMPOUND CONCENTRATIONS: GALVESTON BAY ENTRANCE CHANNEL BUOYS 9 AND 11 ELUTRIATE TESTS

(mg N/1)

Sample	Organic N		Ammonium		Nitrate	
Designation	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD
Disposal Site Water	0.55	0.16	0.25	0	0.18	0.01
Buoy 9:						
Buoy Site Water	0.27	0.09	0.13	0.01	0.20	0.01
5% Oxic*	0.07	0.09	0.69	0.01	0.22	0.01
5% Oxic**	0.44	0.21	0.80	0.01	0.13	0.01
20% Oxic**	0.58	0.26	1.6	0.02	0.14	0.01
Buoy 11:						
Buoy Site Water	0.38	0.08	0.10	0	0.24	0.01
5% Oxic*	0.07	0.29	0.93	0.01	0.21	0.02
5% Oxic**	0.24	0.28	0.92	0	0.13	0.01
20% Oxic**	0.42	0.17	1.7	0.01	0.13	0.01

Organic N and ammonium mean and standard deviation calculated from duplicate analyses. Nitrate mean and standard deviation calculated from triplicate analyses. All samples were centrifuged.

*Elutriated with buoy site water.

**Elutriated with disposal site water.

of GBEC sediments are presented in Table 8. The greatest total phosphorus concentration found was at Buoy 11, the location farthest up the channel. Soluble orthophosphate concentrations found in elutriates from GBEC tests are presented in Table 9. The site waters from Buoys 1, 9, and 11 and the GBEC Disposal Site all appeared to have about the same concentration of soluble orthophosphate (generally between 0.005 and 0.013 mg P/l). Total phosphorus was greatest at Buoy 11, lowest at Buoy 1, and intermediate at Buoy 9 and the GBEC Disposal Site. The overall total phosphorus concentration range was small, however.

TABLE 8. TOTAL PHOSPHORUS CONCENTRATIONS: GALVESTON BAY ENTRANCE CHANNEL SEDIMENT

(mg P/kg dry weight)

Sample Designation	Total Phosphorus	
	$\bar{X}$	SD
Buoy 1	136	4
Buoy 9	111	2
Buoy 11	247	4

Mean and standard deviation calculated from triplicate analyses of one digested sample.

TABLE 9. SOLUBLE ORTHOPHOSPHATE AND TOTAL PHOSPHORUS CONCENTRATIONS: GALVESTON BAY ENTRANCE CHANNEL BUOYS 1, 9, AND 11 ELUTRIATE TESTS

(mg P/l)

Sample* Designation	Soluble Ortho P		Total Phosphorus	
	$\bar{X}$	SD	$\bar{X}$	SD
Disposal Site Water	0.009	0.002	0.05	0.001
Buoy 1:				
Buoy Site Water	< 0.02	0.001	< 0.02	0
5% Oxic*	< 0.02	0	0.03	0.01
20% Oxic*	0.064	0.004	0.07	0.01
Buoy 9:				
Buoy Site Water	0.005	0.001	0.05	0
5% Oxic*	0.024	0.001	0.10	0.01
5% Oxic**	0.023	0.002	0.09	0.01
20% Oxic**	0.023	0.002	0.09	0.01
Buoy 11:				
Buoy Site Water	0.013	0.002	0.09	0
5% Oxic*	0.028	0.002	0.09	0.01
5% Oxic**	0.029	0.001	0.10	0.01
20% Oxic**	0.038	0.003	0.11	0.01

Mean and standard deviation calculated from duplicate or triplicate analyses of one sample.

* Elutriated with buoy site water.

** Elutriated with disposal site water.

The Buoy 9 and 11 elutriate phosphorus concentrations appeared to be independent of the site water source. Buoy 9 elutriate tests all showed increases in soluble orthophosphate and total phosphorus; all elutriates (both 5 and 20%) contained about the same total P concentrations and about the same soluble ortho P concentrations. The 5% and 20% elutriates from GBEC Buoy 11 sediments showed slightly greater release of soluble orthophosphate than those from Buoy 9 but total P concentrations were the same. The total phosphorus concentration in the GBEC Buoy 11 5% oxic elutriate run with buoy site water was the same as that in the site water. The Buoy 11 20% elutriate had a greater concentration of soluble orthophosphate than the 5% elutriates; the total phosphorus concentration was the same as it had been in the 5% elutriates, however.

The soluble orthophosphate and total phosphorus concentrations in the Buoy 1 5% oxic elutriate was not discernibly different from those in the site water. Of all the GBEC elutriates, the 20% oxic Buoy 1 elutriate showed the greatest soluble orthophosphate release. The Buoy 1 site water had a lower total phosphorus concentration than any of the other sampling locations. The total phosphorus concentrations in these Buoy 1 elutriates were also lower than the others.

Organic Compounds. A 5% oxic elutriate test was run for evaluation of the release of selected organic compounds, using sediment and water taken from the GBEC at Buoy 1.

Aldrin, pp'DDT, op'DDE, dieldrin, lindane, and PCBs were detected in the sediment. Sixteen peaks present in the sediment chromatogram were also present in the water chromatogram. Except for dieldrin which was not detected in the water, the same group of chlorinated compounds detected in the sediment was also detected in the water. The profile of organic residues in the elutriate chromatograms showed fifteen of the peaks detected in the site water. The magnitudes of the peaks were lower than those of the site water, however, indicating a tendency for sorption of organic residues during elutriation.

The analytical results for this sediment, water, and elutriate are presented in Table 10. The pesticide levels in the sediment were low, usually below detection; the elutriate showed no detectable change in the concentrations of most of the pesticides measured, although the concentration of aldrin was lower in the elutriate than in the site water. The PCB concentration showed a 19% decrease in concentration after elutriation. The oil and grease levels in this sediment were low compared to others evaluated in the overall study.

Bioassays

Bioassays of the dredged sediment elutriates were conducted using *P. pugio* (grass shrimp). These tests involved placing the organisms in the elutriate waters after the one-hour settling time. The test systems thus still contained

TABLE 10. DATA FOR ORGANIC COMPOUNDS: GALVESTON BAY ENTRANCE CHANNEL BUOY 1, 5% OXIC ELUTRIATE TEST

Parameter	Sediment	Water	Elutriate
	(µg/kg)	(ng/l)	(ng/l)
<u>Chlorinated Hydrocarbons</u>:			
Aldrin	2.5	4.1	2.8
op'DDT	< 1.6	< 3.0	< 3.0
pp'DDT	< 2.2*	< 3.0*	< 3.0*
op'DDD	< 2.0	< 2.0	< 2.0
pp'DDD	< 2.0	< 2.0	< 2.0
op'DDE	2.2	5.2	5.2
pp'DDE	< 1.4	< 2.0	< 2.0
Dieldrin	< 0.8*	< 1.2	< 1.2*
Endosulfan I	< 0.9	< 1.2	< 1.2
Endosulfan II	< 3.4	< 4.4	< 4.4
Endrin	< 1.2	< 1.6	< 1.6
Heptachlor	< 0.3	< 0.4	< 0.4
Lindane	0.8	2.5	2.5
PCBs	192	210	170
<u>Other Organic Compounds</u>			
Oil and Grease	23.5 mg/kg	-	-
Total Inorganic Carbon	1.12%	-	-

*Compound indicated on two columns but below detection limit.

Dash (-) indicates not measured.

the settled as well as suspended sediment; therefore the contaminant release that occurred over the four-day testing period was also included in the bioassay test results. Results of the bioassay tests using P. pugio and GBEC Buoy 1 elutriates (5%, 10%, and 20% sediment) are presented in Table 11. Only a slight toxicity to P. pugio was found. One and two deaths occurred after 36 hrs in the two 20% sediment test; one death occurred in each of the 5% tests after 70 to 80 hrs exposure; no deaths occurred in the 10% test or in the controls.

Toxicity results of the Buoy 11 elutriate bioassays for P. pugio are presented in Table 12. These tests showed no toxicity to P. pugio in the 5 or 10% elutriates. There was one organism death after 24 hrs in one of the two 20% tests and another after 48 hrs in the other replicate.

GBEC Disposal Operations - Dump No. 4

Background. Lee et al. (1977) monitored seven disposal operations involving sediment dredged from the GBEC. This section summarizes the results of these studies, with results from one representative operation, GBEC Dump No. 4, presented. A complete description of the experimental procedures, dump nomenclature, and results are presented in Lee et al. (1977, 1978).

GBEC Dump No. 4 took place on August 29, 1975 at Buoy D in the Gulf of Mexico (Figure 2). Approximately 850 m^3

TABLE 11. RESPONSE OF P. PUGIO TO GALVESTON BAY ENTRANCE CHANNEL BUOY 1 BIOASSAY ELUTRIATES

Time (hr)	Control		Number of P. pugio Living at Varying Sediment Percentages					
			5%		10%		20%	
	A	B	A	B	A	B	A	B
0	10	10	10	10	10	10	10	10
12	10	10	10	10	10	10	10	10
24	10	10	10	10	10	10	10	10
36	10	10	10	10	10	10	9	8
48	10	10	10	10	10	10	9	8
60	10	10	10	10	10	10	9	8
72	10	10	9	10	10	10	9	8
84	10	10	9	9	10	10	9	8
96	10	10	9	9	10	10	9	8

A and B are replicates.

TABLE 12. RESPONSE OF *P. PUGIO* TO GALVESTON BAY ENTRANCE CHANNEL BUOY 11 BIOASSAY ELUTRIATES

Time (hr)	Control		Number of *P. pugio* Living at Varying Sediment Percentages					
			5%		10%		20%	
	A	B	A	B	A	B	A	B
0	10	10	10	10	10	10	10	10
12	10	10	10	10	10	10	10	10
24	10	10	10	10	10	10	9	10
36	10	10	10	10	10	10	9	10
48	10	10	10	10	10	10	9	9
60	10	10	10	10	10	10	9	9
72	10	10	10	10	10	10	9	9
84	10	10	10	10	10	10	9	9
96	10	10	10	10	10	10	9	9

A and B are replicates.

($1100 \ yd^3$) of sediment dredged between GBEC Buoys 1 and 3 was dumped during this hopper-dredge operation. The sampling vessel was positioned approximately 30 m down surface current from the established dumping point; surface water current was found to be about 0.5 knot (25 cm/sec). The dump event occurred at 1258 hrs.

