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STABILITY AND FATE OF DREDGED SEDIMENT[1]

by H. J. Bokuniewicz[2]

ABSTRACT

Hopper dredges often remove fine-grained sediment from navigation channels and release this material at disposal sites into water about 20 m deep. Such operations involving dredges with a capacity of 703 m^3 were studied at two locations in the Great Lakes. The bulk density of the material in the hoppers was 1.3 Mgm/m^3. The dredged material behaved as a fluid and, when the hopper doors were opened, it was driven out by the excess pressure head at speeds up to 4 m/sec. Almost all of the sediment released is deposited from a thin, radially spreading, bottom surge in a ring between 50 and 160 m from the point of impact with the lake floor. The layer formed by a single discharge has a thickness of about 3 mm. The minimum radius of a deposit that is formed by sedimentation from turbidity currents is determined by the range of the surge (<300 m) and the deposit cannot have side slopes greater than 0.05. These conditions control the capacity of a designated disposal area. The surface layer of the deposit is in contact with the overlying water and may be dispersed. The thickness of this layer depends upon the depth of resuspension or bioturbation. In coastal waters of the northeastern U.S., resuspension of the top few millimeters of sediment is typical, and bioturbation may mix sediment to a depth of about 0.1 m. Under these conditions, dredged sediment

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in a deposit containing less than 10^4 m^3 will be almost entirely exposed to the water column; if no net erosion occurs, containment is favored in deposits containing more than 10^6 m^3. Conditions in mined, submarine pits in New York Harbor favor the containment of dredged sediment. The side slopes and roughness of the pit floors will restrict the spread of the bottom surge. The pits act as traps for fine-grained sediments and the naturally high sedimentation rates would make net erosion of the dredged sediment deposit unlikely.

Introduction

Dredged sediment released from a scow or hopper dredge at the water surface descends through the water column and may be deposited on the bottom. The fate of this material depends upon the dredging and disposal techniques, upon the characteristics of the disposal site, and upon the form of the deposit of dredged sediment (Bokuniewicz and Gordon, 1978 a, b). The probability of containment of dredged sediment in a permanent deposit, or the dispersal of dredged sediment from the deposit, may be enhanced by the proper management of the dredging and disposal operation and the judicious choice of the disposal site.

This paper deals with the formation of deposits of dredged sediments on subaqueous disposal sites and then discusses the long-run behavior of the dredged material. The disposal operation may result in either containment or dispersal of dredged sediment and these alternative, disposal strategies will be considered here for a certain class of operations. The discussion will be restricted to the disposal of fine-grained sediment from a hopper dredge in shallow water under slow to moderate currents. This type of operation is the kind usually undertaken by the U.S. Army Corps of Engineers' hopper dredges, and the behavior of the fine fraction of the dredged sediment is of special interest because the most troublesome contaminants are often associated with fine-grained particles. It is this fraction also whose behavior is most difficult to predict.

As examples, two special situations will be considered. The first situation will be one in which it is desired to allow the dredged sediment to disperse from the disposal deposit over a long time. This may be the case for the disposal of relatively uncontaminated sediments in well flushed areas. If the dredged sediment may be safely dispersed, the disposal site could be used continuously as long as the rate of application does not exceed the dispersal rate. The second situation is one in which it is intended to isolate and contain the dredged material by burial in a subaqueous pit, specifically, a pit as might be created during a sand-mining operation. This use of artificial pits as containment sites for dredged sediment is an attractive option. In some areas, mined pits could have an adverse effect on the marine environment. Deep holes in New York Harbor, for example, trap fine-grained sediment with a high organic content; the consumption of dissolved oxygen by this material may affect the quality of the overlying water. The possible adverse effects due to the presence of the pits may be minimized by back-filling. The combination of disposal and mining operation presents a possibility of solving two problems at the same time - the need to dispose of dredged sediment and the need to back-fill mined pits.

The Disposal Operation

Material released from a scow or hopper dredge is emplaced on the disposal site in three steps that were first described by Gordon (1974). Upon release, the dredged material descends rapidly through the water column. Impact with the bottom occurs. The released material then spreads quickly away from the impact point as a well-defined toroidal, density surge only a few meters thick. These three steps--descent, impact and spread of the bottom surge have been observed under a wide range of hydrographic conditions, dredged material characteristics, and dredging equipment (Bokuniewicz et al., 1978; Custar and Wakeman, 1977). The limiting conditions under which these steps will occur have not been determined, but they have been documented in water depths to 67 m and currents to 4 knots.

A small fraction of the released material may be found in the water column above the bottom surge. This is material that has been left behind during the descent phase, has spilled over the top of the hopper before discharge, or has been washed out of the hopper after the discharge. This diffuse cloud of particles drifts with the currents and settles slowly. While the cloud of turbid water may be very noticeable around the dredge, the drifting material accounts for only 1 to 5% of the released material. This fraction will be neglected in this paper.

The data to best describe the disposed processes come primarily from research conducted at two disposal operations in the Great Lakes. One was at Ashtabula, Ohio, in Lake Erie, and the other was at Rochester, New York, in Lake Ontario. The dredging was performed by the U.S. Army Corps of Engineers' hopper dredges (the Hoffman and the Lyman). These vessels have a hopper capacity of 703 m^3. The dredged sediment was predominantly silt or sandy silt, and the water at the disposal sites ranged from 15 to 46 m in depth. This research is discussed in detail by Bokuniewicz, et al. (1978), and Bokuniewicz and Gordon (1978 b,c), and the following description of the disposal operation is a summary of this work.

During dredging, sediment and water were pumped into the hoppers for a fixed time (1 hour). The hoppers were allowed to overflow while sediment accumulated on the hopper floor. After dredging, the bulk density of the material in the hoppers was 1.3 Mgm/m^3, but the sediment is distributed in two layers (figure 1). The upper layer has a density of about 1.1 Mgm/m^3. This dense fluid overlies a layer with a density of 1.7 Mgm/m^3. For the

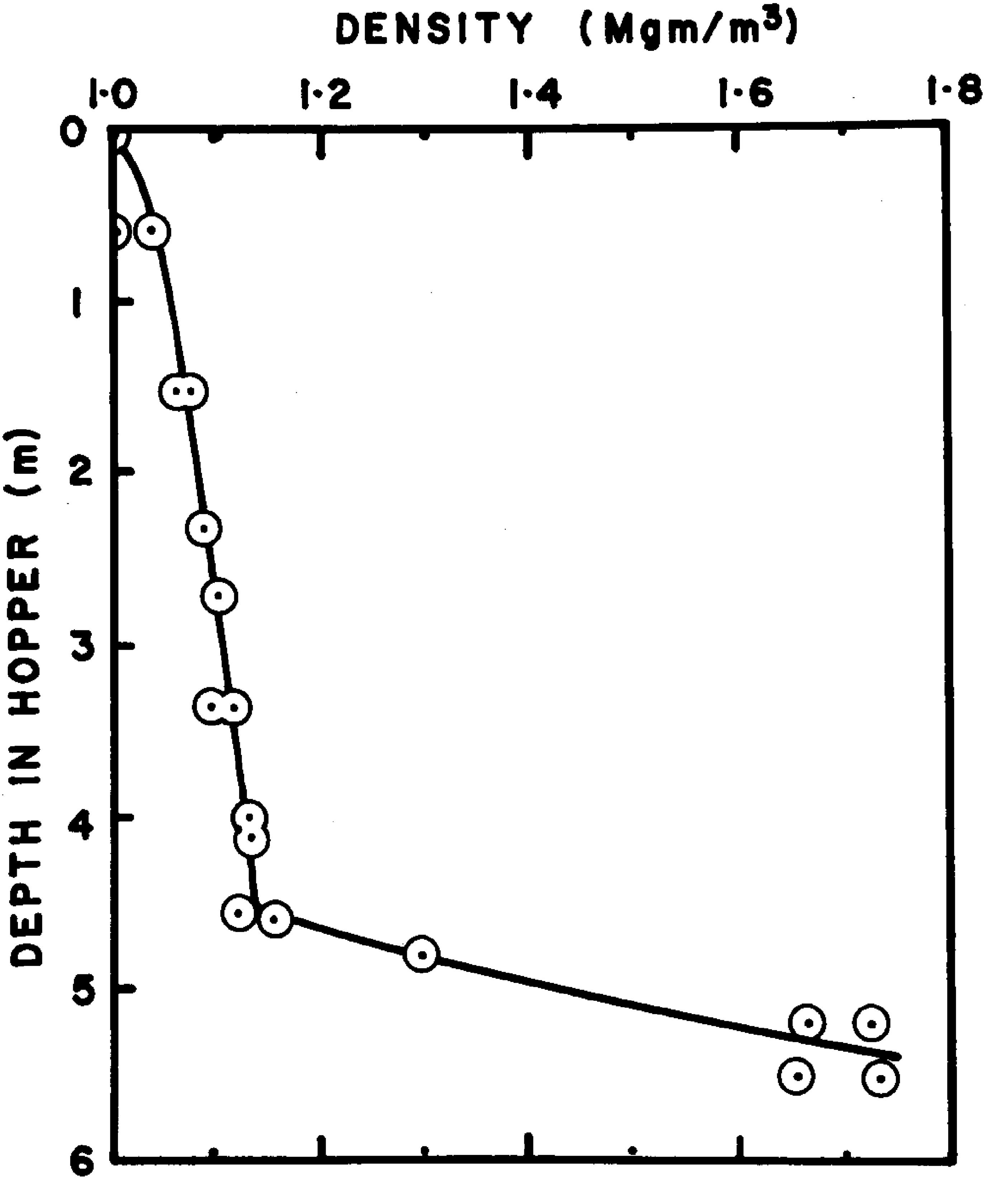

Figure 1. Density profile of the hopper water and sediment from the dredge Lyman.

distribution shown in Figure 1, about 30% of the mass in the hoppers is contained in about 12% of the volume. The shear strength of the bottom layer is only about 10^4 dynes/cm^2.

The dredged material behaves as a liquid, and when the hopper doors are opened it is driven out at high speed by the excess pressure head in the hoppers. The injection process may be described by a simple energy balance, similar to Bernoulli's equation. Figure 2 compares the observed liquid level in the hoppers during the discharge with predicted values using a rate of frictional energy loss of 0.1 Mjoules. The injection speeds may be as high as 4.3 m/sec. The rapidly moving jet of material descends to the lake floor. The speed of this jet has been measured at about 1.0 m/sec. Ambient water is entrained during descent and the total volume impacting on the bottom may be increased to about 72 times the volume released.

Impact of the descending jet with the bottom deflects the flow of dredged material and entrained water to form a bottom surge or density current which spreads away from the impact point across the lake floor. Because of the sharp density contrast between the surge and the overlying water, the surge is easily detected with a standard recording fathometer. Figure 3 illustrates the development of the surge as it was detected simultaneously by acoustic transducers spaced outward from the side of the dredge. The surge slows and thins as it travels radially outward. The evolution of the surge is shown by a series of sections in Figure 4. These sections represent a generalized composite of all the data collected at the two sites. These data include not only acoustic observations but also measurements of the near-bottom currents and the optical transmittance. The concentration of suspended solids in the surge was determined from timed, pumped water samples. Within the surge, concentrations may reach 11 gms/m^3. Combining the measured

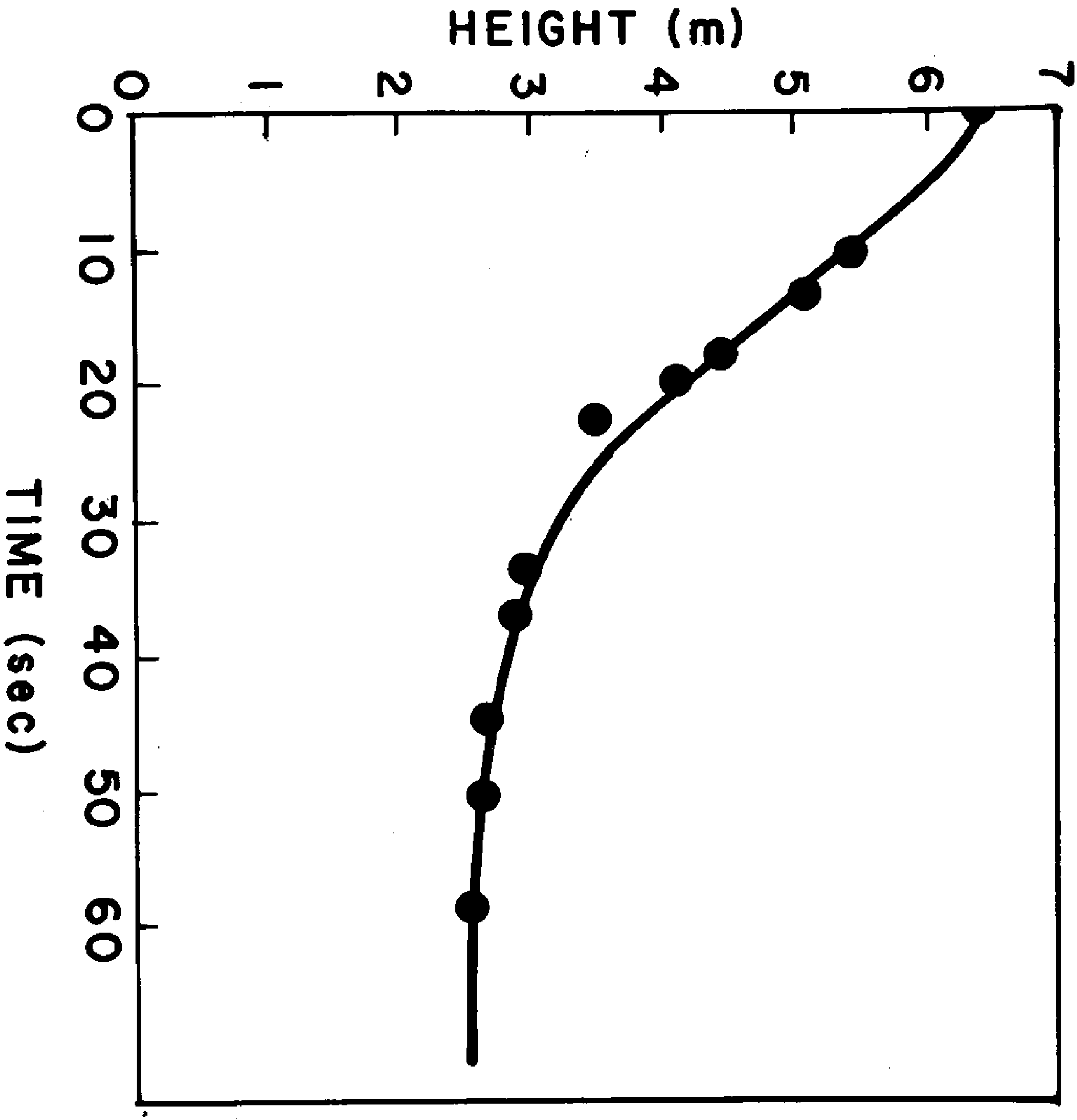

Figure 2. The height of dredged material measured in the hoppers during the discharge (solid circles) compared with a predicted height based on an energy balance with a frictional energy loss rate of 0.1 Mjoules.

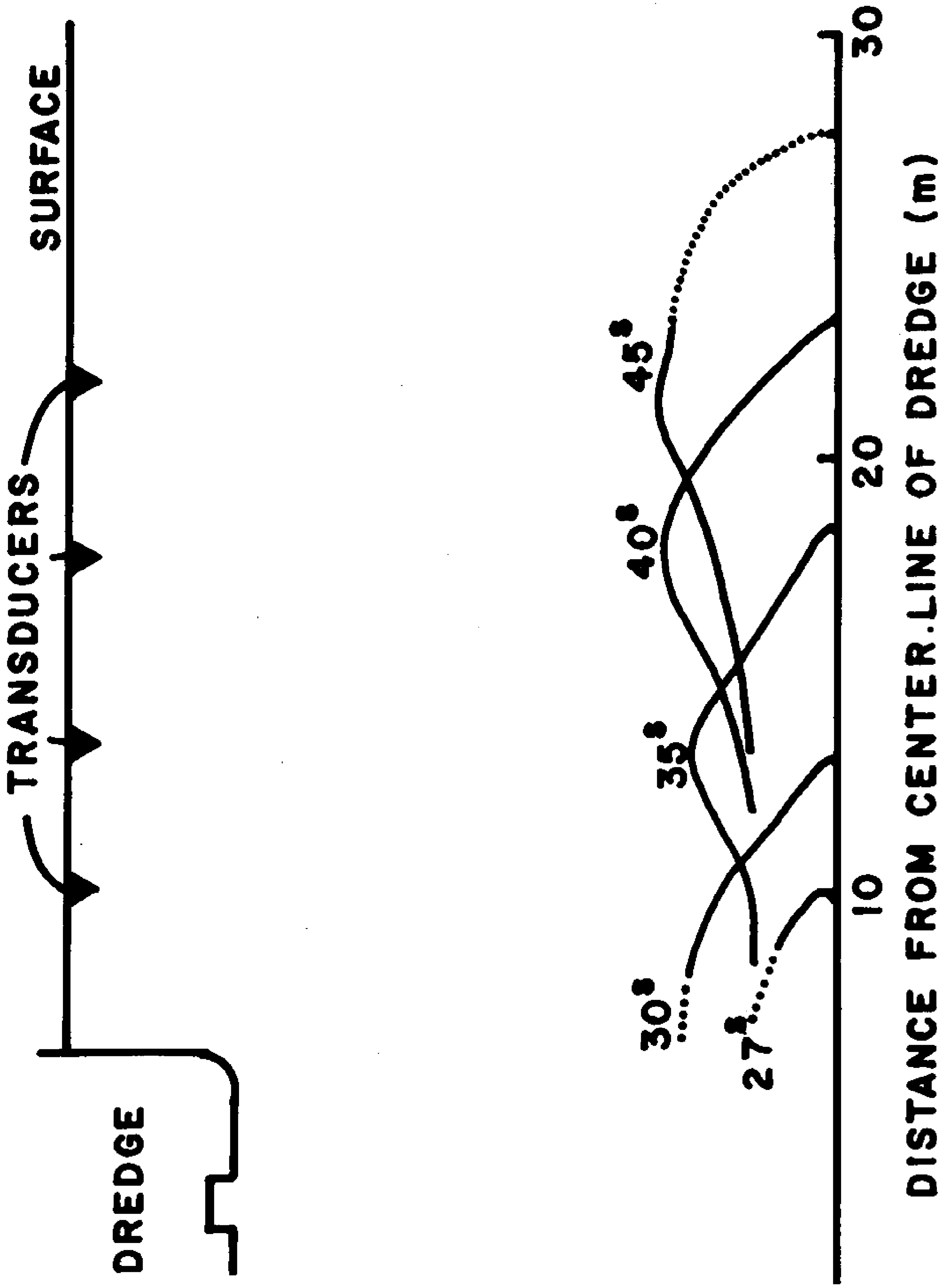

Figure 3. Development of the bottom surge as detected an array of acoustic transducers (fathometers) at intervals after the start of the discharge.

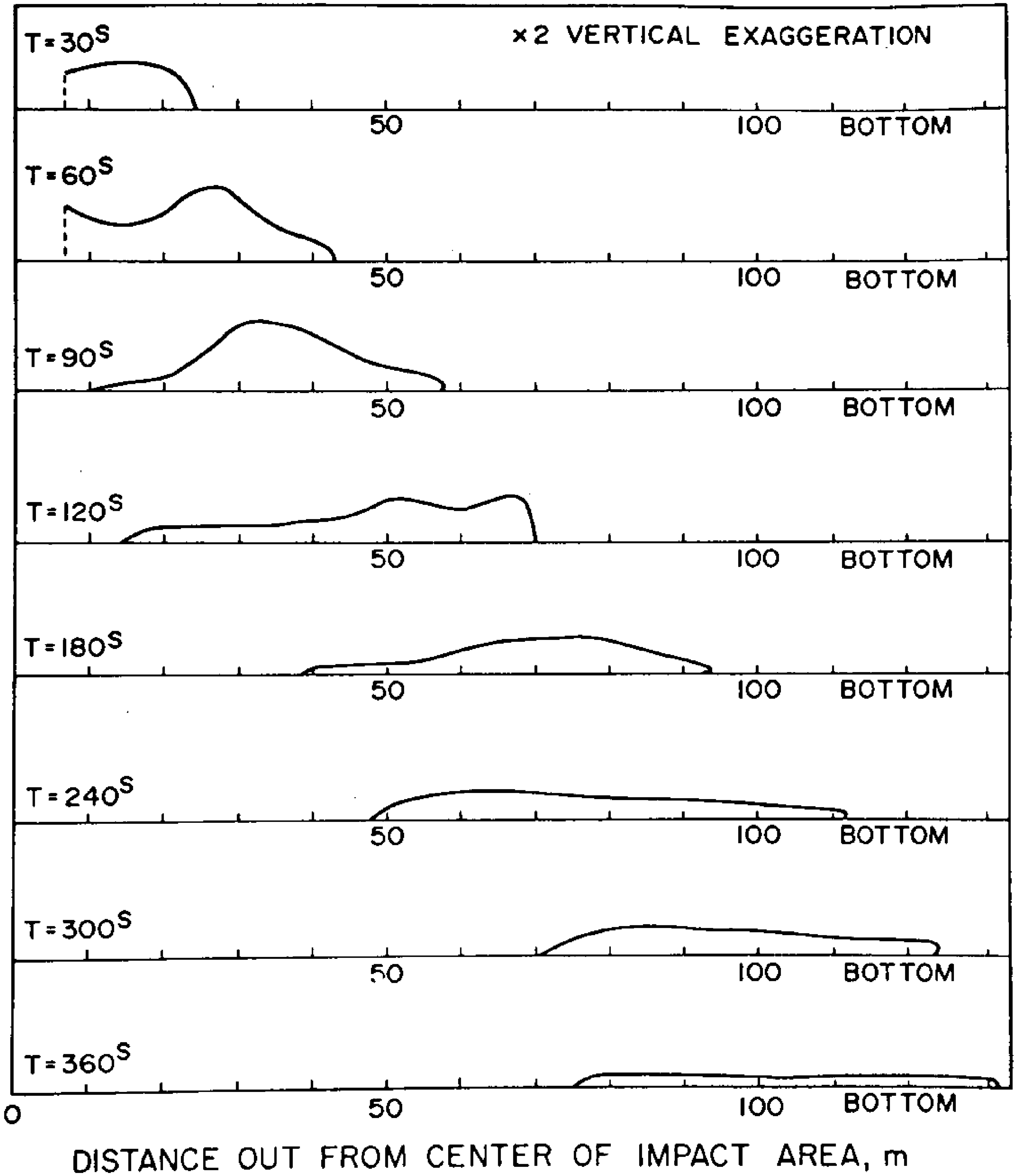

Figure 4. Cross-sections showing the evolution of the bottom surge at intervals after the start of the discharge (from Bokuniewicz and Gordon, 1978 c).

distribution of suspended material with the velocity measurements, and assuming radial symmetry, the total mass of solids in motion may be determined. This is shown in Figure 5. The velocities within the surge are initially high ($\gtrsim$ 1 m/sec). At these speeds no suspended material is deposited and some of the lake floor sediments may be eroded. The total amount of suspended solids increases slightly from the end of the impact phase, 90 seconds after the hopper doors were opened, to a time 120 seconds after discharge while the surge is between 15 and 69 m from the impact point. Not until the turbidity current has traveled to 93 m from the impact point does the mass of suspended material begin to decrease. At this time, 180 seconds after discharge, suspended sediment is lost from the surge at a rate of 10^3 kg/sec and settles to the lake floor at a rate of about 0.01 m/sec. If there is no lateral drift of the material during settling, it will be deposited in a ring around the impact point. In radial cross-section the deposit would look as shown in figure 6. Erosion has taken place within 50 m of the impact point to a depth of about 1 mm. Deposition has occurred in a ring between 50 and 160 m with a maximum thickness of 2.8 mm.

Many repeated discharges would most likely result in a thin deposit of low relief. The minimum radius of the deposit is determined by the range of the bottom surge. On a smooth flat bottom, a conical mound with a radius less than about 300 m cannot be formed by deposition from turbidity currents (Bokuniewicz and Gordon, 1978 a). The deposit must also have side slopes of less than 0.05. This is because a current running down this slope has its empirically determined rate of energy dissipation equal to the change in its potential energy due to the decrease in its elevation. While this is the case, the initial velocity of the surge will not diminish and no deposition will occur. For volumes of sediment less than 10^6 m^3 the radius is the limiting

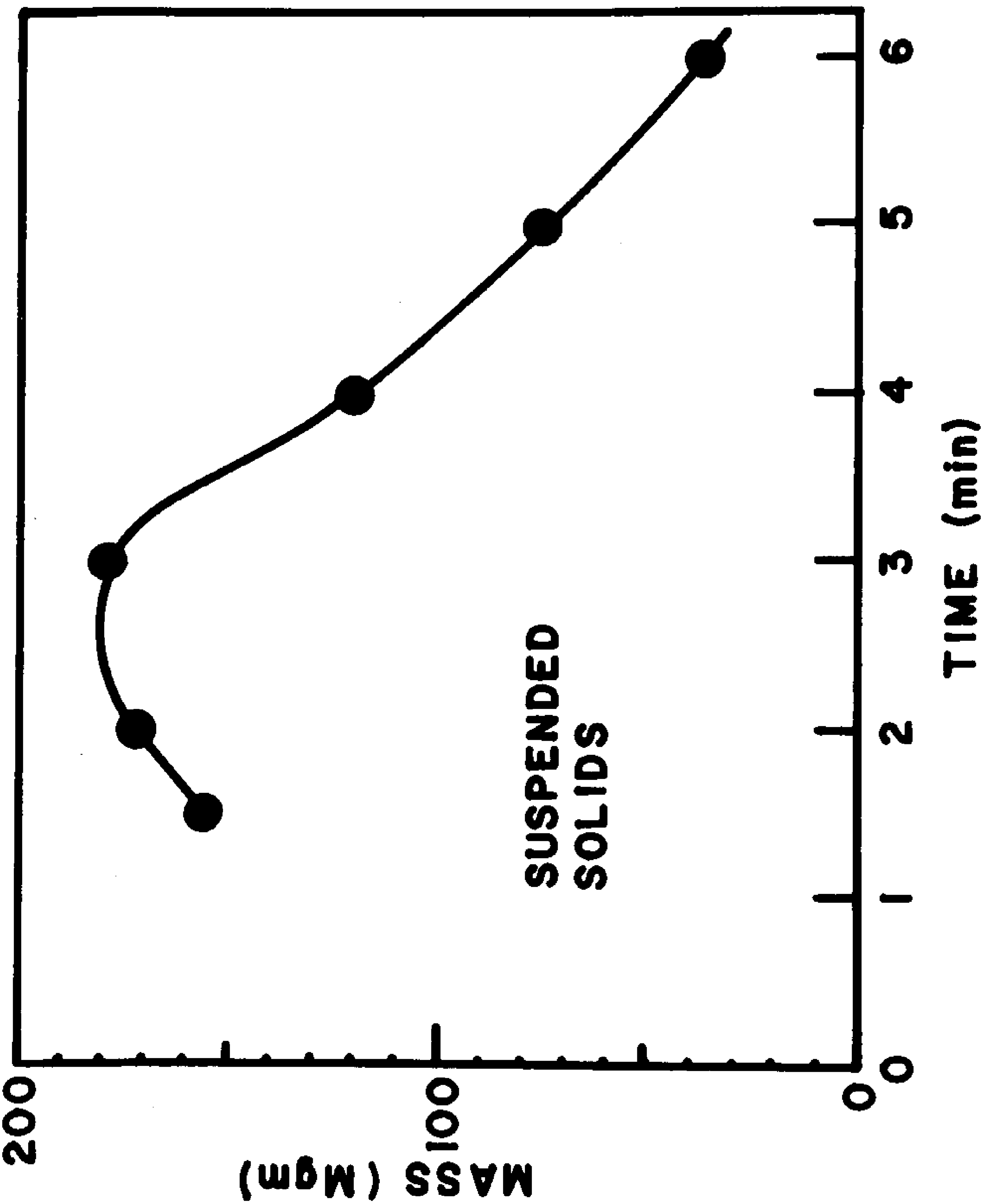

Figure 5. Total mass of solids in motion in the bottom surge at intervals after the discharge.

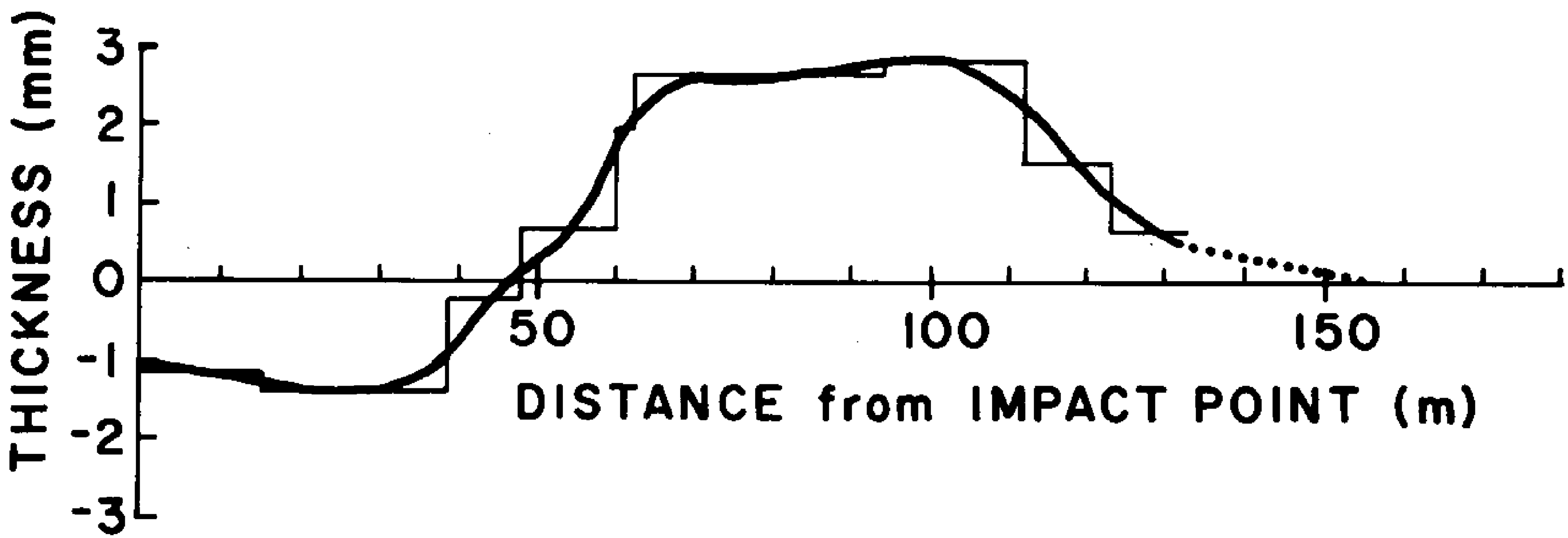

Figure 6. Cross-section through the deposit formed by a single discharge as calculated from the data in figure 5.

factor. With this constraint a deposit containing 10^5 m^3 has a maximum height of only 1.5 m.

A bathymetric survey was made of the Rochester disposal site. The spoiled area could be detected on the basis of its rough microtopography and changes in the reflectance of the bottom, but the relief was too small to be measured (Bokuniewicz, et al., 1978). At Ashtabula, the extent of the deposit of dredged sediment was measured with both sediment traps and graduated rods that were fixed in the lake floor before the disposal (Danek, et al., 1977). Seventy percent (18,000 m^3) of the released material was found to cover the 160,000 m^2 disposal area. Some of the missing material (an unspecified amount) was not found at the site because it had been released at another location. The average thickness would be less than 0.1 m and the maximum, observed thickness was less than 0.5 m. Such deposits present a large surface area for interaction with the overlying water column. This situation favors the subsequent dispersal of the material.

Dispersal from the Disposal Deposit

In areas where the currents are sufficient to regularly resuspend dredged sediment a large fraction of the dredged sediment could be resuspended and exchanged with the ambient sediments between successive discharges. In the coastal zone, for example, tidal currents typically disturb the top millimeter of sediment and the layer formed by a single hopper load is only a few millimeters thick. During the dredging of Mare Island Strait in San Pablo Bay, large quantities of dredged samples were found over an 8 x 10^7 m^2 area and about 10% of the dredged sediment was estimated to have returned to the channel (Custar and Wakeman, 1977).

Under certain conditions, however, the deposit should be able to be maintained indefinitely (Bukuniewicz and Gordon, 1978 a). This will be the case if, for example, the surface of the deposit is armored to resist erosion, or if the natural rate of deposition is sufficiently high so that there is no net erosion at the disposal site. Since the deposit has low side slopes, mechanical instabilities, such as slumps or slides, are unlikely. The deposit will, however, slowly consolidate with the expulsion of pore water.

The exchange of particulates between the deposit and the water will be limited to a thin layer at the sediment water interface. Initially, the thickness of this layer is the depth to which the sediment is disturbed by currents or wave activity, usually a few millimeters. After several months, however, the deposit may be recolonized by benthic fauna (Rhoads, Aller and Goldhaber, 1976; Saila, 1976). When this happens, the fraction of the dredged sediment that is mobilized is increased as the result of bioturbation. The sediment in the bioturbated layer will be put in direct contact with the water. This material may be subject to dispersion while the sediment below this depth will remain undisturbed and retained in the deposit. For an ideal conical deposit, the fraction of material, f, that is subject to mobilization is:

$$f = 1 - (1-d/h)^3$$

whew h is the height of the cone and d is the thickness of the bioturbated layer. In New England waters the bioturbated layer is typically 0.1 m thick. For this situation, even if net erosion does not occur and the deposit is constructed with the minimum possible surface area, complete exposure of the dredged material to the water may be expected in deposits containing less than 10^4 m^3.

Sub-bottom Containment of Dredged Sediment

Slow dispersion of dredged sediment from the disposal site results in dilute concentrations of dredged sediment distributed over a wide area. Dispersion accompanied by net erosion is useful because it permits a particular site to be re-used and therefore limits the area of the sea or lake floor that needs to be committed to the disposal of dredged material. Near-shore water, however, have a limited capacity to absorb this material without degradation (Carpenter, 1975). In addition, the long-run effects of sustained, low levels of contamination or of increased suspended sediment loads on the eco-system are not well known, although investigations are beginning to address this problem (e.g. Schubel, Auld and Schmidt, 1973). As a result, it is often desired to minimize the contact between the dredged sediment and the ambient water rather than to disperse and dilute the dredged material. This is especially the case for highly contaminated material.

Mined submarine pits may be useful as containment sites for fine-grained dredged sediment. There are several reasons for this. Offshore sands are being mined for construction aggregates in New York, New Jersey, Rhode Island, and California, and it appears that these offshore resources will be further developed in the future to avert serious shortages (Cruikshank and Hess, 1975). Simultaneously, new disposal sites must be designated in coastal areas to handle quantities of dredged materials now produced and likely to be produced in the future. In New York Harbor, for example, several large holes from previous mining operations already exist, and it is estimated that some 27×10^6 m^3 will be needed over the next three years (J. Marotta, New York Office of General Services, personal communication); in the New York area the maintenance of navigable waterways requires the removal of about 10×10^6 m^3 of sediment annually, most of which is fine-grained sediment, and

10 to 20% is highly contaminated (D. Suszkowski, Army Corps of Engineers, New York District, personal communication). The technology needed to carry out a back-filling operation is available (Johanson, et al., 1977) and, as mentioned earlier, the pits are natural traps for fine-grained sediment. Dredged sediment can be accurately sent to the pit floor although the bathymetry of the pits will modify the placement processes and deserves further attention.

Figure 7 shows two fathometer records across mined pits in Lower New York Harbor. The larger hole could contain a volume of about 20×10^6 m^3; the smaller, 1.6×10^6 m^3. The side walls of the pits are steep (10° to 15°), but below the angle of repose for sand (32°). The pit floors are irregular as a result of the dredging. Some parts of the record are characterized by a flat, diffuse reflection. These areas were identified as mud by bottom sampling. Core samples show a layer of mud overlying sand on the pit floor. In the smaller hole, this mud layer is about 0.45 m thick. In the larger hole the mud layer has been found to be as thick as 0.90 m (B. Brinkhuis, Marine Sciences Research Center, State University of New York, personal communication). This thickness of mud has accumulated since 1968 when mining operations in the pit were completed (B. Brinkhuis, per. com.), which means that the average sedimentation rate has been very rapid. Mud has accumulated at a rate of 0.045 m/yr in the smaller hole and about 0.09 m/yr in the larger pit.

The side slopes of the pits are sufficient to substantially limit the spread of the bottom surge. From the data used to construct the generalized sections shown in Figure 4, the total energy in the surge has been calculated (Bokuniewicz, et al., 1978) This is shown in Figure 8. Over a flat bottom, the spreading surge loses energy at a rate of about 0.044 Mjoules/m. The gain in the potential energy of the surge in running up various slopes has been calculated and is also shown in Figure 9. For a given slope, the inter-

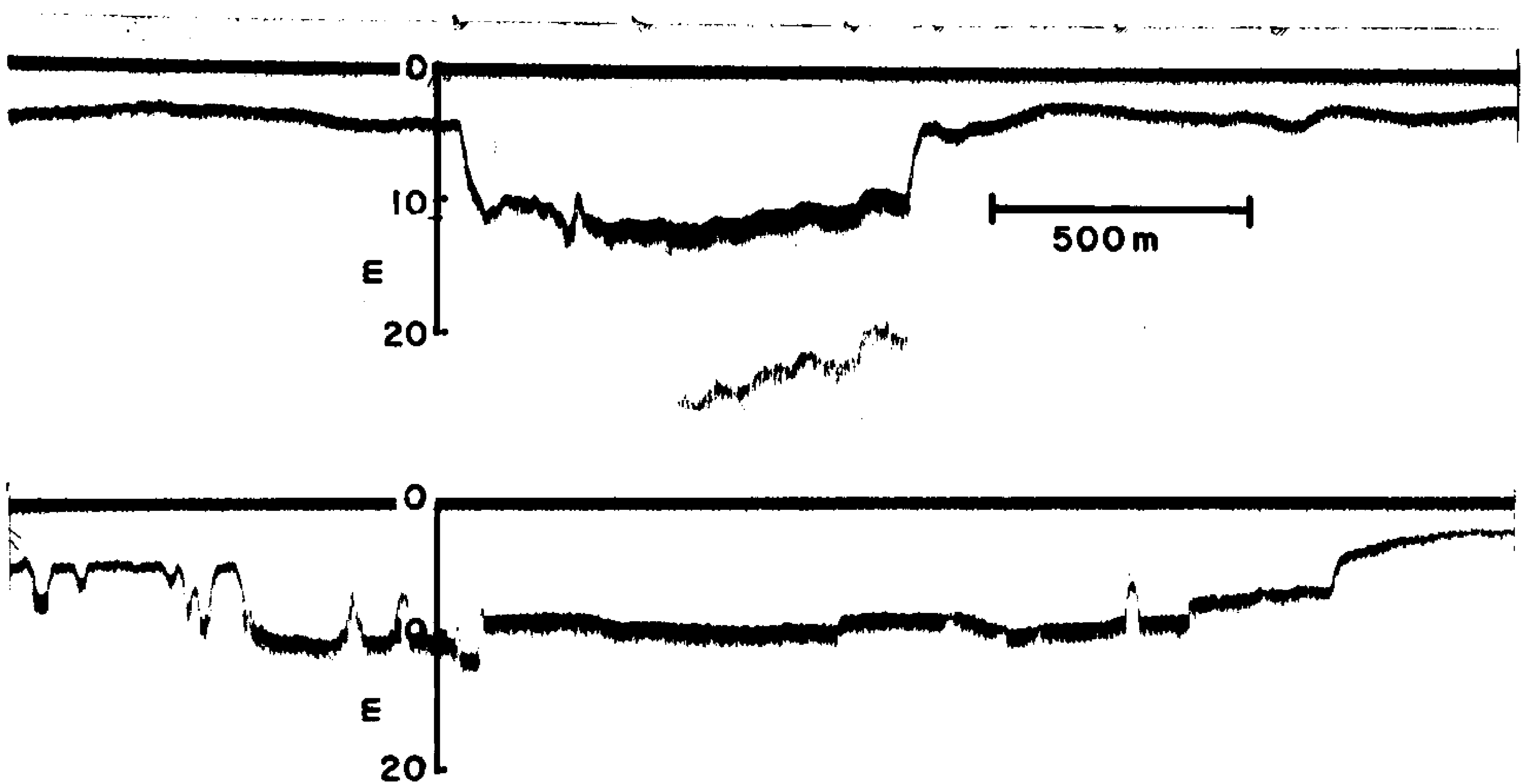

Figure 7. Fathometer records of pits in the floor of New York Harbor. These holes are the result of sand-and-gravel mining operations.

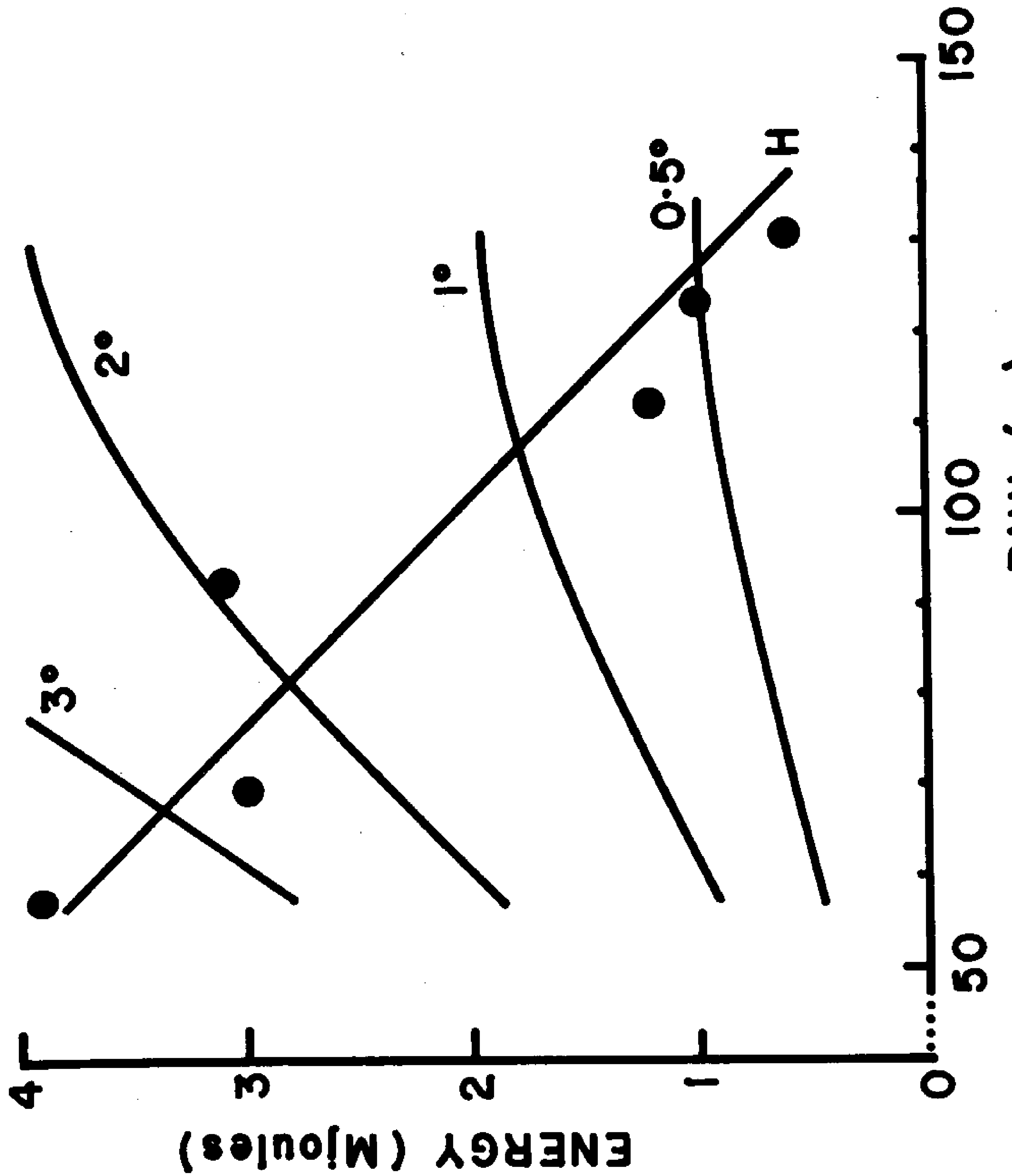

Figure 8. The solid circles indicate the total energy of the surge as a function of the position of the head of the surge. These data are approximated, for convenience, by the line H. Also shown is the calculated increase in the potential energy of the surge as it runs up various slopes.

section of the two curves indicates the greatest distance that the surge may travel up that slope. For example, a surge initiated at the center of a conical pit could not climb out of the pit if the walls had a slope of 1° and a length of 105 m. Of course, the curves shown in this figure will not be unaffected by changes in the discharge conditions. Any modification that will change the energy of the surge will affect its lateral spread. The discharge of a larger volume of material at higher speeds will, for example, extend the range of the surge (Bokuniewicz, et al., 1978; Bokuniewicz and Gordon, 1978 c). It seems clear, however, that the spread of the surge will be severely limited by the slopes of a few degrees.

The roughness of the bottom may also limit the travel of the surge due to increased energy dissipation at the sediment-water interface. If the densiometric Froude number, Fr, is small, there will be little mixing at the top surface of the surge (Middleton, 1966) and the velocity of the surge is expected to be directly proportional to some measure of the bottom friction like the Chezy coefficient (Kuenen, 1952). This seems to have been the case for the disposal operations in the Great Lakes, where $Fr \sim 1$ (Bokuniewicz, et al., 1978; Bokuniewicz and Gordon, 1978 c). If the surge encounters large abrupt bumps, however, the top surface may be disrupted; and suspended, dredged sediment may be injected into the overlying water. On the pit floors, the irregularities are typically less than a meter high and have side slopes of less than 10°. The question of whether or not these bumps would be sufficient to disrupt the spread of the bottom surge deserves further attention.

After deposition on the pit floor, the dredged sediments are subject to resuspension bioturbation and mechanical adjustment. If the natural rate of sedimentation is high, it is likely that no net erosion of the deposit will occur. In mined pits on the floor of New York Harbor, the sedimentation rate of fine-grained sediment is extremely high. In addition to reducing the net erosion of any deposits on the pit floors, this high sedimentation rate would discourage recolonization of the deposit by benthic animals. Bioturbation of dredged sediment would be reduced or eliminated so that the fraction of material in contact with the water would be limited to the thin, surface layer. These conditions enhance the probability of containment of dredged sediment on the disposal site.

Conclusion

The capacity of the coastal zone to absorb dredged material depends upon two elements. Part of the total capacity represents that volume of material that may be safely isolated and contained on the sea floor. The other part is that volume which may be acceptably suspended in the water column and the rate at which new suspended material may be introduced. As more is learned about the processes involved in the disposal operation, techniques may be better developed for the management of the discharge of dredged sediment in order to achieve dispersal or containment. For low levels of contaimination, it may be desirable to minimize the environmental impacts by diluting and dispersing the sediment over a wide area so that each location receives a negligible addition. For heavily contaminated material even small doses may be harmful, and it has been the policy to sacrifice some small area in order to isolate and contain this material there, Intermediate cases may also be considered in which a site is chosen and the disposal operation controlled to result in the slow release of dredged sediment from a deposit on the site by natural processes.

The release of fine-grained material from hopper dredges result in a deposit of low relief and a large surface-to-volume ratio. For volumes less than about 10^4 m^3 this situation favors the dispersal of the dredged sediment. The relative containment capacity increases as the volume increases. The long-run fate of the material depends also on the physical conditions at the site, specifically, the depth of resuspension and bioturbation and the natural sedimentation rate. Sites, such as mined pits, which have naturally high rates of sedimentation are potential containment sites. Of course, many other options are also available for controlling the fate of dredged sediment. For example, more complete isolation might be achieved by covering the deposit with clean material, probably sand. A sand cover would protect the deposit from disturbances during extreme hydrologic conditions, such as might occur during storms or floods. Capping operations, however, pose additional disposal problems. The first problem involves the emplacement of the capping material. Hopper discharges would result in a high-energy impact of the sand with the deposit of dredged sediment. Consequent erosion in the impact area would intermix the spoil with the cover material (Bokuniewicz and Gordon, 1978 c). Special equipment may be needed to cover the deposit (Johanson, et al., 1977). Even after the sand layer is emplaced, there may be difficulties in maintaining the cover due to biological mixing or mechanical instabilities of the sand-over-mud stratification.

As more is learned about the disposal processes, a wider range of management options is available. This type of planning, however, will require more control of the disposal operation than has been normally exerted. The disposal operation should be handled as an engineering project for building a deposit of dredged sediment on the sea floor.