Optical Properties. Figure 4 shows the passage of the turbid plume at the three depths monitored during Dump No. 4, as measured by percent light transmission. The turbid plume took 2.75, 3.25, and 2.25 min to reach the monitoring location at surface, mid-depth, and bottom, respectively. As in other dumps, turbidity was greatest near the bottom where the elevated turbidity conditions persisted for about 10 min. The Secchi depth was 2.5 m both prior to disposal and 37 min after Galveston Dump No. 4.

Dissolved Oxygen. Figure 5 shows that just after the turbid bottom plume arrived at the sampling vessel, there was a 1.7 mg/l decrease in the bottom water DO concentration to about 4.8 mg/l. Before the turbid bottom plume was past the vessel, DO concentrations were near pre-disposal levels; by the time it was past the sampling boat, the initial DO levels were re-established.

Heavy Metals. Table 13 presents the heavy metal concentrations at the disposal site during Dump No. 4. Although few pre-disposal samples were taken, concentrations of the heavy metals were in line with what had been found prior to

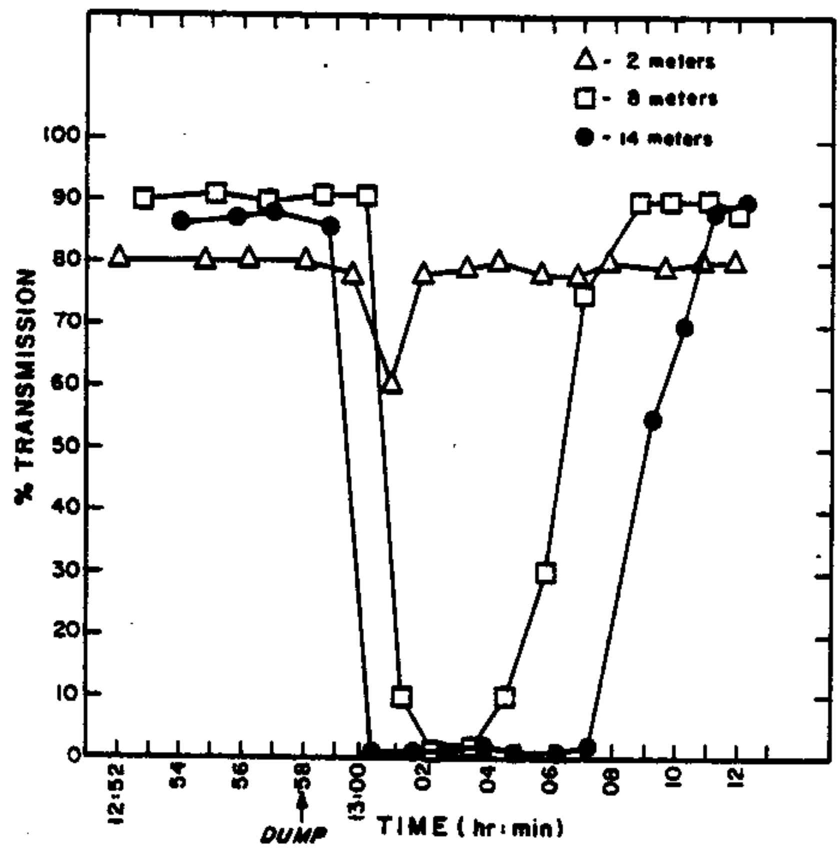

Figure 4. Percent Light Transmission: Galveston Dump No. 4

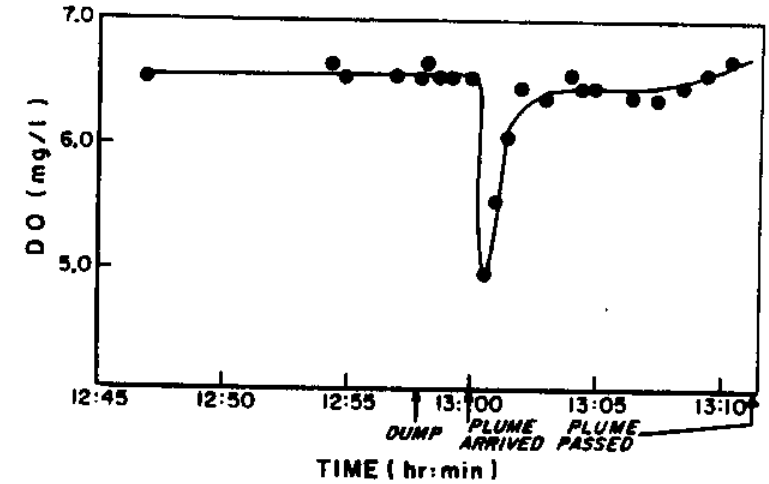

Figure 5. Dissolved Oxygen Concentrations: Galveston Dump No. 4 Near-Bottom Waters

disposal there the day before. No manganese release was observed in Galveston Dump No. 4; in fact, soluble manganese concentrations were about an order of magnitude lower with the passage of the turbid plume, possibly because of precipitation or sorption. Decreases in concentrations of soluble lead, zinc, cadmium, nickel, copper, and iron also apparently occurred with the passage of the turbid plume.

Four samples, two from 8 m and two from 14 m depths, showed increased mercury concentrations. These elevated concentrations, ranging from 0.010 to 0.048 µg/l (0.003 µg/l was observed prior to disposal), occurred while the turbid

TABLE 13. HEAVY METAL* CONCENTRATIONS: GALVESTON DUMP NO. 4

(µg/l)

Time of Collection (hr:min:sec)	Depth (m)	Mn SOL	Mn TOT	Cr SOL	Cr TOT	Cd SOL	Cd TOT	Ni SOL	Ni TOT	Pb SOL	Pb TOT	Zn SOL	Zn TOT	Cu SOL	Cu TOT	Fe SOL	Fe TOT	Hg SOL	Hg TOT	As SOL	As TOT
12:54:00	14	186	-	<2	<2	1.7	-	11.3	-	9.3	-	17.8	-	7.2	-	58	431	0.003	0.003	< 2	<2
				12:58 - Dump occurred																	
12:59:30	14	201	-	<2	<2	1.5	-	12.5	-	14.7	-	13.8	-	5.6	-	35	344	<0.001	0.003	< 2	<2
				13:00 - Surface turbid plume arrived at sampling station																	
13:00:30	1	172	-	<2	<2	1.5	-	10.0	-	12.7	-	12.8	-	8.3	-	18	46	<0.001	<0.001	< 2	<2
13:01:30	8	186	-	<2	<2	1.7	-	10.0	-	10.0	-	7.2	-	4.4	-	13	103	0.003	0.003	< 2	<2
				13:02 - Surface turbid plume passed sampling station																	
13:02:30	1	123	160	<2	<2	2.1	-	-	-	3.6	59.8	13.2	92.1	6.1	28.1	37	1100	0.003	0.039	< 2	< 2
13:03:00	8	105	-	<2	<2	<0.5	-	-	-	3.1	-	7.3	-	2.6	-	9	-	<0.001	0.023	< 2	< 2
13:03:30	8	-	-	-	<2	-	<0.5	-	2.4	-	-	-	-	-	7.2	-	480	<0.001	0.003	< 2	< 2
13:03:50	14	105	182	<2	<2	<0.5	<0.5	0.7	16.8	3.1	37.9	8.5	59.9	3.5	21.2	25	1700	0.003	0.016	< 2	< 2
13:04:15	8	88	111	<2	<2	<0.5	<0.5	6.6	-	2.0	88.0	9.6	30.9	3.5	12.7	13	900	0.016	-	< 2	< 2
13:04:30	14	-	147	<2	<2	<0.5	2.2	3.3	5.9	4.0	14.6	7.0	22.0	2.6	6.7	227	900	0.010	-	< 2	< 2
13:05:30	8	35	200	<2	<2	<0.5	<0.5	2.7	4.5	1.7	13.1	7.9	24.7	5.7	8.2	155	900	0.048	-	< 2	< 2
13:06:00	14	18	200	<2	<2	<0.5	<0.5	4.1	12.9	<1	26.3	2.5	44.2	2.8	15.2	<5	1100	0.023	0.023	< 2	< 2
13:06:30	1	14	126	<2	<2	0.7	-	3.6	-	<1	7.3	2.4	10.8	4.0	4.2	<5	-	0.006	0.010	< 2	< 2
13:07:15	14	25	200	<2	<2	0.6	-	2.8	14.0	<1	30.6	1.4	54.3	2.8	18.7	8	1100	0.006	0.016	< 2	< 2
13:09:45	14	56	165	<2	<2	0.7	-	3.6	7.7	<1	14.6	2.3	18.8	3.6	7.2	15	1000	0.006	0.006	< 2	< 2
13:11:30	8	29	108	<2	<2	<0.5	<0.5	2.8	-	<1	5.8	2.0	5.4	2.8	-	<5	22	0.006	0.026	< 2	< 2
13:12:15	1	40	133	<2	<2	<0.5	<0.5	4.1	-	<1	5.8	1.6	3.7	2.3	3.7	<5	55	0.003	0.023	< 2	< 2

*SOL and TOT represent soluble and total concentrations, respectively.

Dash (-) indicates not measured.

plume was still present at these depths, and appeared to persist for 4 to 6 min. Samples taken when the turbid plume had passed showed that mercury levels at these depths had returned to pre-disposal concentrations. There appeared to be a slight elevation in mercury concentrations in the surface waters, but this was declining at the end of the monitoring period.

Nitrogen Compounds. The results of nitrogen compound analyses during GBEC Dump No. 4 are presented in Table 14. An increase in ammonium concentrations in the surface and mid-depth water samples was observed with the passage of the turbid plume. Before light transmission values returned to pre-disposal levels, ammonium concentrations were at ambient levels. Bottom water samples showed that ammonium concentrations did not change there during the disposal operation. A sharp increase in organic N concentration (to 7.2 mg N/l) was observed in the surface water at 13:02:30, however when the next sample was collected 4 min later, the concentration was about 0.4 mg N/l. Mid-depth organic N concentrations appeared to increase somewhat while the bottom water organic N concentrations rose to as high as 2 mg N/l and remained elevated for the duration of sampling. Sampling was terminated just as the turbid bottom plume was passing beyond the sampling vessel.

In the bottom water, organic N concentrations appeared to show two maxima. There was a sharp increase (to 1.3 mg N/l)

TABLE 14. NITROGEN COMPOUND DATA: GALVESTON DUMP NO. 4

Time of Collection (hr:min:sec)	Depth (m)	Organic N (mg N/l)	Ammonium (mg N/l)	Nitrate (mg N/l)
12:52:00	1	-	0.01	-
12:53:00	8	-	0.01	-
12:54:00	14	0.30	0.02	0.05
	12:58 - Disposal occurred.			
12:58:30	1	-	0.01	-
12:58:45	8	-	0.02	-
12:59:30	14	0.35	0.01	0.10
	13:00 - Surface turbid plume arrived at sampling location.			
13:00:30	1	0.87	0.01	0.06
13:01:30	8	0.30	0.06	0.05
	13:02 - Surface turbid plume passed sampling location.			
13:02:30	1	7.2	0.36	0.09
13:03:00	8	-	0.15	0.06
13:03:30	8	0.27	0.01	0.05
13:03:45	1	-	0.01	-
13:03:50	14	1.3	0.01	0.05
13:04:00	1	-	0.01	-
13:04:15	8	0.66	0.02	0.07
13:04:30	14	0.65	0.03	0.08
13:05:30	8	0.63	0.01	0.08
13:06:00	14	2.0	0.01	0.05
13:06:30	1	0.39	0.01	0.13
13:06:45	8	-	0.01	-
13:07:15	14	2.1	0.01	0.10
13:08:30	1	-	0.02	-
13:09:00	8	-	0.01	-
13:09:45	14	1.0	0.02	0.07
13:11:00	14	-	0.01	-
13:11:30	8	0.26	0.02	0.07
13:12:15	1	0.43	<0.01	0.05

Dash (-) indicates no analysis made.

5 min after the disposal, followed by a decrease. About 8 min after the disposal, a second increase (to 2.1 mg N/l) was observed.

Nitrate concentrations started increasing in the surface water about 5 min after disposal but returned to ambient concentrations by 14 min after the dump. In the bottom and mid-depth water, nitrate concentrations did not show any clear pattern of variation, although concentrations fluctuated at both depths.

Phosphorus Compounds. Data in Table 15 show that the soluble ortho P concentrations during Dump No. 4 appeared to increase at all three depths during this disposal operation. Concentrations increased 25 to 55-fold but returned to ambient levels within a 6 to 8 min period. These increases occurred shortly after the rapid decrease in light transmission (Figure 4). It appears that the smallest reduction in percent light transmission and the greatest release of soluble ortho P occurred in the surface waters. The sample taken at 14 m at 12:54:00 prior to the dump also showed a higher concentration (0.45 mg P/l) of soluble ortho P. This value could be related to residual effects from previous dumps or pre-disposal leaking of the hopper-dredge.

Organic Compounds. Soluble total organic carbon concentrations in composite surface water samples collected before, during, and after the passage of the surface turbid plume from Galveston Dump No. 4 were 11.2, 23, and 9.8 mg/l,

TABLE 15. SOLUBLE ORTHOPHOSPHATE CONCENTRATIONS: GALVESTON DUMP NO. 4

Time of Collection (hr:min:sec)	Depth (m)	Soluble Orthophosphate (mg P/l)
12:54:00	14	0.45
12:58 - Dump occurred.		
12:59:30	14	<0.01
13:00 - Surface turbid plume arrived at sampling location.		
13:00:30	1	0.29
13:01:30	8	<0.01
13:02 - Surface turbid plume passed sampling location.		
13:02:30	1	0.55
13:03:00	8	0.018
13:03:30	8	<0.01
13:03:50	14	0.25
13:04:15	8	0.077
13:04:30	14	0.19
13:05:30	8	0.26
13:06:00	14	0.013
13:06:30	1	<0.01
13:07:15	14	0.012
13:09:45	14	<0.01
13:11:30	8	<0.01
13:12:15	1	<0.01

Concentration based on one analysis of one sample.

respectively. Total organic carbon in those composites were 25.8, 38.4, and 38.3 mg/l, respectively. The total oil and grease, and soluble oil and grease concentrations in those composites were all less than 0.5 mg/l.

TEXAS CITY CHANNEL STUDY

Since the sediments from Galveston Bay Entrance Channel would be considered relatively "clean" compared to sediments from industrialized areas, it was decided that a comparison study should be conducted on the nearby Texas City Channel sediments. Ordinarily the CE would not dispose of Texas City Channel sediments at the GBEC disposal site in the Gulf of Mexico because of the great distance (approximately 50 km) between the dredging and disposal sites. However, permission was granted for the Corps to dredge Texas City Channel sediments with the hopper-dredge McFarland and transport them to GBEC the disposal site so that a comparison could be made between the contaminant release from the relatively "clean" Galveston Bay Entrance Channel sediments and that found when the highly contaminated Texas City Channel sediments are disposed of at the same site by the same disposal method.

Texas City Channel Sediment Elutriate Tests

Sample Sites. Two sets of sediment and water samples were collected in the Texas City Channel for elutriate tests.

The first set of Texas City Channel samples was collected on March 28, 1975 at sites designated as TCC-1, TCC-2, and TCC-3. Their locations are shown in Figure 2. The second set of three sites (TCC-4, TCC-5, and TCC-6) was sampled September 20, 1975 (Figure 2).

General Sediment Characteristics and Oxygen Demand. The results of oxidation reduction potential (Eh), sulfide concentration, and percent dry weight determinations on sediment samples from TCC-1, 2, 4, 5, and 6 are presented in Table 16. Sediments from TCC-1 and 2 had similar Eh values (-38 mv and -20 mv), sulfide concentrations (1205 mg/kg and 1251 mg/kg), and dry weight (53% and 51%). There was also a general similarity between the Eh and percent dry weight characteristics of the sediments from TCC-4, 5, and 6. The Eh readings were -100, -136, and -124 mv, respectively. The dry weights were 38, 44, and 40%, respectively. The sulfide concentrations did not show the same trend; in fact, the three values were quite dissimilar: 2519, 237, and 1557 mg/kg for TCC-4, 5, and 6, respectively. The pattern of similarities in sediment characteristics corresponded more to date of sampling than to the location of the sampling sites in the channel. However, in general, percent dry weight values were all about the same at the five locations; except for TCC-4 and 5 sediments, the sulfide values were also all about the same. As expected, the TCC sediments were far more reducing in nature as assessed by Eh than the GBEC Buoy 1 sediment evaluated, and were also considerably higher

TABLE 16. GENERAL SEDIMENT CHARACTERISTICS: TEXAS CITY CHANNEL SEDIMENTS

Sampling Location* (TCC Site)	Eh (mv)	Sulfide (mg/kg)		Percent Dry Weight		Oxygen Uptake First Hour Per Cubic Meter ($g\ O_2$)	Oxygen Uptake First Hour Per Gram Dry Wt. ($mg\ O_2$)
		$\bar{X}$	SD	$\bar{X}$	SD		
1	- 38	1205	108	53	0.7	5.0×10^2	0.66
2	- 20	1251	95	51	2.1	4.8×10^2	0.64
4	-100	2519	10	38	0.3	6.0×10^2	1.21
5	-136	237	10	44	0.9	4.5×10^2	1.17
6	-124	1557	140	40	0.1	6.1×10^2	0.76

Mean and standard deviation calculated from triplicate analysis.

*Site 3 sediment was not analyzed.

TABLE 17. TOTAL HEAVY METAL CONCENTRATIONS: TEXAS CITY CHANNEL SEDIMENTS (mg/kg dry wt)*

Sampling Site and Date (1975)	Mn		Cr		Cd		Ni		Pb		Zn		Cu		Fe*		Hg		As	
	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD
March 28:																				
TCC-1	840	40	650	20	4.3	0.6	720	20	50	30	240	60	40	1	23	0.3	0.68	0.04	**	
TCC-2	400	20	340	180	4.5	1.3	10	1	60	20	80	10	30	10	17	4.7	0.68	0.04	**	
TCC-3	*680	150	170	90	3.3	0.3	20	1	90	20	50	10	10	1	33	2.3	0.40	0.07	**	
September 20:																				
TCC-4	556	8	77	5	<0.5	-	24	4	<1	-	160	3	83	1	26	0.3	0.041	0.002	12.8	0
TCC-5	557	21	58	4	<0.5	-	21	2	<1	-	114	6	43	1	26	0.3	0.031	0.002	11.8	0
TCC-6	671	13	90	3	<0.5	-	25	1	<1	-	141	2	60	1	26	0.5	0.032	0	13.0	0.2

Dash (-) indicates single analysis.

Mean and standard deviation calculated from triplicate analyses.

*Iron concentrations in g/kg.

**Analyses not performed.

in sulfide.

The data from the oxygen demand test of TCC-1 sediment are plotted in Figure 6. Table 16 presents the first hour oxygen uptake rates for the four sediments evaluated, on both volume and dry weight bases. First hour uptake rates on a sediment volume basis ranged from 480 to 610 g O_2/m^3 and appeared to decrease with increasing distance from the Turning Basin. The demand was somewhat higher at TCC-6 than at TCC-2, however. This pattern was not found for the oxygen uptake computed on a per dry weight basis, which appeared to vary more with sampling date than location. The sediment oxygen uptake rates at TCC-4 and 5 were about twice those found for TCC-1 and 2 sediments. The per volume and in general the per dry weight uptake rates were considerably greater than those found for the GBEC sediments evaluated, although the per dry weight uptake rates for TCC-1 and GBEC Buoy 1 sediments were about the same.