Acknowledgements

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IMPACTS OF OPEN-WATER

DREDGED MATERIAL DISCHARGE

by

Richard K. Peddicord[1]

ABSTRACT

The concept and structure of the study of environmental impacts of aquatic disposal under the U.S. Army Corps of Engineers Dredged Material Research Program are outlined. Results of a number of field and laboratory research projects indicate that almost all the discharge material immediately impacts the bottom, perhaps forming fluid mud, leaving only a small percentage suspended in a turbidity plume. Potentially toxic contaminants and nutrients usually are not released to the water in quantities sufficient to cause concern. A possible exception is ammonia, which could reseach undesirable levels under some conditions. Turbidity and dissolved oxygen are very unlikely to be of ecological concern except on coral reef areas, although turbidity is a very real aesthetic problem. The material that impacts the bottom has an immediate and perhaps substantial physical impact. Some of the buried animals can exhume themselves, and recolonization of the site by larvae and/or mobile adults beings soon after disposal ceases. As this progresses and physical forces tend to return the site to its original condition, the evidence of environmental impact

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decreases and within a period of months to perhaps a year or two, the community structure at the disposal sites is usually similar to surrounding areas; although the species may be somewhat different. The uptake of metals and chlorinated hydrocarbons from deposited sediments and their incorporation into animal tissues was shown to be possible, but to be the definite exception, rather than the rule. Impacts at most aquatic disposal sites are physical and relatively short-term, a possible exception being those comparatively few sediments contaminated with more than several ppm PCB's.

INTRODUCTION

The River and Harbor Act of 1970 (Public Law 91-611, Section 123) authorized the Corps of Engineers to initiate and conduct a comprehensive nationwide study of dredging and dredged material disposal operations. Of particular interest were environmental impacts, productive uses of dredged material, and new or improved dredging and disposal practices. The U. S. Army Engineer Waterways Experiment Station (WES) was assigned responsibility for the research program, designated as the Dredged Material Research Program (DMRP).

The planning and implementation of the DMRP were the responsibility of an interdisciplinary team established at WES as part of the Environmental Laboratory (EL). The thrust of the program involved four major research projects:

a. Environmental Impacts and Criteria Development Project (EICDP).

b. Habitat Development Project.

c. Disposal Operations Project.

d. Productive Uses Project.

This report is primarily concerned with the findings of the EICDP, which was divided into four research task areas. These were: Aquatic Disposal Field Investigations (Task 1A), Movements of Dredged Material

(Task 1B), Effects of Dredging and Disposal on Water Quality (Task 1C), and Effects of Dredging and Disposal on Aquatic Organisms (Task 1D). The research for the last three of these tasks was, for the most part, carried out in the laboratory under controlled conditions. The results are useful, therefore, for understanding known impacts and for predicting others that may occur. They cannot, however, be directly applied to field conditions without verification, but can be considered as "worst case" evaluations. As such, they are useful in defining boundary conditions possible with aquatic discharge. Task 1A was a large-scale field study effort designed to provide definitive information on the environmental impact of dredging and disposal operations and, where undesirable impacts were observed, to suggest means of eliminating or reducing such impacts. This included studies on water and sediment quality, impacts on bottom animals, and the rate and extent of the recolonization of disposal sites by bottom organisms, and responses of swimming and free-floating organisms to disposal.

The field studies were viewed somewhat as demonstration cases to verify in the natural environment the responses studied in detail in the laboratory research. Thus, in a sense they tie the entire Environmental Impacts and Criteria Development Project together. This paper summarizes the findings of the field studies with supporting detail from the other tasks as appropriate.

The basic approach involved the selection of field sites on the basis of representativeness of different geographic regions (environments) and disposal operations. Appropriate strategies were then developed for the collection and analysis of biological, chemical, and physical samples. Samples were taken during controlled disposal operations and compared to samples obtained under baseline conditions and from reference sites.

Although there were site-specific modifications, the general schedule consisted of predisposal surveys to establish baseline (ambient) conditions, one or more disposal operations with frequent sampling to determine acute impacts, and postdisposal monitoring to assess chronic impacts, recolonization by benthic organisms, and the rate of return to predisposal conditions. Whenever possible, physical, chemical, and biological data were obtained concomitantly so that cause-and-effect correlations could be investigated.

The research at each site was conducted through interagency agreements and contracts by various agencies, institutions, and private firms. This resulted in a series of site-specific reports; these reports were published as appendices to summary reports for each site. These summary reports and their appended contract reports, together with the other EICDP reports are available as cited in this paper.[2] Because of the diverse audience for which it was prepared, this paper is nontechnical in the sense of presenting detailed information. Those desiring more specific information are encouraged to consult the field study site summary reports and the technical reports of the other EICDP and DMRP studies.

DISCUSSION OF FINDINGS

As dredged material is discharged, a number of complex chemical and physical events occur which, prior to the completion of aquatic disposal

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[2] U. S. Army Engineer Waterways Experiment Station, P. O. Box 631, Vicksburg, MS 39180, Attn: Ms. D. P. Booth. When supplies are exhausted, copies will be obtainable from the National Technical Information Service, 5205 Port Royal Road, Springfield, VA 22151.

research in the DMRP, were poorly understood. Indeed, many could only be hypothesized from a theoretical standpoint although some had been demonstrated in laboratory studies.

Upon discharge from a hopper or barge dredged material usually falls as a coherent unit that entrains ambient water and descends as a dense mass. Water column interaction is minimal as descent to the bottom occurs in a matter of seconds.

In those unusual circumstances where the material does not descend to the bottom as a cohesive mass, the opportunity exists for it to interact with the water column. This usually occurs only in cases of extreme water depth where the dense mass may entrain enough ambient water to create a neutrally buoyant plume. In this case, maximum water column interaction occurs with relatively little bottom impact. Such interaction may result in the formation of a turbid plume and the exchange of chemical substances between the dredged material and the water column. This interchange depends on a number of variable factors such as particle-size distribution, the chemical nature of the sediment and the water column, the presence of currents, and variable water density. These interactions will tend to be minimized if the sediment is of such a nature as to descend as a more or less cohesive unit.

All disposal operations create a turbid plume of some description, the duration of which depends on particle size, currents, turbulent mixing, and similar phenomena. A turbid plume composed of very fine particles will persist longer than one made up of coarser particles. Depth can be a factor as, in many instances, bottom waters are more dense than surface waters. A plume which has disappeared from the surface may persist at intermediate depths or near the bottom because of differential rates of particle settling.

Ultimately, the disposed sediment will reach the bottom. If it is cohesive and falls as a mass it may produce a mound and existing sediment may be displaced with a turbidity and/or shock wave which travels outward from the impact point.

If the material is not cohesive, it will tend to settle more uniformly upon the bottom. A pronounced mound may not be present and a greater area will be covered with a lesser thickness of material. Under most field conditions, a combination of these two types of impact is expected because the dredged material is generally heterogeneous.

The material discharged from a hydraulic pipeline dredge is a slurry which usually disperses in three modes. Any coarse material, such as gravel, clay balls, or coarse sand, will immediately settle to the bottom of the disposal area and usually accumulate directly beneath the end of the discharge pipe. The vast majority of the fine-grained material in the slurry also descends rapidly to the bottom where it forms a low gradient circular or elliptical fluid mud mound (Nichols et al., 1978). A small percentage (1 to 3 percent) of the discharged material is stripped away from the outside of the slurry jet as it hits the water surface and descends through the water column and remains suspended in the water column as a turbidity plume (Schubel et al., 1978).

The levels of suspended solids in the turbidity plume created in the water column above the fluid mud layer generally range from a few tens of milligrams per litre to a few hundred milligrams per litre. Concentrations rapidly decrease with increasing distance downstream from the discharge point and laterally away from the plume center line due to settling and horizontal dispersion of the suspended solids (Barnard, 1978). Under tidal conditions, the plume will extend inland during the incoming (flood) tide and seaward during the outgoing (ebb) tide. The plume length will seldom be more than slightly longer than the maximum distance of one tidal excursion (i.e., the distance that the suspended sediment is transported during an ebb or flood tide). In rivers where the flow is unidirectional, the plume length is controlled by the strength of the current and the settling properties of the suspended material. In both estuarine and riverine environments the natural levels of turbulence and the fluctuations in the rate of slurry discharge will usually cause the idealized teardrop-shaped plume to be distorted by gyres or eddylike patterns (Barnard, 1978).

Whereas a small percentage of the fine-grained dredged material slurry discharged during open-water pipeline disposal operations is dispersed in the water column as a turbidity plume (Schubel et al., 1968), the vast majority rapidly descends to the bottom of the disposal area where it accumulates under the discharge point in the form of a low gradient mound of fluid mud, commonly referred to as "fluff," overlying the existing bottom sediment (Nichols et al., 1978). Although there is no universally accepted definition of fluid mud, at the overlying water/fluid mud interface concentrations are approximately 10 g/ℓ. This material may be stationary or may freely flow outward, away from the discharge point of an open-water pipeline disposal operation, like syrup poured on a platter, or downslope as a mudflow. At an approximate solids concentration of 200 g/ℓ fluid mud possesses a certain degree of rigidity and will not normally flow freely as low-density fluid mud may (Migniot, 1968). If the discharge is moved as the dredge advances, a series of mounds will develop. The majority of the mounded material is usually high-density (nonflowing) fluid mud that is covered by a surface layer of low-density (flowing or nonflowing) fluid mud. The short- and long-term dispersion characteristics of the discharged slurry depend on many factors, including the nature and rate of slurry discharge, the discharge configuration, and the hydrodynamic regime and bottom topography in the disposal area. With time fluid mud gradually consolidates to densities typical of fine-grained sediments and becomes indistinguishable from the natural bottom.

Regardless of the method of dredging and disposal, once the material impacts the bottom and fluid mud consolidation begins, the material may remain in place for a long period of time or may undergo relatively rapid erosion and dispersal. Which event (or combination) occurs depends on the nature of the material and bottom currents. The latter, of course, are influenced by depth and the adjacent subaqueous topography. After deposition, whether or not extensive erosion and movement occurs, the dredged material may become mixed and incorporated with the underlying natural sediment.

These events are of concern because of the potential effects thay they may have upon biological communities. To discuss these in proper perspective, the general nature of the various communities involved and the components of disposal which may impact them is required.

The pelagic community might conceptually be expected to receive the initial impact of disposal. This community consists of plants and animals which have low mobility and which tend to drift with currents (plankton) as well as organisms with moderate to high mobility (such as fish). If disposal did release contaminants (such as metals, ammonia, pesticides, etc.) pelagic organisms in the plume might suffer adverse impacts. This would be of greater significance to planktonic organisms than to more mobile ones because the latter, (if they could detect the toxic material,) could leave the area. If the plume is moving, planktonic organisms may be carried with it and experience a longer exposure time than mobile animals.

Laboratory studies (Burks and Engler, 1978) indicated that soluble metal release to the water column during disposal is generally small, because metal oxides are relatively insoluble. In some cases, hydrous iron oxide scavenges other heavy metals from the water column and reduces their concentrations. Only manganese was observed to be released in solution to the water column to any extent during disposal. The release was transient, however, and a return to ambient conditions usually occurred within minutes to hours. There did not appear to be any effects on the pelagic community as a result of the increase in manganese.

Some of the heavy metals appeared to be released to a slight degree at some of the sites. These releases did not follow a consistent pattern and the importance of the observations is not clear. As with manganese, the releases were small and did not persist.

With the exception of PCB's at one site, there was no significant release of oil and grease or chlorinated hydrocarbons into the water column. These compounds are quite insoluble in water and readily sorb upon particulate matter, so little release was expected (DiSalvo et al., 1977). In the case of PCB's in Elliott Bay, the EPA criterion for these compounds was exceeded; however, the background concentration in the receiving water also exceeded the criterion. Actual increases due to disposal over the high background values were quite small and transient and did not appear to be of particular biological significance.

Bioaccumulation phenomena conceptually could also affect pelagic organisms. These consist of the accumulation or concentration of substances from the external environment to higher concentrations within an organism. Although commonly referred to as "food-web magnification," this concept is generally misapplied to aquatic organisms. Unlike terrestrial organisms, which do concentrate substances from lower to higher trophic levels, aquatic organisms tend to bioaccumulate directly from the environment through respiratory and other external body surfaces. Hence, if soluble substances were released into the water column during disposal then they could be incorporated into the body tissues of aquatic organisms. Such effects were not demonstrated to occur at any of the field study sites with any toxic material (Wright, 1978). The laboratory studies indicated that while such effects are possible, they must definitely be expected to be the exception rather than the rule (Hirsch et al., 1978; Neff et al., 1978; Brannon, 1978).

Because a significant component of the pelagic community consists of plants (phytoplankton), the potential impact of nutrients is of concern. An excess of plant nutrients (especially phosphorus or nitrogen) above a limiting concentration can bring about a "bloom" or shift in species dominance. As these plants are planktonic, they will tend to move with the impacted portion of the water column and have a maximum opportunity to react to the presence of excessive nutrients. Phosphorus is generally limiting (in short supply) in freshwater while marine systems are most often limited by nitrogen. In an estuary, where marine and freshwater systems mix, either element may be limiting, and the controlling factor may change on an almost daily basis.

The plant nutrients, phosphorus and nitrogen, were released to the water column at most of the ADFI sites. Phosphorus release was quite common but persisted only for minutes to hours. Similar releases have been reported by other investigators (Sly, 1977) in evaluations of dredged material disposal.

Nitrogen was released at most of the ADFI sites in the form of ammonium (NH_4^+-N). This converted to ammonia (NH_3) in the disposal site water at a pH near 8. Although plants can use ammonia as a source of nitrogen, primary concern centered on the toxic effects of ammonia. As with phosphorus,, the elevated levels of ammonia in the water column were of short duration. As the oxidation of ammonia to nitrite and nitrate is quite slow and since ammonia is not readily sorbed by particulate matter, the observed return to ambient conditions most probably resulted from dilution. It is thought that the concentration-exposure time relationships (Brannon, 1978) were such that no damage occurred to pelagic organisms. Because of ammonia's potential toxicity, ammonia concentrations should be carefully monitored during disposal (Burks and Engler, 1978).

As with nutrients, turbidity induced by disposal could affect the phytoplankton by decreasing the amount of light that is available to them. Such a decrease, if it persisted for a significant period of time or over a large area, could reduce photosynthesis and decrease the productivity of the system because phytoplankton, rather than rooted plants, are the basic primary producers for open-water communities. Such effects, however, have not been demonstrated relative to dredged material disposal either by the DMRP field studies (Wright, 1978), or in the open literature (Stern and Stickle, 1978). The latter authors reviewed the literature and found little basis for the conceptual fears of direct or indirect impact of turbidity from dredging operations on aquatic ecosystems. This is supported by the laboratory research of Peddicord and McFarland (1978). Although dredging-induced turbidity may often be an aesthetic problem, rarely is it likely to be an ecological one, except in the proximity of coral reefs (Stern and Stickle, 1978).

The pelagic community could also conceptually be affected by reduction in dissolved oxygen if the disposed sediment has a high immediate oxygen demand. As with toxicity, this effect in part depends upon concentration-time of exposure relationships, as most organisms can withstand a moderate decrease in dissolved oxygen for a relatively long period of time whereas a slightly greater decrease may not be at all tolerable. Because of a variety of chemical interactions, if anoxic conditions were to occur, they could increase the damage potential of toxic substances. Decreases in dissolved oxygen in the receiving water were insignificant at all field study sites (Wright, 1978) and only rarely are likely to be of any consequence (Brannon, 1978).

Essentially no biological effects were demonstrated at any DMRP field study site as a result of water column changes during disposal operations. There were a number of physical and chemical changes which, when they occurred, were of low magnitude, short duration, or both. Only in rare cases were existing criteria exceeded and, even then, these "worst situation" instances were such that concentration-time of exposure considerations (Brannon, 1978) seem to preclude significant biological impacts.

When the disposed material settles upon the bottom the benthic community may be impacted. This community consists of mobile and nonmobile organisms. Among the former are fish and some invertebrates, while the latter consist almost entirely of invertebrates. Sessile organisms may either burrow in the sediment or live primarily at the sediment-water interface. The bottom-dwelling invertebrates are often of direct commercial importance (shrimp, crabs, lobsters, mollusks, etc.), and, even when they are not, they form an extremely important component of the food of sport and commercial fish. Most of the significant impacts associated with disposal at the DMRP field study sites occurred in the benthic community and primarily affected invertebrate organisms. Demersal finfish were little affected (Wright, 1978).

Whether dredged material impacts the bottom in solid form or as fluid mud, it buries those benthic organisms which it covers. Depending on the nature of the material, a drastic habitat change can occur. This will be most severe when the disposed sediment is quite different from the existing bottom as, for example, when fine material is placed on coarse sand or vice versa.

Most species of organisms normally found on sandy or muddy bottoms are more or less mobile, especially as juveniles. Few mud or sand dwellers are sessile (fixed to the bottom). The mobile organisms have various capabilities for moving through newly deposited dredged material, to reoccupy positions relative to the sediment-water interface similar to those maintained prior to burial by the disposal activity. Benthic organisms such as mud crabs and amphipods having morphological and physiological adaptations for crawling through sediments are able to migrate vertically through deposits of tens of centimetres. Vertical migration ability is greatest in dredged material similar to that in which the animals normally occur and is minimal in sediments of dissimilar particle-size distribution. However, results also showed broad variability in migratory abilities, suggesting physiological status and environmental variables to be of great importance to vertical migration ability (Maurer, 1978).

In general, many disposal sites are in areas which would be considered to be subject to natural physical stresses because normal environmental conditions are variable rather than stable. Organisms which occur under such conditions are generally able to better withstand physical stresses and recover more rapidly than those in physically stable environments (Oliver et al., 1977). Estuaries are typical of naturally stressed environments because of the high variability which results from the interactions of fresh and marine waters. Likewise, nearshore or shallow areas which are subject to wave action and/or high current velocities present a physically stressful environment for many organisms. This is particularly true when the substrate consists of relatively coarse sand which is constantly being shifted about by waves and currents.

If toxic substances are present in the disposed material in a biologically active and/or available form, the benthic community may be adversely affected. Such substances can include metals, pesticides, oil and grease, PCB's, ammonia, sulfides, and similar elements and compounds. Dredging and disposal do not introduce new contaminants to the aquatic environment, but at worst simply redistribute the sediments which are the natural depository of contaminants introduced from other sources. After disposal, these substances may conceptually remain in toxic forms in the sediment and may also move across the sediment-water interface into the water column.

The potential for bioaccumulation is of considerable concern in regard to the benthic community because the organisms present are for extended periods in close proximity to substances which have uptake potential. Unlike pelagic organisms, where exposure time is apt to be of short duration and transient, benthic organisms which burrow in or live upon the surface of the disposed material may undergo lifetime exposure. In addition, many benthic organisms are deposit feeders; that is, they ingest large quantities of sediment. While the sediments are passing through the digestive tract of these organisms, changes in pH, digestive enzymes, and other factors may conceptually increase the mobility of some substances (especially metals) and perhaps cause them to be absorbed into the tissues. Moreover, as carbon dioxide is given off, a microzone of reduced (acidic) pH is often observed. This could enhance release of metals and other substances.

For most metals studied in both the laboratory and the field, uptake by organisms was not evident. In the more detailed laboratory studies when uptake was shown to occur, the levels often varied from one sample period to another and were quantitatively marginal, usually being less than one order of magnitude greater than levels in the control organisms even after 1 month of exposure (Neff et al., 1978). It is invalid to compare metals levels in organisms to total or bulk sediment concentration since only a variable amount of the sediment-associated metal is biologically available (Brannon, 1978).

Of a total of 168 animal-sediment-salinity combinations evaluated in tests carried out by Neff et al. (1978), only 22 percent showed significant accumulation due to sediment exposure. The largest uptake was of iron, a metal generally known for its low degree of toxicity in biological systems. Their literature search showed that heavy metals in solution vary over several orders of magnitude in availability to benthic invertebrates. Although extensive accumulation of heavy metals by organisms from the water has been documented, the literature shows no such clear evidence for accumulation of metals from the sediments.

Both Neff et al. (1978), in the laboratory, and Anderlini et al. (1976a), in field work and back-up laboratory experiments, have found the same heavy metal phenomena. The accumulation and release of certain heavy metals seems to vary with the metal, with the species, between sampling times, between sampling sites, and within controls. These variable results have not been directly correlated with dredging operations or sediment loading.

Results indicated that selected estuarine and freshwater organisms can be exposed to dredged material that is contaminated with thousands of parts per million oil and grease and experience minor mortality for periods up to 30 days. Uptake of hydrocarbons from the heavily contaminated sediments appears minor when compared to the hydrocarbon content of the test sediments, and when compared to uncontaminated organisms (DiSalvo, et al., 1977).

Studies conducted on the adsorption and desorption of chlorinated hydrocarbons and PCB's by sediments have generally indicated that these materials are much more readily sorbed than desorbed. On the basis of laboratory studies, it appears that release of these water-insoluble pesticides will not occur to an appreciable extent during disposal (Fulk et al., 1975). Anderlini et al. (1976b) monitored release from sediments and uptake by organisms of PCB's and compounds of the DDT group during a disposal operation in San Francisco Bay. Some uptake of p,p'-DDE was observed but the levels of the other chlorinated hydrocarbons remained constant in bay mussels.

The environmental interpretation of bioaccumulation data is very difficult because in most cases it is impossible to quantify either the ecological consequences of a given tissue concentration of a constituent that is bioaccumulated, or even the consequences of that body burden to the animal whose tissues contain it.

In general, disposal of dredged material at the ADFI sites demonstrated few significant impacts. This is not surprising, as many of the laboratory studies and other investigations of dredged material disposal under conditions similar to those at the ADFI sites also failed to demonstrate that many of the conceptually anticipated impacts actually occurred.

It was not possible to establish a cause-and-effect relationship between the biological changes that did occur at the field study sites and the disposal of dredged material, with the possible exception of benthic community changes resulting from direct burial. In general, the abundance and number of species decreased temporarily immediately following disposal. It appears that this effect was caused by burial although the influence of chemical factors cannot be completely discounted.

Disposal did not appear to have any lasting effect on the sediment chemistry. There were some small changes in dissolved oxygen, metals, and nutrients but these did not appear to be large enough to have a significant impact on the benthic community. There was little evidence of biological uptake of oil and grease (DiSalvo et al., 1977) or heavy metals (Neff et al., 1978) in the laboratory. Likewise, there was virtually no evidence of contaminant bioaccumulation under field conditions at the ADFI sites.

There appeared to be some degree of short-term avoidance of the disposal site by finfish at several of the sites; at another, however, there was evidence of greater numbers of finfish after disposal. Some question exists as to whether this behavior represented avoidance of the material or was a result of the normal seasonal movements of fish (Wright, 1978).

A degree of interpretative judgement was required in evaluating the overall ecological significance of the observed changes in the benthic community. Little is known of the role that many of the organisms play in the entire ecosystem. Although recolonization of the impacted area usually took place within months, the colonizing organisms were often different from those which had been present prior to disposal. This change probably represents successional phenomena, and, if the sites were to be revisited in 2 to 5 years, the original communities may be found to have returned. Alternately, habitat alteration (i.e., a change in the physical nature of the substrate) by disposal may favor the more or less permanent establishment of a community quite different from that which previously existed. Hirsch et al. (1978) documented a number of instances where habitat change and succession have taken place following dredged material disposal.

The physical habitat alteration resulting from dredged material disposal may persist for long or short periods of time (Holliday, 1978). This depends on the nature of the material and the effectiveness of natural phenomena in restoring predisposal conditions. At one study site, dredged material migrated outward from the center of the disposal area; as it did, benthic communities were affected. At other sites, there was a reasonably rapid return to predisposal conditions so far as physical and chemical characteristics of the sediment were concerned, but this was not accompanied by a concurrent return of the benthic community to predisposal conditions.

Where changes in the benthic community did occur as a presumed effect of dredged material disposal, there is little that can be said as to whether these changes were adverse. As noted above, many of the communities are poorly understood and the substitution of one species assemblage for another cannot be easily evaluated. In general, a decrease in biomass or in the number of organisms present would be considered undesirable as would the establishment of a completely different community from that which existed prior to disposal. On the other hand, it appears that many years of disposal at the ADFI site in Long Island Sound was, at least in part, responsible for the creation of conditons which have led to increased populations of lobsters. Likewise, open-water disposal in Lake Superior resulted (at least on a short-term basis) in an increase of organisms which are considered to be an important component of the diet of fish species of recreational and commercial importance (Wright et al., 1975). In the former instance an enhancement seemed to result from the dredged material providing a more suitable substrate for burrowing animals such as lobsters, and, in the latter, the deposition of organic material upon a relatively sterile bottom increased the population of detritus feeders.

The ADFI were primarily concerned with impacts within a designated disposal area. This focus is important in application of the results since impacts <u>not only are expected</u> but also <u>are permitted</u> within a disposal area. To prohibit impacts within a disposal area would be as irrational as prohibiting impacts of solid waste disposal within a sanitary landfill site; it is recognized that disposal will have an impact and that such an impact may be deleterious within the disposal area. In essence, a worst-case approach was employed in that it was assumed that, if impacts were minimal within the disposal area, they would almost certainly be less outside of the disposal area. There is no firm reason to suspect that this was not the case, but it should be recognized that a lack of effects outside the disposal area is, in general, assumed and has not been exhaustively demonstrated.

CONCLUSIONS

It appears that open-water disposal of dredged material may generally have a negligible impact upon physical, chemical, and biological variables. However,the impacts that were observed in the field or indicated in the laboratory studies were usually site-specific, suggesting that the results cannot be universally applied or cited as being conclusive in all situations.

The release of manganese and ammonia during and after disposal may pose a problem in some cases, and there is limited evidence that this conclusion may also apply to iron, mercury, and PCB's. This factor must be addressed by adequate biochemical evaluation prior to dredging and through the use of the appropriate regulations concerning discharge evaluation procedures. Aquatic disposal does cause temporary physical effects on the benthic community, but the ecological significance of the effects is not clear. There is a general lack of understanding concerning the ecological role of most benthic organisms; a shift in community structure, organism abundance, or other parameters is almost impossible to categorize as good, bad, or indifferent. Most of the impacts appeared to be physical in nature (burial or smothering) although it was not possible to completely rule out chemical (toxic) effects.

Overall, most impacts seemed to be relatively short-term. The condition of the water column associated with disposal generally returned to ambient within minutes to hours. Chemical changes in the sediment persisted for days to weeks (where they occurred at all), while physical changes often lasted for several months. An exception concerned PCB's; however, PCB's are a rather unusual constituent of dredged sediment, and the fact that they were detectable long after disposal at some sites is not an indication that other contaminants behave in a similar manner.

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DREDGED MATERIAL: A MANAGEABLE RESOURCE

By Thomas R. Patin[1]

ABSTRACT

For the most part, dredged material produced as a result of dredging operations to maintain the nation's navigable waterways has been treated as a waste product. However, the Corps of Engineers (CE) has changed its perspective towards dredged material to view it as a potentially productive resource. To foster this new philosophy, the Productive Uses Project has been included as an integral part of the Dredged Material Research Program (DMRP) being conducted at the U. S. Army Engineer Waterways Experiment Station in Vicksburg, Mississippi. The objectives of this project are to identify and assess concepts for the productive use of dredged material and dredged material containment areas and to develop specific information and guidance to help district offices implement productive use concepts.

This paper discusses the findings of the Productive Uses Project. The major topic areas to be covered will include:

a. Upland disposal concepts development

b. Land improvement concepts

c. Products development

d. Disposal area land use concepts.

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INTRODUCTION

In March 1978, the Environmental Laboratory at the U. S. Army Waterway Experiment Station completed the 5-year Dredged Material Research Program (DMRP). The overall objective of the DMRP was to determine the environmental impacts of dredged material disposal and to develop feasible alternatives to enhance the beneficial and reduce the adverse impacts of both land and water disposal (4).

Productive Uses

A major concern of the DMRP is to consider dredged material a manageable resource. This consideration, explicitly stated in the program's objective, is the guiding philosophy of the Productive Uses Project (PUP), one of four project areas within the DMRP (Figure 1). In considering dredged material a resource, a dual objective is achieved. Not only is dredged material disposal conducted in an environmentally compatible manner, but a resource normally wasted is put to productive use. This productive use includes not only the use of the disposal site but also the use of the dredged material.

The basic objective of the PUP is to provide definitive information to Corps' Districts and other interested parties on the feasibility of using dredged material productively as an alternative disposal method. The approach was to identify the potential productive uses, then through

DREDGED MATERIAL RESEARCH PROGRAM
TECHNICAL STRUCTURE

Project/Task		Objective
Environmental Impacts and Criteria Development Project		
1A	Aquatic Disposal Field Investigations	Determine the magnitude and extent of effects of disposal sites on organisms and the quality of surrounding water, and the rate, diversity, and extent such sites are recolonized by benthic flora and fauna.
1B	Movements of Dredged Material	Develop techniques for determining the spatial and temporal distribution of dredged material discharged into various hydrologic regimes.
1C	Effects of Dredging and Disposal on Water Quality	Determine on a regional basis the short- and long-term effects on water quality due to dredging and discharging bottom sediment containing pollutants.
1D	Effects of Dredging and Disposal on Aquatic Organisms	Determine on a regional basis the direct and indirect effects on aquatic organisms due to dredging and disposal operations.
1E	Pollution Status of Dredged Material	Develop techniques for determining the pollutional properties of various dredged material types on a regional basis.
2D	Confined Disposal Area Effluent and Leachate Control	To characterize the effluent and leachate from confined disposal facilities, determine the magnitude and extent of contamination of surrounding areas, and evaluate methods of control.
Habitat Development Project		
2A	Effects of Marsh and Terrestrial Disposal	Identification, evaluation, and monitoring of specific short-term and more general long-term effects of confined and unconfined disposal of dredged material on uplands, marsh, and wetland habitats.
4A	Marsh Development	Development, testing, and evaluation of the environmental, economic, and engineering feasibility of using dredged material as a substrate for marsh development.
4B	Terrestrial Habitat Development	Development and application of habitat management methodologies to upland disposal areas for purposes of planned habitat creation, reclamation, and mitigation.
4E	Aquatic Habitat Development	Evaluation and testing of the environmental, economic, and engineering feasibility of using dredged material as a substrate for aquatic habitat development.
4F	Island Habitat Development	Investigation, evaluation, and testing of methodologies for habitat creation and management on dredged material islands.
Disposal Operations Project		
2C	Containment Area Operations	Development of new or improved methods for the operation and management of confined disposal areas and associated facilities.
5A	Dredged Material Densification	Development and testing of promising techniques for dewatering or densifying dredged material using mechanical, biological, and/or chemical techniques prior to, during, and after placement in containment areas.
5C	Disposal Area Reuse	Investigation of dredged material improvement and rehandling procedures aimed at permitting the removal of material from containment areas for landfill or other uses elsewhere.
6B	Treatment of Contaminated Dredged Material	Evaluation of physical, chemical, and/or biological methods for the removal and recycling of dredged material constituents.
6C	Turbidity Prediction and Control	Investigation of the problem of turbidity and development of a predictive capability as well as physical and chemical control methods for employment in both dredging and disposal operations.
Productive Uses Project		
3B	Upland Disposal Concepts Development	Evaluation of new disposal possibilities such as using abandoned pits and mines and investigation of systems involving long-distance transport to large inland disposal facilities.
4C	Land Improvement Concepts	Evaluation of the use of dredged material for the development, enhancement, or restoration of land for agriculture and other uses.
4D	Products Development	Investigation of technical and economic aspects of the manufacture of marketable products.
5D	Disposal Area Land-Use Concepts	Assessment of the technical and economic aspects of the development of disposal areas as landfill sites and the development of recreation-oriented and other public or private land-use concepts.

NOTE: This technical structure reflects the second major program reevaluation made after the second full year of research accomplishment and is effective as of August 1975.

Figure 1

research determine technical, environmental, and economic feasibility, and finally, provide guidelines for their implementation.

To achieve these goals the PUP was divided into four task areas: (1) Upland Disposal; (2) Land Improvement; (3) Products Development; and (4) Disposal Area Land Use. The purpose of this paper is to describe the research that has been conducted in these task areas and present some of the findings and conclusions that have been drawn.

Upland Disposal

Locating new land disposal sites for dredged material disposal is traditionally a function of the various Corps District offices or sponsoring agencies. Additionally these selections have generally been based on the economics involved and resulted in sites near the dredging operation and were normally located in the coastal or lacustrine zones. Because of various social, economic, and legal considerations, these zones have, in many cases, become practically "off limits" for dredged material disposal. This realization has prompted the DMRP to look into the feasibility of using upland disposal sites some distance inland from the dredge site. Once inland, the material could possibly be used productively for reclaiming quarries and sand pits or strip-mined land, etc.

An array of technical, economic, environmental, social, and institutional factors must be addressed for successful inland disposal and productive uses. SCS Engineers, under contract to the Waterways Experiment Station (WES) (18), conducted an in-depth literature review with the primary objective being the feasibility of inland disposal. According to SCS, inland disposal and finally productive uses are feasible, although care should be taken that all factors be addressed properly. The report delineates these factors and provides guidance on properly addressing them.

Additionally, the report contains a comprehensive checklist that decision-makers could use as a basis for determining potential inland disposal sites. The checklist is designed to identify all factors involved and is designed to be used for (a) site selection, (b) project planning, and (c) identifying major problem areas.

The second research effort conducted under this task was a comprehensive evaluation of transportation alternatives available for movement of dredged material inland. Souder et al. (16) conducted a cost and engineering evaluation of long-distance transport of dredged material by pipeline (both hydraulic and pneumatic), barge, rail, truck, and conveyor belt systems. Cost and "how to" data were developed, and although hypothetical, these should offer sound information for planning and designing for long-distance transport systems. Concept systems are developed for each transportation mode to guide the planner or designer in evaluation. It is realized that many major cost items are very site-specific and could not be adequately covered within the text. Therefore, such items as specific terrain conditions, transportation route, right-of-way, water quality standards, noise standards, etc., are flagged, discussed, and referenced for more in-depth study.

Land Improvement

The land improvement task within the PUP was basically directed at determining the technical feasibility of enhancing nonproductive land with dredged material. The emphasis was on fine-grained dredged material and the specific topics addressed were:

a. Dredged material as a soil for reclamation of strip-mined land.

b. Dredged material as an agriculture soil and/or soil amendment.

c. Dredged material in conjunction with solid waste management.

Although legislation exists today which directs reclamation of all future strip-mined land, there still exist thousands of acres of barren strip-mined land from previous mining operations. It is felt that the Corps could provide a valuable resource through the use of fine-grained dredged material as a cover for barren acid-producing strip-mined land (17).

A field demonstration of strip-mined land reclamation with dredged material has been conducted within the PUP (9). The demonstration area is near Ottawa, Ill., and had been mined in the 1930's and was essentially devoid of any vegetation (Figure 2). Fine-grained dewatered dredged material was transported 70 miles from a confined disposal site in Chicago, Ill., to the Ottawa site. Three feet of dredged material was placed on the barren strip-mined land to establish vegetation in the area and reduce acid runoff by shutting off the supply of oxygen and water to the underlying pyrite material.

The dredged material and the strip-mined material were chemically and physically characterized prior to movement; after movement grasses were planted to stabilize the dredged material and leachates; runoff and vegetative growth were and are still being monitored. Figure 3 is a July 1978 photo of one of the test plots.

A second important research effort under the land improvement tasks is that of determining the potential of dredged material as an agriculture soil or as an amendment to a nonproductive soil.

It is realized that the soils of the rich deltas of such rivers as the Mississippi and the Nile are essentially dredged material deposited at some earlier date. It is also known that agriculture has been practiced on dredged material sites at a number of areas in the United States and

Figure 2: Strip-mine site, Ottawa, Ill., before dredged material application.

Figure 3: Strip-mine site after application of fine-grained dredged material.

Europe. For instance crops have been grown on disposal sites in Savannah and Charleston Corps Districts. Cattle have grazed on disposal sites in the Tulsa District. The problem however was that these were essentially happenstance and could offer no concrete guidance for large-scale future use of dredged material in agriculture.

In an effort to produce such guidelines the PUP contracted the USDA Science and Education Administration for the purpose of evaluating dredged material as an agriculture soil and/or soil amendment. Dredged material from ten disposal sites within the eastern and central United States was used in greenhouse plant growth experiments. After collection the samples were physically and chemically analyzed, mixed, and seeded with rye and barley (6). The crops were harvested three times for plant productivity measurements.

Generally, the addition of a fine-grained dredged material to a coarse-grained nonproductive soil will increase plant productivity over that of the nonproductive soil. Therefore, it was concluded that dredged material can be used for increasing agricultural production when mixed with marginal agricultural soils. However, caution should be exercised in using dredged material which has weeds, is high in soluble salts, and with higher than normal concentrations of heavy metals.

A third possible productive use pursued in the Land Improvement Task is that of using dredged material in solid waste management. This research effort, conducted in-house at WES, examined the physical properties of dredged material to evaluate its potential in sanitary landfills (2).

It was found that once dredged material is dewatered it is essentially a soil and can be treated as such. The fine-grained dredged material could be used for covers and liners whereas the coarse-grained materials could be used as gas vents and leachate drains. The study also takes the idea one step further and develops some concepts for the actual use of dredged material in sanitary landfills.

The final research effort in this task was the production of a synthesis report or guidelines for the use of dredged material in land improvement. These guidelines cover the use of dredged material in strip-mine reclamation, agriculture use and sanitary landfills (17) and are basically a culmination of the findings of the Upland Disposal Concept Development and Land Use Concept Tasks within the PUP plus input from other selected DMRP work units. Other DMRP tasks (Figure 1) that supplied major input to final feasibility are: Dredged Material Densification and Disposal Area Reuse Tasks in the Disposal Operations Project and the Confined Disposal Area Effluent and Leachate Control Task in the Environmental Impacts and Criteria Development Project.

Products Development

The objective of the Products Development Task was to determine if marketable products can be developed from dredged material, whether it be the use of the material itself or possibly the use of the disposal site. It was reasoned that if it can be shown that a product of substantial economic value can be produced from dredged material or the disposal site, the incentive for a landowner to allow his land to be used for dredged material disposal purposes may result. The approach was to identify the products and determine their feasibility.

In 1974, a concept study was conducted to determine the potential of using containment areas for the production of lawn sod, horticultural crops, etc. (1) The study concluded that commercial production of horticultural crops on disposal sites is possible although subject to a number of constraints such as reuse of the site for disposal of dredged material, soil conditions such as salinity, marketing problems, site size, site location, etc.

A second products study was conducted to determine the feasibility of using active and/or inactive disposal areas for mariculture purposes (11). Part of the study was a literature review in which it was concluded that approximately 400 different species of animals and plants could possibly be raised if the correct environmental setting could be developed. The second part of the study was a pilot shrimp mariculture experiment in which brown shrimp (Penaeus aztecus) were successfully raised in two small ponds in which fine-grained dredged material had been placed.

Based on the findings of the pilot study, a semi-prototype shrimp mariculture field project was conducted (10). Twenty acres of an existing 158-acre disposal site in Freeport, Texas, were diked off and seeded with approximately 700,000 juvenile shrimp in September 1976. Results have shown that even without feeding the shrimp grew from approximately ½ in. to about 5 in. in 40 days. This is comparable to growth of shrimp in their natural environment. Complete grow-out was not possible because of the unseasonably severe cold weather in the fall of 1976. On the other hand, it is felt that if the shrimp could have been put in the pond earlier, much larger shrimp could have been harvested. An economic as well as technical analysis is included in the final report.

Bricks and synthetic aggregates also hold some possible application for products development from dredged material. No specific research efforts were pursued in this area within the PUP because of ongoing research elsewhere within the United States. On the other hand, these efforts were closely monitored. The basic conclusion was that these products could be produced although market conditions for such products presently do not provide sufficient incentive for commercial development (12).

Disposal Area Land Use Concepts

The objective of the fourth task within the DMRP was to obtain information to facilitate planning and implementation of concepts for the ultimate use of dredged material containment areas. It would be hard to find a major port today that has not used material dredged from harbors or waterways to create new land for development. However, in many past cases where dredged material has been confined, there has been little thought given to the subsequent land use of containment areas. Often, the dredged material containment area is left as a wasted resource, or haphazardly developed, not in harmony with nearby land use. Research in this task generally addressed the "softer" issues associated with the ultimate use of dredged material containment areas. The task examined existing concepts for the use of dredged material to create land and concurrently assessed the economic, technical, environmental, institutional, legal, and social incentives and constraints to the development of dredged material containment areas. From this research, a rational basis for deciding upon the site selection, ultimate land use, and management of the created land can be attained (19).

The research first documented the many different examples for the productive use of existing dredged material containment areas. The examples were obtained from published literature, project descriptions, and discussions with persons knowledgeable about aspects of dredged material disposal. All non-habitat uses of dredged material containment areas were covered in the study (3).

The following land use categories are recognized as existing and/or potential categories where productive uses of dredged material containment areas are possible:

- recreational
- industrial/commercial
- agricultural
- material transfer
- waterway related
- multiple purpose
- public service/municipal

The results further indicate that the more successful recent ventures had better site selection methodologies and overall planning management.

The second and perhaps one of the most important research efforts conducted in the task was a study of 12 selected cases where land use of dredged material containment areas was a specific objective (7). This study sought to discover what issues were raised during the projects, why some issues were more important than others, and how the issues were addressed. Along with the issues the study identified physical planning elements affecting disposal facility and productive land use planning.

This study produced a list of implementation factors for the disposal and productive use of dredged material containment areas. These factors were both substantive and procedural, addressing the full range of planning and engineering problems. The factors are broken down into: environmental, technical, economic/financial, legal, institutional, and planning implementation categories. These 37 factors address the full range of substantive and procedural considerations that are necessary when contemplating a productive use of dredged material containment areas. Table 2 lists the implementation factors.

The documentation of examples of dredged material land use showed that the most prevalent use of dredged material containment areas was recreational. An earlier study conducted under the task examined the potential for creating recreation land from dredged material (15). In particular, the research looked at whether land created from dredged material could fulfill urban

Table 2: Factors Affecting Disposal-Productive Land Use Projects (7)

ENVIRONMENTAL

1. Ecological characteristics of proposed disposal area location.
2. Environmental impacts of disposal-productive use projects.
3. Dredged material pollution properties.

TECHNICAL

1. Dredged material structural properties.
2. Disposal area subsurface conditions.
3. Disposal facility design and operating characteristics.
4. Site size and configuration (as related to productive use).
5. Technical coordination of disposal plan with productive use plan.

ECONOMIC/FINANCIAL

1. Economic or social benefits (costs of the disposal-productive use project).
2. Engineering and construction costs.
3. Dredged material transport costs.
4. Fees or taxes on dredged material.
5. Project sponsor capability to assume financial responsibilities.

LEGAL

1. Conformance with regulatory requirements.
2. Adequacy of environmental impact assessment or statement.
3. Disposal rights to the site.
4. Site ownership authorities (as related to productive use).
5. Land use restrictions.

INSTITUTIONAL

1. Public participation in disposal-productive use planning.
2. Coordination with project sponsor.
3. Coordination with review-regulatory agencies.
4. Coordination with planning agencies.
5. Procedures for identifying and resolving objections to the project.
6. Corps and other participant attitudes.
7. Political, business, and public support.

PLANNING/IMPLEMENTATION

1. Long-range Corps disposal planning.
2. Long-range waterway/environmental planning.
3. Dredging project specification.
4. Temporal coordination of disposal plan with productive use plan.
5. Availability of environmental data.
6. Evaluation of alternative disposal areas.
7. Impacts of disposal-productive use project on existing water uses.
8. Proposed use compatibility with adjacent land uses.
9. Proposed use compatibility with master plans.
10. Proposed use compatibility with available transportation systems and infrastructure.
11. Proposed site plan compatibility with site physical features and user requirements.
12. Commitment to proposed land use plan.

demand for recreation. Most urban areas suffer from a severe lack of recreational opportunities, especially for the poor and the aged. Possibly, the productive use of dredged material to create land could alleviate this lack of recreational opportunity. The socio-economic aspects of such an undertaking were examined during numerous visits to CE Districts and planning agencies with jurisdiction in waterfront areas.

The study found that dredged material containment areas in urban areas could be developed into needed recreational facilities despite constraints associated with the quality of the material, the single purpose nature of dredging projects, and competing land uses. Proper employment of the multi-objective planning process in conjunction with local authorities including projects sponsors, port commissions, and similar agencies would be required.

A more complex question arises as to the funding of such recreational projects. The study concludes that present funding regulations do not appear adequate to foster the development of dredged material containment areas as recreational sites. Perhaps, provisions for modifications to present policy and regulations (5) can be made to promote such a concept.

The fourth study conducted under the task was designed to identify and evaluate laws and regulations affecting the land use of dredged material containment areas (13). All Federal statues and a selected number of state and local statutes were included in the scope of the study.

The Federal legislations considered to be the most important impacting the land use of dredged material containment areas are:

- The National Environmental Policy Act.
- The Federal Water Pollution Control Act Amendments of 1972.
- The Coastal Zone Management Act.
- The Fish and Wildlife Coordination Act.
- The Endangered Species Act of 1973.

Of these, the Coastal Zone Management Act is of particular interest. Under this act, each state is required to develop a plan for its own coastal zone. Once this plan is developed and approved under the act, all Federal agencies must comply with the intent of the plan (8). Thus, the CE may have to work with the Coastal Zone Management agencies of individual states to effectively plan for the land use of dredged material containment areas.

The report also speculates about future trends in both state and Federal law. The impact of these trends on the CE is discussed along with an evaluation of their effect on the land use of dredged material containment areas. It is also an excellent compendium of existing laws and regulations, although these laws are subject to rapid change.

Despite the recent environmental upswing, economics still plays a vital role in the success of any project. Therefore, the final study conducted in the task attempts to develop a methodology for determining the value and associated benefits of the land use of dredged material containment areas (14). Again, the case study approach was used to document the value of existing projects and to extrapolate this knowledge to a general methodology for predicting the value and associated benefits of future projects. Thus far, a framework methodology has been developed that can be used by planners and engineers to predict the value of a proposed land use of dredged material containment ares.

The study has documented a number of case studies in which the land value created from dredged material is significant when compared with other project benefits. A preliminary suggestion is that the CE should consider the ultimate land use of the dredged material obtained from navigation projects in the cost-benefit analysis of such projects.

The final and most important output of this task is the set of guidelines for the productive uses of dredged material containment areas. These guidelines are aimed at the planning and implementation considerations and are essentially a culmination of the most significant and relevant information of the other research efforts in this task.

The audience for this report is not only the engineer and scientist, but the planner as well. Many of the findings are in terms of general planning considerations that should be of concern to all disciplines. The planning considerations presented are based on results of the research and are most important for the success of a productive land use product.

The "soft issues" (social, economic, legal, and institutional) more often than not determine the success or failure of a dredging and/or disposal project, and to properly address all these factors a systems approach should be used. In conjunction with the systems approach, a planning process is required that would allow CE planners and engineers to work more closely together. This multi-objective planning process approach already adopted by the CE for use with other water and related land resource projects (5) should be used for the productive land use of dredged material containment areas.

The research work accomplished under the DMRP has helped to identify and provide solutions to many of these problems. However, the proper application of a multi-objective planning process is needed to ensure that all factors are properly addressed and the concerns of all interested parties are included. The combination of a systems approach and the multi-objective planning process can bring about the productive land use of dredged material.

SUMMARY AND CONCLUSIONS

The disposal of dredged material is a major problem confronting coastal and waterway areas in the United States. However, numerous examples of the productive uses of dredged material show that it can indeed be a valuable resource.

The PUP and the DMRP have established the technical feasibility of productive uses of dredged material. On the other hand, there are a number of policy and planning issues that must be addresssed to enhance the product use alternative to conventional disposal. These policy and planning issues were developed after examination of the myriad of problems that impede the wide use of productive use options. Until they are addressed the productive use of dredged material will not be fully realized (20).

These issues are listed below:

Policy Issues

1. Corps advocacy role in disposal-productive use planning.

2. Corps advisory role in disposal-productive use planning.

3. Evaluation criteria for disposal-productive use alternatives.

4. Financing of disposal-productive use projects.

5. Application of the Principals and Standards and Corps multi-objective planning procedures to disposal-productive use planning.

6. Expansion of Corps role in Corps-sponsor relationships for operations and management.

7. Legislative recognition of disposal-productive use concepts.

Planning Issues

1. A multidisciplinary team approach to disposal planning in Corps District offices.

2. Encourage more cooperative interagency, intergroup participation in planning disposal-productive use options.

3. Development and application of a holistic or systems approach to dredging-disposal-productive use project planning.

4. Establishment of long-term, regional, comprehensive plans for dredged material disposal-productive use alternatives.

5. Development of land use planning expertise within the Corps.

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DREDGINGS CONTAINMENT AREAS AS SEDIMENTATION BASINS

by

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ABSTRACT

The sedimentation regime that exists in a dredged material containment facility affords one of the primary means by which suspended solids can be controlled in the supernatants that are discharged from the disposal site. The concentration and nature of the suspended solids in the effluent supernatants depend on a multitude of factors, including the concentration and nature of the inflow slurry, the size of the disposal area, the relative locations of the discharge pipe and the effluent sluicing device, the degree of channelization in the flow, the retention time of the fluid, the direction and velocity of the wind, and the extent of the vegetation.

The solids retention capability of eleven disposal areas was determined on the basis of the suspended solids concentration in the influent slurry and the effluent waters. The performance of containment areas as sedimentation basins is strongly correlated with the hydraulic efficiency of the area. Insufficient retention time due to shallow ponding depth, small surface area, and excessive inflow rate has a detrimental effect on the settling effectiveness of disposal areas. Predictions based on conventional sedimentation theories tend to underestimate the performance of the dredgings containment areas that were studied.

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SETTLING PROCESSES

Sedimentation basins for the removal of suspended particles have been used extensively for the treatment of water, sewage, and industrial wastes, and they are now finding applications in other areas, such as the disposal of dredged material. In general, the phenomenon of sedimentation depends on the characteristics of the suspended particles and the sedimentation basin. Theories to describe sedimentation in an ideal regime of horizontal laminar flow were developed over 70 years ago (Hazen, 1904) and subsequently modified and extended (Camp , 1946). Methodologies were advanced to account for the effect of non-ideal mixing and dispersion in real basins, and significant effort has been put forth in recent years to develop models that predict sedimentation patterns for cases of discrete or flocculated particle settling.

Settling Velocity of Suspended Particles

Particles settling out of suspension acquire a velocity which depends on their size, shape, and volumetric concentration, as well as the viscosity and density of the fluid. Appropriate flocculating agents are often used to increase settling velocities by causing particle agglomeration. A discrete particle is one that does not alter its size, shape, and weight during settling. When such a particle settles through a quiescent fluid, it acquires a uniform settling velocity, v_s, which, for low Reynolds numbers ($R \leq 1$), can be written as

$$v_s = \frac{\gamma_s - \gamma_f}{18\mu} D^2 \quad \text{(Stoke's Law)} \qquad (1)$$

where γ_s is the unit weight of the solid particle, γ_f is the unit weight of the fluid, μ is the absolute (dynamic) viscosity of the fluid, and D is the equivalent diameter of the settling particle.

The settling of a group of particles is different from that of a single particle due to interference among the individual particles. For low concentrations of solids this effect is insignificant, and the settling may be reasonably assumed to be discrete. When the concentration increases, the particles reduce the area through which the displaced fluid moves upward, and this results in an increased fluid velocity and a lower settling velocity. The terminal settling velocity of a particle in hindered settling, v_h, is usually expressed in terms of its terminal velocity in discrete particle settling, v_s, as

$$v_h = v_s \, f(C_v) \tag{2}$$

where $f(C_v)$ is a function of the volumetric concentration of suspended particles. Many attempts have been made to take hindered settling into consideration, and a variety of formulae have been proposed to calculate values for the function, $f(C_v)$ (Hauksby, 1951; Richardson and Zaki, 1954; Loeffler and Ruth, 1959; Oliver, 1961). Although most of these formulae are quite complicated, Richardson and Zaki (1954) proposed the following expression, which is based on extensive experimental investigations and widely used because of its simplicity:

$$f(C_v) = (1 - C_v)^n \tag{3}$$

where the exponent, n, is determined experimentally; a value of 4.65 is recommended (Lin, 1976) for settling of fine soil particles.

Various theoretical models have been developed to describe the kinetics of particle aggregation during flocculation (Overbeck, 1952; Jovanovic, 1965). In general, these studies have ignored the influence of certain important factors by introducing simplifying assumptions, thereby casting suspicion on the ensuing results and recommendations. Available experimental information on flocculent settling (Camp, Root, and Bhoota, 1940; McLaughlin, 1961; Hamin, 1967) is limited and somewhat unreliable, due primarily to the poor replication of flocs with identical properties.

Zone settling frequently occurs in wastewater treatment plants that employ activated sludge and flocculation processes. If the concentration of the activated sludge and flocculated chemical suspensions exceeds 500 mg/ℓ, the floc particles will adhere together and the suspended mass will settle as a blanket (Eckenfelder, 1966) which forms a distinct interface between the flocculated sediment and the supernatant liquid. Zone settling and hindered settling are frequently used interchangeably, although there exists a distinct difference between the two. Strictly speaking, hindered settling develops if the concentration of discrete particles is high, while zone settling is intimately related to flocculent suspensions.

Settling Basin Performance

The performance of sedimentation tanks or basins is evaluated in terms of the suspended particle removal effectiveness that can be achieved during the time that waters are retained in them. Under ideal conditions the retention time of waters is equal to the theoretical retention time, but conditions is sedimentation basins often deviate from ideal and their removal effectiveness may be reduced accordingly. In order to devise a framework for the formulation of discrete particle settling in continuous-flow basins, the following simplifying assumptions must be introduced (Fair, Geyer, and Okun, 1968): (a) Within the settling zone of the basin, sedimentation takes place exactly as in a quiescent suspension of equal depth, (b) the flow is steady, and, upon entering the settling zone, the concentration of suspended particles of each size is uniform throughout the cross-section normal to the flow, and (c) a particle that settles is not resuspended. The proportion, P, of removed particles with a settling velocity, v_s, is given (Hazen, 1904; Fair, Geyer, and Okun, 1968) by

$$P = \frac{v_s}{Q/A} \tag{4}$$

where Q is the mass flow rate, A is the surface area of the basin, and Q/A is called the surface loading.

The effectiveness of settling basins is reduced by (a) eddy currents caused by the inertia of the incoming fluid, (b) wind-induced currents when basins are not covered, (c) thermal convection currents, and (d) density currents (cold heavy water flows beneath the warm lighter water of a basin surface). Each of these currents may contribute to upsetting the settling process. According to the classical Hazen theory, the proportion, P, of particles removed in a real basin, where currents reduce the efficiency, is given (Fair, Geyer, and Okun, 1968) by

$$P = 1 - \left[1 + \frac{nv_s}{Q/A}\right]^{-1/n} \tag{5}$$

where n is a performance coefficient for the basin and ranges from zero (best performance) to unity (very poor performance).

In an ideal basin fluid displacement is steady and uniform, and each unit volume of fluid is theoretically retained for the same period of time. However, even in well-designed settling basins, some of the inflow reaches the outlet in less than the theoretical retention time and some takes much longer. Thus, only a certain portion of the surface area of the basin is active, while little or no flow passes through the remaining space. This phenomenon (short-circuiting) may be caused by one or more of the following factors: (a) inadequate mixing in the basin, (b) high inlet and outlet velocities, as compared with the translational velocity in the basin, and (c) density or temperature differences. The analytical description of short-circuiting is difficult due to the complex nature of the phenomenon. Although model studies can be used, the scale effects distort the results and create difficulties in applying them to prototype conditions. Tracer studies in the prototype itself produce the most reliable results (Camp, 1946; Fair, Geyer, and Okun, 1968), but such studies, conducted after a basin has been actually constructed, lead only to remedial measures.

Based on assumptions similar to those introduced to determine basin effectiveness in discrete particle settling, O'Connor and Eckenfelder (1958) developed a method to estimate the percentage of suspended particles removed by flocculent settling in sedimentation basins. Subsequently, a model describing the processes of flocculent settling was developed by Vold (1963) and supported by the findings of other investigators (Sutherland, 1967; Lagvankar and Gemmell, 1968; and Javaheri and Dick, 1969). A general simulation model for discrete and flocculent settling in ideal basins has been developed by Chang (1972), and Lin (1976) extended this model to include complete-mix conditions with or without hindered settling and short circuiting. However, the results of these studies are primarily qualitative and do not lead to a simple methodology or approach for the design of real sedimentation basins.

APPLICATION TO DREDGINGS CONTAINMENT AREAS

A dredged material containment facility can be visualized as consisting of three zones. In the first zone, which consists of the vicinity around the discharge pipe, the fill surface varies significantly, channelization of flow occurs, slurry concentration is very high, and sediments are frequently disturbed and resuspended because of disposal operations. In the second zone, which can be considered to act as a sedimentation basin, a slow, essentially horizontal flow prevails in a completely and continuously inundated area with a relatively constant width. In the third zone, which consists of the vicinity around a sluicing device (usually an overflow weir), flow converges, both horizontally and vertically, towards the area of release, and flow velocities increase with decreasing distance from the sluicing device.

Dredged material consists of particles that range in size from gravel and sand to silt and fine clay. Correspondingly, the shape of dredged material particles varies from well-rounded to rod-like and disk-like. Equation 1 can be used for spherical particles that are less than 100 μ in diameter, because Reynolds numbers are less than unity. For rod-like and disk-like spheroidal particles, this velocity is reduced by up to 25% (Fair, Geyer, and Okun 1968). Thus, the discrete settling velocity of particles in the clay-size portion of dredged material would be somewhat smaller than the velocity computed by use of Equation 1, but adequate documentation is not available to substantiate a definitive modification to this equation.

It is evident that the settling velocity of particles, as determined by Stoke's Law, is always reduced due to the effect of hindrance. According to Equation 3, for a volumetric concentration of suspended solids equal to 0.100, 0.010, and 0.001, the reduction of particle settling velocity with respect to discrete settling is 38.7 percent, 5.4 percent, and 0.5 percent, respectively. Dredged material slurries pumped into a disposal area seldom have a solids content of more than 20 percent to 25 percent by weight, which corresponds to a volumetric concentration of about 0.10. However, all coarse particles (such as gravel, sand, shells, and coarse debris) and large clay lumps settle out of suspension quickly, and the concentration of suspended solids in the waters that are flowing toward the outflow is substantially reduced. Thus, for the major portion of a given disposal area the reduction in the discrete settling velocity of suspended particles would probably not exceed 10 percent, and for areas close to the overflow weirs, where supernatants have been substantially clarified, the reduction should be less than 1 percent.

In the case of dredged material disposal areas where sedimentation is the only means by which supernatants are clarified, flocculent settling is involved when fine-grained bottom sediments discharged into the area are in a natural flocculent condition. The degree of flocculation of fine-grained sediments depends on numerous factors (such as water salinity and particle mineralogy), and it is very difficult, if not impossible, to determine the hydrodynamic behavior of such material in a sedimentation basin. A simple and convenient approach (Krizek, Fitzpatrick, and Atmatzidis, 1976) involves conducting hydrometer tests without the addition of a dispersing agent, assuming that the grain sizes resulting therefrom are representative of the material, and using these grain sizes to estimate the removal efficiency of a given disposal area. It should be pointed out that flocculent settling becomes a significant factor if the formation of flocs is generated artificially by introducing coagulants into the water.

Artificially induced flocculation is widely practiced in wastewater treatment and more recently in surface mine siltation control. However, such a practice is still considered uneconomical in dredged material disposal operations, and it is used only if no other alternative is feasible. Since the available sites for dredged material disposal are in short supply and the effluent quality requirement is becoming increasingly stringent, it is anticipated that artificial flocculation could become an integral part of containment area operations.

The process of zone settling may develop only when flocculated suspensions of high solids concentration are encountered. Since dredged materials in their natural state may sometimes exhibit the characteristics of a flocculated suspension (perhaps in a salt-water environment), zone settling may be a significant process in certain sedimentation basins. However, no simple means are presently available to incorporate zone settling into a mathematical model describing the sedimentation regime in a disposal area, and, since the development of appropriate theory is not within the scope of this study, no pertinent recommendations regarding zone settling will be advanced.

In the absence of any documented or accurate sophisticated methodology for predicting the sedimentation regime in dredged material containment facilities, a simplified approach can be utilized to obtain first-order approximations. Experience indicates that it is practically impossible to prevent the creation of currents (eddy, surface, convection, and density currents) in disposal areas. Inevitably, the clarification effectiveness of a disposal area will be reduced by currents, but the extent of this effect cannot be predicted with any degree of confidence.

To expedite the determination of the concentration of suspended solids in disposal area effluents, Krizek, FitzPatrick, and Atmatzidis (1976) combined Equations 1 and 5 and developed a nomograph presented in Figure 1 on the basis of the assumptions that (a) the performance coefficient, n, is equal to unity, (b) the mass of particles with equivalent diameters smaller than 0.1 micron is negligible, (c) the masses of particles smaller than 1 micron and 10 microns are not more than 20% and 50% by weight, respectively, (d) all particles larger than 10 microns will be removed by sedimentation, and (e) the gradation curve between the 10 microns and 1 micron sizes and between the 1 micron and 0.1 micron sizes is a straight line, if grain sizes are plotted on an arithmetic scale. Based on the second and third assumptions, the ten gradation curves shown in Figure 1 were selected to cover the range of gradations expected in dredged bottom sediments. Assumption (e) was made in order to provide a standard basis for performing the computations necessary to develop the nomograph. The use of the nomograph is explained as follows:

(a) Determine the value of the surface loading from a knowledge of the disposal area size and its expected flow rate.

(b) Identify the grain size distribution curve of the bottom sediments to be dredged and select from Figure 1 the representative gradation curve that gives the best fit; if hydrometer tests are employed to obtain the distribution curve, they should be performed without the use of a dispersing agent.

(c) Estimate the suspended solids content of the slurry to be pumped into the disposal area.

(d) Enter the nomograph from the left and move to the right until the selected representative gradation curve is encountered; then, move up or

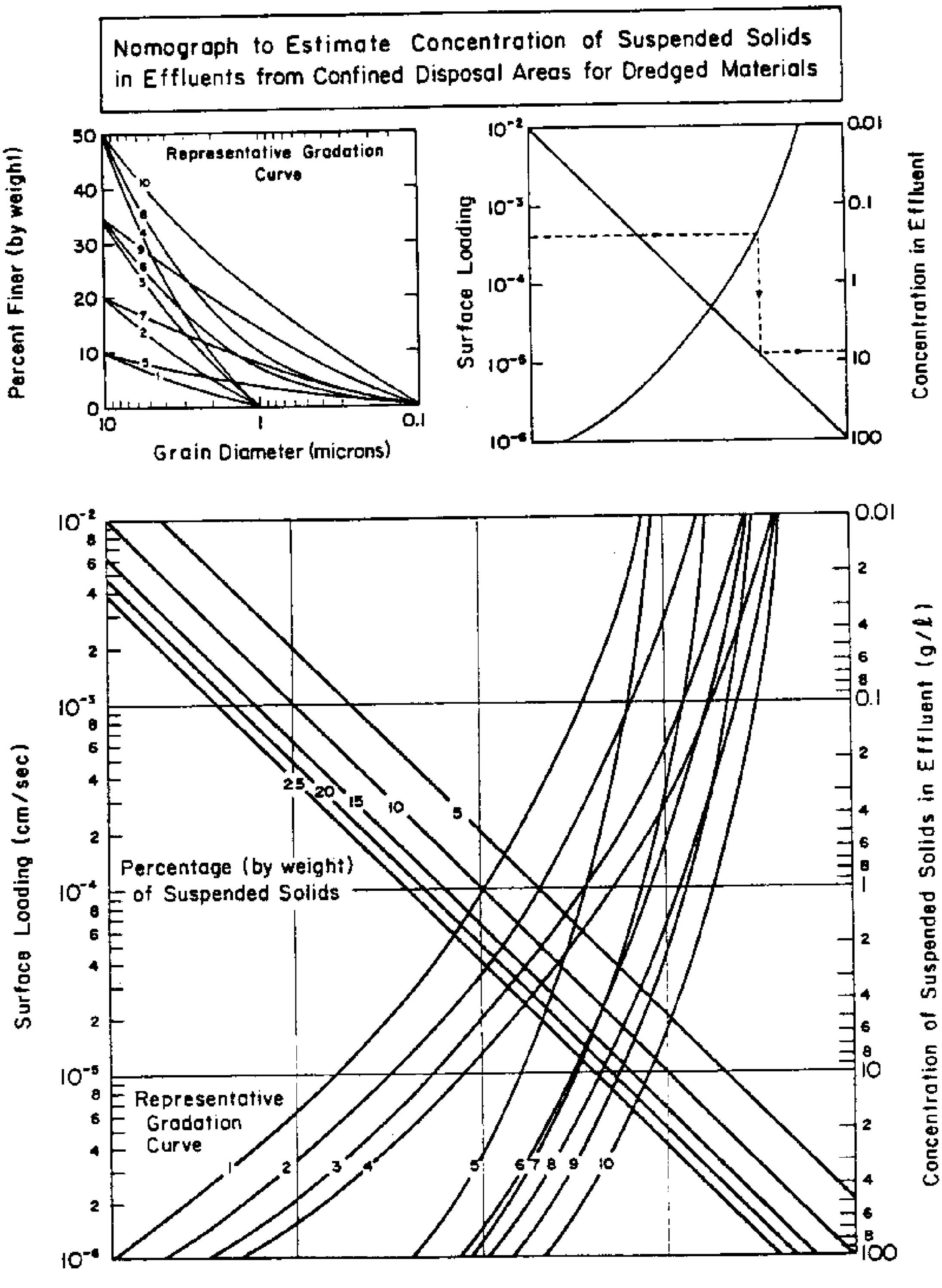

Figure 1. Nomograph to Estimate the Concentration of Suspended Solids in Effluents (after Krizek, FitzPatrick, and and Atmatzidis, 1976)

down until the appropriate line representing the percentage of suspended solids in the slurry is met; finally, exit the right side of the nomograph and read the concentration of suspended solids in the effluent.

This approach does not take into consideration the effects of hindered settling, particle flocculation, and short circuiting on the removal effectiveness of the sedimentation basin. Some or all of these effects exist in disposal areas and can affect the removal effectiveness of suspended solids. Since hindrance has been shown to reduce the settling velocity of discrete particles in disposal areas by not more than 10 percent and most probably by only 1 percent to 2 percent, and considering the fact that the foregoing approach allows only a first-order approximation of the removal effectiveness, a correction of the particle settling velocity, v_s, by a factor to account for hindered settling would not realistically improve the accuracy of the results. It is realized that flocculent and zone settling processes are much different than discrete or hindered settling processes. However, flocculent settling will become a significant factor in the design of disposal areas only when artificial flocculation becomes a standard practice.

PERFORMANCE OF CONTAINMENT AREAS

Samples of influent slurries and effluent waters were obtained from eleven dredgings disposal areas during site visits which were conducted as parts of extensive research programs (Krizek, FitzPatrick, and Atmatzidis, 1976; Gallagher, et al., 1978). According to field personnel, the conditions at the time of sampling could be considered representative of the prevailing conditions during dredging and disposal operations at each site. Pertinent information on the operational conditions of each containment area at the time of sampling is summarized in Table 1. When

Table 1

Characteristics and Performance of Dredgings Containment Areas

Location	Effective Area (acres)	Ponding Depth (feet)	Inflow Rate (cfs)	Ideal Retention Time (hours)	Dredgings Gradation: Percent by Weight Finer than 50μ	10μ	1μ	Suspended Solids: Influent Waters (g/ℓ)	Effluent Waters (g/ℓ)	Solids Retention: Observed (%)	Predicted (%)
Ilwaco, Wash.	10	6	6	121	80	8	3	311.5	0.14	99.9	97
Mobile, Ala.	275	2	60	111	68	35	20	96.1	0.20	99.8	90
Savannah, Ga.	1000	½	60	101	60	25	10	142.7	1.71	98.8	96
Charleston, S.C.	250	1	30	101	62	45	29	147.1	6.24	95.8	92
Yazoo City, Miss. - 1	15	8	18	81	78	10	4	158.5	2.24	98.6	98
Yazoo City, Miss. - 2	11	2	18	15	76	11	4	159.0	11.09	93.0	94
Freeport, Tex.	45	3	45	36	50	40	12	317.0	8.06	97.5	83
Willapa, Wash.	12	3	32	14	58	18	9	147.6	16.95	88.5	89
Seattle, Wash.	20	4	32	30	46	31	8	137.2	0.66	99.5	87
Wilmington, N.C.	600	½	45	81	68	42	14	100.1	1.45	98.6	95
Jacksonville, Fla.	40	2	18	54	88	60	22	114.5	1.59	98.6	92

not accurately known, effective areas were estimated as a fraction of the total surface area of the disposal site. Ponding depth is an estimate of the average depth of water in the containment area. The rate of inflow was computed as a function of the pipeline diameter. The amount of total suspended solids in the influent and effluent waters was determined gravimetrically. Gradations were obtained according to ASTM specifications with the exception that no dispersing agent was used when conducting hydrometer analyses.

The ideal, plug flow, retention time, t, of each area was computed as

$$t = \frac{Ah}{Q} \tag{6}$$

where A is the effective surface area of the basin, h is the average ponding depth, and Q is the influent flow rate. Observed or predicted solids retention effectiveness, E, was computed as

$$E = \frac{S_i - S_o}{S_i} \times 100 \tag{7}$$

where S_i and S_o are the total amount of suspended solids in the influent and effluent waters, respectively. Based on the information presented in Table 1 the following observations can be advanced:

(a) Good to excellent suspended solids retention was being achieved at all sites that were sampled; the retention effectiveness ranged from 88 percent to 99.9 percent with a remarkably high average of 97 percent.

(b) Low solids retention effectiveness is associated with low retention time; increasing retention time appears to improve the solids retention effectiveness.

(c) Predictions based on the application of classical sedimentation theories tend to underestimate the solids retention effectiveness of containment areas.

According to Equation 5, the maximum retention associated with a value of $v_s/(Q/A)$ equal to unity is 63 percent, rather than 100 percent which is expected for ideal basins (Equation 4). For 85 percent removal of suspended particles, the values of $v_v/(Q/A)$ range from 1.9 for best basin performance to 6.0 for very poor basin performance; these values imply that, to achieve 85 percent removal effectiveness, the retention time or the surface area of the basin should be from two to six times that required under ideal basin conditions. To achieve higher removal effectiveness would require even more excessive oversizing of settling basins to compensate for the adverse effects of currents. Therefore, it is believed that use of performance predictions which are made on the basis of classical sedimentation theory formulations would result in very conservative designs which, although safe, would obviously be very costly. This is substantiated by the fact that observed and predicted solids retention effectiveness are in agreement for only three of the eleven sampled areas while predictions underestimate actual performance for the other eight areas.

Furthermore, consideration should be given to the fact that predictions based on the application of the nomograph in Figure 1 assume poorly performing (n=1) sedimentation basins. However, excellent performance (n=0) should not be considered as indicating ideal (plug flow) conditions, but rather as the case where the adverse effects of currents have been reduced to a realistic minimum. Thus, it should be expected that predictions based on good to excellent performance for the sedimentation basin would approximate better the observed effectiveness of dredgings containment areas.

The actual retention time of a containment area is a fraction of the ideal retention time, and a simple relationship is difficult, if not impossible, to establish. Since numerous factors affect the solids retention effectiveness of a disposal area, it is possible that areas with higher retention times could have lower retention effectiveness than areas with lower retention times. The Charleston and Yazoo City-1 disposal areas can be considered as an example. Although the retention time of the former is higher than that of the latter (101 hours versus 81 hours), the Yazoo City-1 area had a better retention effectiveness (98.6% versus 95.8%). This effect could be due to a number of factors, including (a) substantially more fine-grained suspended solids in the influent to the Charleston area, and (b) very shallow ponding depth in the Charleston area, which could result in scour and resuspension of bottom sediments.

It appears, therefore, that retention time and ponding depth are strongly correlated with the solids retention effectiveness of dredgings containment areas. This condition is best exemplified by the performance of the Yazoo City-1 and the Yazoo City-2 disposal sites. Both sites have almost identical influent characteristics (flow rate, and concentration and gradation of suspended solids), but Yazoo City-1 had a ponding depth of 8 feet and an ideal retention time of 81 hours, while Yazoo City-2 had a ponding depth of 2 feet and a corresponding retention time of only 15 hours. The solids retention effectiveness was 98.6 percent and 93.0 percent for Yazoo City-1 and Yazoo City-2, respectively. It can be concluded, therefore, that increased ponding depth and/or retention time improves substantially the solids retention effectiveness of a dredgings containment area.

The effect of ponding depth can further be studied by considering three of the disposal sites which had the largest effective surface areas; these are the sites at Savannah, Wilmington, and Mobile which had effective surface areas of 1000 acres, 600 acres, and 275 acres, and ponding depths of 0.5 feet, 0.5 feet, and 2 feet, respectively. It can be observed that, although the Mobile site had the largest amount of submicron particles suspended in the influent slurry (19 g/ℓ versus 14 g/ℓ for the other two sites), the achieved solids retention effectiveness was higher than at either the Savannah or the Wilmington sites (99.8 percent versus 98.8 percent and 98.6 percent). It can be concluded, therefore, that the beneficial effects on sedimentation of using an oversized area can be offset by the maintenance of very shallow ponding depths, which enhance the possibility of scour and resuspension of bottom sediments.

The retention time is also directly proportional to the rate of flow through the containment area. The Willapa disposal site clearly demonstrates this effect. Although the amount of fines was rather low in the influent slurry (18 percent by weight finer than 10 microns), the solids retention effectiveness was very low (88.5 percent). It appears that the inflow rate of 32 cfs was incompatible with the size of the area (12 acres) and resulted in a very small retention time. The diameter of the pipeline is often selected on the basis of the volume of material to be dredged within certain time constraints or simply in accordance with the equipment available to the dredging contractor. It must be emphasized, however, that the pipeline diameter (and consequently the rate of slurry discharge) must be compatible with the flow rate that corresponds to the required settling effectiveness of the containment area. The Ilwaco disposal site had a very low rate of inflow (6 cfs) which, coupled with a relatively large ponding depth of 6 feet, resulted in excellent solids retention effectiveness, although the concentration of suspended solids in the influent slurry was very high (311.5 g/ℓ).

CONCLUSIONS

On the basis of the foregoing information and discussions, the following conclusions may be advanced:

1. Good solids retention capability is frequently realized in dredgings containment areas.

2. Sedimentation is one of the primary means by which suspended solids can be controlled in the supernatants that are discharged from a dredgings containment area.

3. A simple methodology for accurately describing the sedimentation regime in a disposal area is not available.

4. In the absence of a reliable methodology for predicting the process of sedimentation in dredged material containment facilities, available theoretical expressions for the solids removal effectiveness of non-ideal basins can be used to estimate their performance, but designs based on these formulations may be unduly conservative.

5. Poor containment area performance is usually due to insufficient retention time (low hydraulic efficiency).

6. In some cases high solids retention effectiveness is achieved by use of oversized areas.

7. There is a strong correlation between hydraulic efficiency and suspended solids retention; improving the hydraulic efficiency improves the solids retention effectiveness of an area.

8. In a given containment area, the largest possible ponding depths should be realized in order to increase the retention time and improve solids retention.

9. Pipeline diameter (inflow rate) should be compatible with the geometric characteristics of the containment area (ponding depth and effective surface area) to avoid deterioration of the hydraulic efficiency (retention time) of the area.

ACKNOWLEDGMENTS

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DESIGN OF WEIRS FOR MAINTENANCE OF EFFLUENT QUALITY

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Abstract

The suspended solids concentration in the effluent water from an upland containment area being filled with fine-grained dredged material can be significantly influenced by the length of the weir and the depth of the ponded water. This report presents a procedure for designing and operating the weir to maintain good effluent quality, given a flow and dredged material type.

The Waterways Experiment Station's selective withdrawal model developed by Bohan and Grace, modified to fit observed data, was selected as the basis of the design procedure. Using this model, nomograms were developed for the design of weirs. The nomogram relates the flow, weir length, ponding depth, and effluent suspended solids concentration. The designer manipulates these four variables until he reaches a satisfactory balance between weir length and ponding depth, based on his design flow and effluent goal. Proper operation of the weir can ameliorate the effects of short-circuiting or an undersized basin.

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Introduction

The quality of the effluent from a dredged material containment area can be strongly affected by the design and operation of the discharge weir. The purpose of this study was to develop a weir design and operating procedure for containment areas to maintain good effluent quality. The procedure was based on a density-stratified flow hydraulic model. The model indicated that, for a given dredged material type and discharge flow rate, the weir length and ponding depth control the effluent quality. These two parameters provide the designer with two alternate means of improving the effluent quality. Other factors, including the weir location, shape, and type, were evaluated and used in the design procedure, quantitatively in the velocity profile and weir length, and qualitatively in the form of guidance and recommended procedure.

This report contains a design procedure to aid in selection of weir length and ponding depth for containment areas. The design procedure is based on a nomogram which, given a design flow, weir length, and ponding depth, will predict the effluent suspended solids concentration from a properly designed basin at the end of the basin's service life (worst case). The method was based on data collected at several small sites (13 to 20 acres) and is applicable for fine-grained dredged material from both saline and freshwater environments.

Concepts in Weir Design for Containment Areas

It is assumed that the reader is familiar with basic definitions pertaining to containment areas. Some concepts which are crucial to understanding this report will be discussed below.

Containment areas

The design procedure is for confined disposal areas. A confined disposal area is a diked area with an inlet pipe from the dredge and an overflow weir. The diked area is often referred to as a basin. The plan and profile views for a typical basin are presented in Figure 1.

Suspended solids and density profiles

When the dredged material is discharged into the basin a high percentage of the suspended solids settle to the bottom of the basin. These will be referred to as settled solids. Some of the solids remain suspended and will be referred to as unsettled solids.

Since suspended solids are constantly moving downward, the suspended solids concentration is highest at the bottom of the basin and is lowest at the surface. A graph showing the change of concentration with depth is shown in Figure 1. This type of graph is referred to as suspended solids concentration profile, or a concentration profile. The slope of the concentration profile is said to be the concentration gradient.

The density (mass per unit volume) of the fluid is dependent on the suspended solids concentration, dissolved solids concentration, specific gravity of the solids, and temperature. In a containment area only the suspended solids concentration varies significantly with depth. The density gradient can therefore be directly related to the suspended solids gradient. Since the density and suspended solids concentration profiles are so closely related, they are often used interchangeably. Temperature and dissolved solids concentration do not vary with depth.

The fluid in the containment area is said to be stratified if the density increases with depth. (The term fluid in this report refers to all water and unconsolidated solids above the bottom of the basin.)

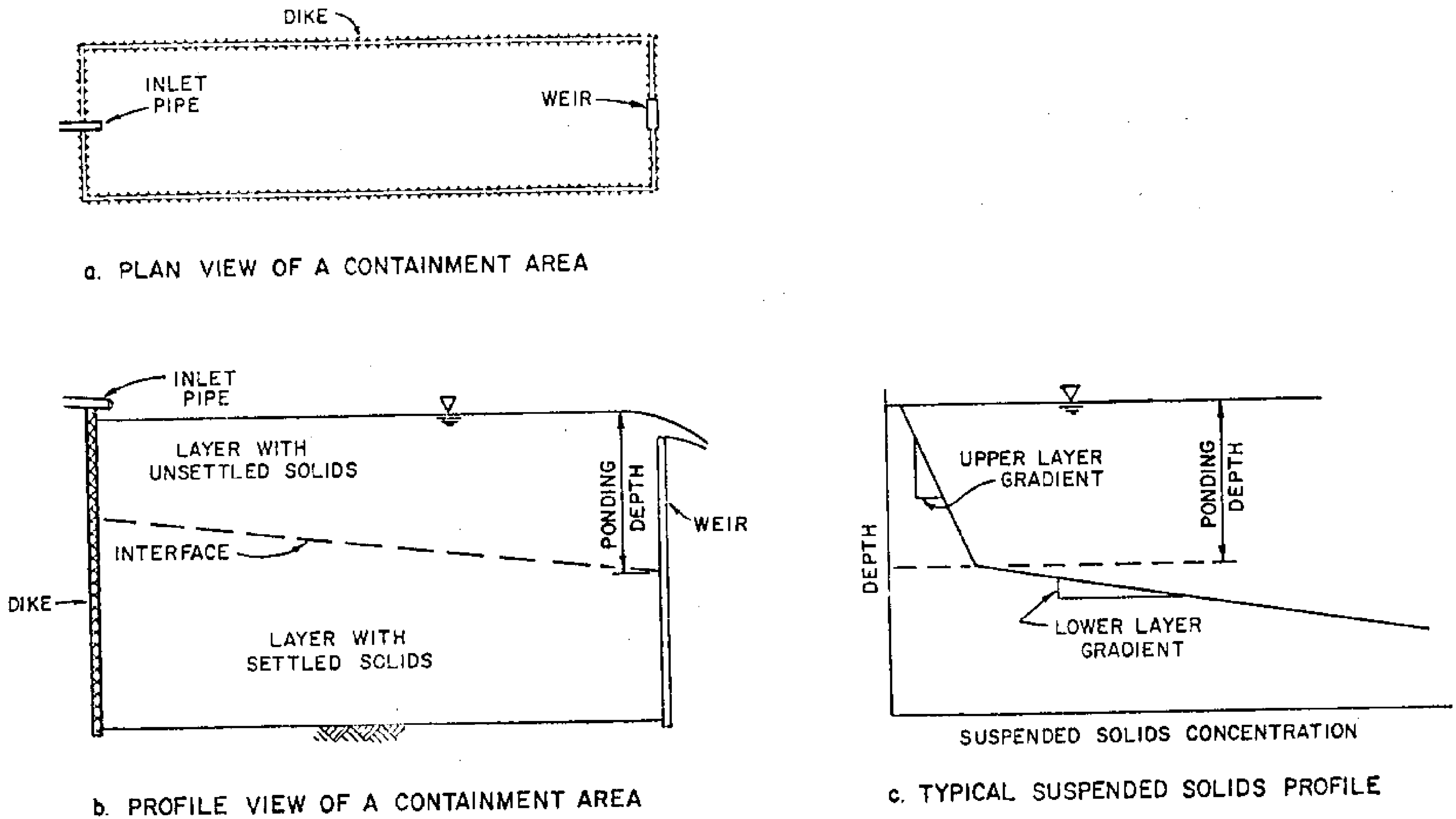


Figure 1. Basin and settling descriptions

Ponding depth

In typical suspended solids concentration profiles from dredged material containment areas, the gradient will be fairly constant in the top layer which contains unsettled solids. At a depth where the suspended solids concentration is approximately 20 g/ℓ, the gradient increases sharply as shown in Figure 1. Below this depth, the suspended solids are considered to be settled. This depth is the interface between the settled and unsettled solids and is simply referred to as the interface. The interface is not perfectly horizontal but slopes slightly (about 1:500) from the inlet pipe toward the weir. The depth of water and unsettled solids above the interface is referred to as the ponding depth or depth of ponded water.

Weir concepts

The weirs utilized in containment areas are sharp-crested rectangular weirs. Sharp-crested means that the thickness of the weirs (T) is small in comparison to the depth of the flow over the weir (h) (see Figure 2; $h/T > 1.5$)[8]. Rectangular means that the weir is straight and flow over the weir is perpendicular to the weir. The flow over the weir (Q), static head (H), and weir length (B) can be related by the following equation:

$$Q = C_D BH^{3/2}$$

where C_D is the weir discharge coefficient, which is usually 3.3 for sharp-crested weirs. H is the difference in elevation from the weir crest to the water surface at a point sufficiently far from the weir so that the flow velocity caused by the weir is negligible (i.e. total head = static head). The above equation is not applicable for polygonal weirs.

The term Q/B is referred to as the weir loading rate or unit flow rate, and is a very important design parameter for weir design. The static head, H , can be related to the depth of flow over the weir, h , for sharp-crested weirs by:

$$h = 0.85H$$

h must be measured directly above the weir crest.

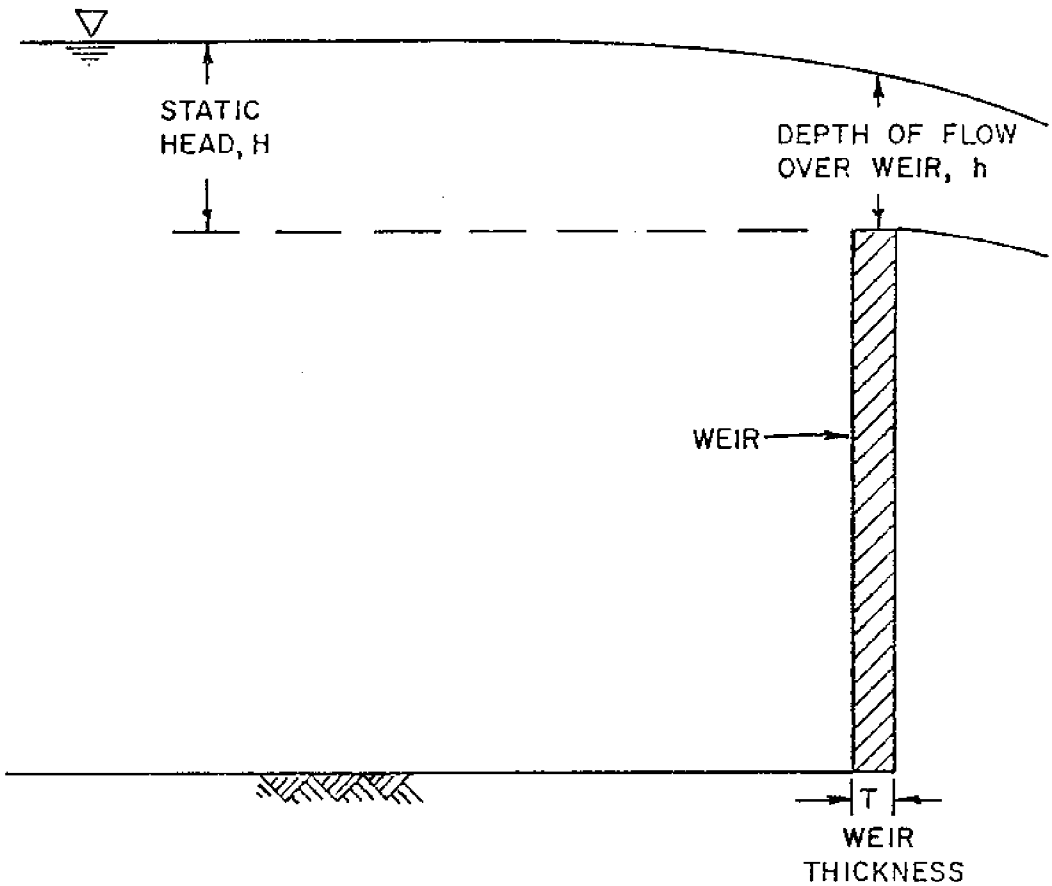


Figure 2. Weir characteristics

Withdrawal zone

The withdrawal zone is the area through which fluid is effectively discharged over the weir. The depth of the withdrawal zone or withdrawal depth is the depth below the water surface from which water is withdrawn over the weir. The size of the withdrawal zone affects the approach velocity of the flow. The approach velocity is the speed at which the fluid is moving toward the weir. Figure 3 illustrates the concept of withdrawal depth and flow velocity. The approach velocity, in conjunction with the density profile, controls the depth of the withdrawal zone.

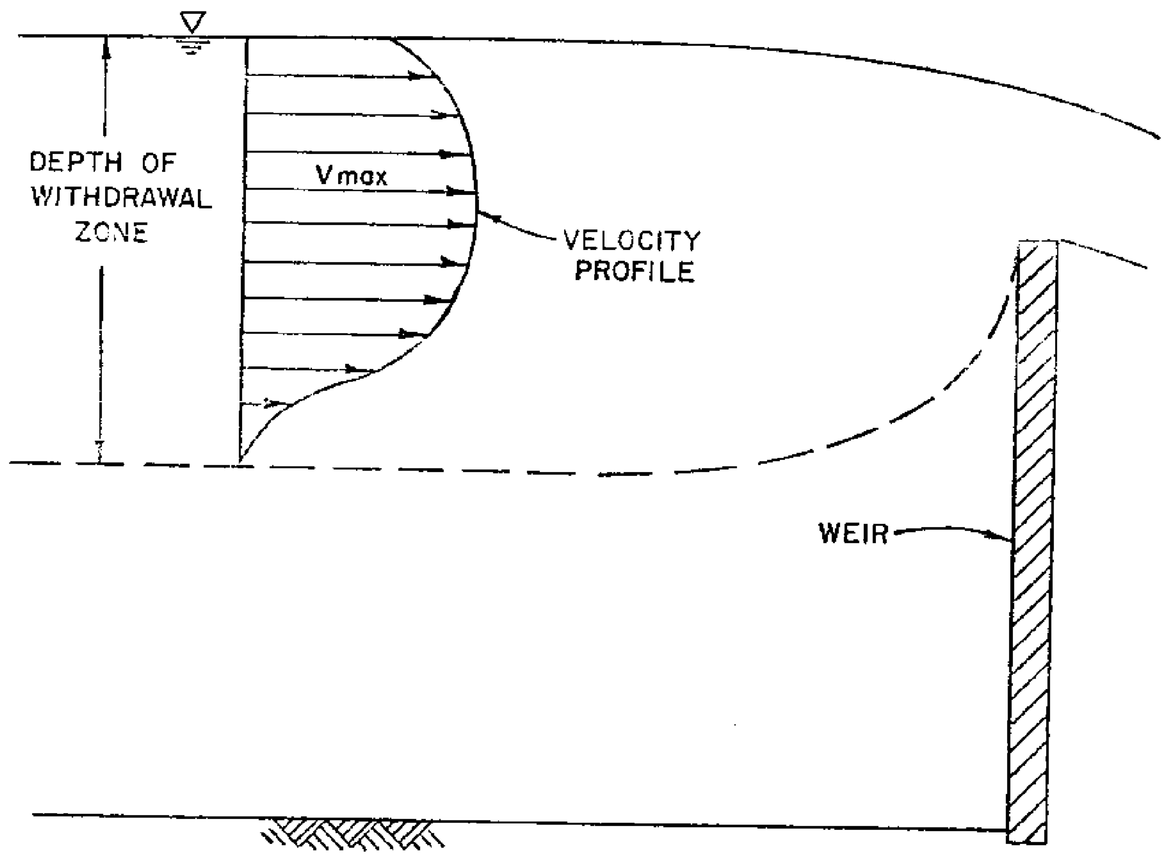


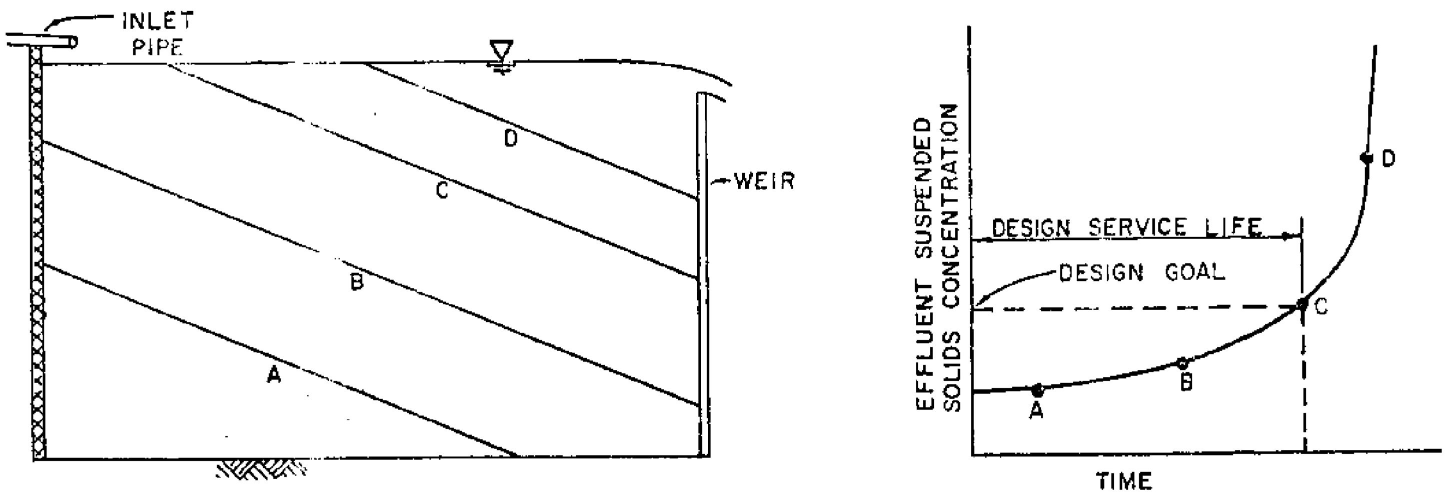
Figure 3. Withdrawal depth and velocity profile

Design description

For a given suspended solids concentration profile and flow, a longer weir reduces the withdrawal depth and improves the effluent quality. The same improvement can be achieved by maintaining the same weir length and increasing the ponding depth. The method for designing weirs to maintain adequate effluent quality is to optimize the tradeoff between increased weir lengths and increased ponding depths.

Service life of basin

During the life of a containment area the interface moves upward and toward the weir. In Figure 4 the lines A, B, C, and D represent the interface at different times in the basin life. (The vertical scale is greatly



a. POSITION OF THE INTERFACE AT FOUR PERIODS IN THE SERVICE LIFE

b. EFFECT OF REMAINING SERVICE LIFE ON THE EFFLUENT SUSPENDED SOLIDS CONCENTRATION

Figure 4. Effects of time on ponding depth and effluent quality

exaggerated in Figure 4.) As the basin fills, the ponding depth decreases. As this happens, more solids are withdrawn over the weir. This is shown in the graph of effluent solids versus time in Figure 4. Sufficient ponding depth must be provided so that the dredging job can be completed before the effluent quality deteriorates as it does between times C and D.

Because of the sloping interface, the ponding depth is not constant throughout the basin but increases away from the inlet pipe. The ponding depth of concern in weir design is the final ponding depth immediately in front of the weir.

Mathematical Modeling

Two steps were involved in mathematically modeling the flows over weirs: the first consisted of a thorough literature review of models for describing such flows; the second involved collecting field data which served as a basis for model selection and verification. Detailed information on the literature review and sampling can be found in a previous study by the authors.[9]

Field trips were made to dredging sites in Yazoo City, Mississippi, Fowl River, Alabama, and Oyster Bay, Alabama. During these field trips, measurements were made of velocity profiles and density profiles in the vicinity of the weirs. The material classification, and salinity were determined. The effluent quality was also sampled for a number of different flow rates.

Based on the data collected during the field trip, the WES Selective Withdrawal model[1] was chosen since it matched the data fairly accurately, had a sound experimental basis and could be easily developed into a design procedure.

The WES selective withdrawal model is a one-dimensional model developed from laboratory flume studies. The flume studies were conducted for the case where the weir extended across the entire width of the flume. The depth of a dimensionless fully developed withdrawal zone was correlated with a densimetric Froude number. The following equation was developed from the correlation for weir flow by using dimensionless variables for the depth of the withdrawal zone and the density profile[1].

$$\frac{V_w}{\sqrt{\frac{\Delta\rho_w}{\rho_w}(gZ_o)}} = 0.60 \quad \frac{Z_o + H_w}{H_w}$$

V_w = average velocity over the weir, fps

$\Delta\rho_w$ = density difference of fluid between the elevations of the weir crest and the lower limit of the zone of withdrawal, g/cm^3

ρ_w = density of fluid at the elevation of the weir crest, g/cm^3

g = acceleration due to gravity, ft/sec^2

Z_o = vertical distance from the elevation of the weir crest to the lower limit of the zone of withdrawal, ft

H_w = static head over weir, ft

Nomogram Development

The field study indicated that there were five important parameters in weir design, namely: (1) flow, (2) weir length, (3) ponding depth, (4) effluent quality goal, and (5) material type. It was the goal of this study to produce a simple nomogram which related these parameters and could be used for design.

Since different fine-grained materials behave differently in containment areas, a separate nomogram could be developed for each material based on its grain-size distribution, plasticity index, etc. To simplify matters, it was decided that for practical purposes, fine-grained material could be divided into two categories: (1) clays in freshwater, and (2) clays in saltwater and silts. Therefore, two nomograms were developed.

The nomograms were developed by running the WES selective withdrawal model program for an array of flows, weir lengths, and ponding depths for each material category. The model then predicted the effluent quality. This information was plotted to make up the nomogram. The nomogram for freshwater clays is shown in Figure 5 while the nomogram for silts and saltwater clays is given in Figure 6.

Figure 7 is included for the case of freshwater materials which settle well. It is based on the assumption that there are no suspended solids above the interface.

Design Procedure

Sufficient weir length and ponding depth near the weir must be provided in a containment area to prevent water with high suspended solids concentrations from flowing out of the basin. The following section provides a design procedure that uses nomograms for selecting weir length and ponding depth at the weir to maintain effluent quality, given the material type and design flows. The design procedure is based on the principles of selective withdrawal of stratified fluids by Bohan and Grace[1] as discussed in the previous report by the authors.[9] The procedure is applicable for fine-grained dredged material containment areas. The performance of a basin for dredged material that is exclusively sands and gravels will not be significantly influenced by the weir design.

Data required

The data required for this design procedure consist of the dredged material type, salinity, design flow, and effluent quality desired.