Heavy Metals. Table 17 shows that there were some major differences in heavy metal concentrations between the two sets of Texas City Channel sediments. The TCC-1, 2, and 3 sediment samples contained 3.3 to 4.5 mg/kg Cd and 50 to 90 mg/kg Pb. In the samples collected in September, concentrations of these metals were below the respective detection limits of 0.5 and 1 mg/kg. Mercury concentrations, which ranged from 0.4 to 0.68 mg/kg in sediments from TCC-1, 2, and 3, were an order of magnitude lower in the TCC-4, 5, and 6 samples. Similarly, Cr concentrations were substantially

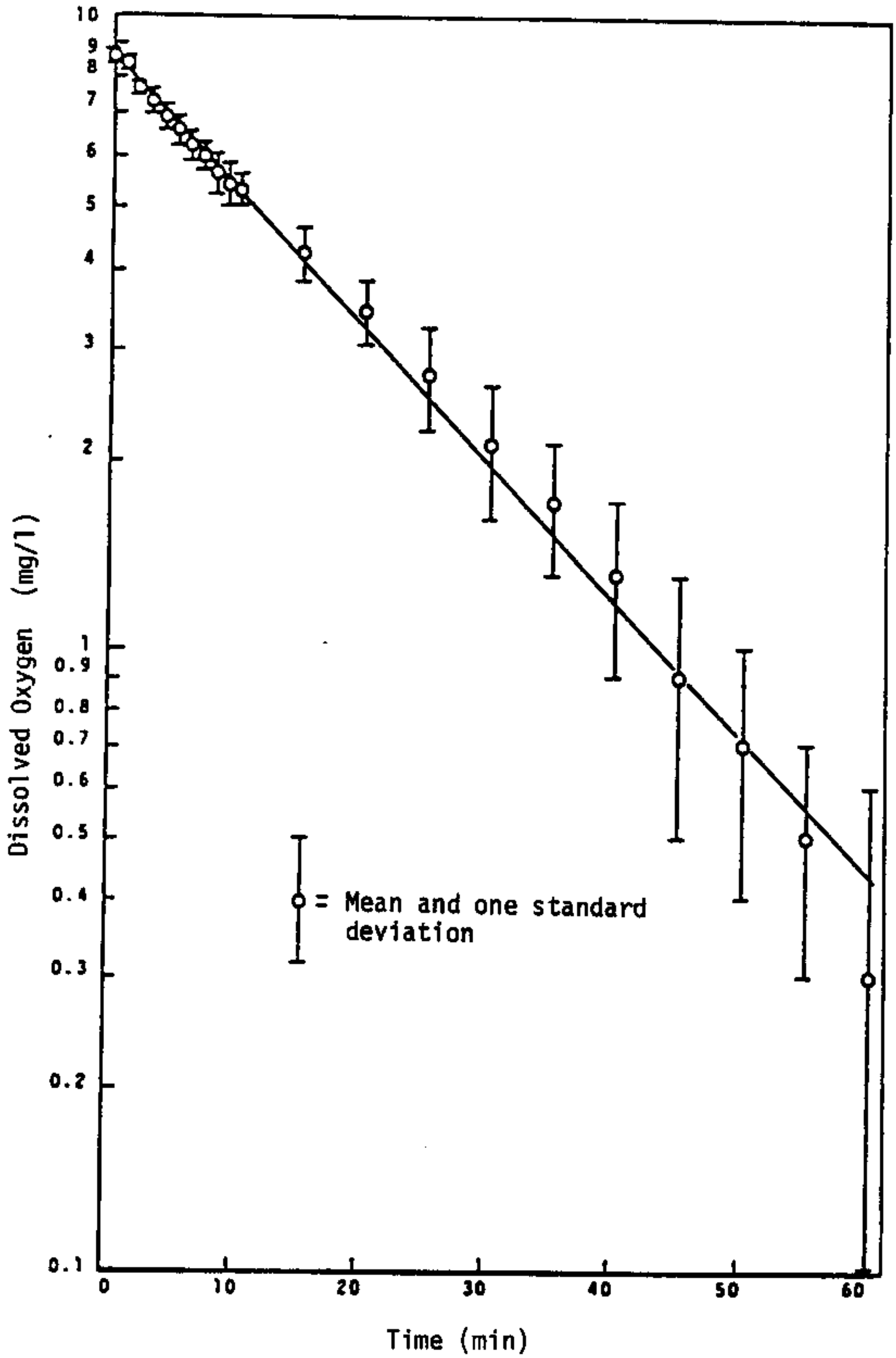

Figure 6. Oxygen Demand Test: Texas City Site 1 Sediment (Sample Size - 5 cc)

higher at TCC-1, 2, and 3 than they were at TCC-4, 5, and 6. However, the ranges of Ni, Cu, Mn, Zn, and Fe were similar for the two sampling periods. For most of the heavy metals measured on TCC-1, 2, and 3 sediment, the concentrations decreased with increasing distance from the Turning Basin area. This relationship was not generally found for the TCC-4, 5, and 6 sediments.

Table 18 presents the results of the soluble heavy metals analyses for the TCC-1 through 6 elutriate tests which were run using both 5% and 20% sediment and some of which were allowed to settle for both 1 hr and 24 hrs. Large amounts of Mn were released from all sediments during elutriation; the greatest release was found from TCC-3 sediments, resulting in elutriate concentrations of about 4000 and 6000 μg/l. There was also measurable release of Cd, Cr, and Fe from TCC-1 sediments although no apparent Cd or Fe release occurred from TCC-2 or 3 sediments. It appeared that there was a decrease in the Zn and Hg concentrations as a result of elutriation of TCC-1 and 3 sediments; release or uptake from TCC-2 sediments was unclear. The statistical validity of the apparent changes in concentrations of other heavy metals was uncertain because of the standard deviations for the analyses. It appears that the only heavy metals that have a potential for release from Turning Basin area sediments (TCC-1, and 4) are Mn and Cr, and possibly Ni and Fe.

Concentrations of Mn were greater in the elutriates

TABLE 18. SOLUBLE HEAVY METAL CONCENTRATIONS: TEXAS CITY CHANNEL OXIC ELUTRIATE TESTS

Sample Designation	Mn		Cr		Cd		Ni		Pb		Zn		Cu		Fe		Hg		As	
	X̄	SD	X̄	SD	X̄	SD	X̄	SD	X̄	SD	X̄	SD	X̄	SD	X̄	SD	X̄	SD	X̄	SD
TCC-1:																				
Site Water	52	8	11.1	2.5	1.8	0.1	4.1	1.5	3.6	1.2	13.3	0.6	5.4	0	5	3	0.27	0.01	*	
5% (1 hr)	262	52	108	8.1	5.5	1.9	11.4	3.1	3.9	2.4	4.8	3.9	4.6	4.0	18	1	0.08	0.01	*	
(24 hr)	551	13	41.4	5.6	3.8	0.6	9.5	1.9	4.1	0.7	5.1	0.9	4.2	1.6	15	8	0.01	0.01	*	
20% (1 hr)	626	48	159	10	5.9	1.1	14.7	0.6	4.5	1.9	4.7	2.6	2.2	0.2	20	5	0.22	0.01	*	
(24 hr)	1063	35	135	14	3.9	1.2	9.5	1.9	5.2	2.9	3.2	1.0	1.8	0.4	27	5	0.10	0.01	*	
TCC-2:																				
Site Water	5	-	<2.0	-	<0.3	-	7.8	-	<1.0	-	8.7	-	<1.0	-	<1	-	0.028	-	*	
5%	296	11	10.3	7.0	<0.3	~0	12.9	0.3	2.0	0	14.3	0.6	<1.0	~0	2	1	0.048	-	*	
10%	472	53	3.7	2.3	<0.3	~0	9.8	0	2.0	0	10.7	0.8	<1.0	~0	2	0	0.014	-	*	
TCC-3:																				
Site Water	12	-	<2.0	-	<0.3	-	4.6	-	1.0	-	19.3	-	1.7	-	<1	-	0.099	-	*	
5%	3970	17	<2.0	~0	<0.3	~0	6.9	1.1	3.2	2.3	15.7	0.4	<1.0	~0	<1	~0	0.034	-	*	
20%	5760	170	4.0	0	<0.3	~0	8.1	0	1.6	0	17.0	2.7	1.2	0.8	<1	~0	0.034	-	*	
TCC-4:																				
Site Water	74	7.8	<2	-	1.1	0	4.9	0.7	7.2	1.1	1.3	-	5.7	0.2	5	1.0	0.007	0	<2	-
5%	315	79	<2	-	1.5	0.1	6.8	0.9	8.6	1.0	1.4	-	4.2	0.1	12	0.9	0.009	0.002	<2	-
20%	1200	200	<2	-	1.5	0.1	5.4	0.4	8.6	1.0	<1	-	3.8	0.4	10	1.3	0.007	0	<2	-
TCC-5:																				
Site Water	68	0	<2	-	0.8	0.2	5.1	0.4	15.0	1.0	1.3	0.3	5.7	0.5	10	2.9	0.003	0	<2	-
5%	2750	495	<2	-	0.6	0.1	8.2	0.3	13.9	0.6	<1	-	3.6	0.3	10	0.2	0.003	0	<2	-
20%	1200	2828	<2	-	0.6	0.1	7.1	0.5	16.4	1.0	<1	-	6.4	2.3	16	1.3	0.009	0.002	<2	-
TCC-6:																				
Site Water	51	7.8	<2	-	1.4	0.1	4.8	0	9.7	0.5	3.0	0.6	5.4	0.3	5	1.0	0.007	-	<2	-
5%	905	134	<2	-	0.7	0.1	5.3	1.2	10.7	1.0	<1	-	4.1	0.5	8	1.6	0.007	-	<2	-
20%	1600	-	<2	-	0.5	-	5.7	-	12.1	-	<1	-	3.7	-	20	-	0.007	-	<2	-

Mean and standard deviation calculated from duplicate analyses.
Dash (-) indicates standard deviation could not be calculated.
* Not determined.

that settled for 24 hrs than in those that settled for 1 hr. For all of the other heavy metals measured, the concentrations in the 24 hr test were below those in the 1 hr test (Cr, Hg) or were not detectably different. There was not a clear, consistent difference between contaminant behavior in the 5% elutriate tests and the 20% tests.

Nitrogen Compounds. Total kjeldahl nitrogen (TKN) concentrations were determined for each of the Texas City sediments; ammonium and organic N were only determined on TCC-4, 5, and 6 sediments. As shown in Table 19, total kjeldahl nitrogen concentrations ranged from 643 mg N/kg for TCC-2 sediment to 1820 mg N/kg for the TCC-3 sediment.

TABLE 19. NITROGEN COMPOUND CONCENTRATIONS: TEXAS CITY CHANNEL SEDIMENT SAMPLES (mg N/kg dry wt)

Sample Designation	Total Kjeldahl Nitrogen		Organic N		Ammonium	
	$\bar{X}$	SD	$\bar{X}$	SD	$\bar{X}$	SD
TCC-1	1670	47	-	-	-	-
TCC-2	643	12	-	-	-	-
TCC-3	1820	432	-	-	-	-
TCC-4	1670	47	1376	159	294	112
TCC-5	654	35	618	35	36	0
TCC-6	1249	116	1027	143	222	27

Mean and standard deviation calculated from duplicate analyses.
Dash (-) indicates not measured.