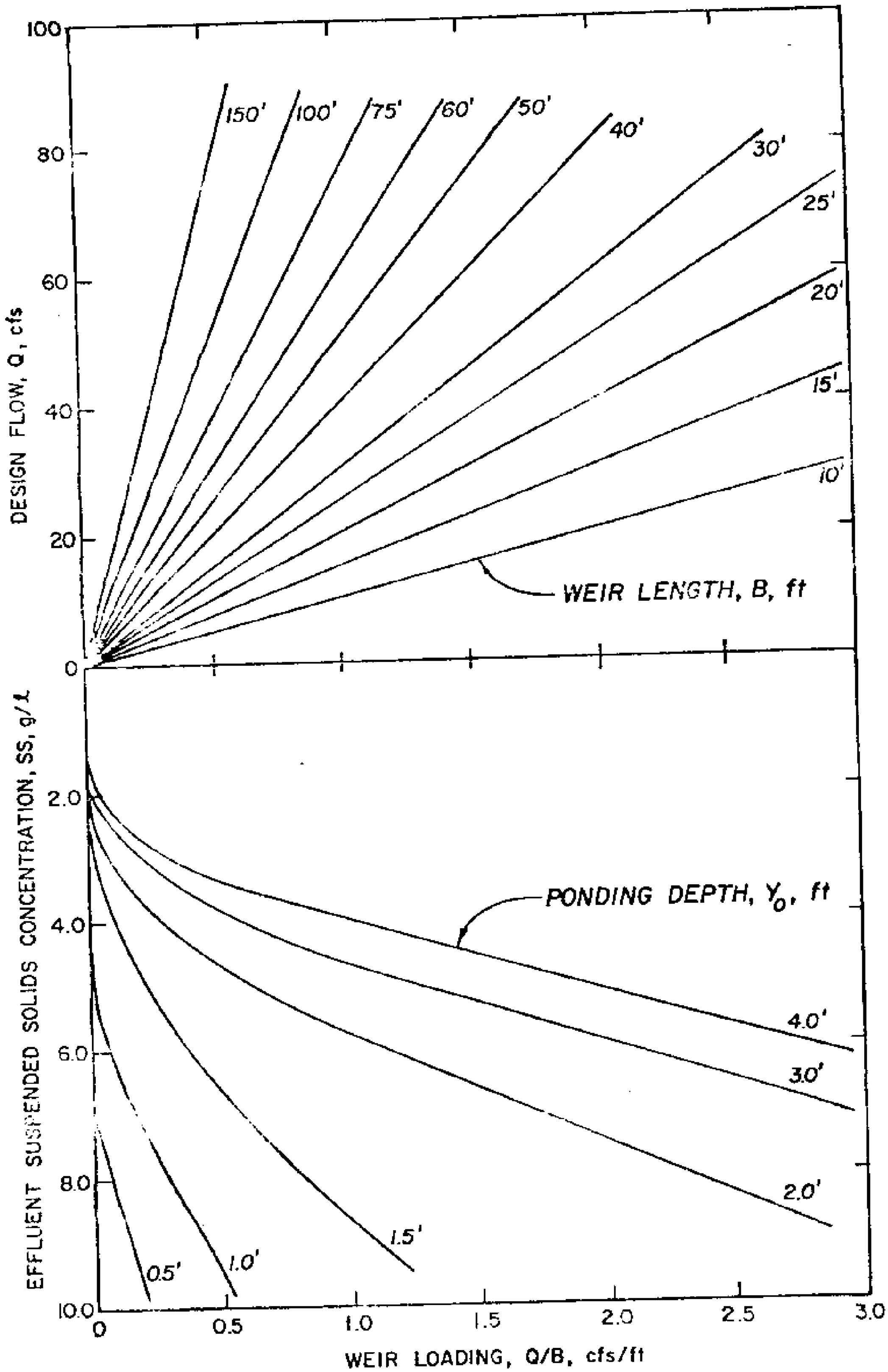


Figure 5. Relationships among design flow, weir length, effluent suspended solids concentration, ponding depth, and weir loading for freshwater clays

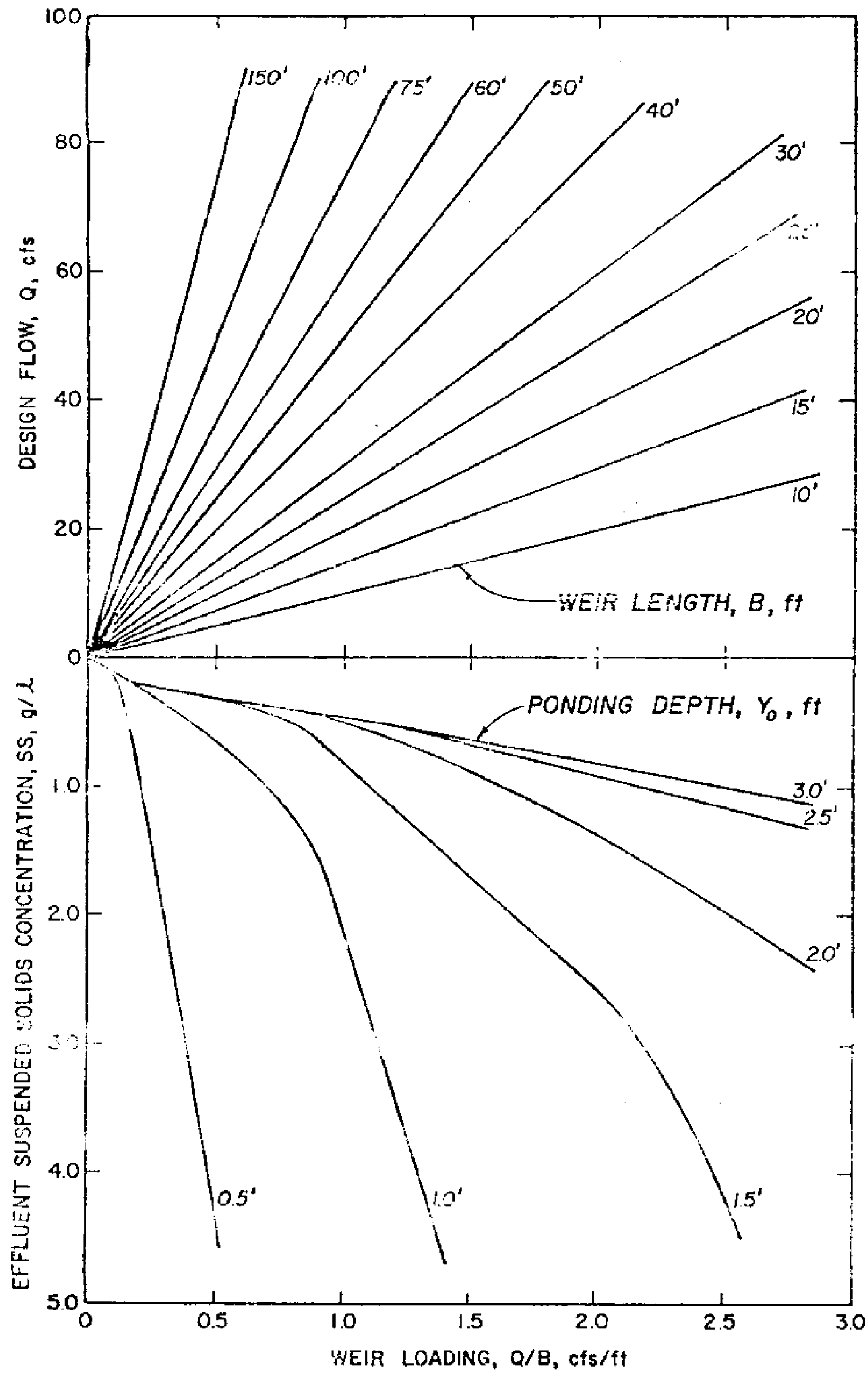


Figure 6. Relationships among design flow, weir length, effluent suspended solids concentration, ponding depth, and weir loading for silts and saltwater clays

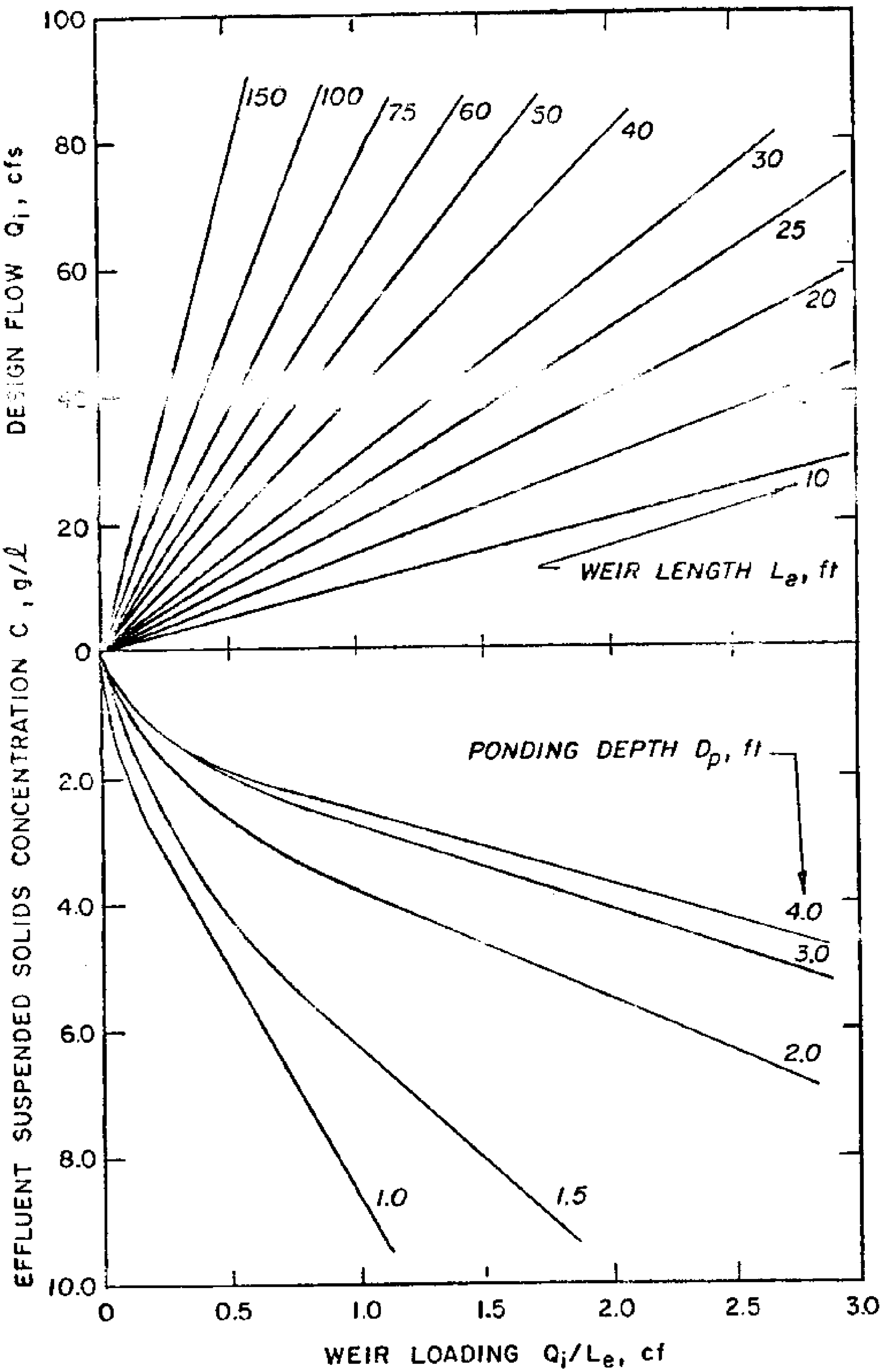


Figure 7. Weir design nomogram for freshwater clays

For the purpose of the design procedure, fine-grained dredged material is classified as either a clay or a silt. To classify the material, the material must first be classified under the Unified Soil Classification System[7]. If the material is classified as a silt or an organic silt (either ML, MH, or OL), then it is classified as a silt in the design procedure. If the material is classified as a matrix of soil types, such as a CL-OL matrix, then the material would be classified as the worst settling type, in this case as a clay since clays settle more slowly than silts. Similarly, if several different types of dredged material are to be disposed in the same basin, the slowest settling type would be used in the design procedure. Not all of the above classes of material have been examined in the field but they were classified as recommended above based on their settling properties.

Clays behave quite differently if the salinity of the dredged slurry water exceeds 2 to 5 ppt because the clay particles flocculate and settle much more quickly. Below 1 ppt of salinity or total dissolved solids, the water is considered to be fresh and the clay particles do not flocculate. Because of the effect of flocculation, a different design nomogram is used for clays in saline water. If the salinity is between 1 and 3 ppt, the clay material will probably behave as an intermediate or transition type for which the effluent suspended solids concentration will be better than that predicted for freshwater clays but not as good as that predicted for saltwater clays. The designer must use judgment, or past experience, with the dredged material to predict the effluent suspended solids concentration for dredged materials in this transition range.

In estuarine areas, the salinity may vary through the year due to differences in the freshwater flow and the location of the saltwater wedge.

Therefore, the lowest probable salinity of a near-bottom water in the area to be dredged during the projected dredging operation should be used since this provides the most conservative design.

Knowing the salinity and the soil type, the designer can select the correct nomogram from Table 1. The nomogram in Figure 5 is for freshwater clays. The nomogram in Figure 6 is for silts and all saltwater fine-grained dredged material. The nomogram in Figure 5 is for dredged material that settles slowly. The nomogram in Figure 6 is for dredged material that settles more rapidly.

Table 1

Nomogram Selection

<table>
<tr><th></th><th>Clays</th><th>Silts</th></tr>
<tr><td>Salinity <1 ppt</td><td>Figure 5</td><td>Figure 6</td></tr>
<tr><td>Salinity 1-3 ppt</td><td>Transition Range</td><td>Figure 6</td></tr>
<tr><td>Salinity >3 ppt</td><td colspan="2">Figure 6</td></tr>
</table>

The design flow refers to the peak flow over the weir during the design life of the basin. If the dredge is not operating for a considerable period of time, the flow rate over the weir may be less than the peak inflows. The actual flow rate will be a function of the dredge, the head loss in the pipe, and the elevation of the discharge pipe at the basin.

The designer must determine the appropriate effluent suspended solids limit for his dredging operation based on effluent standards, the water quality of the stream, and environmental concerns. The effluent suspended solids concentrations predicted by the nomograms are average values. If the designer wants to design for worst conditions, he must assume a value for the ratio of the maximum to average effluent suspended solids concentration for a given weir loading (Q/B) and ponding depth. A ratio of 1.5 to 2.0 was observed in the field data.

Use of nomogram

The design procedure using the nomogram should be an iterative procedure. There are four variables that the user can manipulate to achieve an optimal design. These are design flow (Q), weir length (B), ponding depth (y_o), and the effluent suspended solids (SS). The designer can select any three variables (Q, B, y_o, or SS) and solve for the fourth. To minimize cost, both the weir length and the ponding depth should be minimized. But for a given flow, soil classification, and effluent goal, the weir length is inversely related to the ponding depth, that is, a shorter weir requires a larger ponding depth. By evaluating various weir lengths and ponding depths, the designer can arrive at a design that meets his needs.

The weir loading (Q/B), the flow in cfs per ft of weir length, is the principal design parameter. If the designer wishes to use a low ponding depth, the weir loading must be kept small. Lower weir loadings will produce better effluent quality at the cost of a longer weir. The weir loading should be kept between 0.1 and 3.0 cfs/ft to maintain good effluent quality without requiring excessively long weirs or deep basins. This corresponds to a range of static heads of 1 to 12 in. or a range of depths of flow over the weir of 0.8 to 10 in.

The ponding depth also provides the designer with a parameter through which he can control effluent quality. The optimal range for this parameter is from 1 to 3 ft. Ponding depths of greater than 3 ft will result in high and hence expensive dikes, while not considerably improving the effluent quality. Depths of less than 1 ft will result in poor effluent quality. Ideally, the ponding depth and depth of withdrawal zone will be equal at the end of the basin's service life.

A trial design using the nomograms consists of a single line that starts at the flow (Q) axis and proceeds horizontally right until it intersects a desired weir length (B) line. From there it drops vertically through the weir loading (Q/B) line until it intersects the desired ponding depth (y_o) line. From there it proceeds horizontally left until it intersects the effluent suspended solids (SS) line. The designer should make a number of trial designs until he feels he has optimized the design.

Example designs

The use of the nomograms can best be illustrated by the following example problem.

In the problem, a weir is to be designed for a freshwater dredging site. The dredged material is classified as a CL clay. The design flow is 30 cfs and the effluent standard is 8 g/ℓ.

The designer first selects the proper nomogram from Table 1. Since the material is a freshwater clay, the nomogram in Figure 5 should be used. The deisgner then decides to maintain an average effluent suspended solids concentration of 5 g/ℓ at the end of the basin's service life in order to insure that the maximum effluent suspended solids concentration will not exceed the 8 g/ℓ effluent standard, despite fluctuations in conditions. The designer is now ready to use the nomogram.

The designer draws horizontal lines on the nomogram at his design flow, 30 cfs, and his effluent suspended solids concentration, 5 g/ℓ. These parameters are shown as solid lines A and B on Figure 8. The designer can select an infinite number of combinations of weir length, B , and ponding depth, y_o , to meet his design parameters, 30 cfs and 5 g/ℓ. A possible combination is determined by drawing a vertical line connecting the horizontal lines at 30 cfs and 5 g/ℓ. Six combinations that cover the range of feasible alternatives are presented as dashed lines C_1 , C_2 , C_3 , C_4 , C_5 , and C_6 . These alternatives are tabulated

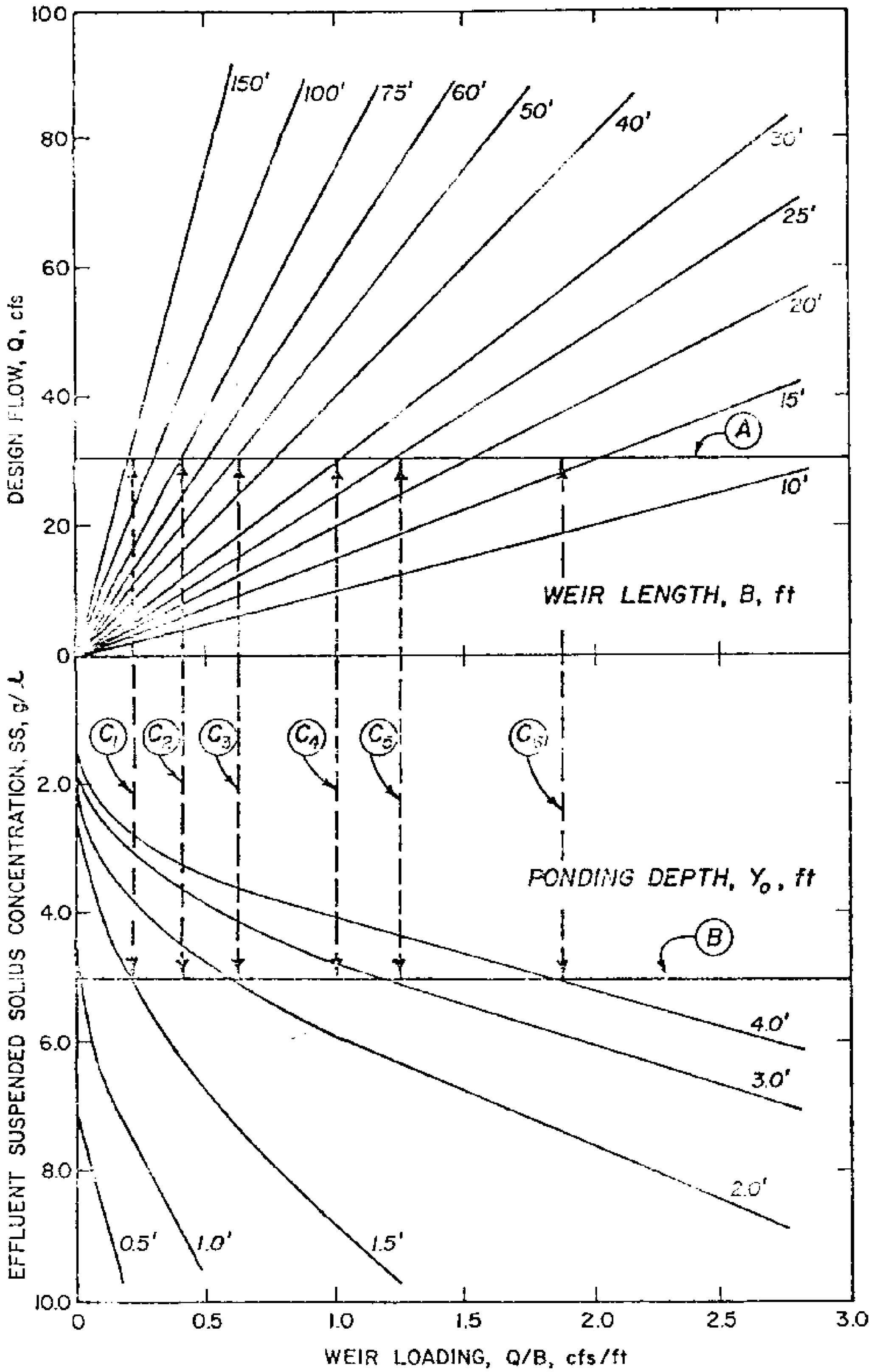


Figure 8. Example Problem 1

below.

Line	Ponding Depth y_o, ft	Weir Length B, ft	Weir Loading Q/B, cfs/ft
C_1	1.5	140	0.21
C_2	1.7	75	0.40
C_3	2.0	48	0.62
C_4	2.7	30	1.00
C_5	3.0	24	1.25
C_6	4.0	16	1.88

Any of the solutions above would be adequate. However, the designer would most likely choose a weir length between 30 and 50 ft since he saves very little ponding depth if he uses a longer weir but may have to add a great deal of ponding depth for a shorter weir. If the designer is not satisfied with any of the alternatives, or if he wishes to evaluate the effects of using different design parameters, he may select a different dredge size and design flow. Similarly, he may reevaluate the effluent quality goal and select a more appropriate goal for his design conditions. Then the designer would once again use the nomogram, as illustrated before, to select his new design alternatives.

Other Design Considerations

While the following factors are not explicitly accounted for in the design nomograms, they must be considered in the design procedure.

Weir design and basin sizing

Weir length and ponding depth are only two parameters in the overall containment area. The site must have sufficient area to permit proper settling, sufficient volume to retain all of the dredged material, and a flow pattern to minimize short-circuiting. These topics are addressed in other DMRP reports.[2,4,6] The design procedure developed here is based on the assumption that sufficient area and volume are provided in the basin and that short-circuiting is not excessive.

Safety factors

In the development of the design procedures, conservative values were consistently employed when there was a question as to the magnitude of a given parameter. Designers are advised to use conservative values whenever there is a question about a given design parameter. If this practice is followed, there should be no need to increase the ponding depth or weir length by adding safety factors.

Sharp-crested weirs

Sharp-crested weirs should be used in dredged material confinement basins whenever possible. They require a smaller ponding depth because the depth of their withdrawal zone is smaller. Consequently, the effluent quality will also be better. A weir is considered sharp-crested if the thickness of the weir is less than two-thirds of the depth of flow over the weir.[8] Except for very low flows, a weir made up of 2-in.-thick boards can be treated as a sharp-crested weir.

Shaft-type weirs

In some cases the outflow structure is a four-sided drop inlet or shaft located in the basin. The weir length (B) determined from the nomograms is for a rectangular weir. In converting the values to make them applicable to shaft-type weirs, the approach velocity of the fluid is the key consideration. To minimize the approach velocity and hence the withdrawal depth, the shaft weir should not be placed too near the dike. In Figure 9, location A is the most desirable since flow can approach it from all four sides (four effective sides). Location B is less desirable since flow can only approach from three directions (three effective sides). Location C is the least desirable since it has only two effective sides.

To convert the weir length (B) determined from the nomograms to be length (S) of a side of the square shaft weir, use the following formula:

$$S = \frac{B}{n}$$

where n is the number of effective sides. A side is considered an effective side if it is at least 5S ft away from the nearest dike, mounded area, or other dead zone. This distance, 5S , is generally accepted as being sufficient to prevent the flow restriction caused by the flow contraction and bending due to the walls.

Since effluent pipes must run from the shaft weir under the dike to the receiving stream, a location such as A in Figure 9 may not be optimal since it is far from the dike and will require a longer pipe than B , which is easier to operate.

Polygonal (labyrinth) weirs

Polygonal (labyrinth) weirs have been used to reduce the head over the weir. Such weirs have very little impact on effluent quality since the controlling factor for the depth of withdrawal and consequently the effluent suspended solids concentration is not the head but the approach velocity. For a given flow, even though the depth of flow and velocity over the weir crest are less for a polygonal weir, the approach velocities, and therefore also the depth of withdrawal and effluent quality will be essentially the same as those for a rectangular weir of equal horizontal length along the dike, L , as shown in Figure 10. Figure 10 illustrates the width of the withdrawal zone or effective weir length (B) for three types of weirs. The arrows indicate the approaching flow towards the weir. The minimum width through which the flow must pass is the width of the withdrawal zone or the effective weir length. For a given flow, the approach velocities are the same for different withdrawal zones of equal size. Therefore, the approach

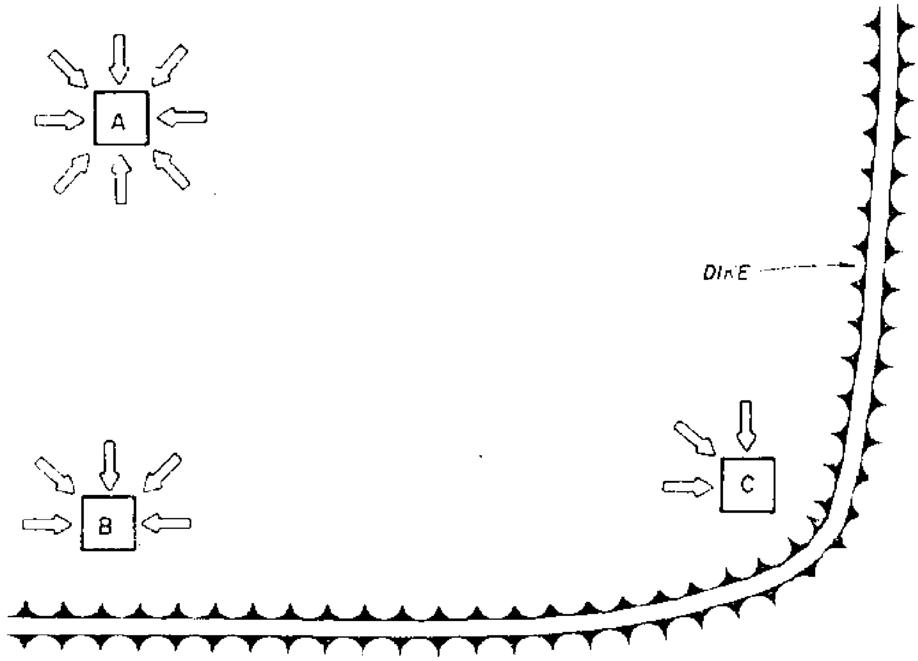


Figure 9. Possible locations for shaft-type weirs

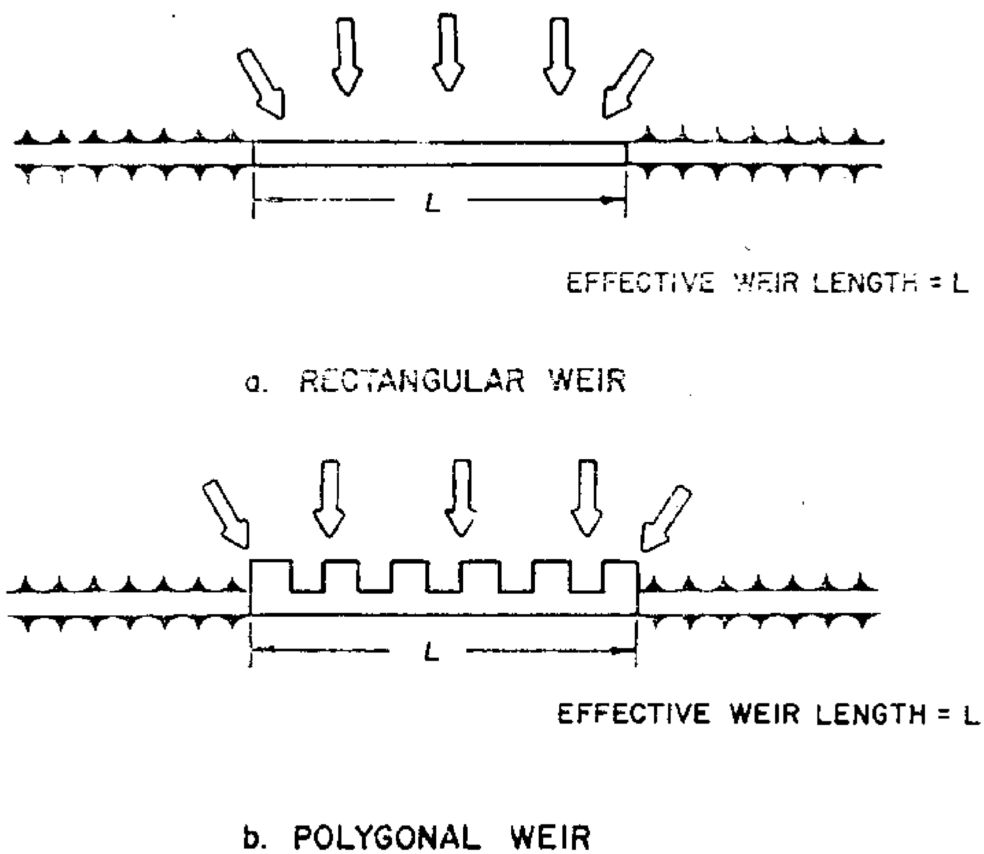


Figure 10. Effective lengths of weirs (plan view)

velocity and the withdrawal depth for the rectangular weir in Figure 10 would be the same as that for the polygonal weir in Figure 10 even though the total weir length for the polygonal weir is considerably greater. Both weirs have the same effective length (B = L).

Since there is no reason to expect an improvement in effluent quality due to polygonal weirs, there is no justification for incurring the greater cost of such weirs.

<u>Weir location</u>

Short-circuiting and dead zones can be reduced by the judicious placement of weirs. Consider the basins shown in Figure 11. The shaded area in Figure 11 indicates dead zones caused by use of one weir. By use of three weirs (each with length one-third that of the weir in Figure 11), the dead zones are reduced in Figure 11. The short-circuiting can also be reduced by use of a spur dike as in Figure 11 as proposed by Gallagher.[2] When several weirs are used in an area, they should be operated with the same weir crest elevation.

<u>Board size</u>

The elevation of the weir crest is controlled by the number of boards placed in the weir. These boards usually range in size from 2 by 4 in. to 2 by 10 in. In order to allow the operator flexibility in controlling the depth of the withdrawal zone and the flow over the weir, small boards should be used near the top of the weir. Use of a large board such as a 2- by 10-in. board at the top of the weir would result in a drastic increase in effluent suspended solids if it is removed. However, the basin could be drawn down slowly without a significant deterioration in the effluent quality by the removal of a small board.

Since some water with high solids concentration may leak between the boards, a small number of larger boards may be preferable to a large number of small boards near the bottom of the weir. Figure 12 shows a weir that will be boarded up to 6 ft but will be operated between 5 and 6 ft. Ten-in. boards are used for the bottom layers with 4-in. boards for the higher zone.

Operational Guidelines

Once the weir is installed and operating, the effluent quality can be controlled only by adjusting the flow or the elevation of the weir crest and hence, the ponding depth. Some basic rules of operation are given below.

General guidelines

The best effluent quality in a dredged material containment area can be achieved if the weir crest is maintained at the highest feasible elevation. This provides the maximum ponding depth at any given time. The weir elevation may need to be lowered to provide the necessary freeboard or to protect the integrity of the dikes. In such a case, the preservation of the dikes is more important than effluent quality, and the boards may be removed quickly.

In operating the weir, it is necessary to keep floating debris from lodging in front of the weir as this will result in more of the flow coming from greater depths with higher suspended solids concentrations. If multiple weirs or a weir with several sections are used in a basin, the crests of all weirs or weir sections should be kept at the same elevation.

If the effluent quality deteriorates below an acceptable limit, the ponding depth (y_o) must be increased by raising the elevation of the weir crest, that is, by adding more boards to the weir. If the weir crest is at the highest possible elevation and the effluent quality is still unacceptable, the weir loading (Q/B) must be decreased by lowering the flow into the basin and over the weir. The flow may be lowered by using a smaller

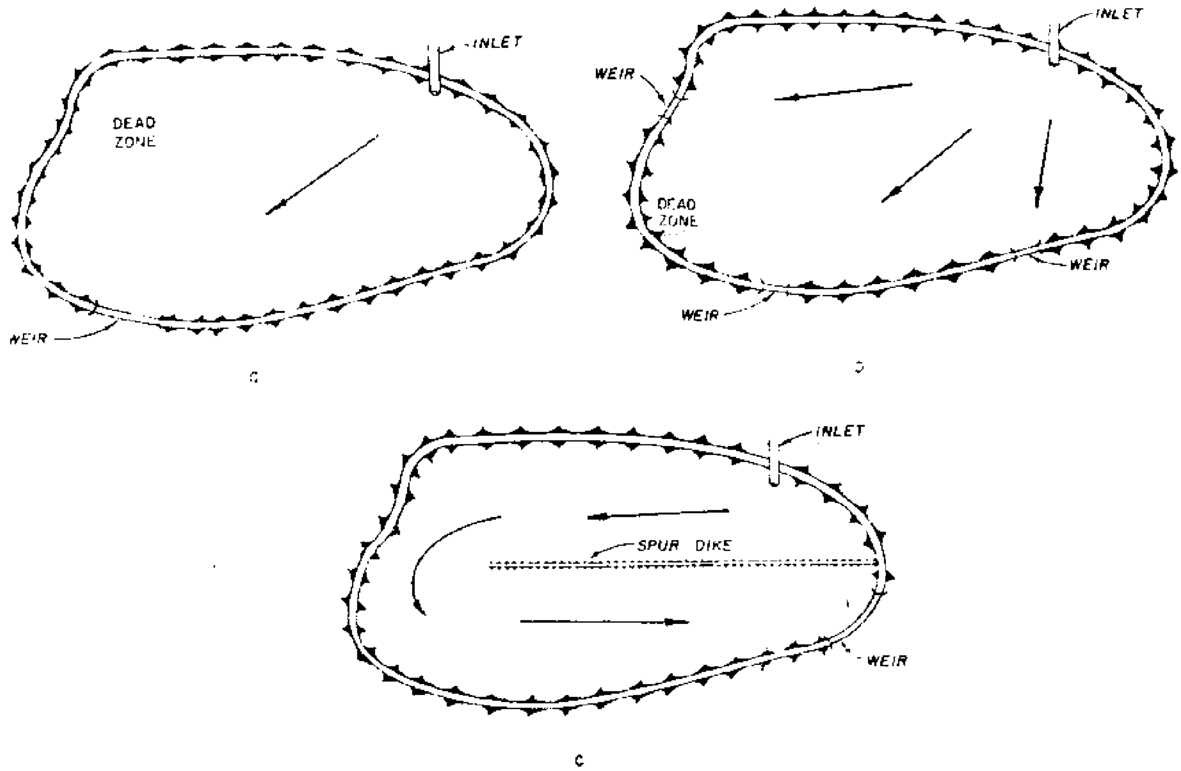


Figure 11. Short-circuiting and dead zones

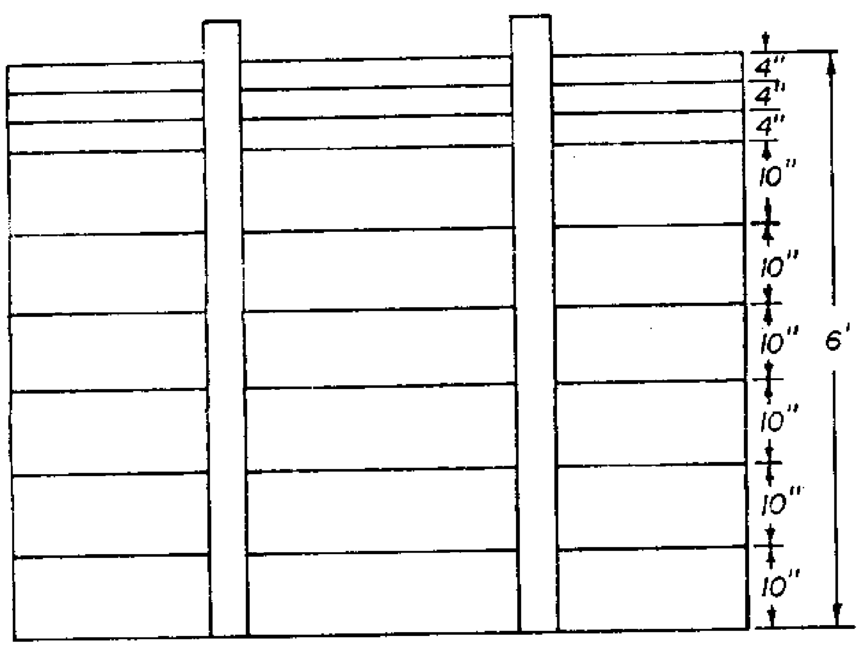


Figure 12. Suggested weir boarding

dredge or by operating the existing dredge intermittently. The new weir loading may be selected by using the nomograms or by measuring the effluent quality for various weir loadings. The weir loading is controlled in the field by using the head over the weir as an operational parameter since the flow over the weir (Q) cannot easily be measured.

<u>Operating head</u>

The head over the weir is the best criterion for weir operation. While the weir loading is a very useful design parameter, the head is the operational parameter used to control weir loading. They are related by the following equation for sharp-crested weirs.

$$H = \left(0.3\ \frac{Q}{B}\right)^{2/3}$$

where

H = static head over the weir, ft
Q = flow over the weir, cfs
B = weir length, ft
Q/B = weir loading, cfs/ft

Using the above equation with the weir loading selected from the nomogram, the operator or designer can determine the maximum allowable head to prevent deterioration of the effluent quality. If the head in the basin exceeds this value, the dredging should be discontinued until sufficient water is discharged from the weir to lower the head to an acceptable level. The dredging should then be performed intermittently to maintain the head within an acceptable range, not exceeding the maximum allowable head. The operator does not need to be concerned with the weir loading or head over the weir if acceptable effluent quality is being maintained.

The head over the weir (static head) can be determined by two methods. First, it can be determined directly by using a stage gage, located in the basin where the velocities caused by the weir are small (at least 10 to 20 ft from the weir), to read the elevation of water surface. The elevation of the weir crest can be read from the weir box providing it is calibrated to the same datum as the stage gage. The difference between the elevations of the water surface and the weir crest will equal the static head (see Figure 2). For example, if the elevation of the weir crest read on the weir box is 68 in. and the elevation of the water surface read on the stage gage is 74 in., then the static head equals 6 in. (74 − 68 = 6).

The static head can also be determined indirectly by measuring the depth of flow over the weir, h (see Figure 2). According to Rehbock,[9] the ratio of depth of flow over the weir to static head (h/H) equals 0.85 for sharp-crested weirs. This ratio approaches 0.67 for broad-crested weirs. Since the depth of flow over the weir is directly proportional to the static head, it may be used directly as an operating parameter. In this case, the weir loading can be controlled by the depth of flow over the weir by using the following equation for sharp-crested weirs.

$$h = 0.85H = 0.85 \left(0.3 \frac{Q}{B}\right)^{2/3}$$

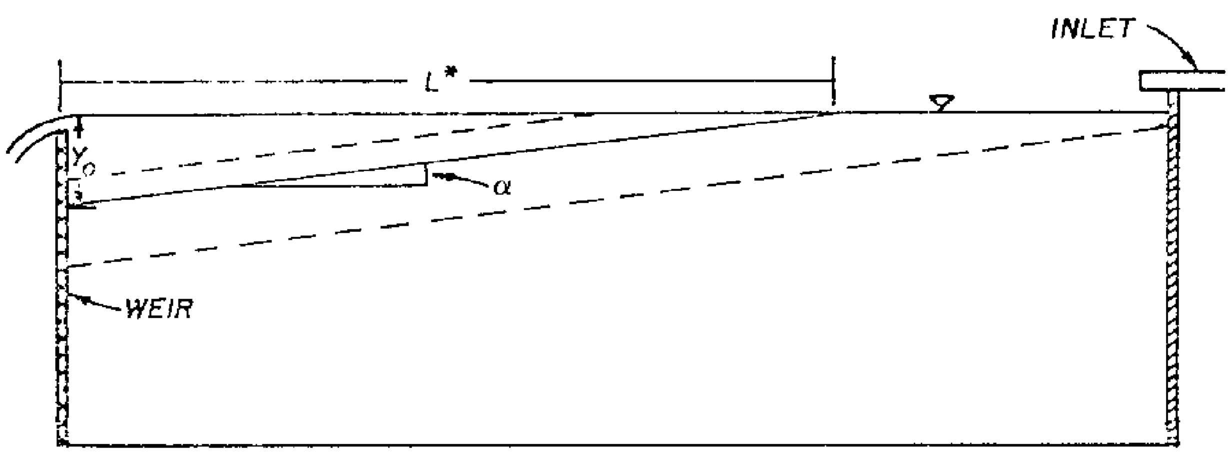
Therefore, using the above equation with the weir loading selected from the nomogram, the operator or designer can determine the maximum allowable depth of flow over the weir to prevent the deterioration of the effluent quality to unacceptable levels. As discussed for the static head, if the maximum allowable depth of flow over the weir is exceeded, the dredge must be operated intermittently to maintain the depth of flow over the weir in a range that does not exceed the maximum allowable value.

The previous equations for the weir loading, static head, and depth of flow over the weir are valid only for sharp-crested weirs. If a different type of weir is used, the above equation must be modified to account for the differences in the coefficient of discharge and the ratio of depth of flow over the weir to static head. Information on polygonal weirs has been documented by Hay and Taylor[3] and Indelkofer and Rouvé.[5]

The length of basin from the weir to the inlet over which water is ponded, hereafter termed the effective basin length (L), can serve as a means for estimating the ponding depth at the weir near the end of the basin's service life. In a basin, the dredged material first settles closer to the inlet and then farther and farther from the inlet. This forms a sloping interface in the basin (see Figure 4). For a given basin with interfacial slope (α) and effective basin length (L), the ponding depth at the weir would be determined by the following equation. (See Figure 13).

$$y_o = \alpha L$$

A typical value for α is 0.002 ft/ft.



L* = CRITICAL EFFECTIVE BASIN LENGTH
Y_0 = DESIGN PONDING DEPTH
α = SLOPE OF INTERFACE

Figure 13. Effective basin length

If the calculated ponding depth from the above equation is less than the design ponding depth, the operator should use the nomogram to select a lower weir loading in order to maintain the effluent quality.

In a similar manner, the equation can be used to solve for the approximate effective basin length needed to maintain the design ponding depth, hereafter termed the critical effective basin length (L*). (See Figure 13.)

$$L^* = \frac{y_o}{\alpha}$$

when the effective basin length approaches the critical effective length, the operator knows the basin is at the end of its service life and the weir loading must be lowered if he wishes to extend the basin's service life without deteriorating the effluent quality.

Basin drawdown

Similarly, once the dredging operation is completed, the ponded water must be removed so that drying can occur. To drain the basin, the weir boards should be removed one row at a time. Preferably, 2- by 4-in. boards should be used in order to minimize the withdrawal of settled solids. The next row of boards should not be removed until the water level is drawn down to the weir crest and the outflow is low. This process should be continued until the interface is reached. It is desirable to eventually remove the boards below the interface so that rainwater can drain from the area. These boards can be removed only after the material has consolidated sufficiently so that it will not flow from the basin. If it begins to do so, the boards should be replaced.

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DREDGED MATERIAL DISPOSAL OPERATIONS RESEARCH

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and

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ABSTRACT

This paper reviews the results of five years of research conducted under the Disposal Operations Project (DOP) of the Dredged Material Research Program. The research was divided into five major areas or tasks:

a. Confined disposal area operations
b. Dredged material densification
c. Disposal area reuse
d. Treatment of contaminated dredged material
e. Turbidity prediction and control.

Methods were developed for sizing containment areas for both capacity and effluent quality. Guidelines were developed for dike design and construction, selection of equipment, weir design, landscaping, and mosquito and odor control. Methods of densifying dredged material to increase the service life of the containment area were developed and evaluated. Concepts for disposal area reuse management were developed when the area serves as a rehandling basin and thus has infinite life. Schemes were also developed where the life of an area is extended by manipulating the material. Methods for treating dredged material to meet effluent quality standards were evaluated. Methods for predicting the extent and duration of turbidity and fluid mud during dredging operations and disposal were developed. Various methods of controlling turbidity and fluid mud were evaluated.

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Background

The $33 million, five-year Dredged Material Research Program (DMRP), the largest and most diverse research program ever undertaken by the Civil Works Directorate of the Corps of Engineers, was concluded in 1978 at the Waterways Experiment Station (WES). The purpose of the program was to determine the environmental impacts associated with dredging and disposal operations and to develop methods for eliminating or minimizing any adverse impacts. The Disposal Operations Project (DOP) of the DMRP was concerned primarily with the engineering and operational aspects of the DMRP. The purpose of this paper is to outline some of the results gained through DOP research and to provide an indication of the guidelines now available for implementation.

The research conducted within the DOP was divided into the following five major research areas or tasks:

Containment Area Operations (2C)

Dredged Material Densification (5A)

Disposal Area Reuse (5C)

Treatment of Contaminated Dredged Material (6B)

Turbidity Prediction and Control (6C)

The objectives of each task are given in Table 1. As shown in this table, each task was managed by a WES engineer or scientist.

Containment Area Operations (Task 2C)

Task 2C was probably the most diverse of all of the tasks within the DOP and included research areas such as dike construction, mosquito and odor control, containment area sizing, weir design, and vehicle mobility on dredged material.

One of the most important research efforts was the design of containment areas for fine-grained dredged material. In the past the size of a confined disposal area was determined using various "rules of thumb" and bulking factors. A bulking factor is simply the ratio of the volume occupied by the dredged material in the containment area to the volume of sediment dredged. In some instances the use of bulking factors was adequate. However, with increasing scarcity and cost of land, as well as the cost of building the facilities, more rational design procedures were required. A properly designed containment area must have sufficient volume to contain the material to be dredged, and must have an area large enough to provide sufficient natural sedimentation of the soil particles to meet existing effluent quality standards. When fine-grained dredged material is hydraulically pumped into a containment area, the final evaluation of the surface depends on the initial settling characteristics of the solids suspended in the slurry, and subsequent consolidation of the settled solids. Many previous studies had addressed the settling characteristics of individual particles and the post-depositional consolidation of fine-grained materials; however, little was known about the sedimentation and consolidation of dredged material slurries with average solids concentrations of 15 percent by weight. Based on the results of

TABLE 1

DISPOSAL OPERATIONS PROJECT

Task	Manager	Objective
2C - Containment Area Operation Research	Mr. N. C. Baker Ms. M. E. Poindexter*	Development of new or improved methods for the operation and management of confined disposal areas and associated facilities.
5A - Dredged Material Densification Research	Dr. T. A. Haliburton	Development and testing of promising techniques for dewatering or densifying dredged material using mechanical, biological, and/or chemical techniques prior to, during, and after placement in containment areas.
5C - Disposal area Reuse Research	Mr. R. L. Montgomery	Investigation of dredged material improvement and rehandling procedures aimed at permitting the removal of material from containment areas for landfill or other uses elsewhere.
6B - Treatment of Contaminated Dredged Material Research	Mr. T. K. Moore Dr. W. D. Barnard*	Evaluation of physical, chemical, and/or biological methods for the removal and recycling of dredged material constituents.
[illegible] - Turbidity Prediction and Control Research	Dr. W. D. Barnard	Investigation of the problem of turbidity and development of a predictive capability as well as physical and chemical control methods for employment in both dredging and disposal operations.

*Task Manager after manager was reassigned.

laboratory tests and field evaluations, procedures were developed for designing containment areas. These procedures were based on the sedimentation and consolidation properties of the dredged material slurry which had been determined from relatively simple laboratory tests.[1]

The level of suspended solids in the effluent from a containment area is a function of not only the settling characteristics of the material, but also the retention time available within the facility. Procedures for determining the retention time required are given by Gallagher.[2] It should be noted that the retention time is not a function of the total volume of the facility, but instead depends on the surface area and shape of the containment area as well as the volume of water ponded above the settled dredged material. It is therefore necessary to pond water at the weir or sluice in order to obtain good effluent quality. The weir does not simply skim water from the top of the ponded water; instead, there is a depth of withdrawal below the weir crest. If the ponding depth is greater than the depth of withdrawal, desired water quality standards can usually be met. However, if the depth of withdrawal is greater than the ponding depth, settled dredged material can be resuspended and carried over the weir. Nomograms have been developed relating the length of the weir to the flow, ponding depth, and the desired solids concentration of the effluent.[3]

The stability of dikes surrounding the containment areas has often been a problem in the past. In many previous cases, dike design and construction were the responsibility of the local sponsor or the dredging contractor. In more and more instances the Corps now either takes the responsibility for dike construction or reviews the design before the containment area is built. The state of the art for designing and constructing dikes has long been sufficient; however, in most cases containment area dikes do not need to be built to the same standards used for major earth embankments, such as dams or mainline levees on major rivers. Consequently, guidelines were developed for the design and construction of dikes using current construction methods.[4] These guidelines also include methods of increasing dike stability through improved construction methods.

During the planning stages of a containment area, factors such as landscaping and odor and mosquito control should be considered. The guidelines developed for landscaping containment areas describe the constraints the architect must work within and also present some landscaping concepts. The guidelines also contain a rather extensive list of plants that may be used in containment area landscaping.[5] Guidelines for odor[6] and mosquito control[7] are also available.

Containment areas must be properly managed to maximize their effectiveness. Proper management often requires that vehicles and equipment be used in and around disposal areas. Since many containment areas are built in low-lying areas where foundation properties are very poor and are often filled with dredged material possessing poor engineering characteristics, vehicle and equipment mobility may have been a problem in the past. Based on an evaluation of over 60 vehicles or pieces of equipment, guidelines were developed for their use.[8] By knowing the strength (i.e., rating cone index or RCI) of the material, both the maximum ground contact pressure and the minimum soil strength for a particular vehicle can be determined. The RCI is easily determined using a hand-held cone penetrometer. Detailed procedures for determining the RCI and other parameters are given by Willoughby.[9] Of all the vehicles evaluated only the Riverine

Utility Craft (RUC) can be operated effectively on the very low-strength dredged material often found in disposal areas.