Except for the sediments from TCC-2, and 5, all Texas City Channel sediments had TKN concentrations above 1000 mg N/kg. The TKN concentrations of the sediments appeared to decrease between TCC-1, and 5, and then increase again beyond; there was no distinct concentration difference between the two sampling dates. The lowest ammonium and organic N concentrations were in the TCC-5 sediment sample; the TCC-4, and 6 sediments had about the same concentrations of ammonium and organic N. Table 20 shows that the TCCTB water had higher organic N concentrations than the other TCC site waters, but had about the same nitrate concentrations. TCC-1 site water had higher ammonium levels than the other site waters; nitrate levels in all samples were similar.

There were substantial amounts of ammonium released in all of the Texas City Channel sediment elutriate tests. Concentrations in the standard 5% elutriates ranged from 1.6 to 4.5 mg N/l, and in the standard 20% elutriates from 3 to 16 mg N/l. The highest concentrations were in TCC-1, 4, and 6 elutriates. With the longer settling, somewhat greater ammonium release occurred. Organic N concentrations appeared to generally decrease during elutriation. Since the TCC-2, and 3 elutriates were centrifuged prior to analysis, however, the organic N concentrations for these samples only reflect the dissolved fraction and any organic N associated with fine particulates. Nitrate concentrations did not change by measurable amounts during elutriation of TCC-1, 2, or 3 sediments, but tended to be released from

TABLE 20. NITROGEN COMPOUND CONCENTRATIONS: TEXAS CITY CHANNEL OXIC ELUTRIATE TESTS
(mg N/1)

Sample Designation	Organic N $\overline{X}$	Organic N SD*	Ammonium $\overline{X}$	Ammonium SD*	Nitrate $\overline{X}$	Nitrate SD**
TCC-1						
Site Water	5.0	1.0	0.59	0.04	0.08	0.01
5% (1 hr)	3.2	1.3	3.1	0.18	0.05	0.01
(24 hr)	2.2	0.7	3.8	0.2	0.06	0.02
20% (1 hr)	0.2	1.5	7.5	0.1	0.05	0
(24 hr)	0.6	0.6	10	-	0.07	0.02
TCC-2:						
Site Water	0.25	0.02	0.15	0.01	0.11	0.01
5%***	< 0.05	-	1.6	0	0.11	0.01
20%***	< 0.05	-	6.0	0.08	0.09	0.01
TCC-3:						
Site Water	0.30	0.21	0.22	0.01	0.11	0.01
5%***	< 0.05	-	3.0	0.01	0.09	0.01
20%***	< 0.05	-	4.7	0.23	0.11	0.01
TCC-4:						
Site Water	0.84	0.30	0.12	0	0.10	0.01
5%	< 0.05	~ 0	4.5	0.09	0.42	0.01
20%	< 0.05	~ 0	10	0.25	0.74	0.02
TCC-5:						
Site Water	0.79	0.22	0.12	0	0.11	0.01
5%	0.93	0.16	1.6	0.01	0.05	0.01
20%	< 0.05	~ 0	3	0.02	0.09	0.01
TCC-6:						
Site Water	0.15	0.09	0.12	0.01	0.04	0.01
5%	0.91	0.01	4.4	0.01	0.50	0.01
20%	2.6	2.3	16	0.42	0.31	0.02

* Standard deviation calculated from duplicate analyses.
** Standard deviation calculated from triplicate analyses.
*** Samples centrifuged prior to analysis.
Dash (-) indicates standard deviation could not be calculated.

TCC-4 and 6 sediments and taken from solution during elutriation of TCC-5 sediments.

Phosphorus Compounds. The total phosphorus concentrations in the Texas City Channel sediments are presented in Table 21. There does not appear to be an overall pattern (considering all six locations) of concentration with regard to sampling location. In the first sampling series (TCC-1, 2, and 3), the greatest concentration was found at TCC-1 (farthest up the turning basin), the lowest at TCC-2, and an intermediate value at TCC-3 (the most seaward site). Concentrations at TCC-4, 5, and 6 were all greater than those from the other TCC sites, but showed a similar pattern,

TABLE 21. TOTAL PHOSPHORUS CONCENTRATIONS: TEXAS CITY CHANNEL SEDIMENT (mg P/kg dry wt)

Sample Designation (TCC Site)	Total Phosphorus	
	$\bar{X}$	SD
1	750	70
2	473	59
3	644	23
4	1468	34
5	937	7
6	1232	14

Mean and standard deviation calculated from duplicate analyses of one sediment sample.

i.e., the greatest concentration was found farthest up the Turning Basin (TCC-4), the lowest was found at the mouth of the Turning Basin (between TCC-1 and 2), and the intermediate concentration at TCC-6, the most seaward site.

The mean soluble orthophosphate and total phosphate concentrations found in Texas City Channel elutriates are presented in Table 22. The site waters from all three locations had essentially the same concentrations of soluble ortho P and total P. The TCC-1 sediment released about the same amount of soluble ortho P in both the 5% and 20% standard elutriate tests; it released greater amounts after the 24 hr settling period than after the 1 hr period. The total P release was also about the same in the 5% and 20% elutriates settled for 1 hr. The settling time did not appear to affect the total P concentration in the 5% elutriate. However, the 20% elutriate settled for 24 hrs had a phosphorus concentration at least four times greater than that settled for 1 hr. The results of the rest of the elutriate tests were mixed, with TCC-4 sediments showing a release of soluble ortho P and the others either no change or an uptake.

Organic Compounds. A 5% oxic elutriate test was run on samples from Texas City Channel Site 1. Aldrin, pp'DDT, dieldrin, lindane, and PCBs were detected in the sediment. The PCB value of 7426 μg/kg is one of the highest PCB concentrations found in sediment in the Lee et al. (1978) studies. This is likely due to a point source PCB discharge that has occurred in this area.

TABLE 22. SOLUBLE ORTHOPHOSPHATE AND TOTAL PHOSPHORUS CONCENTRATIONS: TEXAS CITY CHANNEL OXIC ELUTRIATE TESTS (mg P/l)

Sample Designation	Soluble Ortho P $\bar{X}$	Soluble Ortho P SD	Total P $\bar{X}$	Total P SD
TCC-1				
Site Water	0.095	0.021	0.10	0
5% (1 hr)	0.24	0.005	0.27	0.01
(24 hr)	0.30	0.008	0.30	0.01
20% (1 hr)	0.22	0.008	0.24	0.01
(24 hr)	1.3	0.03	> 1.0*	-
TCC-2				
Site Water	0.11	0.006	0.12	0.01
5% (1 hr)	0.048	0.002	0.09	0.01
20% (1 hr)	0.11	0	0.24	0.01
TCC-3				
Site Water	0.10	0.003	0.13	0.01
5% (1 hr)	0.054	0.005	0.13	0.01
20% (1 hr)	0.074	0.003	0.15	0.01
TCC-4:				
Site Water	0.17	0	-	-
5%	0.28	0.018	-	-
20%	0.26	0.001	-	-
TCC-5:				
Site Water	0.18	0.003	-	-
5%	0.13	0.003	-	-
20%	0.15	0.001	-	-
TCC-6:				
Site Water	0.20	0.002	-	-
5%	0.12	0.001	-	-
20%	0.22	0	-	-

Dash (-) indicates no analysis made.
Mean and standard deviation calculated from duplicate or triplicate analysis of one sample.
*Sample absorbance exceeded that of highest standard; insufficient sample remained to rerun analysis.

Sixteen of the peaks present in the sediment were detected in the water sample. The same chlorinated hydrocarbons identified in the sediment were also identified in the water, but only the aldrin concentration was above the detection limit. The profile of organic residue in the elutriate showed the 16 peaks detected in the water plus five others. Generally, the magnitudes of these peaks were higher in the elutriate than in the water, indicating a tendency for release of those organic residues.

The concentrations of chlorinated hydrocarbon pesticides and PCBs are presented in Table 23. The oil and grease content of the sediment (304 mg/kg) was relatively low which would indicate a low tendency for sorption of PCBs and chlorinated hydrocarbon pesticides by the suspended sediments. The results of the elutriate test showed a possible release of lindane, aldrin, and PCBs.

A 20% elutriate test was run on samples from Texas City Channel Site 4. The profile of organic residues on the chromatograms of the sediment and water showed essentially the same fingerprints found for the TCC-1 sediment and water samples. The heights of the chromatogram peaks for the elutriate were substantially higher than most of the peaks detected in the site water.

Measurable amounts of aldrin, pp'DDT, pp'DDE, lindane, and PCBs were detected in the TCC-4 sediment. Only lindane, aldrin, and PCBs were detected in the site water. In the elutriate there was release of aldrin, pp'DDT, pp'DDE, lindane,

TABLE 23. DATA FOR ORGANIC COMPOUNDS AND RELATED PARAMETERS: TEXAS CITY CHANNEL SITE 1, 5% OXIC ELUTRIATE TEST

Parameter	Sediment (µg/kg)	Water (ng/l)	Elutriate (ng/l)
Chlorinated Hydrocarbons:			
Aldrin	3.4	1.4	3.6
op'DDT	<1.6	<3.0	<3.0
pp'DDT	8.4	<3.0*	<3.0*
op'DDD	<2.0	<2.0	<2.0
pp'DDD	<2.0	<2.0	<2.0
op'DDE	<1.4	<2.0	<2.0
pp'DDE	<1.4	<2.0	<2.0
Dieldrin	2.7	<1.2*	<1.2*
Endosulfan I	<0.9	<1.2	<1.2
Endosulfan II	<3.4	<4.4	<4.4
Endrin	<1.2	<1.6	<1.6
Heptachlor	<0.3	<0.4	<0.4
Lindane	1.0	<0.3*	1.5
PCBs	7426	130	150
Other Organic Compounds			
Oil and Grease	304 mg/kg	-	-
Total Carbon	1.8%	-	-

*Compound indicated on two columns, but below detection limit.
Dash (-) indicates not determined.

and PCBs.

Bioassays

Table 24 presents the results of the 96 hr bioassays of *P. pugio* in 5%, 10%, and 20% sediment elutriates of TCC-1 sediment. Examination of the data shows that no toxicity to *P. pugio* was found for the 5% and 10% sediment elutriates. One organism death was observed in the duplicate 20% sediment elutriates after 24 hrs and another in each after 36 hrs.

Table 25 presents the toxicity response of *P. pugio* to bioassay elutriates of TCC-4 sediment, the other sediment from the TCCTB area. Mortality in the 5% and 20% tests was similar to that in the respective TCC-1 tests. There was somewhat greater toxicity in the TCC-4 10% tests than in the TCC-1 10% test. The TCC-4 sediment showed sufficient toxicity to be of potential concern if disposal were to take place in an area in which there would be limited opportunity for mixing-dilution to take place. Mixing was found to be rapid at the GBEC Disposal Site, so this amount of toxicity in these laboratory bioassays would not be of great concern in this disposal operation.