Dredged Material Densification (Task 5A)

Fine-grained maintenance dredged material pumped into containment areas is usually approximately 85 percent water and 15 percent solids by weight. Even after the solids in the slurry have settled in the containment area and the excess water has been discharged over the weir, the remaining material in the area may have water contents of 200 to 400 percent (on a dry weight basis). Although a thin, dry crust may develop on the surface of the area, the underlying dredged material may be characterized by extremely high water contents for many years. Consequently, these disposal areas may contain large volumes of water and relatively small volumes of solids. The objective of this task was to develop economically feasible methods for removing the water and increasing the effective capacities of the disposal area. For fine-grained material removal of a cubic yard of water often provides close to a cubic yard of additional space in the containment area. Therefore the primary purpose of dewatering or densifying the dredged material is simply to increase the volume within the area and secondarily to improve the engineering properties of the material. With improved engineering properties the fine-grained dewatered dredged material may be productively used for landfill or construction material.[10]

There was never any doubt that fine-grained dredged material could be dewatered. However, techniques commonly used for dewatering foundations for relatively small structures may not be economically feasible for dewatering hundreds to thousands of acres of dredged material. Dewatering techniques investigated in both the laboratory and the field during the DMRP included the use of well points, low-voltage-gradient electro-osmosis, surface-loading consolidation, vegetation, underdrains, and trenching.[11] However, of all the methods evaluated, progressive trenching appears to be the most universally feasible method.[12] In some instances the use of underdrains also appears to be attractive. Other methods may have limited application in some specific cases; but because of the wider applicability of progressive trenching and underdrains much more emphasis was placed on the development and refinement of these methods.

The concept of progressive trenching allows for natural evaporation to dewater and densify the dredged material with minimal input from man. The depth of the crust developed over fine-grained dredged material is largely governed by the net evaporation (i.e., evaporation minus precipitation) in the area. Consequently, if precipitation exceeds evaporation no crust, or at the best, a very thin crust may develop. By placing trenches in the disposal areas any ponded surface water and rainfall can be removed relatively quickly. This increases the net evaporation and causes both the rate and depth of crust development to increase.

The primary problem was to develop methods for trenching very soft, dredged material. Fortunately, surface trenching can be accomplished with the Riverine Utility Craft (RUC). The RUC is propelled by two rotors, which produce two parallel trenches as the RUC moves over the dredged material. Using this technique a network of trenches can be produced in a relatively short period of time. Because of the soft nature of the material, the initial trenches may be only 2-4 in. deep; however, these depressions collect and drain the rainfall and any surface water. A crust and associated desiccation cracks gradually develop down to the bottom of these trenches. By using the RUC to periodically and progressively deepen the trenches, a crust of 1½ - 2 feet can be developed. At this point, more conventional, low-ground pressure equipment can be used to continue deepening the trenches. It should be stressed that the trenches remove only the surface - and not the subsurface-water. The progressive trenching technique has been successfully applied in field demonstrations in the United States. A similar system has been in use in the Netherlands for the past several years.[13]

In areas where sand in large quantities is dredged in combination with fine-grained material, it may be feasible to use the sand to construct drainage blankets in the containment areas prior to disposing of the fine-grained dredged material. Such a blanket must also incorporate collector pipes in order to be effective. The concept of dewatering fine-grained dredged material using underdrains was evaluated during field tests at the Upper Polecat Bay disposal area in the Mobile District. In addition to evaluating a simple gravity flow system, a vacuum was applied to the sand in one test pit. The concept of ponding water over the fine-grained material to produce downward seepage forces and to enhance consolidation was also evaluated with simple gravity flow and with application of a vacuum to the sand layer. Tentative results indicate that underdrain systems may be effective, but final results will not be available until tests are completed in 1979.

The DMRP synthesis report[12] entitled "Guidelines for Dewatering/Densifying Confined Dredged Material" presents guidelines for the progressive trenching techniques and provides a methodology for predicting the volume of space thus produced. This reference also presents interim guidelines for the design and use of underdrain systems.

Disposal Area Reuse (Task 5C)

A completely reusable disposal site is one which serves as a rehandling basin. That is, dredged material is placed in the area and then removed for upland disposal or for a productive use. Since complete removal of all the dredged material from a containment area will rarely occur, a site may be considered to be reusable if the capacity of the site is significantly increased through the rehandling of some of the dredged material.

Earlier studies within this task indicated that there may be a large demand for construction or landfill material in areas close to dredging operations.[14] However, in many cases where only the coarser-grained material is desirable, it may be necessary to selectively remove various fractions of the dredged material.[15] The DMRP synthesis report[16] entitled "Guidelines for Dredged Material Disposal Area Reuse" presents complete guidelines for selecting and developing reusable disposal sites.

The reusable site concept was applied to the Upper Polecat Bay disposal site. Large volumes of dewatered dredged material (i.e., thickened crust) were produced from the field tests previously described and used to raise the dikes of the area. In addition to the dewatered material obtained around the inside perimeter of the area, a system of roads specifically designed for soft foundations was constructed in the disposal area to provide access to additional dried crust. The dried material within the site was "mined" and transported to the dikes with trucks. In addition to removing the dredged material and using it productively to increase the dike height, each cubic yard of dredged material removed provided an additional cubic yard of effective capacity for future disposal operations.

Treatment of Contaminated Dredged Material (Task 6B)

Special emphasis within this task was placed on the treatment of effluent from confined disposal sites. Research within the DMRP indicated that the vast majority of the contaminants associated with effluent from a confined disposal area is closely associated with the suspended solids.[17] Consequently, removal of solids will in most cases produce an effluent of sufficient quality to meet environmental standards.

Primary treatment of the dredged material slurry involves the natural settling of the solids within the disposal area. In most cases effluent from a properly designed and managed disposal area will probably meet applicable effluent standards; however, in some cases additional treatment may be required. Such treatment may take the form of chemical flocculation or filtration.[18]

Laboratory studies[19] indicated that through the use of chemical flocculation nearly all the suspended solids and associated contaminants can be removed from the effluent. Although this is easily accomplished in the laboratory, flocculant application during disposal operations is often difficult and costly. Injection of flocculants into the pipeline produces highly unpredictable results primarily due to the fact that dredged material slurry discharged into a containment area during a hydraulic dredging operation is highly variable. Solids concentrations can vary from 0 to 40 percent solids by weight over a very short period of time. Also there may be a wide variation in the characteristics of the sediment being dredged. Since the dosage of chemicals necessary to provide effective flocculation is highly dependent on both the material and the solids concentrations, the wide variation in both of these parameters makes it very difficult to design a system for treating the dredged material in the pipeline before it is discharged into the containment area. An alternative method involves treating the suspended material in the effluent at the weir. The relative effectiveness of effluent treatment is higher due to the fact that the average solids concentration and variability in the composition of the material is greatly reduced. Guidelines are available for treating dredged material in the dredge pipeline and at the weir.[20]

Filtration is another method for removing suspended solids from the effluent. Guidelines are now available for the design of pervious dikes and other types of filtering systems.[21] The concept of vegetative filtering was also evaluated in a series of field tests; however, no specific guidelines could be developed based on the results of these tests.

Turbidity Prediction and Control (Task 6C)[22]

Certainly one of the most visible effects associated with many dredging and open-water disposal operations is the generation of turbidity. Fortunately, the turbidity generated during dredging and disposal activities usually has little environmental impact; most of the problems associated with turbidity are basically aesthetic in nature. Ninety-seven to 99 percent of any fine-grained material discharged during open-water pipeline disposal operations descends rapidly to the bottom where it accumulates under the discharge point in the form of a low-gradient fluid mud mound. Because the dissolved oxygen levels in the fluid mud layer are close to zero, the impact on benthic organisms covered by the fluid mud may be significant.

During open-water pipeline disposal operations only one to three percent of the material remains suspended in the water column above the fluid mud layer in the form of a turbidity plume. The characteristics of the turbidity plume are highly dependent on the hydrodynamic regime in the area, the sediment being dredged, the production rate of the dredge, the age of the plume, and the water depth. With estimates of these parameters, a prediction of the solids concentrations as a function of distance from the discharge can be made using a series of nomograms.[23]

The turbidity generated by the dredging process is generally very small compared to that generated by the disposal process, particularly when open-water pipeline disposal is used. Downstream of clamshell operations turbidity plumes may be 300 to 500 meters long with suspended solids concentrations in the water column of generally less than 500 mg/ℓ. Within 3 m of the cutter of a cutterhead dredge suspended solids levels may be as high as a few tens of grams per liter (g/ℓ) with concentrations decreasing exponentially with distance from the cutter. Suspended solids levels in the vicinity of the cutter tend to increase with increasing rates of production. Around a hopper dredge draghead turbidity levels are probably less than a few grams per liter. Turbidity levels in the near-surface overflow plume decrease with increasing distance from the discharge ports and quickly reach levels of less than 1 g/ℓ.

A commonly used method for controlling turbidity involves the use of silt curtains or turbidity barriers. These are simply impervious plastic curtains used to surround a source of turbidity. These curtains were extensively evaluated and their effectiveness was determined. In general, the silt curtains are not effective in areas where currents exceed approximately one knot or where high waves can be expected; however, silt curtains may be effective in quiescent areas. Guidelines and specifications were developed for the procurement and use of silt curtains.[24]

Concluding Remarks

As a result of the DMRP many major questions associated with the environmental impact of dredging were evaluated and methods to minimize any adverse effects developed. Of course, during the five-year period of the DMRP all questions could not be answered and, in fact, some were not even addressed. Many of these questions are now being evaluated through research being conducted jointly by the Corps of Engineers and the U. S. Environmental Protection Agency (EPA) as part of their effort to develop criteria and guidelines. Some remaining questions concerning long-term effects of dredged material disposal are also being addressed through the Corps of Engineers Dredging Operations Technical Support (DOTS) program. In this program relatively low-level monitoring of some open-water disposal sites and habitat development sites is continuing. Also verification and refinement of many of the techniques discussed in this paper will continue. As part of the DOTS program the WES provides assistance to the various Corps of Engineers elements in assessing potential problems and in developing methods of minimizing or alleviating adverse environmental impacts associated with dredging and disposal operations.

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DEWATERING DREDGED MATERIAL BY EXPLOSIVE TRENCHING

by

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ABSTRACT

Dewatering confined dredged material using explosive trenching methods has undergone two major trials by the Panama Canal Company's Dredging Division. Ditching unconsolidated dredged material has heretofore been a costly operation requiring either special equipment or costly ancillary cribbing to use standard excavation equipment. The Panama Canal Company, using conventional explosives, has developed a ditching technique that has proven effective and economical. The paper discusses the techniques and methods used in two distinct confined disposal areas and the results obtained thus far in the post ditching evaluation.

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INTRODUCTION

The disposal of dredged material is a major problem found everywhere in the dredging industry. The Panama Canal faces the same problems port and river authorities have throughout the world--what to do with the dredged material and how to manage it.

The Panama Canal Company's Dredging Division, which is tasked with all maintenance and construction dredging throughout the major waterway has developed and tested the use of explosives for the construction of drainage systems to dewater confined dredged material. This paper will briefly discuss the dredging mission, history and equipment as a prelude to a more detailed description of the explosive trenching techniques and the results obtained on two large scale field trials.

When the Canal opened in 1914, 225 million cubic yards of earth and rock had been torn from the jungle and moved to create the waterway. Since the opening of the Canal, four times more material has been moved in maintenance dredging than was moved during the entire construction period.

DREDGING FLEET

In 1913, the Panama Canal's dredging fleet was centralized under one division and headquartered at Paraiso, near Pedro Miguel Locks. In September, 1936, the Dredging Division relocated to Gamboa, at the north end of Gaillard Cut on the east bank of Gatun Lake, in order to have the reserve dredging fleet north of any serious slide that might occur in Gaillard Cut. Gamboa is the geographic center of the Canal and thus the logical center for dredging operations. With the dredging disposal areas used in dipper dredge operations located in Gatun Lake and the equipment moored at Paraiso, such equipment would have been practically useless if a slide had interrupted traffic through the narrow eight-mile-long Gaillard Cut.

The present Panama Canal dredging fleet consists of three large dredges, a multipurpose clamshell dredge, and a drillboat. The U.S. MINDI is a cutterhead suction dredge capable of moving 1,200 cubic yards of earth per hour. She is operated an average of eight months each year in maintenance and capital improvement projects. The MINDI is employed primarily in the Gatun Lake channels, the ocean approaches, the harbors and anchorage areas, but is occasionally used to sweep Gaillard Cut. The MINDI is 295 feet long, 50 feet in beam, and can dig to a 72-foot depth. She was built in 1942 and is being repowered with diesel-electric engines and modified with the addition of a ladder pump and an additional main pump. Cutter horsepower will be 600 and the total installed horsepower will be 11,000.

The U.S. CASCADAS is a 15-cubic-yard steam-powered dipper dredge with a normal production of 500 cubic yards per hour. The CASCADAS was built in 1915 and when placed in operation, set what was believed to be a world record for a day's work by any kind of excavating machine in hard material, excavating a total of 23,305 cubic yards of rock and earth on February 18, 1916, during an actual working time of 23 hours and 15 minutes. From October 31, 1915, to March 20, 1916, slightly over 4 1/2 months, she excavated 1,447,900 cubic yards and was delayed by breakdowns only 77 hours and 35 minutes. A 13-cent Canal Zone stamp was issued on February 23, 1976, commemorating the U.S. CASCADAS and her service to the Panama Canal. Despite her age, the CASCADAS remains a reliable and productive member of the dredging fleet. She has been operating for seven months this present year in dredging blasted rock and earth, primarily from Gaillard Cut. Both of these dredges are operated 24 hours per day, 7 days per week when they are in service.

A contract was awarded in 1976 for a new 15-cubic-yard diesel-electric dipper dredge to replace the inoperable and decommissioned dipper dredge U.S. PARAISO, which has been out of service since 1971. This new dredge, the U.S. RIALTO M. CHRISTENSEN, is the largest diesel-electric dipper dredge in the world and is presently in service widening and deepening the Canal.

The CHRISTENSEN (a 15-cubic-yard dipper dredge), manufactured by the Hakodate Dock Co. in Japan, was delivered in September 1977, and has undergone start-up trials during the previous year. Powered by two 2,150 hp main engines, this diesel-electric dredge can dredge at depths of 60 feet with a cycle of 60 seconds and develops a total excavating force of 320,000 lbs. Initial operations indicate that the CHRISTENSEN will dig harder material faster than the record breaking CASCADAS.

The crane barge/clamshell dredge U.S. GOLIATH, which can be equipped with a 7 1/2 cubic-yard clamshell bucket, was built in 1969 and is primarily used for removing large boulders and small shoals which frequently develop in Gaillard Cut and also provides a floating heavy lift capability of up to 90 tons.

The drillboat U.S. THOR is a diesel, pneumatic-powered floating drill platform which mounts four drill towers. The THOR is employed in subaqueous drilling and blasting. Such blast fracturing is required in rock areas in order for the dipper dredge to excavate the material. The explosive currently used in these underwater blasting operations is 60 percent ammonium nitrate gelatin dynamite. Water-gel explosives are being tested to determine if they are adaptable to the underwater blasting operations in the Canal. The most efficient and economical rock breakage in Gaillard Cut is achieved by drilling on 9- by 12.5-foot centers and using a powder factor of 1.5 pounds per cubic yard.

Supporting this dredging fleet is a variety of tugs, launches, work floats and lighter barges. The frequent movements of this support equipment and the continuous operations of the dredges and drillboat proceed without interference to transiting vessels.

DREDGED MATERIAL DISPOSAL

As material is dredged from the Canal, it must be disposed of rapidly and efficiently. Positioned throughout the length of the Canal are disposal areas for the deposition of dredged material. Many of these areas are located in the man-made fresh water Gatun Lake, a sufficient distance from the canal prism to minimize re-silting of the Canal. Our main disposal areas on the Pacific and Atlantic entrances are located on the banks adjacent to the Canal in confined or diked areas. Due to the accumulation of dredged material over the years, and the continual raising of the confining dikes, we have reached a point where additional raising of the dikes will be very expensive because of slope stability considerations. As a result, the Panama Canal Company is reevaluating its past dredged material management practices in order to maximize the space available inside the confined dredge disposal area.

The material dredged from Panama Canal waters in maintenance dredging operations is uncontaminated by industrial and agricultural pollutants, although there are highly localized small areas of biologically active material near sewerage outfalls. Therefore, the disposal of the material in land and water disposal areas has not generated the adverse environmental impact prevalent in other areas of the world. Sixty years of observation plus a recent study by consultants on the environmental effects of dredging and disposing of dredged material have indicated that the effects of such operations on this largely man-made environment are, if anything, benign.

During the period from July to December, 1976, while performing routine harbor and channel maintenance dredging, the cutter suction dredge U.S. MINDI pumped 1.7 million cubic yards into the main Pacific disposal site, Velasquez, which amounted to a depth of approximately four feet throughout the 260-acre area.

This disposal area was originally established in 1946 and has been used on a recurring basis since that time. The present elevation is some 30 feet over the original swamp and the containment dikes have been successively raised to provide additional volume. As is unfortunately a common practice in such cases, no provision for underdrainage was provided at the outset, therefore the area has become increasingly smaller as the dike construction encroached on the storage area. Land development surrounding the site has limited the expansion of the disposal area. An alternate disposal site has fallen prey to the conservationists and has been designated a wild-life sanctuary, so a very real problem so common to all of us in the dredging industry faced us squarely. On the basis of our close tracking of the extensive WES Dredged Material Disposal Studies it appeared as if dewatering offered some potential relief to our plight. In 1976 our first feeble dewatering efforts commenced at Velasquez with the drilling of six 8-inch horizontal drain holes through the toe of the dike some 100 yards into the dredge material. Gravel was blown into the 8-inch PVC casing in some cases and in others the casing was left unpacked. The results were initially good with heavy flows of highly saline water developing. These results were short lived and dewatering highly localized. The drain pipes were blocked by the fine clay silt and after some further futile efforts the project abandoned. The dike was raised some five feet with a final dike elevation as high as forty feet over original grade in some areas.

The idea of dewatering still remained active in our thought process and results reported at last year's Dredging Technology Seminar reinforced our conviction that there had to be an economic means to efficiently drain at least the surface water from the disposal area in order to enhance evaporative drying and consolidation of the material.

Because of our extensive experience and involvement with explosives it is not surprising that explosive trenching developed as a primary experimental project to establish a surface drainage system. Therefore, in March 1978, explosive operations were begun to construct a series of drainage ditches to provide a fast runoff of water during rains. The benefits of evaporative drying have been well established by previous studies and actual practice so our presentation deals with the mechanics of installing drainage ditches and the results that we have experienced.

ALTERNATIVES CONSIDERED

Light-weight excavation equipment that may be locally available in proximity to United States dredging works was simply not available from Panama Canal resources or from contractors in the Republic of Panama. Company-owned conventional excavation equipment could not be used because it was committed on other jobs in the Canal to support canal operation and for essential work in the community. Contractor excavation costs in the Canal Zone are somewhat higher than in many United States locations for a variety of reasons: i.e., lack of competition, expensive repair parts, inefficient contractor practices due to low experience levels, high demand for equipment during the dry season, higher cost of government work, etc. It was estimated that conventional excavation, if available from contractors (clamshells, backhoes, draglines, all working off pads) would cost between \$5-\$7/yard.

Using explosives was considered and we decided to run some tests. We consulted FM/5-34, the Army Engineer Field Data, and found that using explosives was a proven military method of constructing ditches. The Engineer Field Data did not provide us with the precise information we needed because of the difference in explosives we were using. The standard explosive used in the Panama Canal is 60 percent ammonium-gel dynamite, 3 inches in diameter, 2 feet long, and weighs 8.33 lbs. per stick.

Running a series of test shots showed that a pattern of 1/2 stick or 4 lbs. at 3-foot intervals produced a ditch 5-feet deep by 10-feet wide. Successive blasting to deepen the ditches was not as effective. A pattern of 1/2 stick at 3-foot intervals following the initial ditching resulted only in an additional 1/2 to 1 foot of depth, and this effort has gone back to the drawing board. After these successful trials we decided to perform a large scale field trial in the Pacific diked disposal area.

LAYING OUT OUR DITCH LINE

The low end of the Velasquez disposal area was selected for the ditching since we thought the high end would drain naturally, due to its higher elevation. The high end of the disposal area is located near to the two dredge pipe discharge points and the elevation drops off from these points with the lowest elevations near the single drop inlet spillway. There is a 3- to 5-foot difference in grade between the high and low end of Velasquez.

The ditch lines were laid out more or less perpendicular to the contour lines. Our original ditch line was laid out using two drainage outlets as shown in Figure 1.

We had to somewhat modify this pattern because the southwest part of the disposal area was still too fluid and too unstable to sustain ditching. The explosive just churned up the dredged material. Three days later, however, the immediate area where the explosives were detonated had settled by as much as 1 1/2 feet in some areas. This was caused by the exposure of the underlying very wet material to evaporative drying.

Unable to follow our original pattern, we modified it to meet field conditions, as shown in Figure 2.

Our completed pattern gave us drainage throughout the lower end of the Velasquez disposal area.

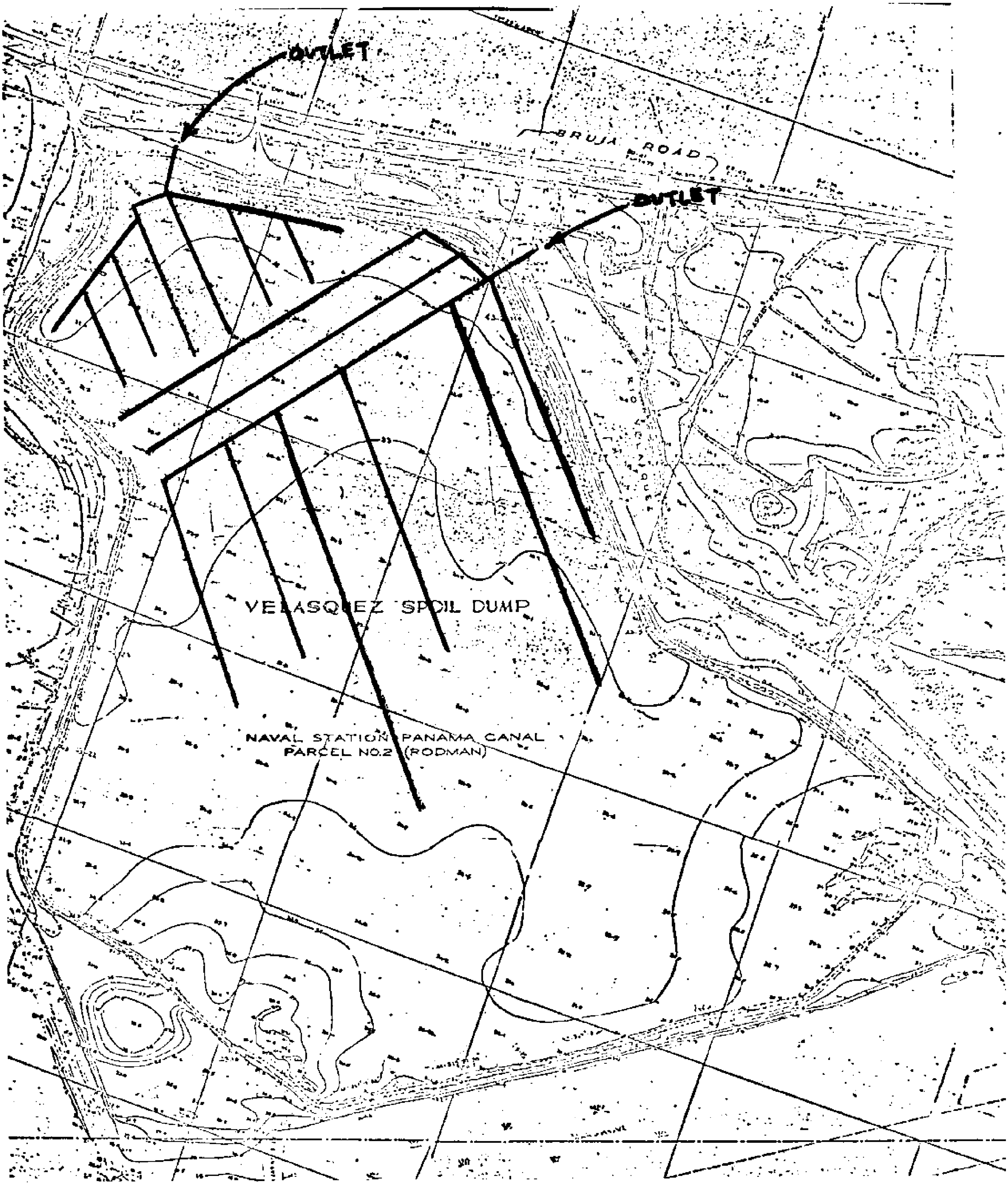

OUTLET
OUTLET
BRUJA ROAD
VELASQUEZ SPOIL DUMP
NAVAL STATION PANAMA CANAL
PARCEL NO.2 (RODMAN)

FIGURE 1

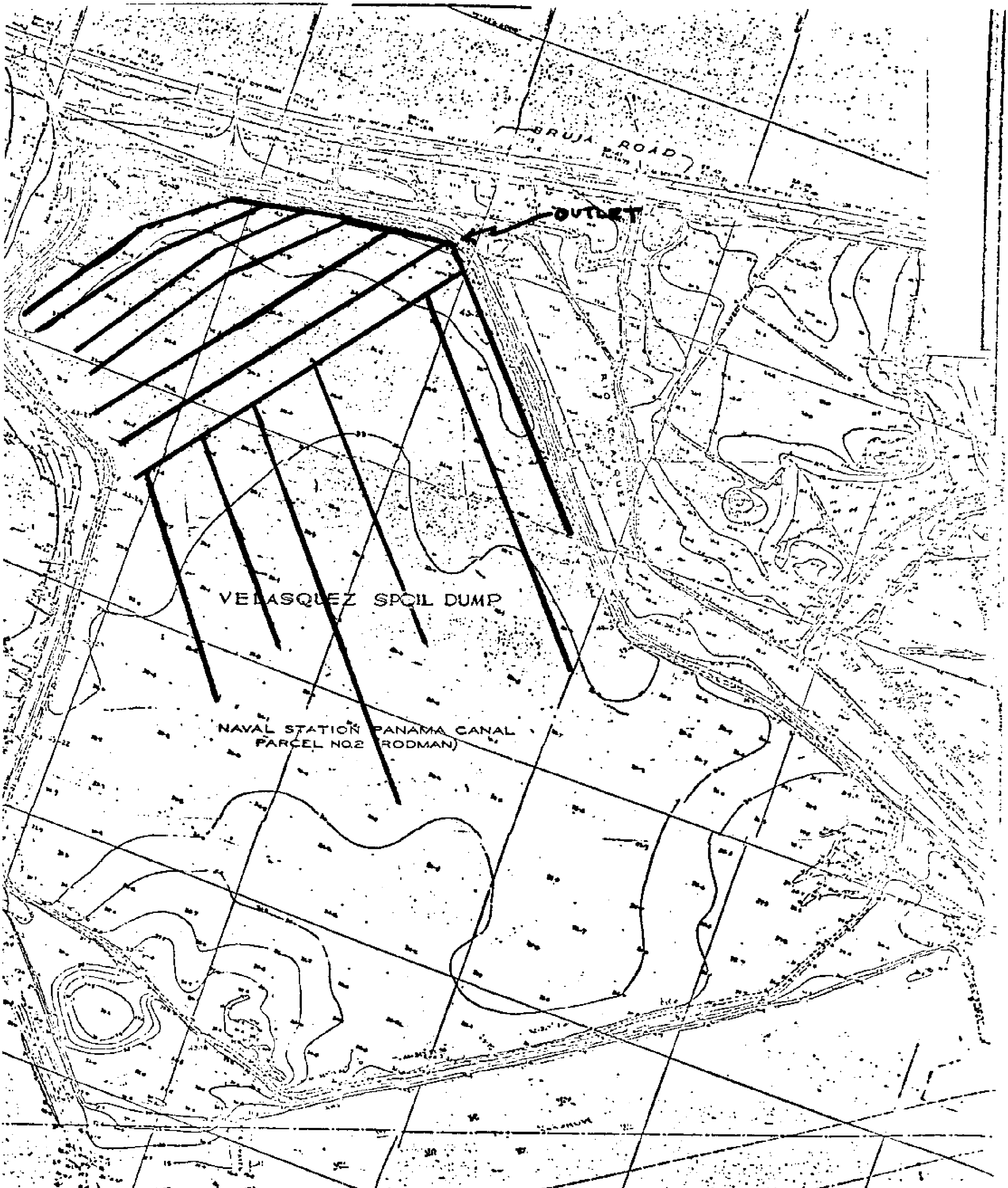

BRUJA ROAD
OUTLET
VELASQUEZ SPOIL DUMP
NAVAL STATION PANAMA CANAL
PARCEL NO.2 (RODMAN)

FIGURE 2

Despite the complete absence of rain during the duration of this project, which was accomplished during the four-month dry season, water began to accumulate and flow in the ditches. There was no water immediately following the blast, but on the following day, water had accumulated and was flowing. Water was draining from the exposed ditch wall, thereby helping the process of consolidation in the vicinity of the ditch line.

The trenching production rate was in excess of 1,000 feet per day and the entire project was completed in three weeks. The total cost was about $21,000.

Dynamite, primacord, caps, etc., were $9,000 and labor was $12,000, or a completed cost of about $1 per lineal foot. The most time-consuming operation was transporting the explosives to the site, which was done manually.

The following sequence was used in constructing the ditches:

1. The ditch line was laid out.
2. Dynamite intervals measured.
3. The hard crust removed by a post-hole digger where necessary.
4. Holes formed using a wood plug forced into the material.
5. Dynamite with attached detonator cord placed.
6. Detonator cord attached.
7. Blast.

PROCEDURE

The first three blasts were approximately 1,000 feet or 1,000 lbs. each and despite the close proximity to the housing area, no damage occurred until the third blast, which had progressed several hundred feet closer to the Navy and Marine housing area, where approximately 45 window panes were broken.

The blasts were reduced to 500-foot lengths and the occupants were instructed to open their windows immediately prior to the blast. This procedure eliminated further damage.

The atmospheric conditions during blasting were characterized by northerly dry season winds of 15-30 m.p.h. and may have contributed to the blast effect. Blasting was done in the afternoon, the time of day of the high winds, because the Nursery School was in session in close proximity during the morning hours. We delayed our shots until there was a momentary wind lull and then shot. Debris was thrown several hundred feet into the air and small particles a distance of about 500 feet. The large clumps of crust and muck were deposited adjacent to the ditch line.

Navy security personnel provided an invaluable assistance to our efforts in providing security. Despite some inconveniences, the entire Navy community gave us their support, which made this type of project possible.

In the seven months since the project was completed, there has been some siltation in the ditches but even during heavy tropical downpours, there is still about a foot of freeboard in the ditches with the ditches flowing full.

Our results are still coming in; i.e., moisture content of the soil and the increase in volume due to drainage and consolidation. However, some things are readily apparent. The high end of the disposal area, despite our assumption that it would be well drained, remains wet and soggy while the low end, where the ditches are constructed, are dry and can be walked on without picking up any mud one day after a heavy rain.

So it appears that in order to get beneficial results throughout the site, the entire area must be provided with drainage channels.

MAINTENANCE OF DITCHES

We have deliberately performed no maintenance on the ditches in order to determine the effects of siltation/erosion.

Our preliminary data indicates that these ditches could be self-cleaning to some extent providing the proper gradient is used. Another factor is the length of the ditch which will provide a self-cleaning action near the outlet (as more and more water flows into it) while having the inevitable siltation near the start of the ditch.

The maximum ditch width has increased to about 16 feet while the depth has decreased to about 3 feet, resulting in a shallower but wider ditch with a greater water carrying capacity. Rainfall from March to October of this year has been 64 inches, and this translates into a volume of over 60 million cubic feet of water that has passed through these ditches.

After the completion of drainage ditches at Velasquez, we moved to Telfer's Island on the Atlantic where a slightly different technique was used. Telfer's Island has a dense concentration of mangrove and roots and the charge was increased to a full stick (8.33 lbs.) at 5-foot intervals. This gave us a slightly wider ditch with gentler sloping sides permitting the ditches to stay open despite the more unstable material in that area as opposed to Velasquez.

OTHER BENEFICIAL RESULTS

The construction of the drainage ditches has had other beneficial effects. Mosquito control has been enhanced and the Canal's sanitation Division has realized savings of possibly as high as $30,000 by not having to cut surface drainage ditches manually for mosquito control and by reducing the amount of insecticides, etc. used for this purpose. These savings alone pay for the outlay of construction.

Let us review some of the advantages of explosive ditching:

ADVANTAGES

1. Zero capital investment: Use existing on-hand tools and methods. Conventional techniques require investment in specialized equipment that may not be fully utilized.
2. Effective: Ditches are blasted to a depth of 5 feet.

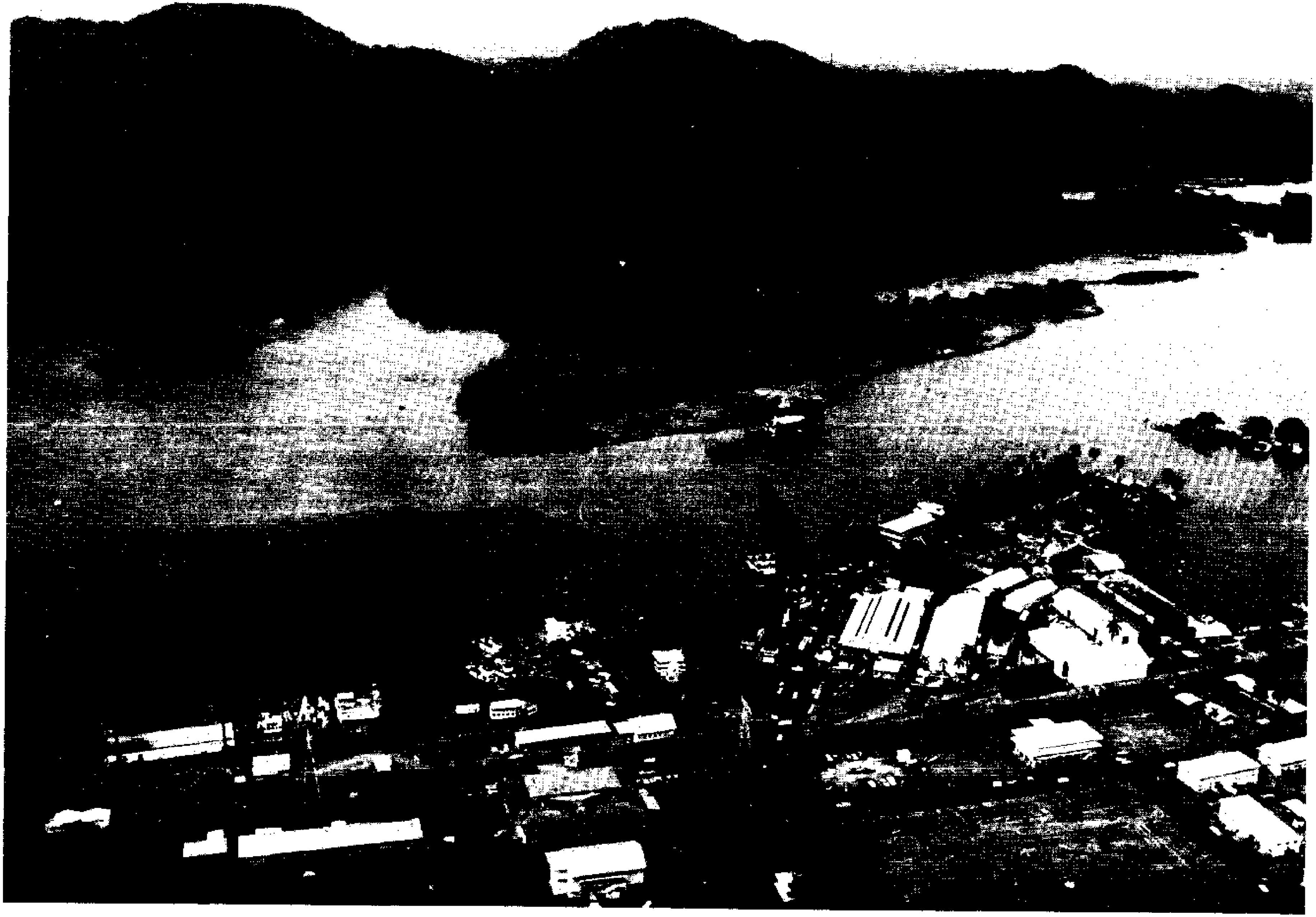
3. Quick: 1,000+ feet of ditch line can be easily constructed in one day.
4. Economical: Cost of $1.00 per lineal foot.
5. Safe: Provided safeguards and safe practices are strictly adhered to.

LIMITATIONS

This new application of an old technique cannot be used in every location. Where industrial facilities, transportation routes, or residential areas are close at hand the local conditions may not permit the use of high explosives. But, for the Panama Canal, explosive ditching has proved itself a safe, efficient, rapid and cost effective technique for dewatering dredged material in confined disposal sites.

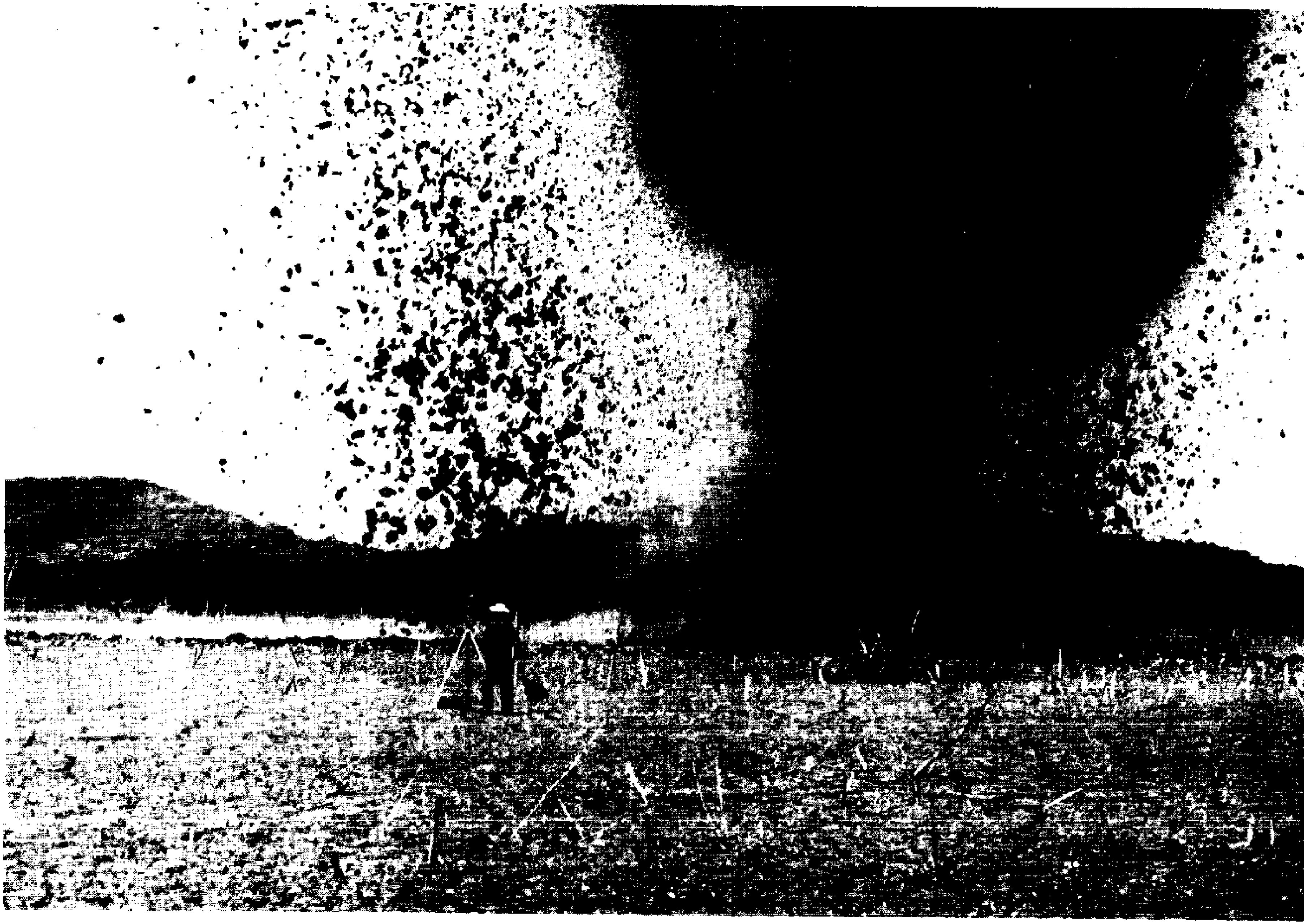
SUMMARY

To summarize, dewatering dredged material is an accepted goal to be striven for and presents manifold benefits. State-of-the-art drainage construction techniques reported in the literature were costly and cumbersome. Establishing an effective and economic drainage system using explosive trenching has shown encouraging initial results in our two major field trials and may provide a useful tool to the dredging industry in suitable situations. We are continuing to develop quantifiable data on our field trials which will include soil analysis, water table elevation changes, net surface drainage and other parameters that will assist in the extrapolation of our results to systems having diverse characteristics.



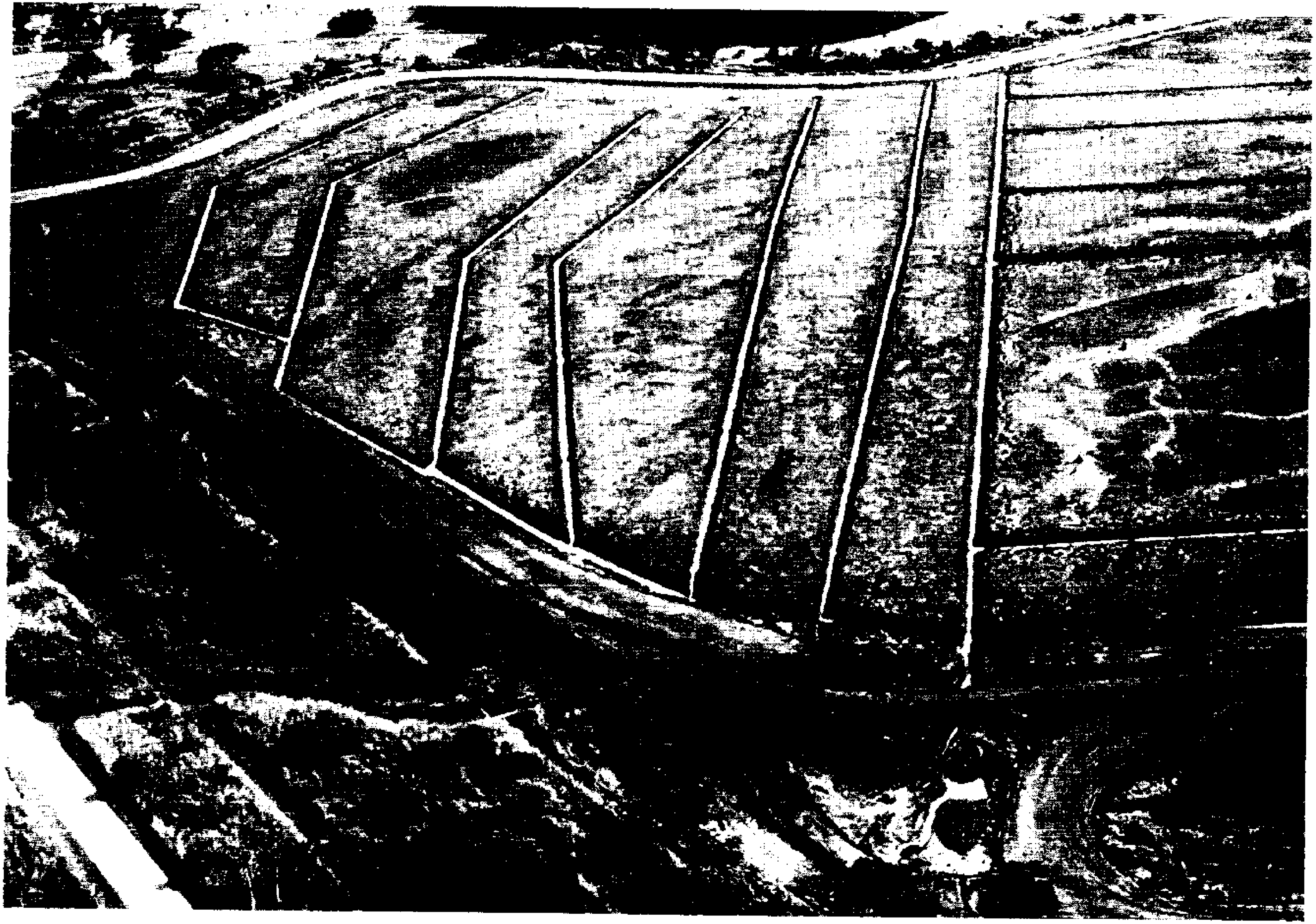
GAMBOA - HOME OF THE PANAMA CANAL COMPANY'S DREDGING DIVISION,
LOCATED ON THE BANKS OF THE CANAL

CONSTRUCTION SEQUENCE - PLUNGER BEING USED TO FORM HOLES
IN SURFACE CRUST AND DYNAMITE LOADING.