Texas City Channel Disposal Operations - Texas City Channel Dump No. 2

Background. In October 1975, Lee *et al.* (1977) monitored two disposal operations in which sediments dredged

TABLE 24. RESPONSE OF P. PUGIO TO TEXAS CITY CHANNNEL SITE 1 BIOASSAY ELUTRIATE

Time (hr)	Control A	Control B	5% A	5% B	10% A	10% B	20% A	20% B
			Number of P. pugio Living at Varying Sediment Percentages					
0	10	10	10	10	10	10	10	10
12	10	10	10	10	10	10	10	10
24	10	10	10	10	10	10	9	9
36	10	10	10	10	10	10	8	8
48	10	10	10	10	10	10	8	8
60	10	10	10	10	10	10	8	8
72	10	10	10	10	10	10	8	8
84	10	10	10	10	10	10	8	8
96	10	10	10	10	10	10	8	8

A and B are replicates.

TABLE 25. RESPONSE OF P. PUGIO TO TEXAS CITY CHANNEL SITE 4 BIOASSAY ELUTRIATE

Time (hr)	Control A	Control B	5% A	5% B	10% A	10% B	20% A	20% B
			Number of P. pugio Living at Varying Sediment Percentages					
0	10	10	10	10	10	10	10	10
12	10	10	10	10	9	10	9	9
24	10	10	10	10	9	10	9	9
36	10	10	10	10	9	8	9	9
48	10	10	10	10	9	8	9	9
60	10	10	9	10	9	7	9	9
72	10	10	8	10	9	7	9	8
84	10	10	8	10	9	7	9	8
96	10	10	8	10	9	7	9	8

A and B are replicates.

from the TCCTB (in the vicinity of TCC-1, 4) were dumped near Buoy B_1 located about 500 m from Buoy B in the GBEC Disposal Site (Figure 2). Although two operations were studied, for only one (Dump No. 2) was there sufficient data collected for a comparison of elutriate test results with those of the disposal operation. Lee et al. (1977, 1978) provide additional data and discussion of the Texas City Channel sediment disposal operations.

Texas City Channel Dump No. 2 occurred at 1014 hrs on October 10, 1975. This disposal of approximately 585 m^3 of sediment occurred approximately 40 m off the bow of the sampling vessel. The current readings were 0.1 knots (5 cm/sec) and 0.2 knots (10 cm/sec) in surface and bottom waters, respectively.

Optical Properties. Figure 7 presents percent light transmission values for the surface, mid-depth, and bottom waters during the Texas City Channel Dump No. 2 and illustrates the passage of the turbid plume. The turbid plume took 4, 5, and 3 min to reach the sampling vessel in surface, mid-depth, and bottom waters, respectively. Turbidity at the 10 m depth (bottom) gradually began to decrease 16 min after disposal but remained above the general pre-disposal level for the duration of the 1.5 hr sampling period. Secchi depth prior to disposal (0922 hrs) was 3.0 m; at 1036 hrs (after the turbid surface plume had passed) Secchi depth was 2.5 m.

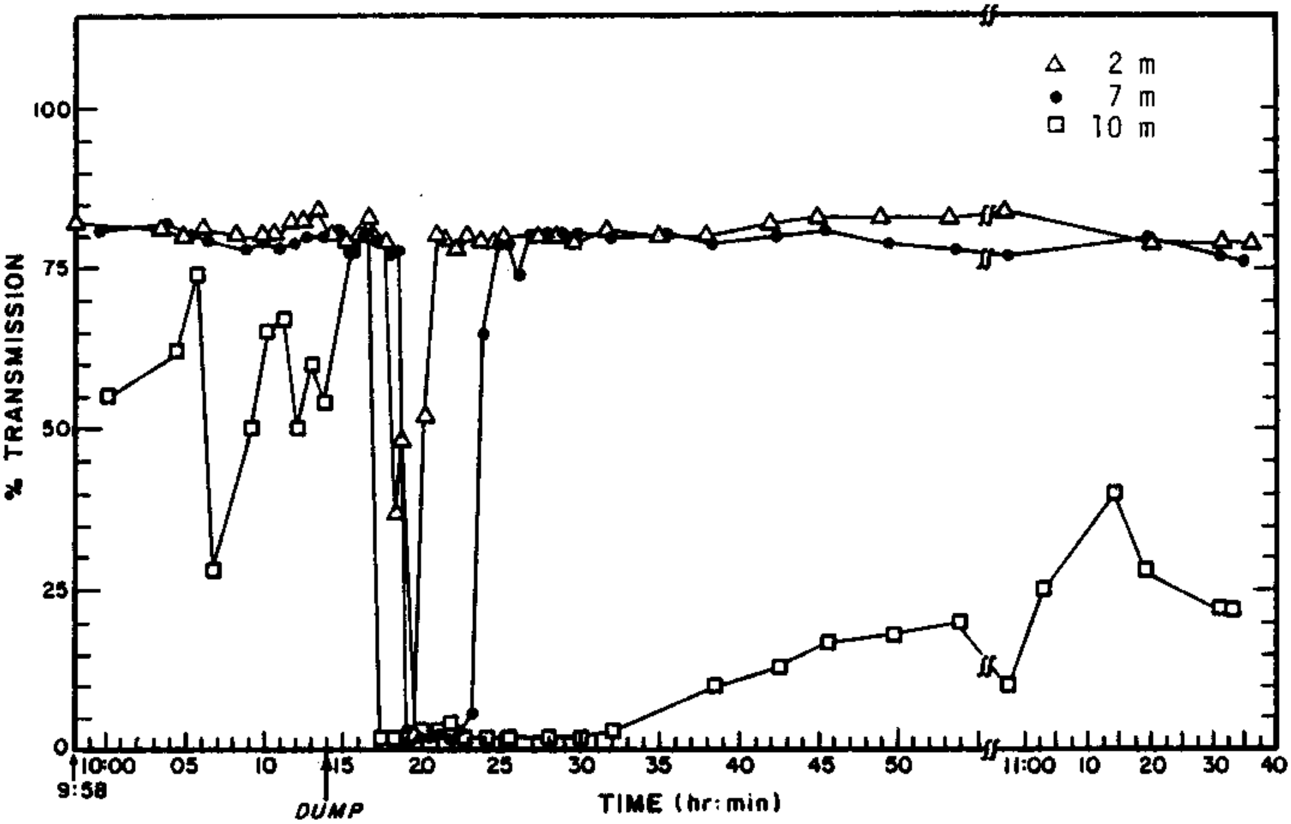

Figure 7. Percent Light Transmission: Texas City Channel Dump No. 2

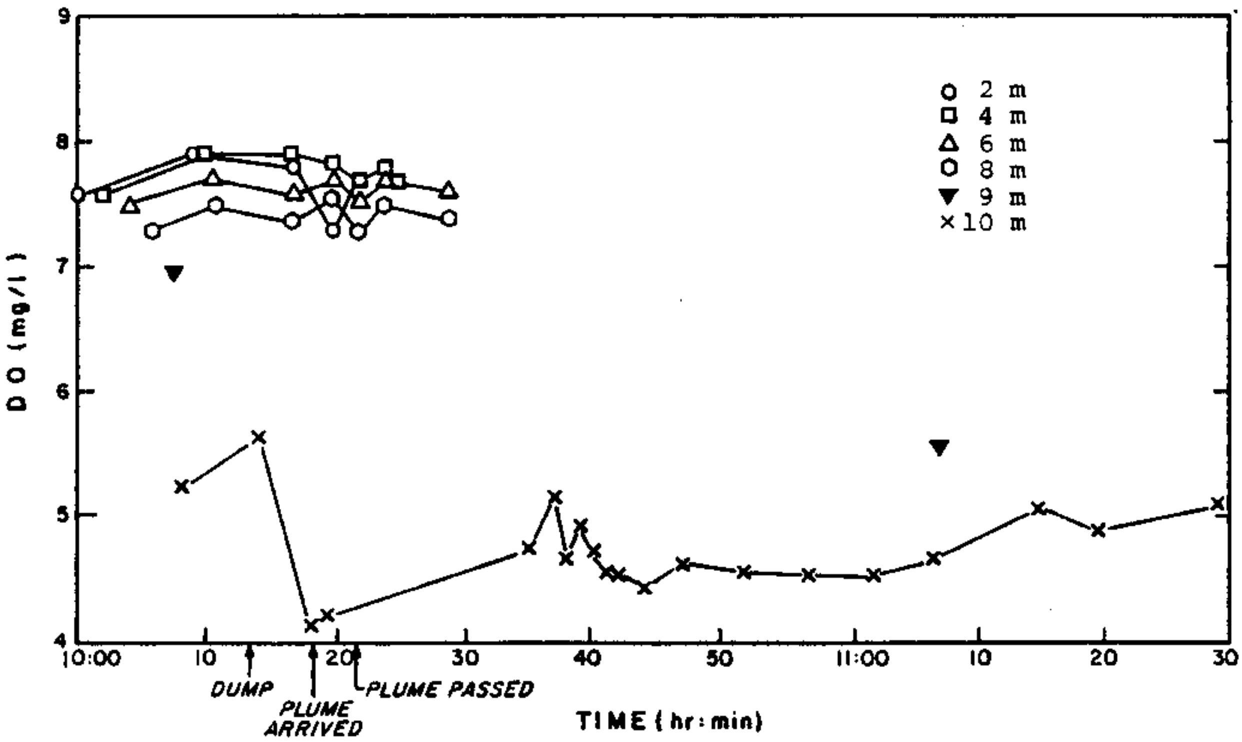

Figure 8. Dissolved Oxygen Concentrations: Texas City Channel Dump No. 2

Dissolved Oxygen. The pattern of dissolved oxygen concentrations at the 2 and 10 m depths (Figure 8) was essentially the same as the percent light transmission pattern during TCC Dump No. 2. The concentration change at the 2 m depth was minor, and the DO returned to pre-disposal level minutes after the disposal, while at the 10 m depth, it stayed below the pre-disposal level. DO readings were made only twice at the 9 m depth. At 1007 hrs the DO was 6.9 mg/l and at 1107 hrs it was 5.5 mg/l. Dissolved oxygen levels at 10 m at those times were 5.2 and 4.6 mg/l, respectively. It appears that measurable oxygen decrease occurred only in the bottom 2 m of the watercolumn.