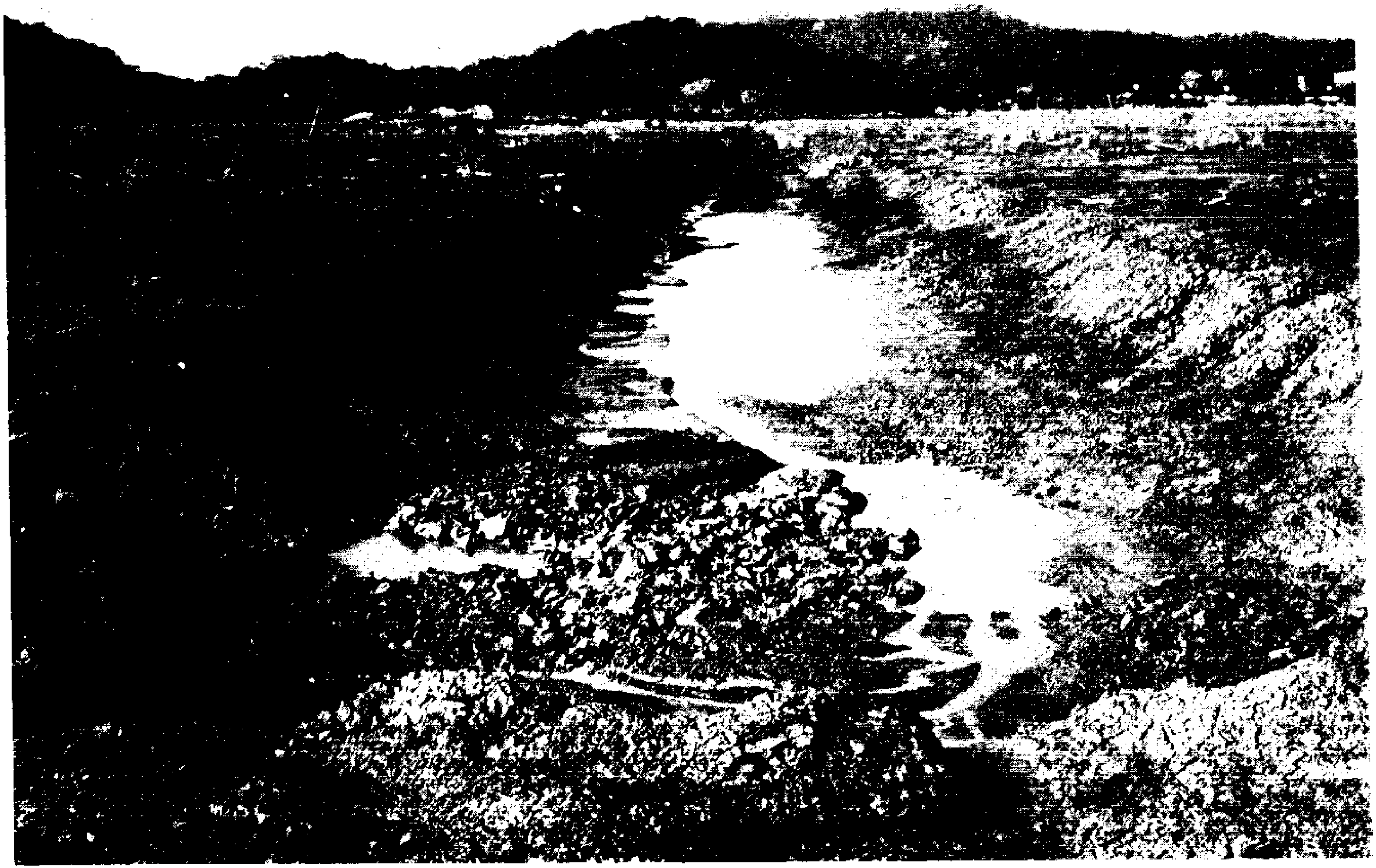


EXPLOSIVE TRENCHING - IN PROGRESS
500 POUNDS OF DYNAMITE LOOKING DOWN AXIS OF SHOT

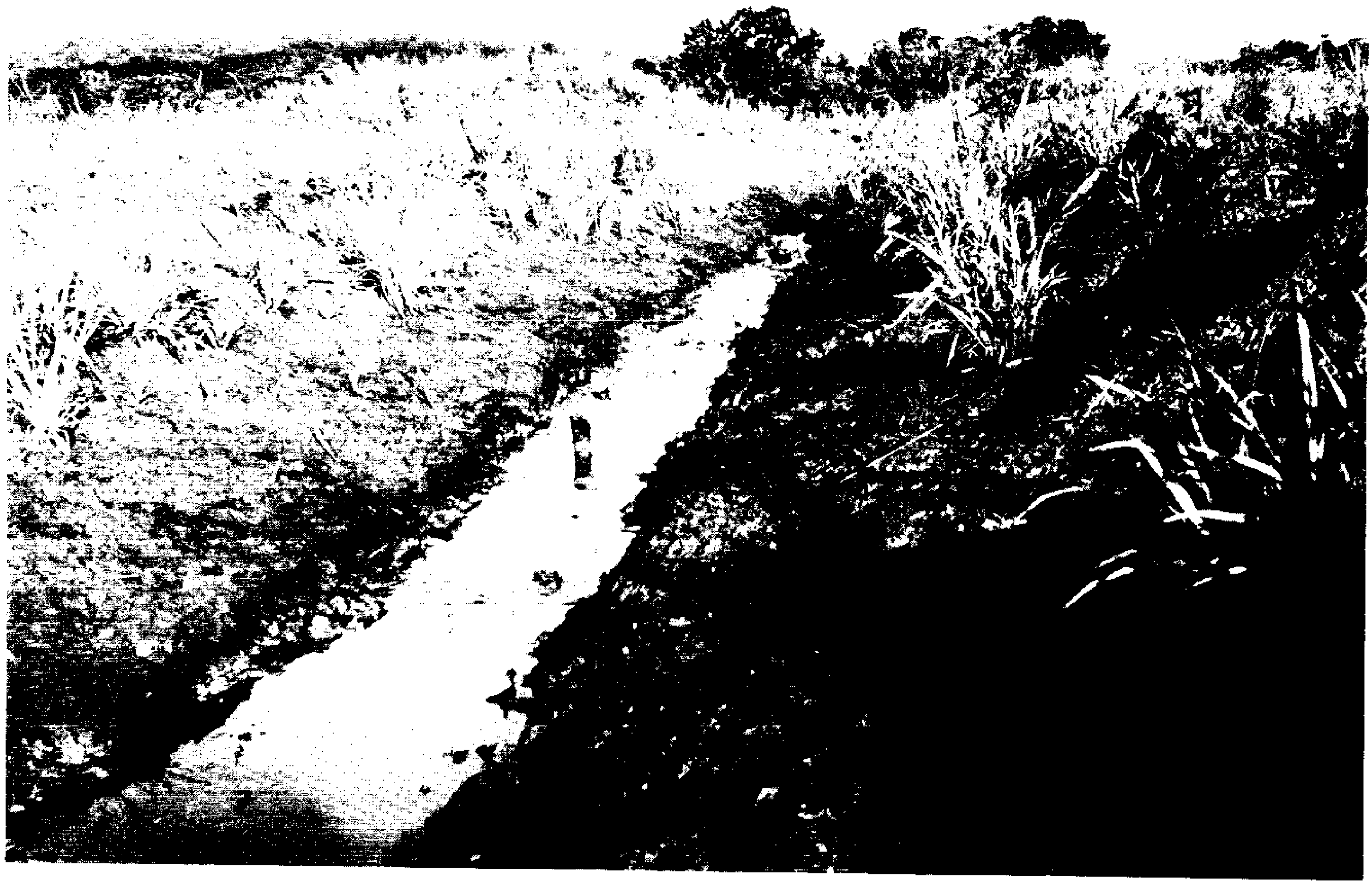
COMPLETED DITCH AFTER BLAST - 10 FT. WIDE BY 5 FT. DEEP,
NOTE EXTREME REGULARITY WITHOUT NEED FOR FURTHER CLEANUP



COMPLETED PROJECT - 21,000 LINEAL FEET OF EXPLOSIVE TRENCHED DRAINAGE
IN THREE WEEKS, VELASQUEZ DISPOSAL AREA



SILTATION AND EROSION EFFECTS ON A TYPICAL DRAINAGE DITCH AFTER EIGHT MONTHS OF RAIN (MARCH - OCTOBER 1978). 16 FT. WIDE X 3 FT. DEEP.



SELF-CLEANING ACTION DUE TO HIGH FLOW RATES HAS
MAINTAINED DITCHES CLEAR OF SEDIMENTATION

HIGH UNDITCHED AREA OF VELASQUEZ DISPOSAL AREA, ONE DAY AFTER HEAVY RAIN, SHOWING STANDING WATER AND WET SURFACE DUE TO LACK OF DRAINAGE.

A HISTORY OF DREDGING AT THE MOUTH OF THE MISSISSIPPI RIVER

by David F. Bastian[1]

ABSTRACT

Almost from the time of its discovery in 1682, the mouth of the Mississippi River has needed dredging. The founding and development of New Orleans was delayed because of natural navigational restrictions. The first dredging was performed in 1729 and was done sporadically thereafter until the 1850's when it became an annual affair until interrupted by the Civil War. Politics, economics and nature all combined to retard the implementation of dredging. Due to a lack of understanding of sediment transport many schemes used to open navigation resulted in failure. The magnitude of the problem promoted many unique types of dredging apparatus.

[1]Engineer, Chief, Chesapeake Bay Model Branch, Chesapeake Bay Model. Stevensville, MD 21666

Following the discovery of the mouth of the Mississippi in 1682, the French had visions of developing the interior. Such efforts were particularly inhibited by limiting depths over the bars guarding the various passes at the mouths of the river.

New Orleans was finally established around 1718 and the need to provide a navigable channel at the mouth of the river became more acute. Harrows were shipped from France the same year but were unloaded and subsequently lost because the engineer who was going to use them died. To aid navigation, a pilot was hired and headquartered at Balize but the need for dredging still remained. Dredging was finally performed in Balize Pass in 1729[1] and was credited with achieving a channel of 17 feet. What type of dredge and how long it was used is not known. Figure 1 shows a type of dredge that the French used at the time and had been suggested for use in Louisiana. It was a dipper dredge using a mill for a winch and a barge to carry off the dredged material.

The entrances (passes) of the river continued to shoal and shift. The shoaling was an annual event generally accompanying the winter floods and January through April were often the worst months. Ship captains continued to complain but no dredging was done. Problems were increased due to the monopolistic pilot system that had developed over the years. These problems continued and increased when Louisiana came under Spanish control in 1767.

The problems of navigation at the mouths of the river became those of the United States when she acquired Louisiana in 1803. Although concerned with the problem there was little if any dredging occurring elsewhere in the United States at the time and certainly none under the federal government.

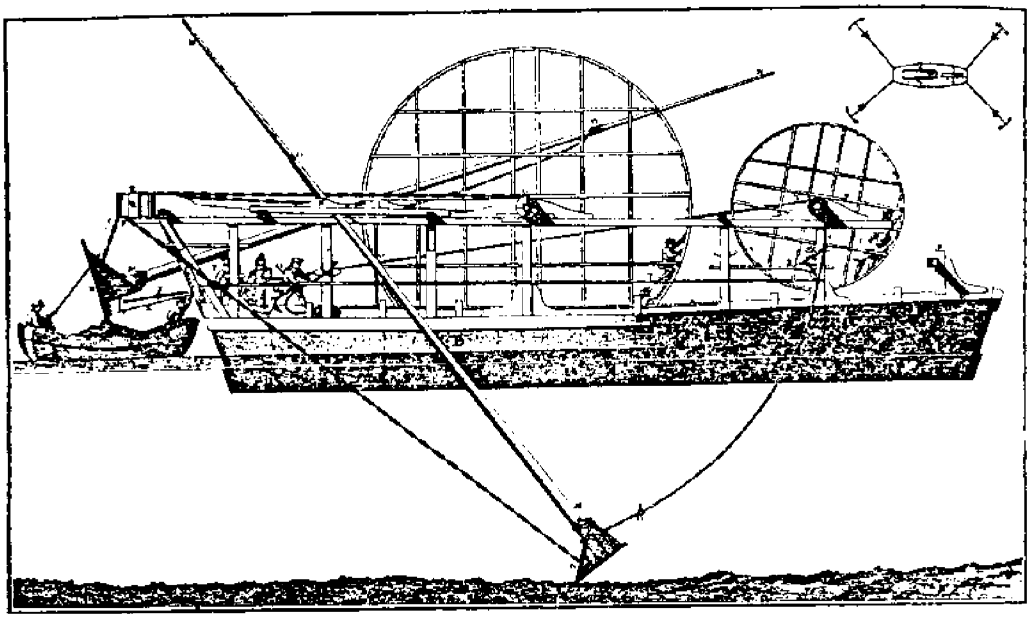


Figure 1. French pontoon-dredger, with power supplied by treadmills, 1745

Lack of action to improve the channel was partially overcome by the design and construction of different hull forms and by usage of towboats to tow ships across the bar.

In 1828, Lieutenant Alexander H. Bowman of the Corps of Engineers, made the first federally sponsored survey of the passes. The intent was to determine the practicability of constructing a breakwater harbor outside the mouth of the river to protect ships until there was sufficient depth to cross the bar. Such a harbor was concluded as unfeasible and a second survey was ordered in 1829. Captain Richard Delafield made this survey but confined it to the Southwest Pass. During his inspection he noted that ships still used warping - a practice introduced by the French over a century earlier.

Delafield's report gave three proposals for improving the channel across the bar at the Southwest Pass which at the time of his survey had a limiting depth of 13 1/2 feet at low water. One proposal was to confine the channel with jetties and let the increased resultant velocities scour the bar. His other two proposals were dredge the bar by plowing or digging and removing the shoal material. The cheapest and easiest would be to plow or scrape the bar into the flanking troughs. None of these ideas were new but more important none of them were acted upon. The desperation of the situation prompted the State Engineer of Louisiana, Benjamin Buison, to propose a canal between the river and the Gulf. The proposed location was a few miles below Fort St. Phillip. His suggestion met the same fate as Delafield's.[2]

It wasn't until 1836 that the Senate finally took up the problem again. The result was another federal survey. This time, 1837, it was directed by Lieutenant Benjamin Poole.

Poole concluded that only two of the passes were potential candidates for improvement. Of the two, he favored the Southwest Pass over the Northeast Pass and believed that by dragging the former for a length of 300 yards he could obtain a navigable depth of 16 feet across the bar.

Captain William H. Chase elaborated upon Poole's report and suggested three options for navigation. One was to close all but one pass. A second was to dredge the bar and confine the channel with jetties. The third, his favorite, was to build a ship canal. He submitted great detail on each. The dredging option included a cost estimate for improving the Southwest Pass whereby five dredges would be used to dredge a channel, 5500 feet long and 300 feet wide, by removing the upper three feet of the bar for $210,000 For another $160,000 and four more dredges, the Northeast Pass could be dredged. Thereafter, the channel could be maintained for $144,000 annually.[3]

George W. Long, the State Engineer, felt that dredging was too short-term but favored it as the easiest and cheapest method provided it was in the form of scraping.

After three surveys and 34 years, the United States authorized dredging in the act of March 3, 1837. Chase was promoted to Major and Chief Engineer of Mississippi Improvement. He placed Captain Andrew Talcott in charge of the dredging.

The Engineer Department had a ladder bucket dredge built in Philadelphia. The dredge was named BELIZE. This self-propelled vessel had a set of buckets on each side. Each 5-foot-wide bucket had a 27-cubic-foot capacity and was spaced eight feet from the adjacent buckets. Due to ice on the Delaware River a trial was delayed until the BELIZE reached the Mississippi in May 1838. The cast-iron chains forming the endless loop connecting the buckets repeatedly broke and interrupted progress. Eventually wrought-iron chains arrived from Philadelphia and dredging resumed in September.[4]

June soundings had revealed that Northeast Pass was the least shoaled so dredging was performed there, and, by January 1839, a channel 900 by 100 by 16 feet had been achieved. Dredging continued intermittently until April when the 1836 appropriation of $75,000 and 1837 appropriation of $210,000 were exhausted. With no more money appropriated the BELIZE was laid up in Mobile and never returned to the Mississippi.[5]

In January 1838, before the arrival of the dredge BELIZE at the passes, Talcott organized two survey parties to conduct a topographical and hydrodynamic survey of the passes. One was headed by William H. Sidell and the other by George G. Meade. Their reports were not submitted until October of that year, or several months following the initiation of dredging. The board of engineers which reviewed Talcott's subsequent report, through a process of elimination, promoted dredging as the best method for deepening the entrance of the Mississippi River. Chief Topographical Engineer, Colonel John J. Abert, reviewed the board's findings from which he made his comments in the annual report to the Secretary of War for 1839. Abert supported dredging but no new appropriations were made.[6]

The cotton trade continued to grow. Strangely enough, the shipping or trading season corresponded with the shoaling season. Each year as the bars would choke the passes, vessels would be delayed, sometimes for weeks. New Orleans was anxious to maintain its position as a leading port and looked to dredging as the one feasible way to provide help.

With no new federal action in 1841 the Louisiana Legislature asked Congress to examine the potential use of a submarine plough invented by

J. R. Putnam[7] to scrape the bars. William B. Davis contacted the Engineer Department directly lobbying for his own contrivance, a submarine wheel. Davis, aware of the stratified flow over bars, proposed that his wheel would raise the bar material above the saltwater wedge and set it in suspension to be removed by the ebbing fresh water.[8]

Accidents, groundings and detentions at the passes continued. In one reported case, the bar shoaled to a limiting depth of 11 feet while as many as seven ships at one time were left waiting inside the bar to cross. The severity was such that in 1844 the State Legislature of Louisiana again solicited Congress for help. Still nothing was done.

In 1850, 2,720 ships took pilots. In the same year Congress acted by appropriating $50,000 to conduct a topographical and hydrodynamic survey of the lower Mississippi for the purposes of planning future flood control and navigational legislation. This study, undertaken by Captain Andrew A. Humphreys and Lieutenant Henry L. Abbott, was not completed until 1861.[9] Louisiana initiated their own survey and hired Professor Caleb Forshey of New Orleans to investigate the passes. Forshey's report was completed in 1852 in which he proposed that the federal government close all but the one pass and confine it with jetties. But, he warned, this would ultimately form a new delta with the same old problem. For immediate consideration he proposed raking the bars. His idea was to employ a steamboat to drag a crescent-shaped harrow 30 feet wide with twelve 18-inch-long teeth. He claimed that this method would provide a navigable depth of 17 feet at the Southwest Pass. In a third study, the Bureau of Topographical Engineers contracted Charles Ellet, Jr. to determine the proper mode of deepening the channels across the bars. His report, entitled The Mississippi and Ohio Rivers, was published in 1853. Ellet conjured up a novel dredge to clear the bars. Basically, he designed an agitation dredge. He proposed mounting six to eight 20-foot-long 18-to 20-inch diameter tubes onto a steamboat. These tubes would be angled into the water and submerged eight to ten inches into the bed. He reasoned that the 6 to 7 mph speed of the vessel would cause the slurry to rise up the tubes and spill out in the fresh water. His scheme was impractical and was roundly attacked in 1852 by Forshey and Albert Stein, a civil engineer from Mobile.[10]

Two major occurrences in 1852 resulted in Congressional action. First, the worst blockade in history occurred at the passes. One ship was stuck 83 days at the Southwest Pass. The second was the news that the Navy turned down a bid by New Orleans for a naval yard because of insufficient depth at the passes. The Chamber of Commerce reacted and solicited the aid of other states in the Mississippi valley. Congress responded to their pleas on August 30, 1852, by authorizing $175,000 for the clearing of the passes.

Of course, no action could be taken without another survey. Major Chase was chosen to head a board to study the appropriate methods of clearing the passes. They reported on the usual and previously proposed methods but unanimously recommended stirring as the easiest and cheapest. G. T. Beauregard, a member of the board, lobbied for his own invention which he called a self-acting bar excavator which was supposed to be able to direct the surface current down onto the bar and scour it. The invention consisted of box or rectangular solid open at both ends and at the bottom. The top sloped downward in the direction of flow reducing the cross-sectional area of flow, thus increasing the velocity and sediment transport capacity.[11]

The board took a conservative approach and signed a contract with the Ocean Tow Boat Company and the Star Tow Boat Company on November 18, 1852, to create a channel across the Southwest Pass 18 feet deep and 300 feet wide. The contract was for $75,000 payable upon completion. The companies supplied two tow boats. The dredging apparatus for each consisted of large triangular harrows, armed with iron teeth or coulters, protruding 15 to 18 inches below the timbers of the harrow. By dragging this along the bottom of the channel, the current was assumed to sweep the loosened material into deep water outside of the bar. The harrows were pulled across the bar except when ship traffic was great during which the dredges towed ships across the bar. Interestingly, Beauregard supervised the contract. The contract was complete in November 1853 but no money was appropriated for maintenance dredging.[12]

In 1856, $330,000 was appropriated by Congress for the opening and maintenance of both Southwest Pass and Pass a l'Outre provided that the work must be done under contract. Advertisements requesting bids were issued. Two of the responsive bidders proposed dredging while a third, Craig and Rightor, of Newport, Kentucky, proposed using wing dams somewhat like what had been used on the Ohio River since 1825. With the wing dams or jetties they claimed that they would provide a channel 20 feet deep and maintain the channels for another five years for 36,000 per year.[13]

A contract was signed with Craig and Rightor November 14, 1856, to provide channels across the bar 18 feet deep and 300 feet wide. The Southwest Pass was to be completed by September 13, 1857, and Pass a l'Outre by February 13, 1858. Lieutenant Colonel Stephen H. Long was in charge of the project.

Craig and Rightor using the patented Meig pile dams began installation of the jetties with a pile driver. Progress was slow and uncertain. Despite no apparent increased depths resulting from the jetties, complaints surfaced from shippers that the currents had increased. Time extentions were requested and granted. With continued questionable results, the contractors began experimenting with dredging. On March 22, 1858, the DOWNS was used as a scraper dredge. When the scraper was lost it was replaced with two small semi-circular buckets. Dipper dredging was very inefficient because of the difficulty in discharging the dredged material from the buckets. Next they resorted to blasting. The blasting was done with powder-filled canisters submerged 7 to 20 feet below the water surface. The canisters were set off electrically rather than by hammer which made the operation more efficient. This work was directed by George L. Baker who had previously been employed by the government in blasting out the rocks in the Hell Gate channel north of Manhattan Island. On May 28, 1858, the contract depth had been achieved at the Southwest Pass and similarly the contract depth was achieved at Pass a l'Outre on September 10, 1858.

No maintenance had been performed in the Southwest Pass since May. By December, surveys showed limiting depths of 14 feet. Craig and Rightor had Baker back blasting in the Southwest Pass in December and January but the new Government supervisor, Charles A. Fuller, reported the effects of the blasting to be unsatisfactory. He claimed that they only produced holes 3 to 4 feet deep and about 10 feet in diameter and these filled within one to two days. Craig and Rightor abandoned the project in January, 1859.

With the maintenance effort a failure, the passes shoaled so badly as to produce the worst blockade in history. By February 1, there were a reported 49 vessels detained at the mouths of the river waiting for sufficient water. Some of these vessels had left New Orleans as early as December 9. Fifty more vessels were loaded and ready to leave New Orleans.

Such an uproar was raised that delegation after delegation made investigations of the passes to determine the validity or extent of the incredible stories of delays and difficulties associated with navigation at the passes. By late February the limiting depths were 12 1/2 feet at Southwest Pass and 11 1/2 feet at Pass a l'Outre. By early March, 55 vessels were detained at the passes where the limiting depth had increased to 15 feet but the channel was too crooked for ships to follow across the bar even with the aid of a towboat.

The Corps of Engineers and the use of jetties came under fire. Numerous editorials blamed both for the difficulties. However, Craig and Rightor blamed the towboat association; the group that had successfully cleared the passes in 1853. They were accused of such action as purposely towing ships with 19-foot drafts across the bar until these ships grounded and then abandoning them causing the shoal to allegedly worsen.

While accusations were being made the shipping rates and cost of trading at the port of New Orleans skyrocketed. The Chamber of Commerce and the Board of Underwriters of New Orleans suffered the most.

Again a series of ideas, both old and new, were advocated for the clearing of the passes. Two of the more novel were E. B. Bishop's patented screw dredge[14] and the using of the drydocks at Algiers as "camels"[15] to ferry the ships across the bars. The State of Mississippi wanted to eliminate the problem altogether and proposed a railroad from the interior to a Gulf Coast port.

Long officially terminated Craig and Rightor, and on March 24, 1859, signed a contract with Charles S. Hyde to dredge for one year for $67,500. By this time a rise in the river affording 17 feet of water at the Southwest Pass had freed most of the ships.

Hyde commenced dredging May 12, 1859, using a side-wheel steamer, P. F. KIMBALL, to drag the bar. The scraping apparatus was attached just in front of the bow and the P. F. KIMBALL would back out with the surface ebb current dragging the bar, raise the scraper and return above the bar to repeat this action. Scrapers were constructed of boiler iron and mounted on the under side of a 24-foot-long horizontal oaken beam. Each scraper was about 4 feet wide and protruded 18 inches below the beam to which it was mounted. The frame supporting the scraper was connected to or pivoted from the sides of the steamer by long arms. The beam to which the scrapers were attached was raised and lowered by chains by means of a small engine and capstan.[16] Hyde then bought the ENOCH TRAIN in Boston which used propellers at the stern to cut and stir up the bottom sediment. Water tanks enabled the captain of the dredge to obtain the desired draft. The problem with the ENOCH TRAIN was that the propellers broke or were stopped by the mud.[17] The two dredges produced little good and the contract was cancelled in October.

A new contract was let on November 19, 1859, with Thomas McLellan, manager of the Crescent Towboat Company, for $4,440 per month. Dredging began January 2, 1860, using a scraper designed by Long, consisting of an oak beam 12 by 24 inches, 18 feet long, onto which were attached five semi-cylindrical scrapers made of boiler iron. The scrapers spanned 15 feet and were positioned a few feet in front of the bow. Each drag was hoped to scrape a depth of 18 inches. First the MOBILE and then the PANTHER served as the dredge boat in a fashion similar to the P. F. KIMBALL.[18]

Like previous dredging efforts there were continued mechanical breakdowns with the dredging apparatus. Thinking something was lodged in the bar, Fuller removed the scrapers and raked the bar for twelve days finding nothing harder than stiff blue clay. He replaced the harrows with a new scraper designed by Long. During the same month a ship drawing 20 feet of water crossed the bar with the aid of one towboat.

Dredging continued at Southwest Pass with few interruptions until February, 1861, when slowness of payment by the Government resulted in the balking of the contractor. Considerable shoaling during the following two weeks prompted action and dredging resumed. Disloyal movements in Louisiana caused the War Department to suspend dredging on March 7.

During the contracted maintenance of Southwest Pass in 1860, Pass a l'Outre was the most heavily trafficked. It is suggested that the scraping by the ships' bottoms and action of the propellers helped keep the channel open with dredging.

After the Civil War, New Orleans was anxious to recapture her old trade; but to become a competitive port, a navigable channel would have to be provided. By January 10, 1866, the passes shoaled to 14 feet which resulted in the worst impediment to traffic since 1859. Not waiting for the Federal Government, two private attempts were initiated to clear the passes The local agent for the Star Line, mail steamers operating between New York and New Orleans, proposed stationing a tow boat at the bar to tow a "heavy revolving wheel 28 or 30 feet in width, armed at all sides with cutters so arranged as to raise the sand and break the clay." General Beauregard had a bill introduced in the Louisiana Legislature which would allow him to form a company to maintain navigable channels by harrowing the bars the money for which would come from tolls. The New Orleans press strongly denounced this idea and the bill died in committee.[19]

By the end of spring and trading season, the bars scoured and local concern subsided. However, on June 23, 1866, $75,000 was appropriated by Congress for the improvement of the mouth of the Mississippi. Brevet Lieutenant Colonel Miles D. McAlester, Captain of Engineers, was ordered to take charge of the project. After inspecting the passes, McAlester reviewed previous dredging attempts at the passes. Of the four types of channel improvements: dipper or bucket dredging, scraping, raking and use of jetties, he felt that scraping was the best. He proposed a new device looking like a medieval mace which would be attached by chains and rope from a steamer. He also wanted to incorporate a double-ended propeller and variable draft like the ENOCH TRAIN to help the stirring process.

The Corps of Engineers advertised for bids to open a channel 18 feet deep and 200 feet wide. To be responsive the bidders had to bid on dredging by scraping or harrowing. Two bids were received. The one from McClintock and Scott was for jet dredging where they would build a dredge with a series of 1-foot diameter hoses to scour the bed material. They added to their design heavy revolving harrows to be drawn over the bar just in front of the jets.[20] Their contrivance called for an adjustable frame so they could control the distance from the center of the revolving stirrer and the jet or pipe openings to the bed. The details of this feature were left to the builder as was the manner in which this apparatus was to be moved. McClintock and Scott claimed to have built a prototype and in a trial at a 50-foot depth excavated a 25-foot hole. The winning bid was to a company headed by Horace Tyler that proposed using Bishop's patent. The contract was signed November 5, 1866, and the contract was to be completed by January 23, 1867.

Construction of the dredge didn't get started until January and already the bars had shoaled enough to cause another blockade. By January 19, 21 ships with drafts of 12 feet and greater were detained at the passes. The passes were now worse because of abandoned wrecks.

While Tyler's dredge was under construction, the New Orleans Lightering and Wrecking Company tried to get the Louisiana Legislature to give them an exclusive charter to lift ships over the bars using the camel or drydock method. Naturally, there was to be a charge for such service. Charles F. Fisher of New Orleans thought he had the solution to the problem with his newly invented bucket dredge which he claimed would dredge 7,000 cubic feet per hour. McAlester, too, had new ideas. He decided that the Government should have its own dredge and proposed a dredge having a screw propeller at each end and a variable draft of 16 to 24 feet. Both propellers would power the vessel across the bar and agitate the sediments at the same time.[21]

Tyler's dredge WIGGINS was completed March 19, 1867. Her dredging machinery consisted of two 20-foot-long conical screws each with a 5-foot diameter base and a helical flange, 12 feet wide at the base tapering to 6 inches at the points. The two screws were 20 feet apart at the base and angled toward each other. The machinery worked well but under-powered engines and too light a vessel made the dredge unfit for the job. The contract was annulled in May.

In March several significant events occurred. First, the Coast Survey started an examination of the passes to be used for future dredging plans. The second was a bill appropriating $200,000 more to the maintenance of a navigable channel. The third was a bill allowing the Corps of

Engineers to build and operate two dredges at the mouth of the river and to apply previously appropriated money to the construction and operation of such dredges.

With approval for one dredge McAlester modified his design slightly to decrease the screw diameter to 14 feet and the maximum variable depth to 22 feet. He also added a Long-type scraper in front of each screw. On October 15, 1867, a contract for the construction of such a dredge was signed with the Atlantic Works of Boston for $233,000. The contracted delivery date was April 10, 1868.[22]

As with the WIGGINS, there were delays with the completion of the ESSAYONS. Finally complete, the ESSAYONS sailed for New Orleans in June and on arrival required repair and was not ready for dredging until September.

The ESSAYONS got off to a dismal start. She broke down during her September 19 trial and returned in October for a second trial. This lasted nine days during which all four blades of the forward screw were lost. After further repairs she returned on November 19 for two more days before breaking down. This continued so that during the first ten months the ESSAYONS only worked 68 days.[23]

In early 1869 another blockade occurred. The ESSAYONS gave no relief. The New Orleans Chamber of Commerce organized a "Committee on Obstructions to Commerce" and tried to get from the Federal Government both the ESSAYONS and the remaining or unexpended funds. Their plan was to modify the ESSAYONS by replacing its dredging apparatus with Bishop's screws.

In April 1869 Brevet Major Charles W. Howell, Captain, Corps of Engineers, took charge of the project, and in May the ESSAYONS had secured a channel 17 feet deep in Pass a l'Outre while there was only 15 feet at the Southwest Pass. Nevertheless, traffic continued to use the latter. Howell politicked with ship owners to get them to use the pass and finally a ship drawing 17 feet successfully passed through Pass a l'Outre. Shortly after this success a ship grounded in the pass while the ESSAYONS was in bad repair and the channel suffered. Howell blamed the pilot for purposely grounding the vessel.

By June, Howell had again obtained 17 feet in Pass a l'Outre. To guard against future sabotage by the pilots, he pressed for the establishment of a bar master to regulate use of the channels, who would also be the engineer in charge of dredging. In the meantime, the ESSAYONS was sent to New Orleans for modification and to be prepared for winter shoaling.

Shoaling problems began earlier than usual with serious problems of groundings and delays in October. The new blades and scrapers arrived in December so after a 6-month delay the ESSAYONS was dredging again only to demonstrate that the improvements broke as easily as their predecessors. The shoals of Southwest Pass cleared in mid-February 1870, while continued dredging at Pass a l'Outre gave only 14 feet of water by June as the Southwest Pass had daily traffic of ships drawing up to 19 feet.

With Southwest Pass still the most popular, Congressional pressure was applied to get dredging switched to Southwest Pass. General Humphreys countered, offering to do so if Congress would appropriate $745,000 to build two more dredges allowing one for each pass and the third to act as a replacement. In July 1870, Congress appropriated $300,000 for the repair of the ESSAYONS and for the construction of a second dredge to be used at the mouth of the river. Repairs to the ESSAYONS were finished in October and Howell, yielding to pressure, put her to work at Southwest Pass. In four days the limiting depth was increased from 16 to 19 feet. An out-break of yellow fever temporarily halted operations as did minor breakdowns in November and December. The ESSAYONS performed well in January and until February 4 when the plow and propeller shaft were badly bent. She was repaired and back at Southwest Pass on March 8. A confrontation with the towboats resulted in an accident and damages to the ESSAYONS. During the spring, the dredge would often achieve a channel 18 feet deep only to find it had shoaled overnight.

The building of the second Government dredge was delayed until Howell was satisfied with the improvements of the ESSAYONS which were to be incorporated in the new dredge. Howell was convinced that the improvements were good by January 1871 and a contract was signed with John Roach and Son of New York to build a dredge for $218,300 and deliver it by January 1, 1872. The McALESTER did not arrive in New Orleans until July, 1872.

The McALESTER incorporated further refinements over the ESSAYONS. The forward screw was for dredging only, was brass with 6 blades, 12 feet in diameter and weighed 23,000 pounds. The aft propelling screw had only three blades.[24]

During the winter of 1873, the passes started deteriorating again. In March, a 19-foot draft ship grounded at the bar in Southwest Pass and the channel quickly shoaled to 13 feet. Howell could get no cooperation from the Towboat Association so he moved the dredges to Pass a l'Outre.

The problem at the passes had really become critical. The number of sailing and steam ships entering and leaving the river had decreased from 3,635 ships in 1869 to 2,478 ships in 1873. More reflective of the times was the fact that the number of more modern and deeper draft steam ships entering and leaving had decreased from 2,117 in 1870 to 829 in 1873.[25] Dredging operations had been a failure. The old idea of building a canal connecting the river to the gulf had gathered tremendous momentum. Even Howell publicly supported it. The St. Louis Merchants' Exchange hosted a convention in May 1873 inviting all Congressmen for the purposes of promoting the canal.

Captain James B. Eads presented a resolution at the convention calling for deepening of the Southwest Pass by building jetties. He rallied sufficient support. A tremendous controversy arose over navigation by canal or improvement through jetties. While the controversy raged, dredging continued.[26]

Eads was able to sell the Government on his scheme to create a navigable channel by use of jetties. Eads was given South Pass and had freedom of design as long as the jetties were no closer than 700 feet apart. Eads was to receive $4,250,000 when a channel 30 feet deep by 350-feet wide was attained. Meanwhile, Howell was still in charge of dredging at the Southwest Pass and the canal idea was tabled.

To expedite progress, Eads contracted for three dipper dredges to remove the shoal at the head of pass to allow more discharge through the South Pass. These worked from November 1876 through February 1877.

Eads decided that a dredge was necessary for maintenance at the end of the jetties. His understanding of tidal hydraulics was such that he could see no way for agitation dredging performed by the ESSAYONS and McALESTER to be successful. He believed that the current already carried a full capacity of suspended load and agitation would only result in shoaling. He therefore contracted for the construction of an hydraulic hopper dredge. The G. W. R. BAYLEY was built by D. W. C. Carroll and Company of Pittsburgh, Pennsylvania, and arrived at Port Eads on November 15, 1877. The hydraulic hopper dredge had a cutter suction head and could dispose of the dredged material both by typical hopper fashion or by pipeline. The BAYLEY could dredge between 1000-1500 cubic yards of material an hour.[27]

In the course of 150 years, the mouths of the Mississippi had experienced a number of different types of dredging from the earliest raking by the French through the ladder bucket, raking and scraping and finally to hydraulic dredging in 1877. The problem of navigation had fostered many schemes such as Beauregard's self-acting excavator, Ellet's tubes and Bishop's screw which are silly in retrospect. The importance of keeping the mouth of the Mississippi open to shipping was great enough to encourage almost any scheme.

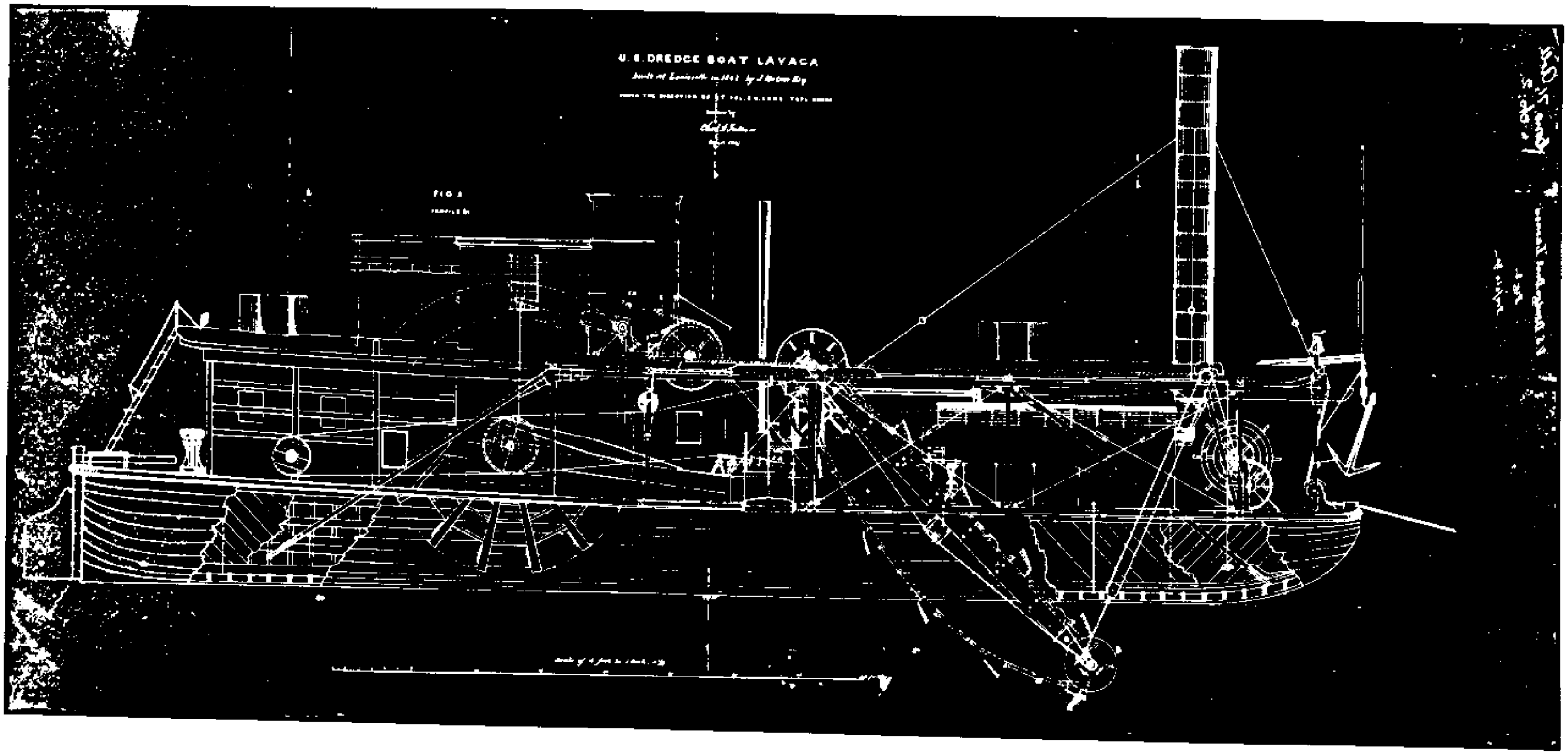


Figure 2. Government dredge similar to BELIZE

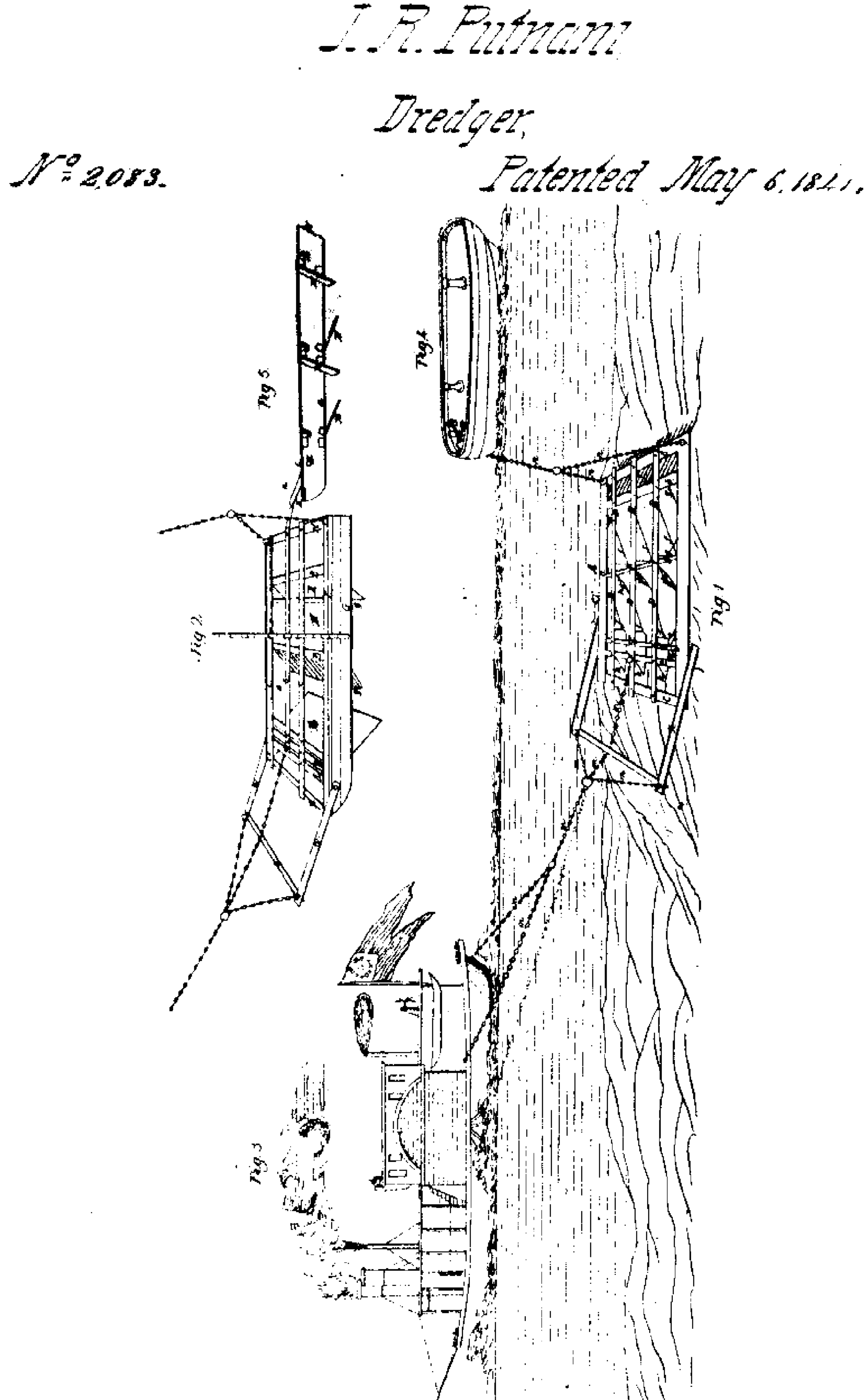


Figure 3. Putnam's scraper dredge, patented 1841

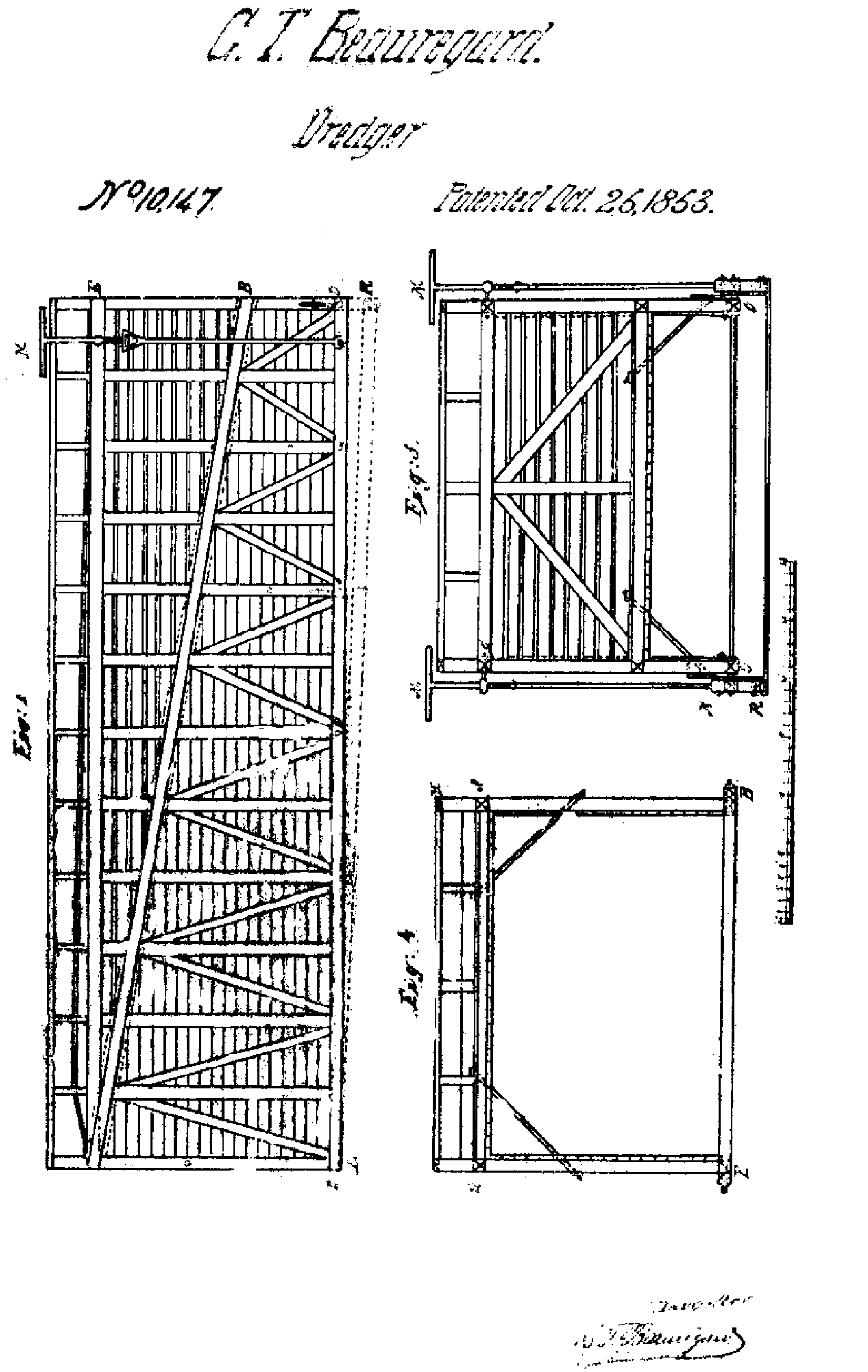


Figure 4. Beauregard's self-acting bar excavator, patented 1853

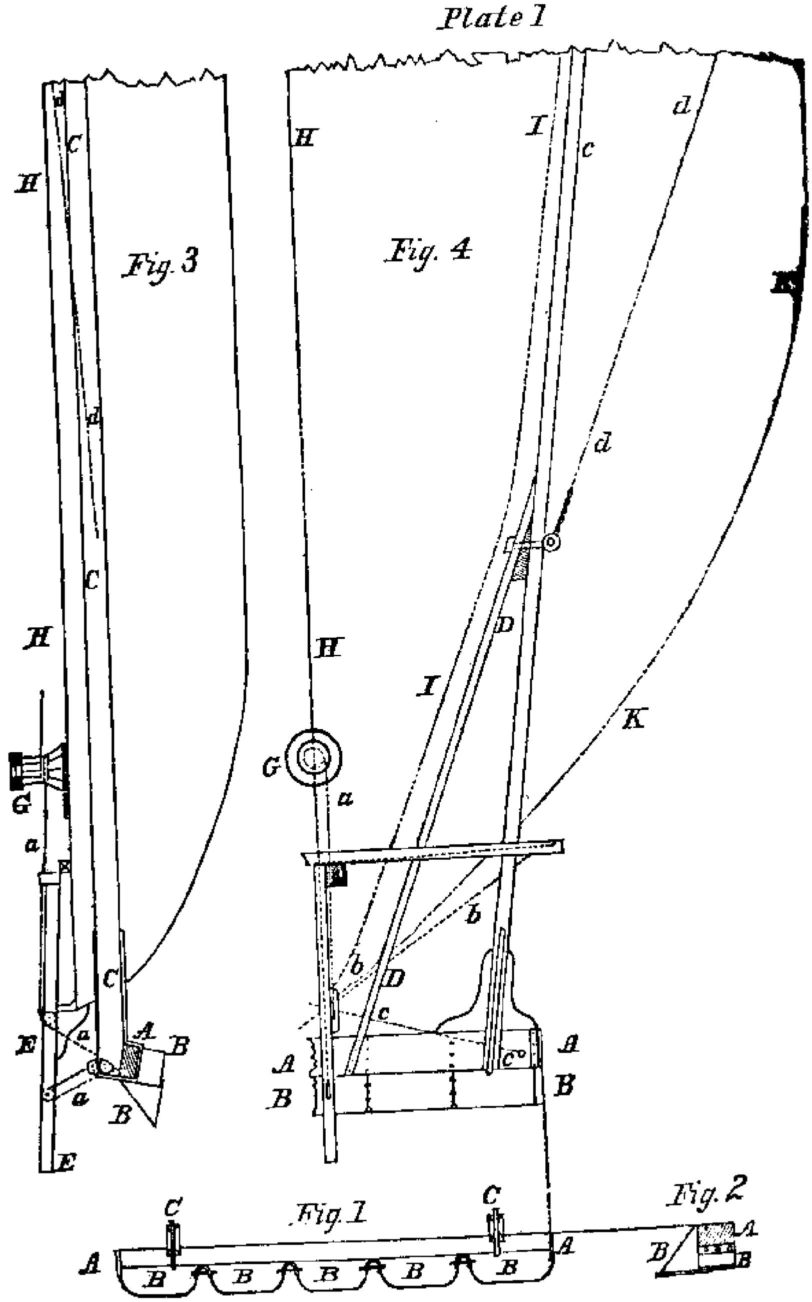


Figure 5. Scraper dredge P. F. KIMBALL, 1859

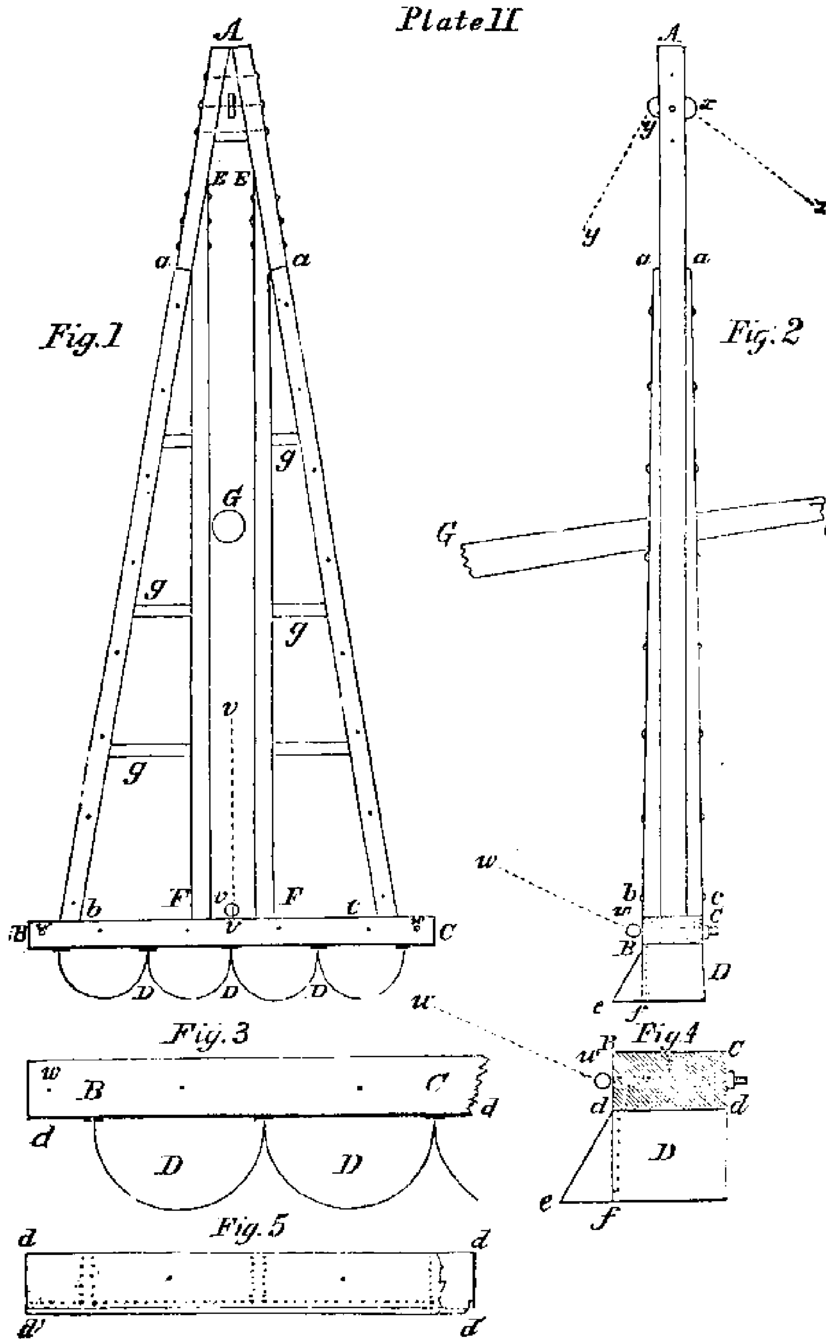


Figure 6. Scraper used on dredges PANTHER and MOBILE, 1860

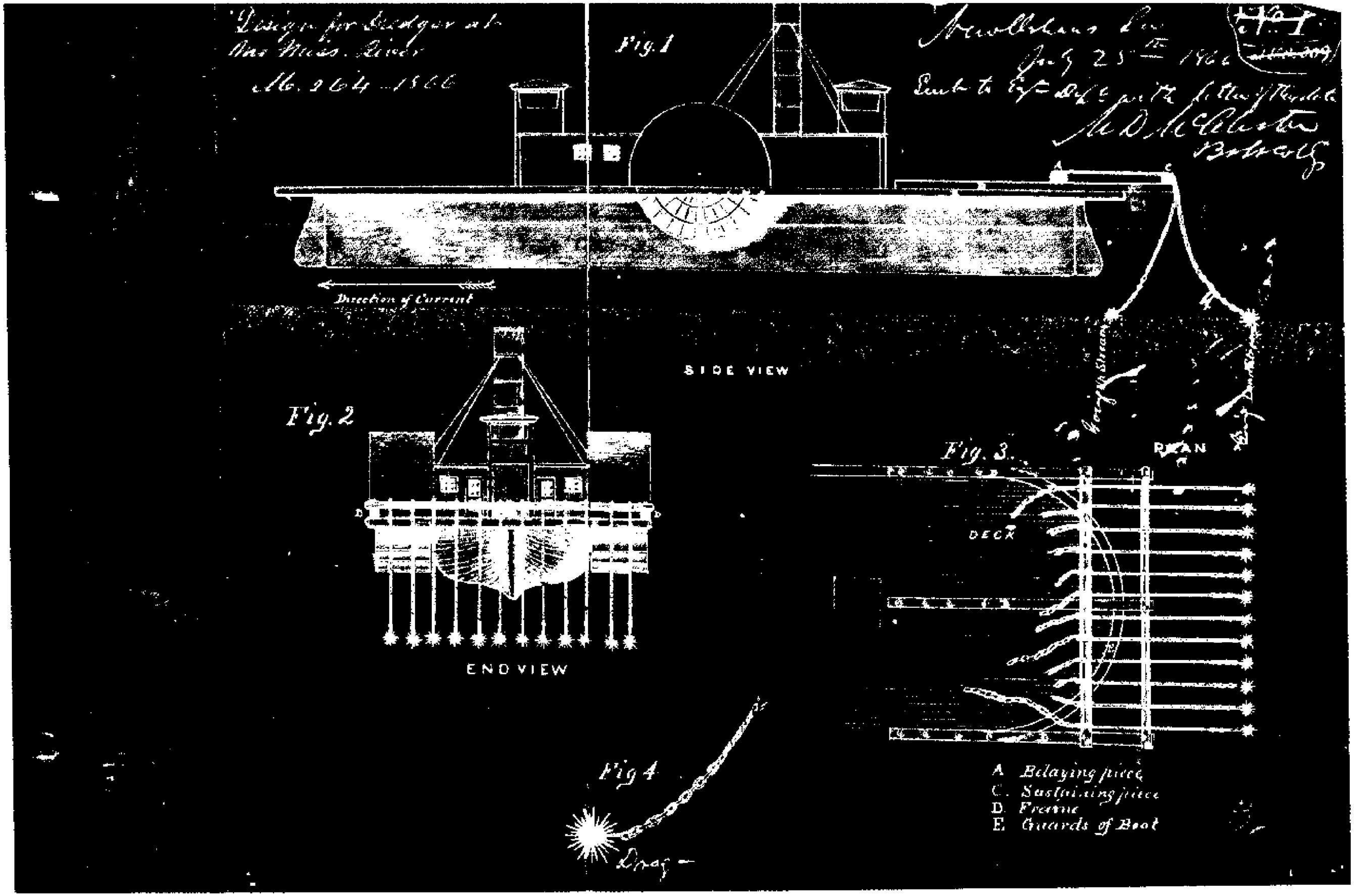


Figure 7. Dredge design of Capt. McAlester, 1866

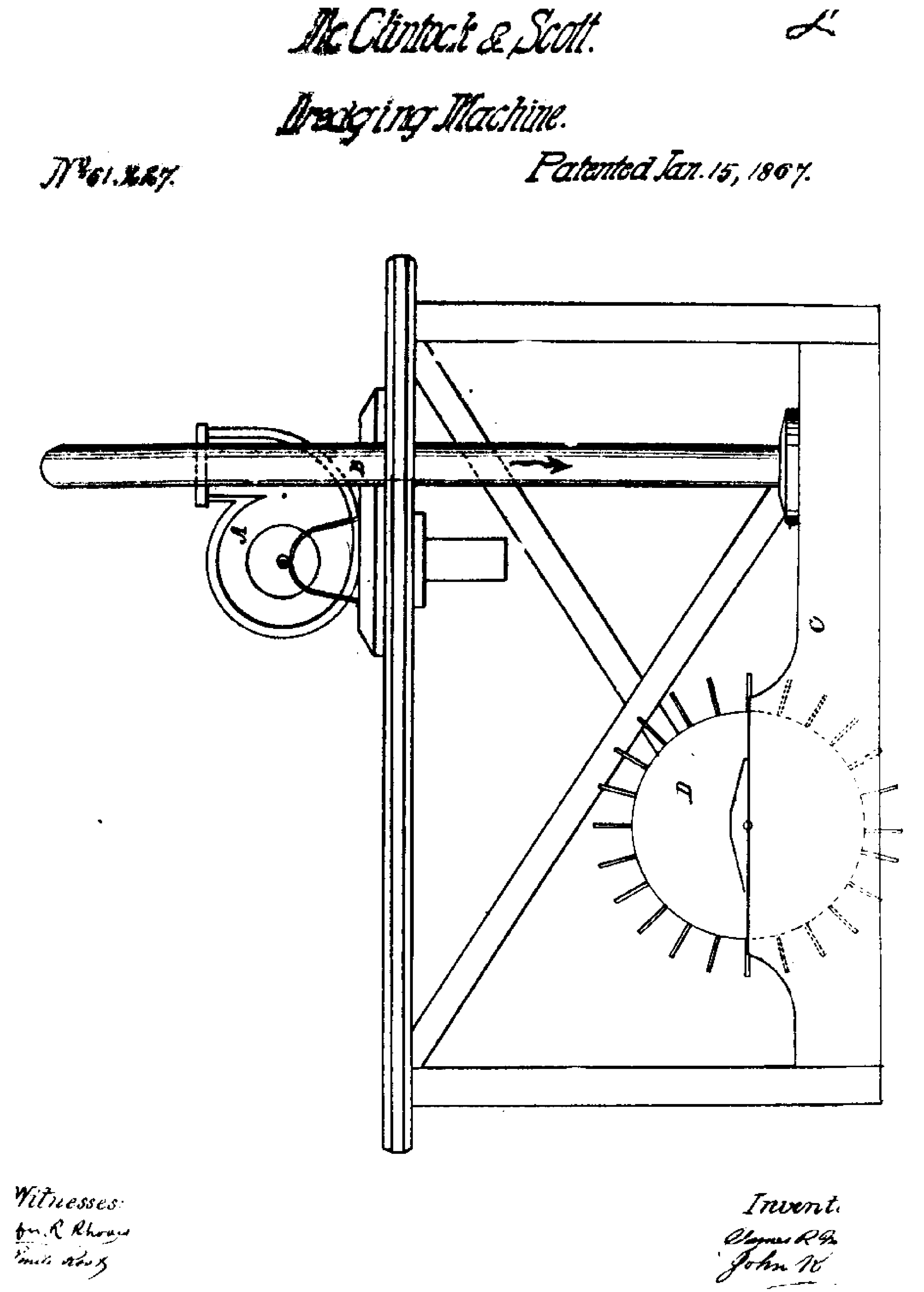


Figure 8. McClintock and Scott jet-agitation dredge, patented 1867

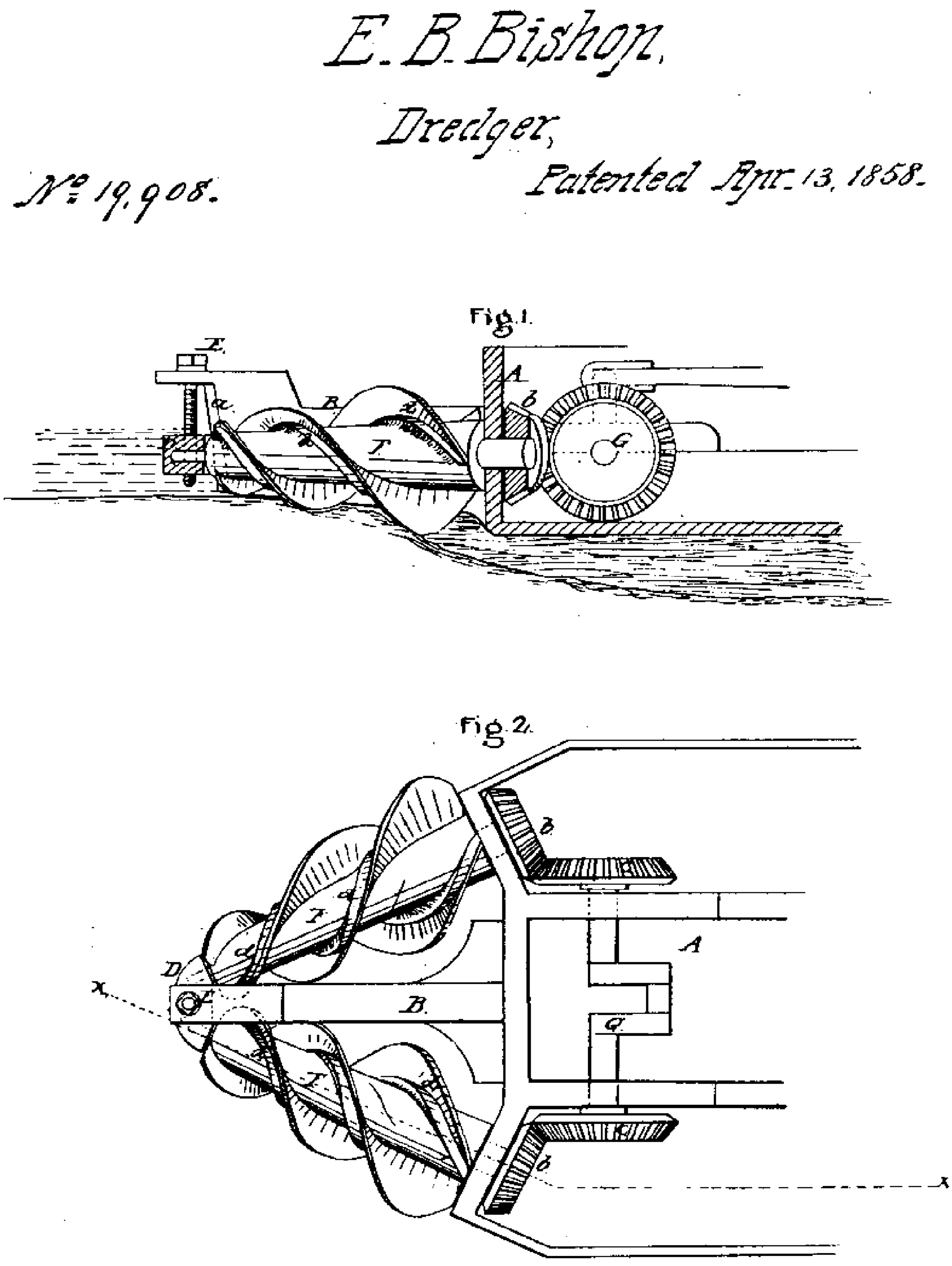


Figure 9. Dredging apparatus for WIGGINS, 1867

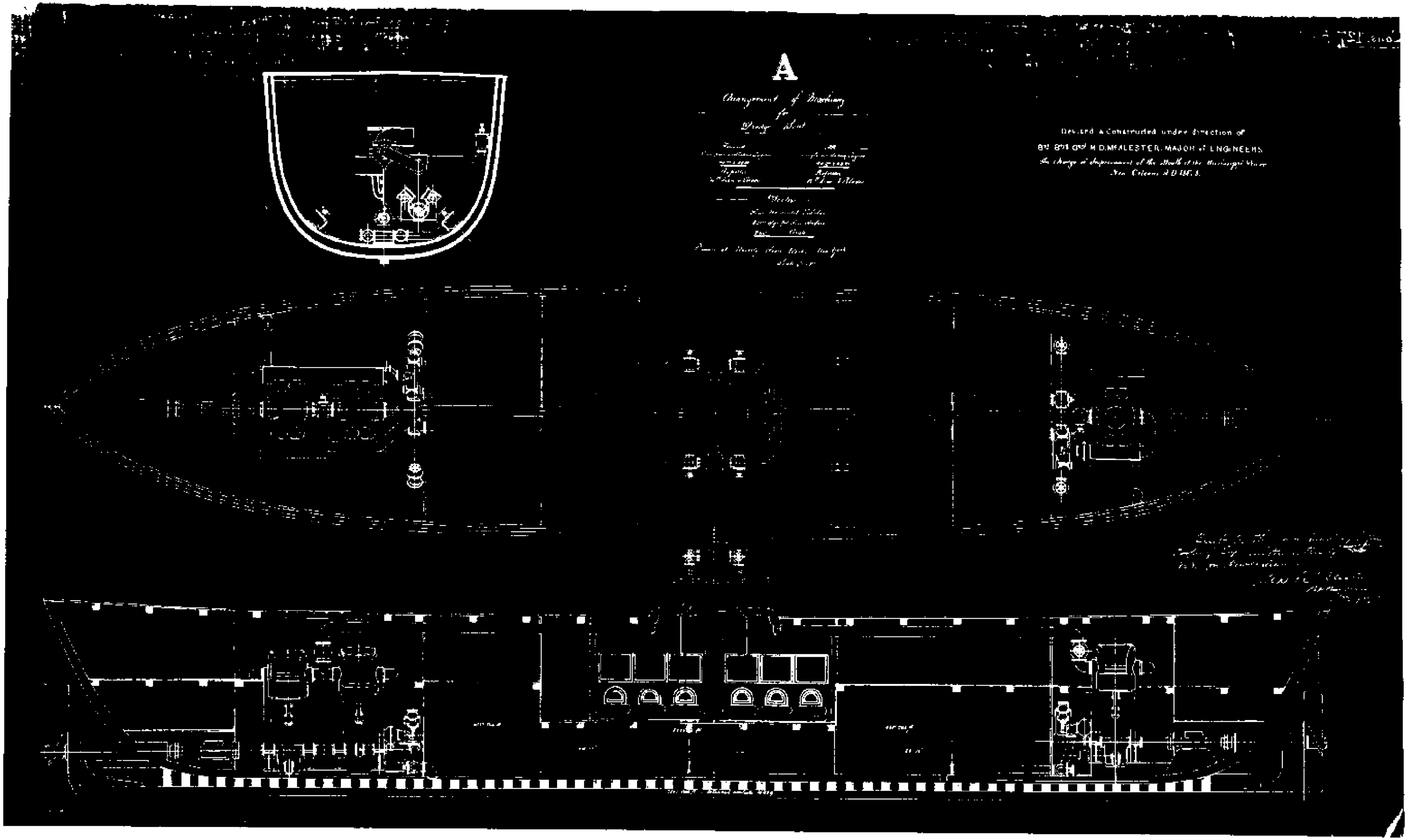


Figure 10. Dredge ESSAYONS, 1868

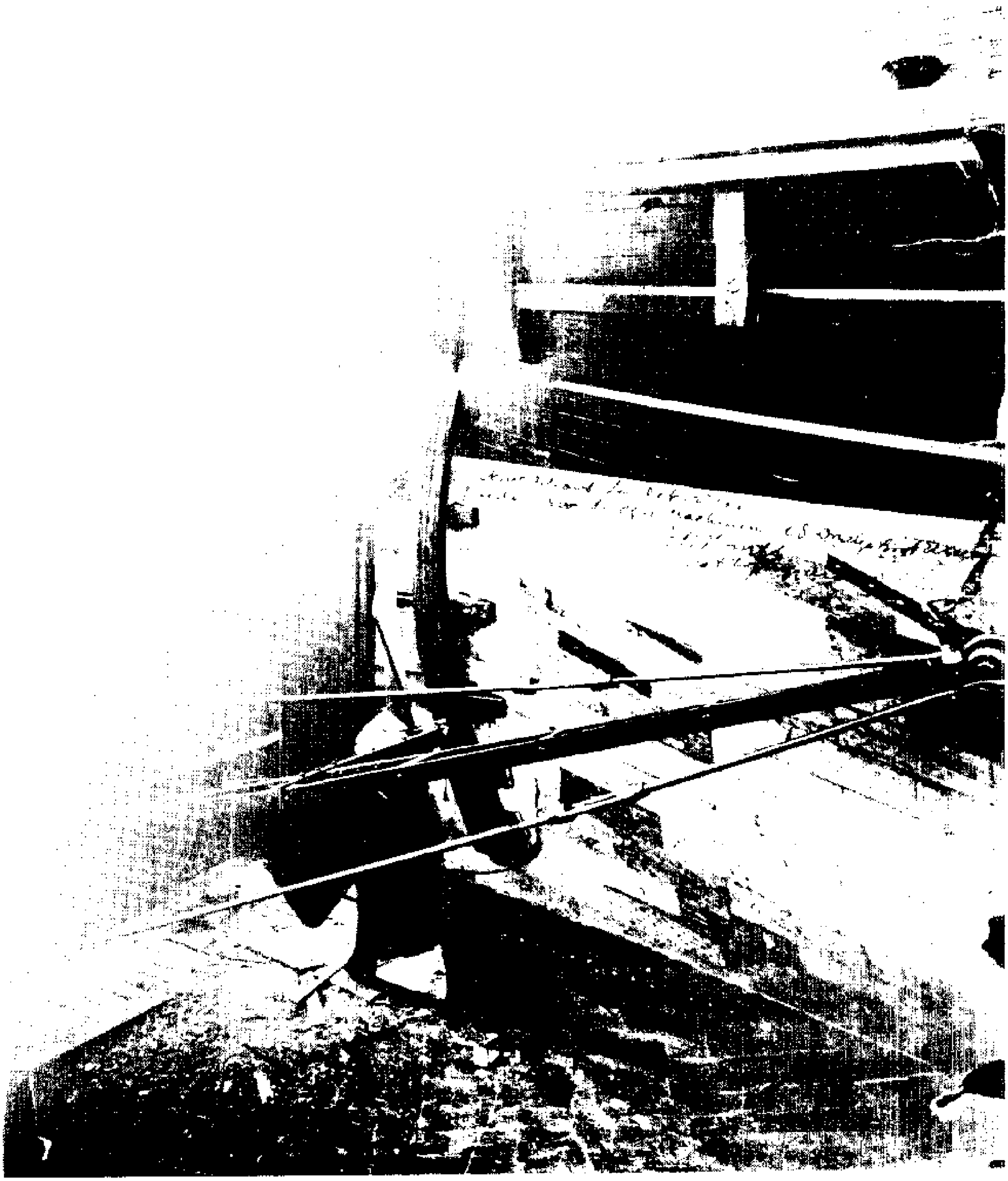


Figure 11. Scraper on ESSAYONS, 1870

DREDGING TECHNOLOGY FOR PCB REMOVAL IN THE HUDSON RIVER

by R. F. Thomas,[1] F. Bryant,[2] T. J. Tofflemire and I. Carcich[3]

ABSTRACT

The New York State Department of Environmental Conservation proposes to remove PCB-contaminated riverbed material from a forty-mile reach of the Hudson River above Albany, New York. The dredging program involves 1,500,000 cubic yards from forty "hot spot" areas (greater than 50 micrograms per gram PCB). A feasibility report on the proposal was completed in January 1978 and a preliminary design report is now being prepared. Dredging systems which have been evaluated are clam shell with mechanical and hydraulic unloading of scowls and hydraulic dredging. A single disposal site of approximately 150 to 200 acres is proposed. The disposal site will meet state and federal requirements for chemical waste landfill. Return dredge flows will receive a high level of treatment through primary settling, coagulation, flocculation and final settling.

The dredging program and related remedial actions will take two to three dredging seasons and have an estimated cost of $250,000,000. Implementation is contingent upon federal funding. A decision is expected in early 1979.

The proposed program may represent a solution for similarly contaminated rivers. The dredging process, however, will

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require careful control and management to minimize loss of hazardous material. Dredging contractors will be encouraged to develop innovative techniques and equipment which will optimize dredging production with minimum loss of contaminant.

Introduction

The purpose of this paper is to describe a program for dredging PCB-contaminated bed materials from portions of the upper Hudson River as proposed by the State of New York (see Figure 1). The program described is based upon a feasibility report published in January 1978 and a preliminary engineering design report now being completed. The purpose of the dredging is to remove PCB-contaminated bed material with a minimal loss of the contaminant and dispose of the dredged material in a hazardous material containment area. The proposed work area involves a 40-mile reach of the Hudson River between Albany and Fort Edward, New York.

The Hudson River is an estuary in the 150-mile reach between New York City and Albany. Above Albany the river has a series of 8 locks and dams that provide navigation for the New York State barge canal system and generation of hydro power. These pools are a significant factor in the characteristics of the upper Hudson River and act as sedimentation tanks which have apparently trapped a significant portion of the PCB discharged to the river. The river in the proposed work area is typically 600 to 1,000 ft. in width with maximum depths on the order of 20 ft. There are extensive shallow areas with depths of several feet. A minimum depth of 12 ft. is maintained in the barge canal. Locks allow for barge tows 43.5 ft. by 300 ft. Minimum vertical clearance through the barge canal system is 15.5 ft.

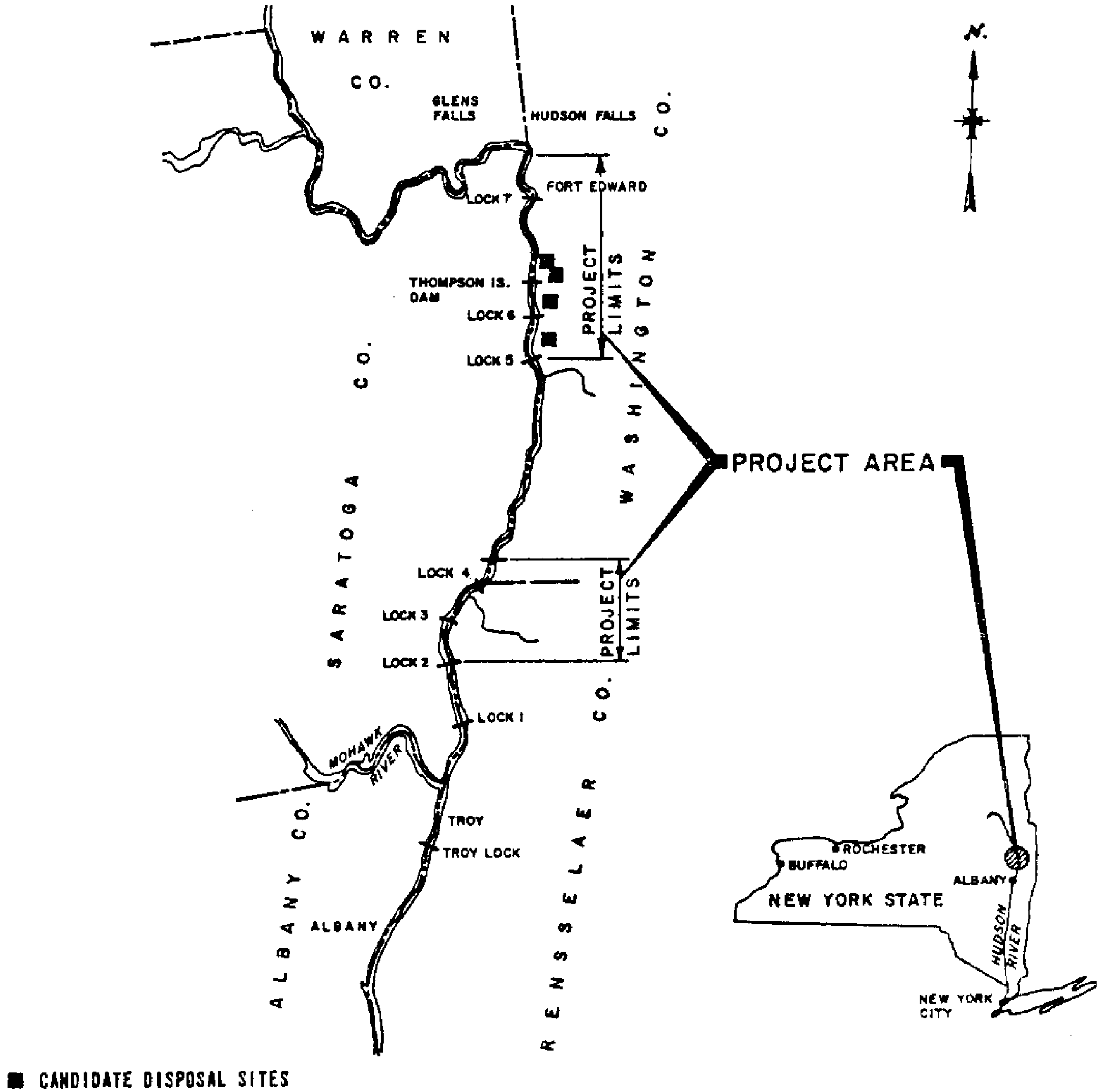

WARREN
CO.
GLENS FALLS
HUDSON FALLS
CO.
FORT EDWARD
LOCK 7
THOMPSON IS. DAM
LOCK 6
LOCK 5
PROJECT LIMITS
WASHINGTON
SARATOGA CO.
PROJECT AREA
LOCK 4
LOCK 3
LOCK 2
PROJECT LIMITS
LOCK 1
MOHAWK RIVER
RENSSELAER CO.
ALBANY CO.
TROY
TROY LOCK
ALBANY
ROCHESTER
BUFFALO
NEW YORK STATE
ALBANY
HUDSON RIVER
NEW YORK CITY
N.
■ CANDIDATE DISPOSAL SITES

FIGURE 1

PCB contamination in the river reach is highly variable both longitudinally and across the river section. PCB contamination also varies with depth in the river bed. Peak values are typically reached at about 10 inches in depth. Generally contamination does not exceed a depth of 24 inches. In the uppermost pool of the study reach (Thompson Island) a contaminated depth of 24 inches is estimated. In the remaining 7 pools contamination depths do not exceed 15 inches. Approximately 10 to 20 percent of the bed surface in the areas to be dredged comprises silt sizes or finer material.

The proposed dredging program will be limited to 40 "Hot Spot" areas where PCB levels exceed 50 μg/g. Hot spot locations in the Thompson Island Pool are indicated in Figure 2. Maximum values of PCB measured are on the order of 3,000 μg/g. Average values in the hot spots are approximately 125 μg/g. It is estimated that approximately 95 percent of the PCB present will be recovered from areas dredged. This is based upon PCB loss rates for material missed by the dredge, lost to the water column at the dredge wood and discharge to the river after treatment of return flows.

A program of full river bed dredging in the 40-mile reach would cost on the order of $200 million. A full dredging program was judged not to be feasible at this time.

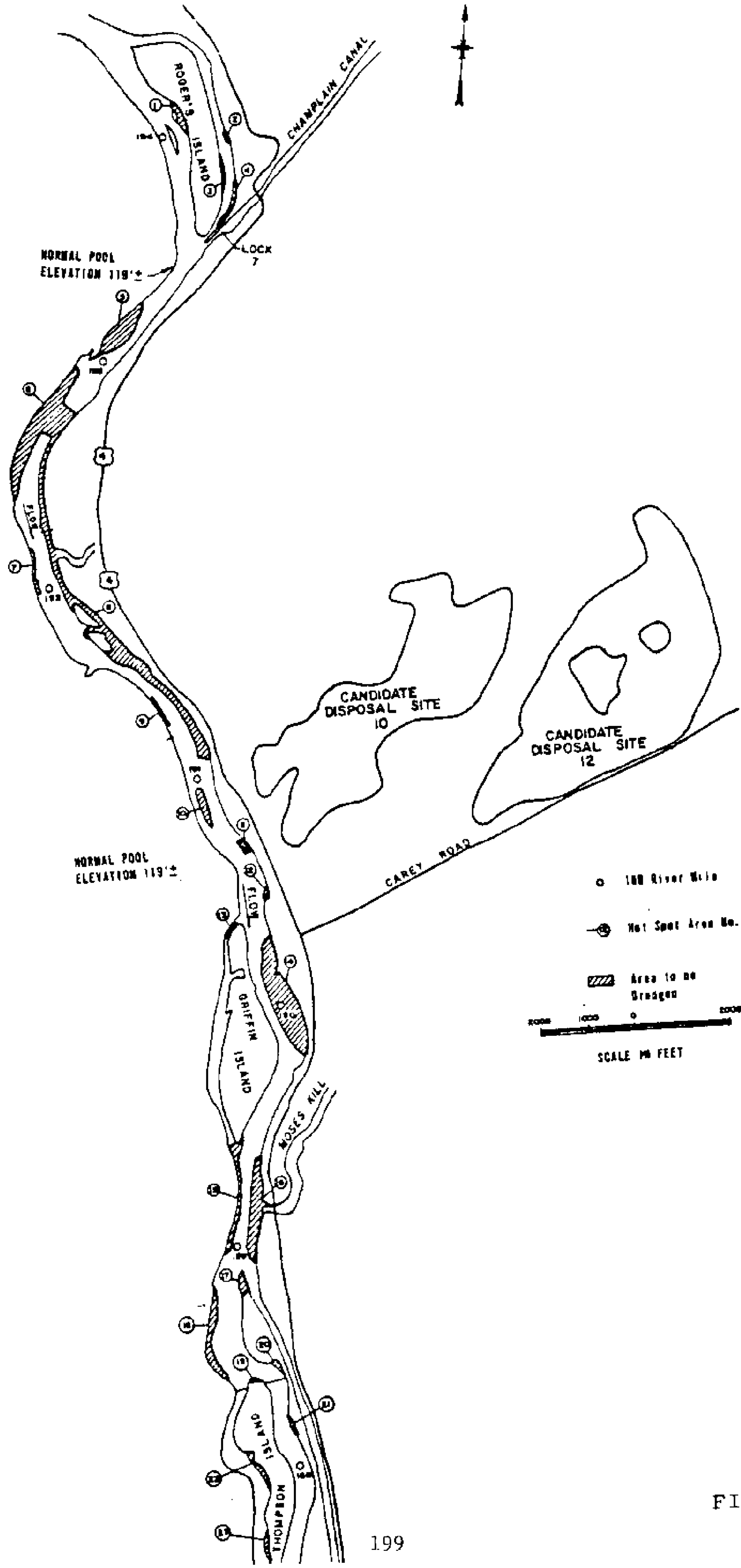

ROGER'S ISLAND
CHAMPLAIN CANAL
LOCK 7
NORMAL POOL ELEVATION 119'±
FLOW
CANDIDATE DISPOSAL SITE 10
CANDIDATE DISPOSAL SITE 12
NORMAL POOL ELEVATION 119'±
CAREY ROAD
FLOW
GRIFFIN ISLAND
MOSES KILL
THOMPSON ISLAND
River Mile
Hot Spot Area No.
Area to be Dredged
SCALE IN FEET

FIGURE 2

Dredging Volumes

Dredge quantities are based upon a minimum cut of two ft. in the areas to be dredged. An additional 1 ft. overcut has been allowed and volume estimates are based on a total cut of 3 feet. The total dredging volume is 1,500,000 cubic yards. Incentives are being considered to limit dredging to cuts of 2 ft. in order to reduce the volumes of material to be disposed and return water to be treated.

Alternative Dredging Systems

Alternative dredging systems investigated for the program consist of 4 major elements:

- Descriptions of the areas to be dredged
- Dredge Transport Systems
- Disposal Site(s)
- Return Flow Treatment

Each of the material-handling elements involves potential losses to the environment.

The performance and cost characteristics of alternate dredging systems have been based on essentially conventional, currently available dredging systems. Information on the pneuma and oozer type dredges is limited and these dredges are not readily available in the United States. Information on the economics of these systems is also not readily available. In the event these conditions change these systems will be given consideration for the dredging program.

Dredging systems which have been examined for the dredging program are 16 in. hydraulic cutter head dredge, 5-cu. yd. clamshell dredge with mechanical unloading of barges, and 5-cu. yd. clamshell with hydraulic pumpout of barges. All dredging systems examined have utilized a single disposal site. The use of one disposal site will minimize local disruptions. A single disposal site also simplifies the monitoring and maintenance requirements. All return flows from the disposal site and materials handling systems including precipitation will be treated prior to discharge to the river.

The water treatment system at the site includes the initial sedimentation which takes place in the primary disposal area followed by secondary or settling ponds prior to addition of polymers with flash mixing and flocculation.

The barge pumpout system includes a provision for recycling of pumpout water. This will minimize the load on the water treatment system as well as the discharge of flows to the river containing some trace of PCB. Water treatment flow for the hydraulic dredge system is estimated to be 10 mgd. Return water flows for the mechanical unloading system and the hydraulic unloading with recirculation will be on the order of 1 million gallons per day. See Figure 3 for a schematic diagram of the 3 alternate dredging systems under consideration.

Figure III-1

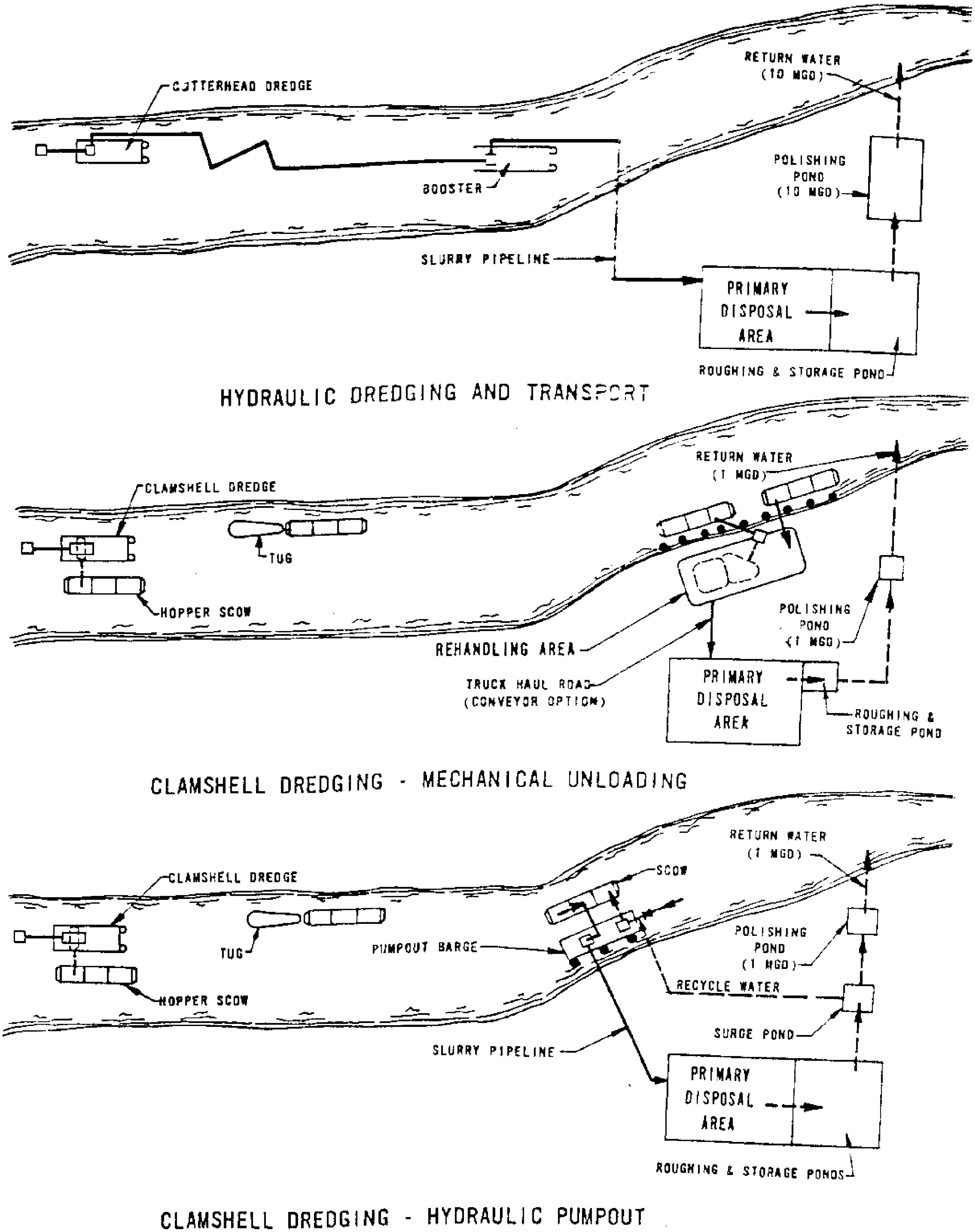


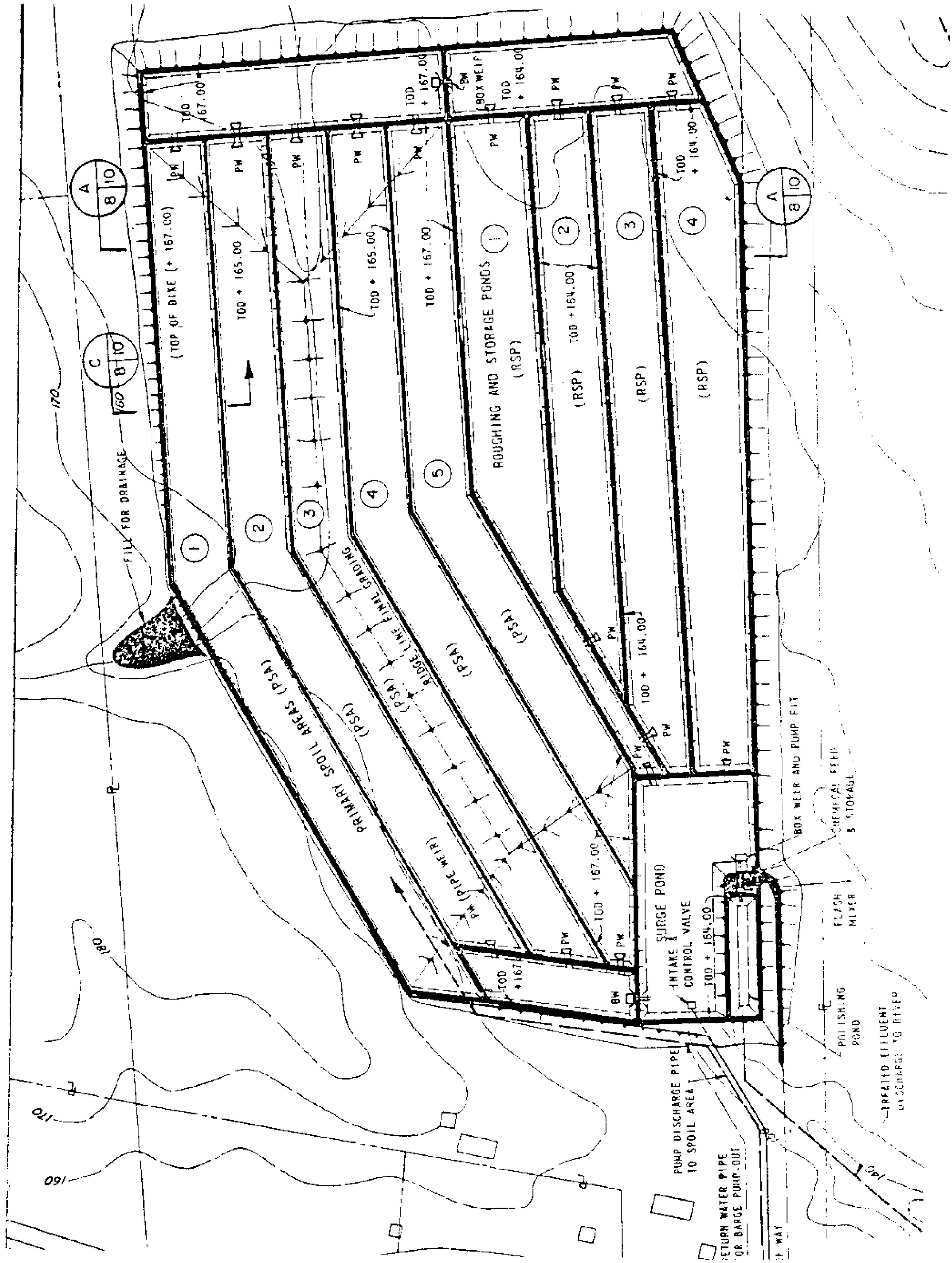
FIGURE 3

ALTERNATIVE DREDGING SYSTEMS

Disposal Site

All dredge material will be placed in a contained disposal area of 100 to 200 acres which meets Federal EPA and New York State criteria for chemical landfills. The essential requirement contained in these criteria is the existence of highly impermeable clay soils with permeability on the order of 1×10^{-7} c/s. The disposal area will have a perimeter containment dike constructed of the clay material to heights of 10 to 15 ft. The outside slopes of these dikes will have a minimum slope of 1 in 4. Internal drainage systems which utilize available coarse materials are provided in the preliminary design. An 18 in. clay cover will be placed on the surface of dredge material and covered with another 18 inches of coarse and soil material for turf establishment. A diagram of the disposal area for hydraulic pumpout is shown in Figure 4. The layout shown provides a rather elaborate system of interior dikes. This system is proposed in anticipation of encountering large percentages of fine-grained materials which may be difficult to dewater and complicate placement to the final clay cover. The diked channels shown will provide an opportunity to segregate fine materials between fingers of coarse, stable materials. If detailed river-bed probing indicates that fine-grained material will not be a problem this complex layout will not be necessary.

Estimated costs of the proposed dredging programs are indicated in the following table.



PRIMARY SPOIL AREAS (PSA)
ROUGHING AND STORAGE PONDS (RSP)
SURGE POND
(TOP OF DIKE (+ 167.00)
TOD + 165.00
TOD + 167.00
TOD + 164.00
TOD 167.00
(RSP)
(PSA)
RIDGE LINE FINAL GRADING
FILL FOR DRAINAGE
PIPE WEIR
PW
BW
BOX WEIR
INTAKE & CONTROL VALVE
BOX WEIR AND PUMP PIT
CHEMICAL FEED & STORAGE
FLASH MIXER
POLISHING POND
TREATED EFFLUENT DISCHARGE TO RIVER
PUMP DISCHARGE PIPE TO SPOIL AREA
RETURN WATER PIPE FOR BARGE PUMP-OUT
170
180
160
140

FIGURE 4

	Cu Yd	Cost	$/Cu Yd
Thompson Island Pool	700,000	$ 9,600,000	13.70
Other Pools	800,000	12,300,000	15.50
Total	1,500,000	$21,900,000	---
Remnant Deposits	290,000	$ 3,000,000	---

The program costs for the Thompson Island Pool are allocated approximately as follows: Dredging/Transport 40%, Disposal Site 5%, Water Treatment 20%, and Contingencies, Engineering Monitoring and Overhead 40%.

The dredging program envisions a two-season program. The first season involves the uppermost or Thompson Island Pool reach of the river. The second season involves five additional river pools. The remnant deposits noted in the table will be removed in the first season dredging. These deposits are remnant areas in the former pool of an old river dam which has been removed. The remnant deposits are essentially above present low river water levels. They will be removed by conventional dredging and truck haul to the single contained disposal area.

The program envisions the ultimate removal and destruction of the PCB contained in the dredged material at such time as destruction becomes technically and economically feasible.

Program Administration

An important requirement for the success of the proposed dredging program will be dredging administration. It is obvious that no matter how sound the engineering design and specifications are for the program, ultimate success depends on the day-by-day dredging operation. It is anticipated that several aspects of dredging administration will involve:

- prequalification of bidders
- preconstruction conference
- improved dredging equipment
- improved dredging techniques
- dredging monitoring

Although essentially conventional dredging equipment and systems will be utilized, it is anticipated that some improvements in actual dredging equipment and dredging techniques may be possible to maximize recovery of PCB-contaminated materials at a reasonable cost. It is anticipated that extensive monitoring of the dredging activities will take place to measure performance as well as to demonstrate that no unacceptable environmental impacts are taking place. The dredge disposal site will also have an extensive monitoring system for detecting any PCB leakage.

Program Schedule

The State of New York intends to implement the program upon the receipt of federal funds. Federal funds are being sought under several provisions of the Federal Water Quality Act as well as the possibility of special federal legislation. It is hoped that some resolution of the funding process can be reached early in 1979. In anticipation of early federal funding the program schedule calls for:

- o disposal site preparation in 1979,
- o Thompson Island Pool dredging and remnant deposits removal in 1980,
- o dredging of remaining pools in 1981.

The dredging program proposed is not a routine project. It will also be under close scrutiny by regulatory and environmental groups to assure that the removal of the PCB contamination from the river is being done in the most efficient manner possible. As such it represents a challenge to the dredging industry to carry out the program in a sound and economic manner.

DREDGING THE PANAMA VS. THE SUEZ:

UNIQUE PROBLEMS FACING EACH OF THESE WATER PASSAGES TO THE WORLD

by Doug Harris[1]

ABSTRACT

The history of the construction of the world's two great canals, the Suez and Panama, is like a novel. Thousands of people worked in adverse conditions, facing disease, starvation, dehydration, burning deserts and steamy jungles. Behind the scenes, political intrigue and high finance often shaped the fate and fortunes of those involved. The story of the canals is also a story of dredging. The two canals presented an early large-scale application of modern dredging techniques. Dredging continues to play an important role in the maintenance and expansion of both canals, and each has its own particular set of problems. The desert still tries to cover the Suez. And in Panama, silting and slides keep dredge crews busy.

With the world's economic and energy problems, it is crucial that such shortcuts remain open. In this paper, comparisons and analyses of geologic, technical, and financial problems of the two canals are presented. Geologic factors involve differences in terrain, soils, and general geographic difficulties in construction. Technical and financial topics discussed include 1) particular applications of dredges used in construction, 2) technical comparisons of the two canals

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including cost comparisons, and 3) maintenance and capital improvement problems facing the canals. In conclusion, the paper discusses the role dredging will play in the future of the canals, and the role DredgeMasters International has taken in the area of major development in the two canals.

Introduction

The history of the construction of the world's two great canals--the Suez and Panama--is like a novel. Thousands of people worked in adverse conditions, facing disease, starvation, dehydration, burning deserts, and steamy jungles. Thousands died. Behind the scenes, political intrigue and high finance often shaped the fate and fortunes of those involved.

It is also the story of dredging. The two canals presented an early large-scale application of modern dredging techniques. In each project, the use of dredges differed. But with both canals today, the use of dredging is still an important factor in maintaining the two navigational shortcuts.

First, let's look at the geography and a little of the history of the two regions.