Heavy Metals. Soluble heavy metal concentrations during TCC Dump No. 2 are presented in Table 26. There were no major releases of heavy metals during disposal. Apparent Ni increases were observed in the bottom water with the arrival of the bottom turbid plume. The concentrations decreased again 10 min later. There appeared to be some release of Zn, Cu, Hg, and Fe most of which occurred primarily in the bottom waters. By the end of sampling about 1.25 hrs after the dump, all concentrations had returned to ambient levels.

The concentration of manganese measured at 10:15:50 at 10.5 m was 838 μg/l. This value was well outside the concentration range of this metal during this dump, and, as the sample was not taken in the turbid plume, it is suspected that the high value was due to contamination during collection or handling.

TABLE 26. SOLUBLE HEAVY METAL CONCENTRATIONS: TEXAS CITY CHANNEL DUMP NO. 2

(μg/l)

Time of Collection (hr:min:sec)	Depth (m)	Mn	Cr	Cd	Ni	Pb	Zn	Cu	Fe	Hg	As
9:58:30	1.5	14	<2	0.8	14.5	1.9	9.7	2.9	<5	0.002	<2
9:58:45	8	14	<2	0.8	14.5	2.4	5.3	2.5	11	0.002	<2
10:01:46	1.5	30	<2	0.8	14.5	1.9	2.3	2.5	5	0.006	<2
10:02:25	10.5	25	<2	0.8	3.1	2.4	13.1	3.0	18	0.016	<2
10:14 - Dump occurred											
10:14:15	8	<5	<2	0.6	12.2	6.8	12.9	4.8	459	0.002	<2
10:15:50	10.5	838	<2	0.9	6.8	0.9	22.7	2.6	13	0.002	<2
10:16:10	8	<5	<2	0.6	10.5	1.9	7.2	2.8	40	0.002	<2
10:16:45	8	14	<2	0.7	14.5	6.8	15.8	4.6	37	0.006	<2
10:18 - Surface turbid plume arrived at sampling station											
10:18:00	1.5	16	<2	0.8	4.1	0.9	4.2	4.3	17	0.002	<2
10:18:25	10.5	25	<2	1.1	4.7	2.4	16.9	6.1	9	0.002	<2
10:19:10	10.5	14	<2	1.0	12.9	1.9	8.7	6.0	109	0.016	<2
10:19:40	8	21	<2	0.7	16.2	2.7	7.4	9.4	212	0.006	<2
10:19:55	1.5	<5	<2	0.6	13.7	1.9	2.2	2.8	13	0.002	<2
10:20:10	10.5	14	<2	0.7	18.6	2.7	8.6	2.9	49	0.002	<2
10:20:50	1.5	14	<2	0.9	15.3	2.4	44.8	2.8	20	0.016	<2
10:21 - Surface turbid plume passed sampling station											
10:21:10	10.5	30	<2	1.2	15.3	2.4	17.7	7.5	74	0.012	<2
10:21:50	1.5	<5	<2	0.8	12.1	2.7	6.7	3.1	50	0.023	<2
10:22:45	1.5	30	<2	0.9	12.9	2.2	7.7	2.9	123	0.012	<2
10:23:45	8	19	<2	0.8	4.4	2.8	5.0	4.6	18	0.006	<2
10:24:35	10.5	31	<2	1.2	5.6	2.8	17.1	9.4	41	0.006	<2
10:25:00	8	14	<2	0.8	10.5	1.6	2.7	2.6	11	0.016	<2
10:26:35	10.5	<5	<2	1.0	11.3	2.4	10.2	6.8	98	0.016	<2
10:27:55	8	14	<2	0.9	15.3	2.2	4.8	2.5	10	0.016	<2
10:29:45	8	26	<2	0.7	3.3	2.0	6.5	3.5	13	0.016	<2
10:30:50	10.5	21	<2	0.7	17.0	2.7	9.3	8.9	22	0.006	<2
10:31:35	1.5	21	<2	0.8	12.1	2.2	10.3	3.4	5	0.016	<2
10:34:45	8	27	<2	0.7	3.6	0.9	1.5	2.4	<5	0.002	<2
10:35:10	10.5	10	<2	0.8	3.6	1.4	30.7	4.0	12	0.002	<2
10:39:50	10.5	27	<2	1.0	4.1	0.9	3.3	3.5	5	0.002	<2
10:43:10	8	25	<2	0.8	3.3	2.8	29.0	3.1	7	0.002	<2
10:43:30	10.5	21	<2	1.0	4.4	0.5	26.2	4.3	7	0.002	<2
10:48:00	1.5	32	<2	0.8	3.1	0.9	7.6	2.6	<5	0.002	<2
10:48:30	8	26	<2	0.8	5.0	2.4	11.8	3.1	10	0.002	<2
10:49:00	10.5	21	<2	0.8	3.4	0.9	4.8	3.2	<5	0.002	<2
10:55:50	10.5	10	<2	0.9	4.4	0.9	3.0	2.6	<5	0.002	<2
11:01:10	10.5	10	<2	0.8	4.4	0.9	1.8	2.6	<5	0.002	<2
11:07:25	10.5	19	<2	0.9	3.6	2.8	7.7	3.2	13	0.002	<2
11:13:30	10.5	10	<2	0.9	3.6	0.9	3.7	3.2	6	0.016	<2
11:17:30	1.5	25	<2	0.9	4.2	2.8	6.3	3.7	7	0.002	<2
11:18:00	8	25	<2	1.0	3.6	2.4	12.7	3.7	6	0.002	<2
11:18:15	10.5	25	<2	0.8	5.0	2.4	7.0	2.8	<5	0.002	<2
11:32:40	10.5	15	<2	0.8	3.3	2.4	12.3	3.8	6	0.002	<2

Nitrogen Compounds. Total ammonium concentrations are plotted as a function of time during TCC Dump No. 2, in Figure 9. Release of ammonium in the bottom water was observed with the passage of the bottom turbid plume (Figure 7), but the concentrations in the overlying water remained below 0.05 mg N/l except for one sample each of surface and

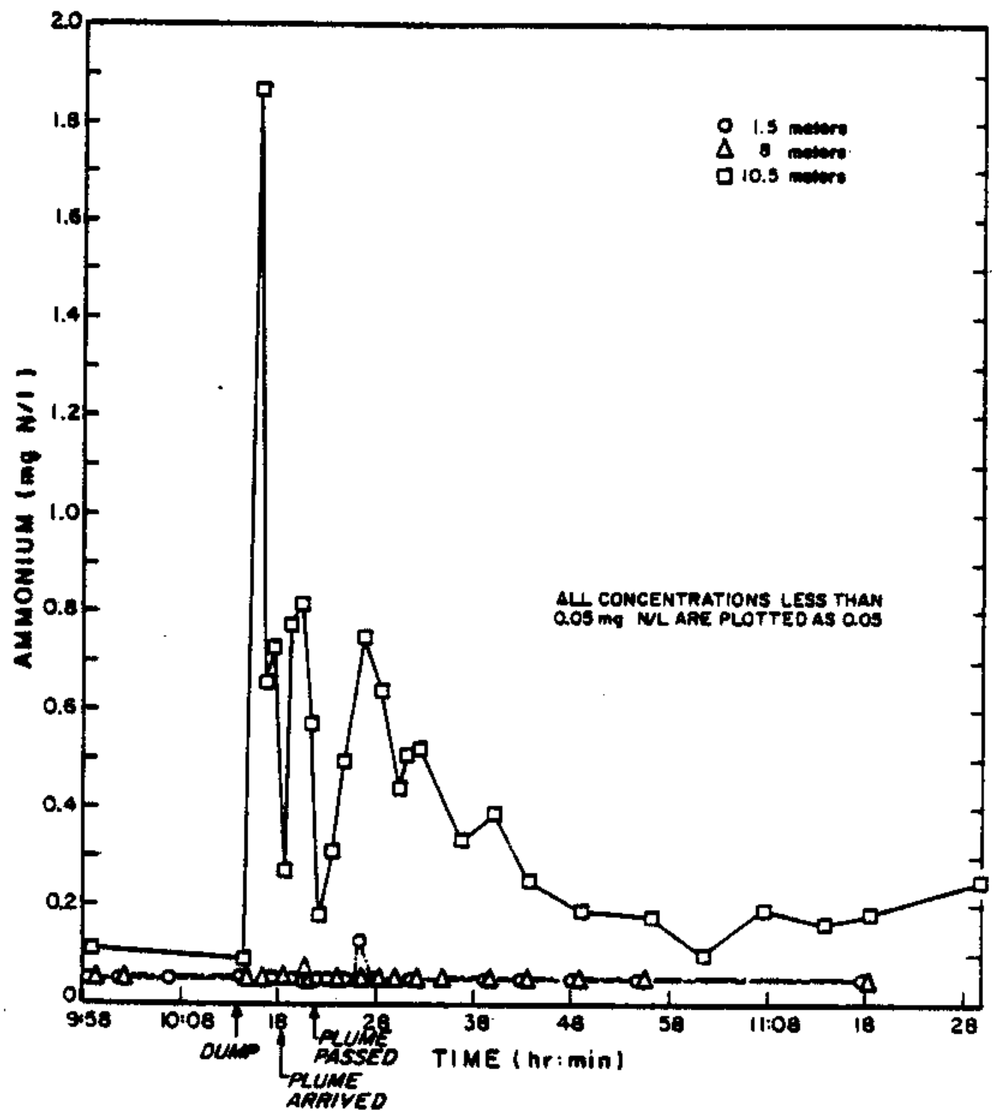

Figure 9. Ammonium Concentrations: Texas City Channel Dump No. 2

mid-depth water. The peak bottom water concentration persisted at the sampling location for less than 1 min. Organic N was apparently released in the bottom waters as a result of the disposal operation but returned to ambient levels 13 min after the surface turbid plume arrived at the sampling location. In the surface and mid-depth water, no discernible pattern could be observed.

Phosphorus Compounds. Concentrations of soluble orthophosphate during TCC Dump No. 2 are presented in Table 27. It appears that surface water soluble ortho P concentrations decreased slightly with the passage of the surface turbid plume, but increased with increasing turbidity at the other two depths monitored; at mid-depth the increase was only seen in one sample during the period of greatest turbidity. The bottom water soluble ortho P levels were variable but were consistently within or below the ambient range within 30 min after the disposal.

Organic Compounds. Seven composites of surface water samples collected before, during, and after the passage of the surface turbid plume were analyzed for soluble TOC, and oil and grease. One composite was of samples collected before the plume arrived; samples collected within the plume made up Composites 2 through 5. Two composites were made from samples collected after the plume had passed. The composites were centrifuged at 9000 rpm for 10 min to remove particulate matter but were not subjected to filtra-

TABLE 27. SOLUBLE ORTHOPHOSPHATE CONCENTRATIONS: TEXAS CITY CHANNEL DUMP NO. 2

Time of Collection (hr:min:sec)	Depth (m)	Soluble Orthophosphate (mg P/l)
9:58:30	1.5	0.070
9:59:00	10.5	0.072
10:01:45	1.5	0.10
10:02:00	8	0.11
10:02:25	10.5	0.097
	10:14:00 - Dump occurred.	
10:14:15	8	0.084
10:14:25	10.5	0.067
10:15:50	10.5	0.56
10:16:10	8	0.019
10:16:45	8	0.053
	10:18:00 - Surface turbid plume arrived at sampling location.	
10:18:00	1	0.096
10:18:25	10.5	0.18
10:19:10	10.5	0.084
10:19:40	8	0.14
10:10:55	1	0.061
10:20:10	10.5	0.35
10:20:50	5	0.062
	10:21:00 - Surface turbid plume passed sampling location.	
10:21:10	10.5	0.054
10:21:50	1.5	0.054
	(Continued)	
10:22:45	1.5	0.044
10:23:45	8	0.076
10:24:35	10.5	0.17
10:25:00	8	0.049
10:26:35	10.5	0.086
10:27:55	8	0.049
10:29:45	8	0.077
10:30:50	10.5	0.14
10:31:35	1.5	0.045
10:34:45	8	0.058
10:35:10	10.5	0.070
10:39:50	10.5	0.069
10:43:10	8	0.10
10:43:30	10.5	0.14
10:48:00	1.5	0.079
10:48:30	8	0.021
10:49:00	10.5	0.041
10:55:50	10.5	0.070
11:01:10	10.5	0.076
11:07:25	10.5	0.072
11:13:30	10.5	0.064
11:17:30	1.5	0.049
11:18:00	8	0.042
11:18:15	10.5	0.026
11:32:40	10.5	0.033

Soluble ortho P concentrations based on a single analysis of one sample.

tion in order to avoid the loss of oil and grease due to adherence to the filter. It is assumed that the total organic carbon (TOC), and oil and grease concentrations measured represent the soluble fraction of these two parameters.

Table 28 shows that the concentration of TOC in the composite of samples collected before the arrival of the plume was about 22 mg/l. The TOC concentrations in Composites 2

TABLE 28. CARBON AND OIL AND GREASE IN THE SURFACE WATER: TEXAS CITY DUMP NO. 2

Composite Number	Time (hr:sec)	Soluble TOC (mg/l)	Oil and Grease (mg/l)
1	10:06 to 10:16	21.8	5.9
	Surface turbid plume arrived at sampling location.		
2	10:16 to 10:17	20.8	7.1
3	10:18 to 10:22	19.7	4.6
4	10:24 to 10:26	15.2	8.6
5	10:27 to 10:31	27.2	6.1
	Surface turbid plume passed sampling location.		
6	10:36 to 10:39	26.1	7.5
7	10:42 to 10:55	26.4	5.7

through 5 ranged from 15.2 to 27.2 mg/l with an average of 20.7. The two post-plume composite samples each contained about 26 mg/l TOC. The oil and grease content before the plume arrived was 5.9 mg/l. During passage of the plume, oil and grease concentrations fluctuated between 4.6 and 8.6 mg/l with an average of 6.6 mg/l. The two post-plume samples contained 7.5 and 5.7 mg/l, with an average of 6.6 mg/l.

Statistical analysis of variance was run on the Texas City TOC and oil and grease data. The results indicated that there was no significant difference, at a 95% confidence level, between concentrations of these two parameters before, during, or after passage of the surface turbid plume.

DISCUSSION

This study has shown that open water disposal-dumping of even highly contaminated dredged sediments, such as those derived from the Texas City Channel, does not represent a threat to watercolumn aquatic life or other beneficial uses of the Gulf of Mexico waters near Galveston, Texas. The conclusions from this study are in accord with the results of several other similar studies that were conducted at various locations in the U.S. as part of the Corps of Engineers' DMRP. As reported by Lee *et al*. (1978) and Jones and Lee (1978), with few exceptions, contaminants associated with U.S. waterway sediments are not released to the watercolumn upon open water disposal of dredged sediments. In general these contaminants are tightly bound to the sediments. As discussed by Jones and Lee (1978), any contaminant release that did occur during a disposal operation was rapidly diluted to levels which are not toxic or otherwise adverse to watercolumn organisms at the disposal site. It would be rare that the open water disposal of dredged sediments would have an adverse impact on beneficial uses of the waters at or near a dredged sediment disposal site.

It is important to note, as discussed by Jones and Lee (1978), that the US EPA water quality criteria of the type released in 1976 (US EPA, 1976), and state water quality standards numerically equal to these criteria, are not valid tools by which to judge the potential adverse impact of contaminants' release in elutriate tests or to the open water disposal site watercolumn. This is because the US EPA criteria are generally chronic exposure, safe concentrations; when adopted as standards, they are generally applied to total contaminant concentrations. It is thereby assumed that aquatic organisms can receive a chronic - lifetime exposure to the contaminant, and that all of the contaminants are in forms that are available to aquatic life. It is well-known that many contaminants exist in aquatic systems in a variety of forms, only some of which are available to aquatic life. This is especially true in the case of the contaminants associated with dredged sediments because in general, particulate contaminants are not available to aquatic life. Therefore, relatively high concentrations of some particulate-associated contaminants in dredged sediment may be present in the water-column without adverse effects on the aquatic life or other beneficial uses of the water.

As discussed by Jones and Lee (1978), Jones et al. (1979), and Lee et al. (1981a), the worst case, chronic exposure assumption used by the US EPA and many states causes US EPA criteria and state water quality standards to have little direct applicability to most dredged sediment disposal operations. In general, depending on the kind of organism, these criteria and standards are based on as much as a year or more exposure of the organism to the contaminant in a 100% available form. At the Galveston disposal site, the maximum exposure that an organism could obtain to contaminants released to the watercolumn during open water disposal would be in the order of a few minutes to an hour. Since aquatic organisms can be exposed to relatively high levels of contaminants for short periods of time without adverse impact, it is almost impossible for aquatic organisms to receive a sufficient exposure to contaminants released to the watercolumn during open water disposal of dredged sediment to be harmed by such disposal practices. For further discussion of these topics, consult Jones and Lee (1978).

It is important to emphasize that the Galveston, Texas studies as reported herein, as well as reported by Lee et al. (1978), all show that the elutriate test is a reliable tool to predict the direction and approximate amount of contaminants that would be released by open water disposal of hydraulically dredged sediments. This test procedure should be used to detect potentially adverse situations. While, as noted above, it will be rare that such adverse situations occur, it is possible, especially when dredging takes place near certain types of industrial waste discharges and disposal is in areas with limited mixing-dilution, that an adverse situation could be encountered. If an elutriate test shows the release of contaminants to levels above the US EPA criteria or state water quality standards, then a hazard assessment evaluation should be performed of the type described by Lee and Jones (1981a) and Lee et al. (1981) to determine whether the apparently "excessive" contaminants represent a hazard to the beneficial use of the receiving waters. Factors such as duration of exposure, concentrations of available forms of contaminants, rate and extent of dilution, etc., must all be considered in determining whether release of contaminants during an elutriate test represents a potential hazard to aquatic life or other beneficial uses of the waters at the dredged sediment disposal site.

The studies reported in this paper are primarily directed toward watercolumn impacts arising during the release of contaminants during open water disposal operations. In addition to the concern about this situation, consideration must be given to the potential impact of the contaminants associated with the dredged sediments at the dredged material disposal site on benthic and epibenthic organisms that have colonized the disposal area sediments. This study, as well as others, has shown that while there is some toxicity to laboratory test organisms associated with some dredged sediments, it is small compared to the toxicity that would occur if appreciable parts of the contaminants present in the sediments were available to the organisms. As discussed by Jones and Lee (1978), the benthic organism bioassay testing of dredged sediments is a tool that can signal potential problems. It is not an indicator of real problems that will occur upon open water disposal of dredged sediment. There is a wide variety of factors that influence the translation of laboratory-based toxicity tests to field conditions. The overall approaches that should be used to evaluate, on a site-specific basis, the significance that a certain level of toxicity found in dredged sediment bioassays in particular, and bioassays in general under laboratory conditions represents to the aquatic life and other beneficial uses at a dredged material disposal site, are discussed by Jones and Lee (1978) and Lee and Jones (1981b).

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Environmental Sciences Program participated in this study; a complete listing of these individuals is provided in the report by Lee *et al*. (1977) and Lee *et al*. (1978). In addition, a number of individuals associated with the Galveston District of the Corps of Engineers and the Corps of Engineers Waterways Experiment Station at Vicksburg, MS contributed to the completion of this study. Of particular note was the assistance of Dr. R. Engler and Dr. R. Peddicord. The completion of this paper was supported by the Department of Civil Engineering, Colorado State University, and G. Fred Lee & Associates - EnviroQual, both of Fort Collins, CO.

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CONTEMPORARY APPROACHES IN BIOLOGICAL MONITORING

by John D. Lunz[1]

ABSTRACT

Impacts may be categorized as physical habitat alterations, secondary effects related to pollution or other special impacts affected by the dredging plant's design. A habitat based ecological viewpoint is useful in designing monitoring strategies and sorting multivariable influences on aquatic biological populations and communities. Physical habitat alterations are thereby viewed as modifications of natural habitat variables; secondary effects related to pollution are viewed as stresses superimposed on natural habitat variability. An organism's performance can be described in an N-dimensional geometric display in which individual performances are represented as vectors and each vector describes the capacity or potential ability of a fish or other aquatic organisms to perform a particular activity. The habitat based approach to impact assessment permits the construction of quantitative impact statements about important aquatic biological resources even when monitoring strategies employing direct observation of the resource are impractical or impossible. The complexity of the issue of pollution effects caused by the large number of known and unknown chemical substances acting individually or together to cause a biological response prescribes the development of an empirical bioassay-type test for making an impact statement. Currently such a test does not exist. Procedures that have been developed and that are being refined to test the suitability of dredged material for aquatic disposal do not claim to indicate larger biological system effects or any correlation with subtle sublethal biological effects.

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