The Suez Isthmus separates the Mediterranean Sea on the north side and the Red Sea and Suez Gulf on the south. Before the canal, it was 100 miles of blistering desert, marshes and stagnant lakes. In prehistoric times, the land was part of either or both bodies of water. Wide expanses of it are below sea level, though in places plateaus 30 feet high stretch across 10 miles of desert.

Senourset III, Pharaoh of Egypt, built a canal through the desert in about 2000 B.C., but the fine, shifting sands kept clogging the channel. In the following centuries, a succession of rulers tried to keep the canal open, but the desert always reclaimed it.

The Panama Isthmus has different characteristics, altogether. A spine of mountains runs down the Isthmus, and along either side are disease-infested jungles, warm tropical lakes and seemingly bottomless swamps. After Vasco de Balboa discovered the Pacific Ocean by crossing over the mountains, he recommended that a fortified trail be cut to facilitate trade from ocean to ocean. As an afterthought, he suggested a search for a straight. If one could not be found, he concluded, "It might not be impossible to build one."

Of the two canals, the Suez was by far the easiest to construct. There were no mountains and very little rock. It was a job made for dredging. The plan was simple: build service canals in the deserts, lakes and swamps; then, let dredges go to work widening and deepening the channel. At the peak of the service canal work, 25,000 conscripted Egyptians labored in the deserts and swamps for the Egyptian government which owned part of the canal along with the private Suez Canal Company.

Horse-drawn wagons riding on rails pulled out some of the dirt. Camels also carried out a good portion. In one lake, men scooped up mud, pressed it against their bodies to squeeze out the water, then deposited it in baskets.

But the lion's share of sand, dirt and gravel moved through French dredges. At one point, a dredge was actually taken apart, carried overland and reassembled in a distant service canal.

In ten years, the Suez Canal was completed. The man who conceived the plan, raised the money and then oversaw construction was Frederick de Lesseps, a diplomat, promoter and lay engineer. He was hailed as a genius, a man who could move mountains. People were eager to invest in his next project, and he cockily predicted he would build the shorter Panama Canal in eight years.

His company did put eight years into the Panama project, but it barely cleared 25 percent of the earth and rock standing between the two oceans. Bankruptcy followed, and the French sold out to the United States. The French failed for several reasons. The venture was rife with corruption. There was no standardization of methods or equipment. For instance, five years into the project the French were still experimenting with several ways to move earth. In a given day, workmen used bucket and suction dredges, elevators, cableways, steam shovels and even hand labor. In addition, eleven different types of flatcars carried dirt on six different gauges of track. They were, in short, working against each other.

Yellow fever and malaria killed thousands and disabled many more.

The French, though, never lacked for equipment. In their first 18 months in 1881 and 1882, they shipped in a staggering amount of gear, especially when compared to the U.S. commitment in hardware during its first year and a half in 1904 and 1905. The French brought four times as many steam shovels, six times as many flatcars, half again as many locomotives, twice as many boats and barges. They laid 10 times as much railroad track. And they doubled the number of dredges, putting 14 to work compared with seven for the U.S.

By far, the French had superior dredging equipment. They made most of their headway, in fact, by dredging. They removed 78 million yards during their fling in Central America. In 1883, they brought in three of the largest, heaviest and most complex dredges ever seen on the Isthmus. Among them was the Compte de Lesseps, one of the first large hydraulic cutter suction dredges. It was 120 feet long, 30 feet wide and drew eight feet. One dredge authority commented that it was so nearly automatic, "that it could be operated by only a dozen men."

America paid de Lesseps' company $40 million in 1902 for all properties and concessions of the canal. Another $10 million went to the newly formed Republic of Panama, plus a guarantee of $250,000 a year beginning in 1913.

Private bids to construct the canal were unsatisfactory, so the U.S. government decided to do the work through the U.S. Army Corps of Engineers. Planning and coordinating fell on the shoulders of Col. George W. Goethals, a methodical engineer who once remarked, "You don't have to be crazy to do a job like this, but it helps." I'm sure this famous statement has been carried down through the years and probably even quoted at one time or another by one of you here today. Goethals was determined to make the most of what he could learn of the French mistakes.

Another Army colonel, Dr. William C. Gorgas, paved the way for construction, so to speak, by identifying mosquitoes as the carrier of yellow fever, as well as malaria. A comprehensive control program knocked out the debilitating disease.

For excavation, Goethals settled on one method: steam shovels would scoop up dirt and place it in rail cars which would haul it away. Simple and to the point. The French had done most of the initial dredging work, leaving the Americans with other challenges. They had to cut through the mountains and also build three sets of locks to compensate for the difference in height of the Atlantic and Pacific. The U.S. still dredged, but it was primarily to keep open already cut channels.

The U.S. moved 219 million yards of earth and rock, blasting through the mountains. And the 1,000-foot-long locks are still considered an engineering masterpiece. The canal officially opened in 1920, and the final price came to $400 million.

In the long run, keeping a canal open can be as big as job as building it in the first place. Dredging continues to play an important role in the maintenance and expansion of both canals, and each has its own particular set of problems. The desert still tries to cover the Suez. And in Panama, silting and slides keep dredge crews busy. With the world's economic and energy problems, it is crucial that such shortcuts remain open.

We have seen what an extended canal closing can mean. The 1967 Middle East War turned the Suez into a glorified tank trap, halting commerce through it for a decade. The U.N. estimates a $13 billion global loss in inflated shipping costs and lost business. It is easy to understand the economics of a canal closing. The Panama Canal cuts 10,000 miles off an Atlantic-Pacific transit. The Suez cuts by half the distance between the Arabian Gulf and South European ports on the Mediterranean. Even on a short-term basis, a canal closing could be expensive. Operating costs for ocean-going ships range from $1,000 to $10,000 a day. That makes dredging vital.

Our company has been fortunate enough to play a role in this effort by supplying dredging equipment to both port authorities.

In Panama, DredgeMasters was awarded a contract to renovate the 35-year-old U.S. MINDI. The MINDI, even by today's standards, is quite a lady. Her specifications include: a 32-inch suction and 28-inch discharge, with power from a 5,000 horsepower steam turbine fired by four boilers. She is 296 feet long from the cutter end to the outside of the spud keeper, with a 52-foot beam and a 14-foot molded depth. With full tanks and raised spuds and ladder, she displaces 3,288 tons. The spud gantry is 94 feet above the main deck, and the ladder is 106 feet long, capable of digging 72 feet down. She can pump 5,000 yards in an eight-hour shift--that is 625 yards an hour.

The MINDI handles 30 to 40 percent of the canal's dredging, dividing her time between maintenance and capital improvements. Three eight-hour shifts work round the clock.

Yes, the MINDI is quite impressive for what she is. But the craft is, nonetheless, antiquated. She is a marvelous platform, though, for improvements, and modernization is much more practical in this case than buying a new dredge.

The Panama Canal Company, the quasi-government owner of the canal, required three major improvements:

First, two main transport pumps in series, driven by diesel engines to replace the single existing pump and steam turbine.

Second, an electric-driven underwater ladder pump and diesel generator for power. This ladder pump is connected in series with the two main transport pumps.

Third, new control panels for operating the system from the pilot house as well as from inside the hull.

With computer technology, we have designed what we believe is a more efficient pump that will operate more economically and smoothly than previous pumps, and it will do so for longer periods of time.

For the two main pumps, all wear parts, except for the cases, are interchangeable. The aft or high-pressure pump has a heavy ribbed case to withstand greater pressure, and it has been hydrostatically tested to 275 psi. The forward pump has been tested to 150 psi. They are a single suction volute-type pump with four-vane impellers. The 80-inch diameter impellers are statically balanced before assembly. The impellers are driven by 11 1/2-inch forged steel shafts set in heavy duty anti-friction radial and thrust bearings. Single-piece bearing housings enclose the mechanisms for maximum oil circulation and cooling to increase service life. The impeller is fully adjustable to compensate for wear on the suction side liner, thus maintaining maximum efficiency throughout the life of the pump.

The pumps have 32-inch suction and 28-inch discharge, capable of providing 30,000 GPM (gallons per minute). They produce a total dynamic head of 370 feet when connected in series and 185 feet in a single pump operation. Two General Electric Electromotive diesels create a total of 7,200 BHP.

The second phase, the underwater ladder pump, is also a single suction, volute-type pump with a 32-inch suction and a 32-inch discharge. It has a three-vane impeller, 60 inches in diameter. The ladder pump helps to overcome the barometric head and removes a major work load from the main pumps by boosting material to the surface. An induction motor in an oil-filled housing powers the pump. It is directly connected to the pump through a water-tight seal. The motor's special windings are impervious to hot oil. The ladder pump is rated at 900 horsepower, and the direct connection of the motor to the impeller eliminates the need for an expensive gearbox.

The total connected system--the two main pumps and the ladder pump--is rated at 8,100 horsepower and 455-foot of total dynamic head.

The equipment is undergoing further testing here in New Orleans before shipment to the Canal Zone.

In Suez, DredgeMasters is supplying a new dredge, our heavy duty model HPC-24DRM DuraMaster. It will be the largest cutter suction dredge in the Suez Canal Company's fleet. Its cutter has 900 horsepower, with 4,405 horsepower for its connected pumps. Suction is 28 inches and discharge 24. The ladder length is 95 feet, with a downward reach of 68 feet @ 45° angle. The dredge will be delivered in January and will go to work immediately on port improvements.

The Suez Canal Company has entered an exciting expansion period after the decade the channel was closed. The canal was infested with mines, other explosives and the hulks of nine ships, including three dredges. Clearing the canal and raising the wrecks required experts from America, England, France and Russia, as well as Egypt. More than 100 Egyptians lost their lives in the clearing operation, most of them dying when recovered ordnance exploded. Now, with the canal cleared, dredges are widening the canal and increasing harbor space, with plans set to triple the total surface area and deepen the channel. Eventually, the canal will accommodate two-way traffic, and all but the largest tankers will be able to pass through fully loaded.

A French engineer reported to Napoleon in the 1830's on the possibility of building a Suez Canal, and he said that there might be enough current in a channel to flush away silt and sand. He was wrong, though. As the canal's area increases, so too will the amount of dredging to maintain it.

And on the other side of the world, the Panama Canal continues to require dredging, both for maintenance and capital improvements. The MINDI and other canal dredges, such as the veteran CASCADAS, will be busy. Since the canal opened in fact, more material has been dredged from it than was originally removed to construct it!

There will be no decrease in the need for dredging equipment, technology and know-how. The Panamanian government regardless of the form it takes, will keep the canal open for the revenue it produces. And in Egypt, the re-opening of the canal--especially to Israeli cargo--was seen as an olive branch long before Camp David. Egyptians also need the canal revenue, along with the commerce, industry and international prestige which go with it.

The oceans have been pulled together by these two great canals, thanks in large part to dredging technology. And the international dredging industry will be doing its part to keep them open.

LUNCHEON ADDRESS

by

Colonel Thomas A. Sands[1]

Good Afternoon:

It's a genuine pleasure to be with you today, mainly because I feel I have some subjects to discuss that are of considerable concern to those of you interested in the dredging industry and its activities.

Since much of what I have to say deals with federal laws and regulations, I thought I'd begin by likening our situation to that of Moses when he was attempting to lead his people out of slavery in Egypt. You'll recall that Moses and his people found themselves stranded precariously on the shores of the Red Sea, with the Egyptian army in hot pursuit.

Moses implored the Lord to part the waters in order to allow his people to pass over the sea and avoid the impending massacre. The Lors is reported to have replied, "I think I can help you by parting the waters, Moses, but before I can take that action you'll have to file an environmental impact statement.'

I guess this story illustrates the sometimes helpless and frustrated feeling all of us have had recently as we've been trying to carry on our normal business while attempting to comply with the maze of federal regulations that have come on the books over the past few years.

Like Moses and his people, you in the dredging industry as well as we in the Corps are trying to accomplish vital tasks. And I mean no blasphemy by this, but I'm likening the federal regulatory agencies' position to that of the Lord in that we want to help you get your job done as efficiently and economically as possible; but it's also our responsibility to follow and implement legislative guidelines aimed at protecting the nation's environmental quality.

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[1] District Engineer, New Orleans District, U.S. Army Corps of Engineers.

But, today, I'd like to put this problem in its proper perspective by giving you my perceptions of where we've been, where we are now, and where we're going in the foreseeable future, when it comes to dredging our waterways.

The passage of the National Environmental Policy Act of 1969 began a chain of events that has intensified enormously over the last few years, seriously altering the way we do our business. NEPA was aimed at redirecting the nation's priorities to strike a balance between our traditional goal of economic development and our newly perceived need to preserve our nation's environmental quality.

Of course after NEPA, we got The Federal Water Pollution Control Act; The Marine Protection, Research and Sanctuaries Act; The Coastal Zone Management Act; The Fish and Wildlife Coordination Act; The Endangered Species Act; The National Historic Preservation Act; and others, not to mention the many subsequent amendments to these acts.

Now I'm sure no one advocates polluting the air or water, killing off birds and furry animals, or destroying the nation's natural wilderness areas and its historic and cultural treasures. In Louisiana, we have some of the nation's most scenic wetland semi-wilderness areas, an abundance of fish and wildlife resources, and a strong cultural heritage going back several hundred years.

But we also make our living in Louisiana, which is one of the nation's largest energy producers as well as the location of the nation's second and fourth largest ports. The major commodities handled at these ports are crude petroleum and grain products. Therefore, we in Louisiana realize the importance of our waterways, not only for our own livelihood but for the nation's energy and food needs.

So here in Louisiana, we, perhaps more than people anywhere else in the nation, understand and appreciate the need to strike a balance between economic growth and environmental quality.

Well, for the last five years or so, we in the New Orleans District have been struggling to understand the new regulations and to apply them judiciously. And I might add we've been doing that with very little increase in our manpower or funding to help us conduct the extremely time-consuming and expensive tasks necessary for compliance with the regulations. And no doubt about it, we are up to our eyeballs in problems right now, which many times means frustrating delays, while we attempt to comply with regulations. In some cases these regulations seem difficult or even impossible to implement.

Let me talk to you about specifics. The main problems we are now having in our maintenance dredging program concern the Environmental Protection Agency's ocean dumping criteria, which we have to satisfy before EPA gives us approval to dump dredged material into the Gulf.

For each of our maintenance dredging projects, we have to file a final environmental impact statement with the President's Council on Environmental Quality. There are then about 14 steps to complete, including public notice, possible public hearings, and preparation of a water quality assessment, before EPA's approval can be sought.

EPA has designated 11 ocean dumping sites for our district. EPA's original ocean dumping criteria required only chemical analyses of the water column and bottom sediments at these sites, and bioassay testing was optional. However, since September 1977, we've been required to do bioassay tests to determine the mortality of marine life before, during, and after ocean dumping at our 11 sites.

After bioassays became mandatory, we immediately had to take three dredging contracts off the market and cancel one that had already been awarded until we could have the bioassays done and hopefully obtain EPA's approval of the proposed dredging.

Subsequently, we have had delays of six months or more while we negotiated contracts to have the bioassays done. One of our worst problems was that there were practically no commercial laboratories with the capability of doing the chemical analyses and bioassays on the large scale we required.

Therefore, most of the sampling for water quality has been done by contract with the U.S. Geological Survey. The cost of collecting samples, water, and sediments, and the analyses for the approximately 40 materials we need to sample has been an averageof $1,000 per sample.

The New Orleans District spent over $800,000 in 1975 and nearly $1 million in 1976 acquiring water quality data necessary to obtain EPA approval to perform maintenance dredging. Resampling was required because EPA increased the level of accuracy of the analyses, especially for pesticides and PCB.

The cost of bioassays has so far been $50,000 for each of our maintenance dredging projects. Now EPA wants us to furnish them with additional bioassay data obtained from simulating summer and winter conditions in the Gulf, so we'll have to spend another $850,000.

Along these lines, we are faced with some severe "state-of-the-art" limitations in our bioassay procedures. What this means is that the methods required for making the bioassays have not been perfected.

We are now faced with our first test case of this problem. Dredging of the Calcasieu Bar Channel is desperately needed at this time to allow ships carrying petroleum products to get in and out of the Port of Lake Charles fully loaded. However, bioassays made for ocean dumping at that site failed to meet EPA criteria. To put it bluntly, "too many of the little critters died" during the bioassay tests.

The regulations say that if the bioassays show that EPA's criteria cannot be met, the regional EPA office in Dallas must deny our request to dredge, and we must then ask the Chief of Engineers to certify to the Secretary of the Army that to do no maintenance dredging would have a serious adverse economic impact on the region, and that no economical or suitable alternative to ocean dumping exists. The regulations stipulate that the Secretary of the Army then seeks a waiver of the EPA criteria from the EPA administrator in Washington.

In the case of the Calcasieu Bar Channel, the regional EPA office has refused our request for ocean dumping because of the results of the bioassays at the Calcasieu dumping site. We are now developing detailed information concerning the cost and environmental acceptability of a number of alternative methods of disposal. This, of course, means further costly delays before dredging of the Calcasieu Bar Channel can be accomplished, and the local shipping people in Lake Charles are understandably quite upset.

Since this is the first time we have had this experience, we are unable to estimate how long it will be before the problem is resolved. In the meantime, we are working as part of a group to revise and refine the ocean dumping criteria. and are reevaluating our bioassay procedures.

Another enormous burden imposed on us by EPA's "ocean dumping" criteria is the requirement that we prepare ocean disposal site baseline or trend assessment surveys for continued use of our 11 sites. EPA's original site designations for ocean dumping were for a 3-year interim period ending in January 1980. It will be impossible for us to complete our surveys and reports by that time unless we can find contractors who can perform in the required time.

We are facing an even more serious problem next year. It looks like we may be required to do bioassays to test the impact of dredging on marine life in our inland waterways as well as our ocean dumping sites. This could come about as early as next spring. If this does happen, we will have to make the expensive and time-consuming bioassay tests for the Mississippi River, the Gulf Intracoastal Waterway, and our many other inland waterways where maintenance dredging is a vital fact of life.

In addition to our problems with ocean and inland material disposal, we are facing further complications because of provisions in the 1977 amendments to the Federal Water Pollution Control Act. These amendments require that states, including local assuring agencies and the public, share an increased percentage of the cost of dredge and fill projects. Our Corps regulations have implemented this law by requiring local interests to pay for the cost of constructing and maintaining dikes to contain dredged material.

We have some doubts about whether or not local people would agree and in some cases be able to afford to pay an increased share of the costs of our projects. Also, in some instances, the local people may feel they receive no direct benefits from the project, and, of course, in the case of maintenance dredging of the Gulf Intracoastal Waterway, this is certainly true.

Let me say further that maintenance dredging is not our only problem: new construction is also affected. After President Carter announced his hit list in 1977, and after his veto of the appropriations bill last month, I guess we all realize that new construction is going to be a lot harder to come by than in the past. All new projects, as you probably know, will now require both a more stringent economic justification and a more thorough environmental assessment than ever before to receive authorization.

After listening to all of this, I hope you aren't taking me for a prophet of doom. I'm certainly not predicting the "dark ages of dredging," a "reign of regulational terror," a "great dredging depression," or any other catastrophe. Let me summarize my feelings about where we are now.

I believe that we are now in a transitional period in our nation's history, as well as in the Corps, as we attempt to strike a balance between our country's original need for rapid expansion and development and our newly perceived need for controlled, planned development that takes into consideration the need to preserve environmental quality.

Right now we are admittedly in a state of flux, frustration, and sometimes seeming confusion as we try to find ways to process and implement the regulations as fast as they are being written, revised, and refined.

And I'll lay it on the line to you when I say that I believe that the trend right now is to strengthen the regulations and to restrict dredging and filling activities while the guidelines are being modified. In the short run, this means we may be doing less work than in previous years while we refine the regulations and develop better scientific techniques for assessing environmental impacts. It's possible that we'll have a few temporary work stoppages, and you can be fairly certain we'll have fewer new construction starts.

But as I said, I'm not a prophet of doom. Quite the contrary. Although for the foreseeable future we'll undoubtedly be preoccupied with learning to live with the existing laws and regulations and adjusting to new or revised ones as they come along, the outlook for you in the dredging industry is very bright.

I am confident that we in the New Orleans District will be allowed to continue to provide the services to the public necessary to keep our vital waterway system functioning. And that of course means regular maintenance dredging of the 40-foot Mississippi River channel, the 36-foot Mississippi River Gulf outlet, the 40-foot Calcasieu Channel, and the Gulf Intracoastal Waterway and its feeder channels.

Let me remind you that the New Orleans District does more maintenance dredging annually than any other district in the Corps or almost 30 percent of the total Corps dredging nationwide. During a normal dredging year, we remove a total of about 70 million cubic yards of material from our waterways, 55 million cubic yards from our three deep-draft channels, and 15 million cubic yards from our shallow-draft waterways.

About 60 percent of this is done by contract. In fiscal year 1978, you people in the dredging industry performed $12 million of our new work and $21 million worth of our maintenance dredging, while our government dredges performed only $9.4 million of the work, all of it maintenance dredging.

The percentage of our dredging done by contract is bound to increase as we further develop our industry capability program, which, as you well know, is designed to determine the dredging industry's capability to perform in a timely manner and at a reasonable cost of the work traditionally performed by government dredges.

In the New Orleans District, the industry capability program is concerned only with hopper and dustpan dredging, since all other dredging has been handled by our contractors for years. In FY 79, our district will advertise two jobs under the industry capability program that have traditionally been done by government dredges. One is for removal of 12 million cubic yards of shoal material from the Calcasieu River Bar Channel. This is ordinarily done by government hopper dredge. The other job is for dredging 7 million cubic yards from the Mississippi River crossings. This work is normally done by a government dustpan dredge.

And as you people build hopper and dustpan dredges and you perfect operation of this equipment, you'll be doing more and more of our dredging, as we wind down the government fleet to the point where we're handling mostly emergencies and national defense work.

Incidentally, along those lines, you might be wondering why we're building a $67.5 million hopper dredge if we're planning to turn more of our work over to you. Well, although we will be reducing the government fleet in the future, we intend to maintain our emergency capability with the most modern, up-to-date vessels, and the dredge Avondale Shipyards will build for the New Orleans District will be of this vintage.

Now . . . More good news. Looking to the future, as you know, we have a study in progress to assess improving deep-draft access to the ports of New Orleans and Baton Rouge. The present 40-foot depth will become increasingly restrictive as commerce grows and because of the well-established trend toward larger ships. An increasing number of ships in the world fleet cannot now navigate the present deep-draft approaches to the ports fully loaded, and this situation will undoubtedly get worse without the deepened channel.

Although our study is not yet complete, we'll probably recommend enlargement of the Mississippi River from Baton Rouge to the Gulf by way of Southwest Pass to a depth of between 50 to 55 feet.

If and when the Mississippi River channel is deepened, our dredging requirements would be more than double what they are now in the entire district. Right now, as I said before, we remove about 70 million cubic yards of material from our waterways each year. We estimate that a 55-foot channel from Baton Rouge to the Gulf would require removal of an additional 75 million cubic yards of material each year which, as I said, is double what we are now doing.

More good news! In the immediate future, we'll be keeping you people busy with our ongoing Red River Waterway Project with its 53 channel realinements and five lock and dam complexes. These will require considerable dredging work over the next few years, so there will be no shortage of work in the New Orleans District.

By the way, in explaining delays to you, I think it only fair to tell you that not all of our delays are caused by new regulations or in-house problems such as manpower shortages.

During the past year, we did not even spend our total maintenance dredging budget because some of our contractors were not able to get their dredges to the job on time. As a result, in some cases, our channels were not maintained to authorized dimensions, and funds not earned by the contractors were revoked. So although work slippages are frequently due to cumbersome regulations and in-house problems they have also been caused by the contractors themselves.

Let me close by making a prediction, and I think I'm on very firm ground when I make this one. We in the New Orleans District will always have plenty of work for you in the dredging industry to do as we continue to provide the public with the vital services necessary to keep our waterways open. I predict that in the future, industry will participate in our maintenance dredging program to an even greater degree than now.

We'll learn to live with the new regulations just as we have learned to live with other cumbersome governmental processes such as project authorization and design, and the advertisement, bidding, and awarding of contracts. I believe that in the not too distant future, we'll be going about our business as usual. according to an orderly and predictable process that provides for efficient, economical construction, and maintenance of our waterways in an environmentally sound manner.

Thank you.

CONSOLIDATION OF CONFINED DREDGED MATERIAL

Marian E. Poindexter[1]

Abstract

Dredged material containment areas must be designed to provide adequate storage capacity to meet dredging requirements for the service life of the facility.

Following a given disposal operation, the dredged material undergoes sedimentation and self-weight consolidation, resulting in gains in storage capacity. The placement of dredged material imposes a loading on the containment area foundation; therefore, additional settlements may result from consolidation of compressible foundation soils. Hence, settlements resulting from consolidation are a major factor in the estimation of long-term storage capacity.

This paper presents guidelines for estimation of gains in long-term storage capacity resulting from settlements within the containment area. The guidelines are based on conventional consolidation theory modified to consider self-weight consolidation behavior of newly-placed dredged material. The effects of foundation consolidation, time-rate of consolidation, and placement of sequential lifts of dredged material are also described.

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Introduction

Dredged material containment areas must be designed to provide adequate storage capacity to meet anticipated maintenance dredging requirements. If the containment area is intended for one-time use, as in the case of some new work projects, estimates of long-term storage capacity are not required. However, containment areas intended for use in conjunction with recurring maintenance work must be sized for long-term storage capacity over the service life of the facility.

A methodology has been developed for estimating the long-term storage capacity of dredged material containment areas.[6] This methodology is based on an initial correlation of the in situ sediment void ratios and the void ratios of material in the containment area at the completion of dredging. Gains in long-term storage capacity resulting from settlement of the dredged material and foundation soils can then be estimated by using the fundamental principles of consolidation theory modified to consider the self-weight consolidation behavior of newly placed dredged material. Use of available computer models is recommended for cases involving repetitive disposal operations and/or intermittent dewatering or removal of material. This method is equally applicable to the design of new containment areas or evaluation of existing sites.

Concepts of Containment Area Design

The design/analysis method presented herein was developed for and is applicable to confined areas used for containment of maintenance-dredged material. Such areas consist of a tract of land enclosed by dikes to form a total confined surface area and volume into which dredged channel sediments are pumped hydraulically. The major components of a dredged material containment area are shown schematically in Figure 1.

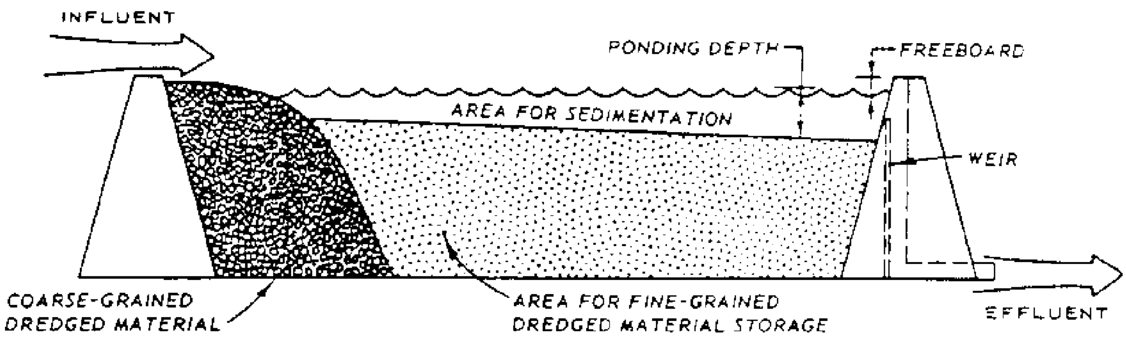

Figure 1. Conceptual diagram of a dredged material containment area

The storage capacity of a containment area is defined as the total volume available to hold additional dredged material. It is equal to the total unoccupied volume minus the volume associated with ponding and freeboard requirements.

Settlement resulting from consolidation of both the dredged material and the foundation soils is the major consideration in estimation of long-term storage capacity. After dredged material is placed in the containment area, it undergoes sedimentation and self-weight consolidation. Placement of the dredged material imposes a loading on the containment area foundation soils which may then undergo settlement as a result of consolidation of the compressible foundation soils. Since the consolidation process is slow, especially in the case of fine-grained materials, it is likely that total settlement will not have occurred before the containment area is required for additional dredged material placement. For this reason, the time-consolidation relationship must be considered in estimating long-term containment area storage capacity. Settlement of the containing dikes may also significantly affect the available storage capacity and should be considered.

Consolidation Testing

Determination of containment area long-term storage capacity requires estimation of settlements resulting from self-weight consolidation of newly placed dredged material and consolidation of compressible foundation soils. Consolidation test results, including time-consolidation data, must be obtained in order to estimate the average void ratios at completion of 100 percent primary consolidation.

Dredged material

The consolidation testing procedure for dredged material samples generally follows that for the fixed-ring test for conventional soils but some modifications are required concerning sample preparation and method of loading. A fixed-ring consolidometer should be used because of the fluid-like consistency of these samples.

Samples to be used for the consolidation test may be taken from the in situ channel sediments since this material has the same engineering properties in the channel as it will have when deposited in the containment area. These consolidation samples must be representative of the fine-grained portion of the material to be dredged. In the case of a relatively homogeneous fine-grained sediment, consolidation samples can be taken directly from the sediment grab-samples obtained during the field investigation phase. For sediments that contain mixtures of sand (>10 percent dry weight), a more representative consolidation sample can be obtained if the sand fraction has previously been separated. For this case the consolidation test should be performed on a sample with initial water content/void ratio approximately equal to that at the end of the dredging operation as determined by the procedures given in Reference 6.

Since sediment samples are essentially without structure, consolidation samples can be placed in the consolidation ring in a remolded condition. The ring should be placed on a flat plate prior to filling. The remolded sample should not be allowed to drain nor should air pockets be allowed to form within the sample while the material is being placed in the ring.

For the dredged material consolidation test, the initial load placed on the consolidation sample should not exceed 0.005 tsf. The relatively low initial load is necessary to adequately define behavior at low effective stresses. The seating load plus the compression load caused by the dial indicator should be considered as the initial loading increment for the test. The dial indicator force can be estimated using a balance reading, in grams, obtained with the indicator compressed to approximately that setting to be used when the test is initiated.

Succeeding load increments may be placed using the normal beam and weight or pneumatic loading devices. The following loading schedule is recommended: 0.005, 0.01, 0.02, 0.05, 0.10, 0.25, 0.50, and 1.0 tsf. A maximum loading of 1.0 tsf should be adequate for most applications. However, the effective stress acting at the bottom of the dredged material layer at the end of the containment area service life should be estimated to determine if a higher maximum load increment is necessary.

Time-consolidation data should be examined while the test is in progress to ensure that 100 percent primary consolidation is reached for each load increment. In some cases, it may be necessary to allow 48 hours for each increment. Rebound loadings are not required.

Foundation soils

Consolidation testing of foundation soils should be performed according to standard soil mechanics procedures. Guidance for performing this test may be obtained from References 4 and 5.

Settlement Due to Consolidation

Dredged material

Settlement resulting from self-weight consolidation of dredged material is estimated by considering the change in void ratio due to the self-weight loading. The average load is assumed to act at midheight of the dredged material layer and is equal to the effective stress due to the buoyant weight of the overlying material.

The following expression is used to compute the average effective stress acting at midheight of the dredged material layer:

$$\overline{p}_f = 1/2\, H_{dm}\, \gamma_w \frac{G_s - 1}{1 + e_o} \tag{1}$$

where

$\overline{p}_f$ = average effective stress acting at midheight of the layer of dredged material solids, psf

H_{dm} = thickness of the dredged material layer at completion of the dredging operation, ft

γ_w = density of water = 62.4 pcf

G_s = specific gravity of solids

e_o = average void ratio of dredged material at completion of dredging

The initial thickness of the dredged material layer H_{dm} is a function of the surface area in use and the volume occupied by the dredged material at the completion of the dredging operation; this value must be determined from sedimentation analyses.[6] When evaluating the remaining long-term storage capacity of existing sites, the surface area will be known and the initial lift thickness for a given disposal operation may be determined directly. However, design of new containment areas to accommodate a given long-term dredging requirement necessitates that the surface area be determined by trial as discussed later in this paper.

The change in thickness of the dredged material layer due to primary consolidation is estimated using the following expression:

$$\Delta H = H_{dm} \frac{e_o - e_f}{1 + e_o} \tag{2}$$

where

ΔH = change in thickness of the layer at completion of primary consolidation, ft

e_f = average void ratio at completion of primary consolidation

The void ratio e_f corresponding to the effective stress $\bar{p}_f$ is determined using an e-log p relationship (Figure 2) which is obtained

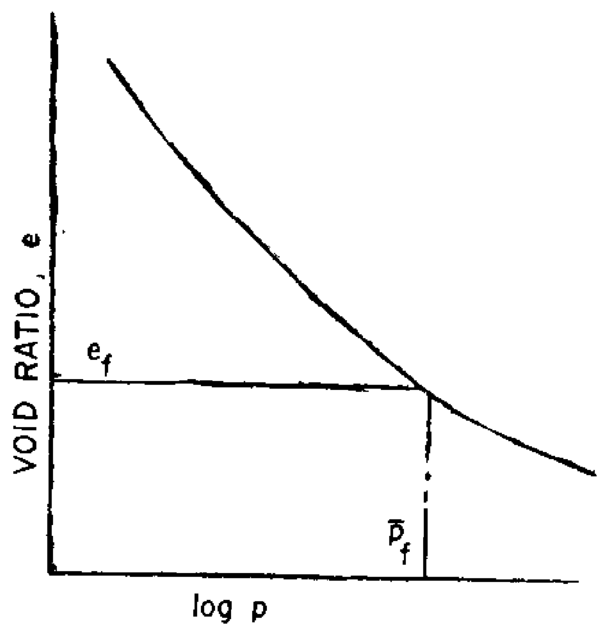

Figure 2. Illustrative plot of void ratio versus log of pressure for newly placed dredged material

from the dredged material consolidation test performed as discussed in Reference 6. This void ratio is representative of the average void ratio of the dredged material layer at the completion of primary consolidation. Since the time required to reach ultimate consolidation may take years, the dredged material layer will probably not reach its final thickness before the containment area is again required for dredged material placement.

Therefore, a relationship for the time-rate of consolidation must be developed before the available storage capacity can be estimated. This will be discussed later.

Foundation soils

Settlement of the foundation soils can be estimated by using conventional soil mechanics principles. Specific considerations related to containment areas are discussed below. Additional guidance for the determination of foundation consolidation and computation of settlement is described in EM 1110-2-1904.[5]

Settlement of the containment area foundation soils is caused by the increased load imposed on these compressible soils by placement of dredged material. The magnitude of this load is dependent upon the volume of dredged material deposited and the water table conditions existing during and following the disposal operation.

The total load on the foundation soils caused by placement of dredged material is initially dependent upon the weight of the layers of solids and the ponded water maintained in the containment area during the disposal operation. Following disposal, the ponded water should be decanted, thus reducing the total load. However, the groundwater table conditions (i.e., perched or continuous) existing during and after disposal will dictate the effective loading placed upon foundation soils. An evaluation of the groundwater conditions must be made based on the foundation stratification and initial water table conditions as revealed by the preliminary field exploration program. Since the imposed loads are uniformly distributed over an area which is usually large compared to the depth of the compressible layers, the increase in loading Δp on the foundation may be considered as constant for full depth.

The ultimate settlement of each foundation soil stratum for a given load Δp can be estimated by the expression:

$$\Delta H = H \frac{e_1 - e_2}{1 + e_1} \tag{3}$$

where

ΔH = change in thickness of layer at completion of primary consolidation, ft

e_1 = initial void ratio of soil layer at pressure p_1

e_2 = final void ratio of soil layer at pressure $p_2 = p_1 + \Delta p$

H = initial thickness of layer, ft

From the pressure-void ratio relationship developed from consolidation tests performed on the foundation soils, the values of e_1 and e_2 are obtained as shown in Figure 3 using the average loads p_1 and p_2 existing before and after the disposal operation, respectively.

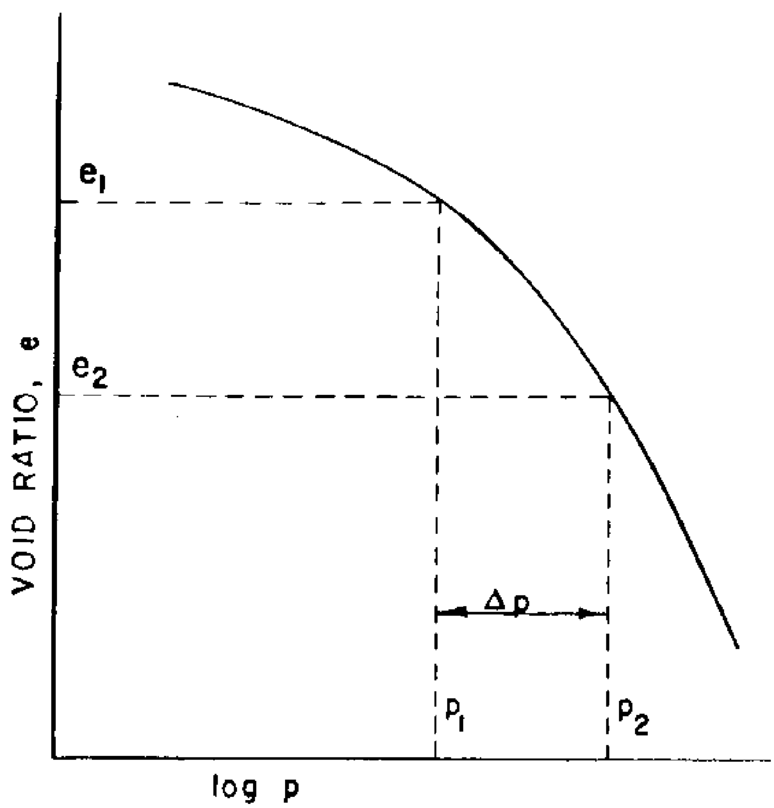

Figure 3. Illustrative plot of void ratio versus log of pressure for foundation soils

After the ultimate settlement has been determined for each foundation soil stratum under a particular load Δp , the total settlement of the foundation may be determined by summing the settlements of the individual soil strata.

Time-Rate of Consolidation

Since the consolidation of dredged material and compressible foundation soils may require significant periods to reach completion, the time-rate of consolidation must be considered in order for the storage capacity available at any time to be determined. The procedure for estimating the time-rate of consolidation described in this section generally follows those methods found in EM 1110-2-1904[5] and is applicable to both self-weight consolidation of dredged material and consolidation of foundation soils. Values for the coefficient of consolidation c_v may be determined from the consolidation-time data using the Log-Time Method.[5] The values for c_v should be determined for each consolidation pressure used in the consolidation tests and a graph of c_v versus consolidation pressure should be constructed as shown in Figure 4. The coefficient of consolidation c_{vf} corresponding to the

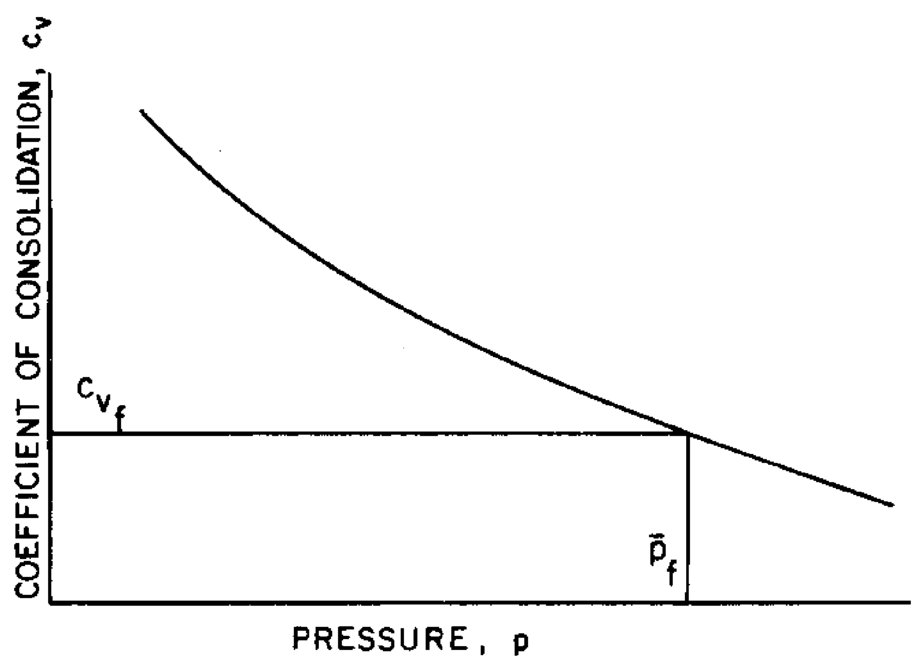

Figure 4. Illustrative relationship of the coefficient of consolidation versus consolidation pressure

average effective stress at midheight of the layer under consideration can then be determined using the graph.

Times required for an individual layer to reach various percentages of ultimate consolidation U can be estimated using the following expression:

$$t_u = \frac{T_u H_d^2}{c_{vf}} \quad (144) \qquad (4)$$

where

t_u = time to reach degree of consolidation U , min

T_u = time factor for degree of ultimate consolidation, U (see Figure 4)

H_d = effective drainage height, ft

c_{vf} = coefficient of consolidation corresponding to the average effective stress at midheight of the layer, in^2/min

Time factors for various percentages of total consolidation are shown in Figure 5; the two curves are required since the distribution of pore

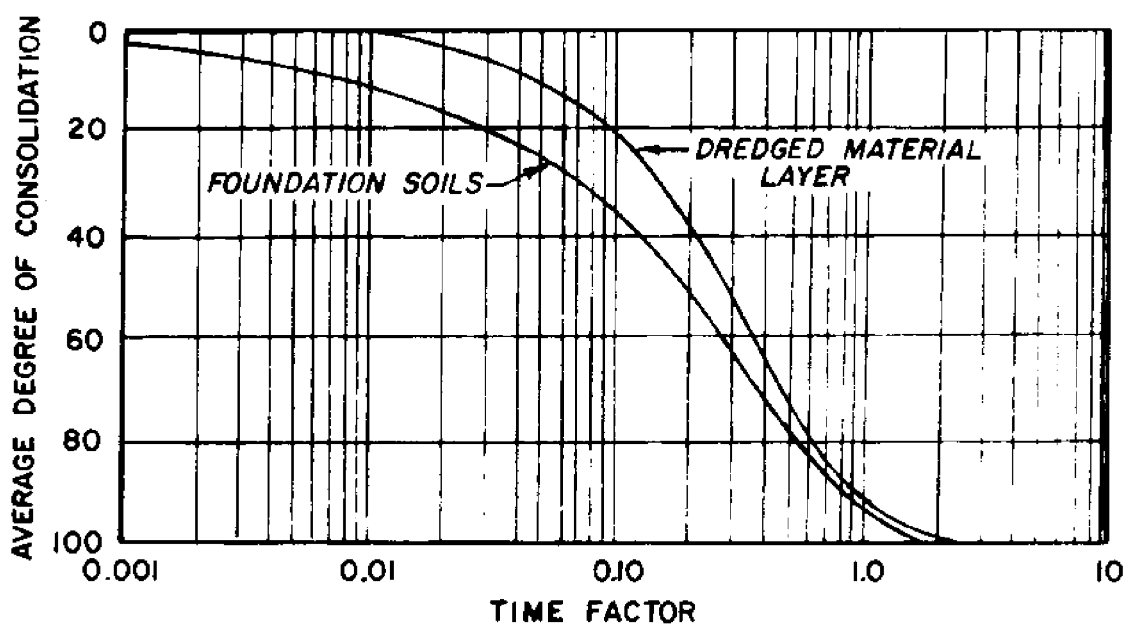

Figure 5. Time factors for consolidation analysis (adapted from NAVFAC DM-7[3])

pressures, and thus the rate of consolidation, are significantly different for dredged material and foundation soils. Values for H_d will be equal to the total layer thickness if single drainage occurs or one-half the layer thickness of double drainage prevails.

The settlement of the layer at time t_u may be estimated by the expression:

$$\Delta H_{t_u} = \frac{U}{100} (\Delta H) \tag{5}$$

where

ΔH_{t_u} = settlement of the layer at time t_u, ft

Using the settlement values ΔH_{t_u} calculated, a curve representing the time-settlement relationship can be constructed for each layer as shown in Figure 6. By combining the time-settlement curves for the dredged material

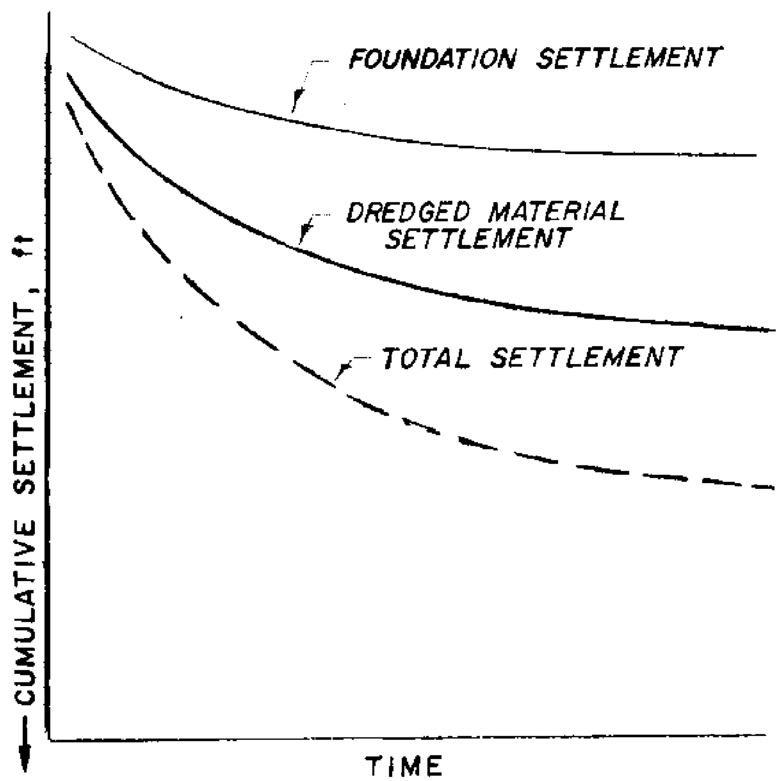

Figure 6. Illustrative time-settlement relationships

and foundation soil(s), the time-total settlement relationship resulting from placement of a single lift of dredged material can be obtained. Using this curve, the long-term storage capacity versus time relationship for a single lift or the short-term storage capacity versus time relationship for sequential lifts can be estimated. The effects of surface drying on storage capacity must be separately determined as described in Reference 1.

Placement of Sequential Lifts

Estimates of settlement caused by placement of subsequent lifts of dredged material should consider the continued consolidation of previously placed lifts and additional foundation consolidation as well as consolidation of the newly placed dredged material. This is most effectively done by considering the previously placed dredged material layer as an additional foundation soil layer.

Storage Capacity - Time Relationship

If the containment area is to be used for long-term placement of subsequent lifts, a graph of projected dredged material surface height versus time should be developed. This graph can be developed using the time-settlement relationships for sequential lifts combined as shown in Figure 7. Such data

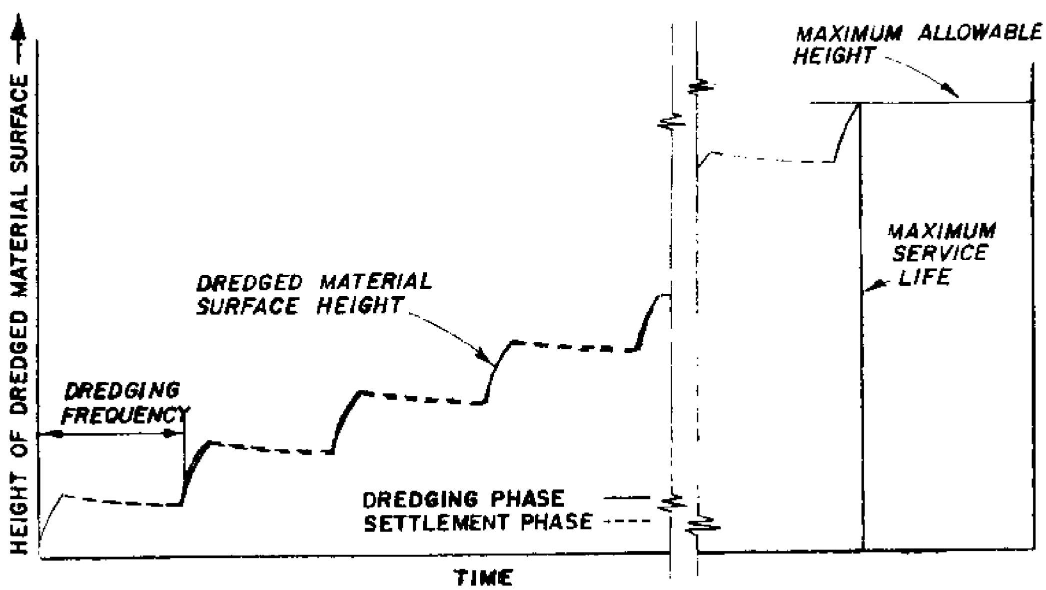

Figure 7. Projected surface height for determination of containment service life

may be used for preliminary estimates of the long-term service life of the containment area.

The maximum allowable height of dredged material at a particular site is dependent upon the allowable dike height and the ponding and freeboard requirements.[6] The maximum dike height as determined by foundation conditions or other constraints and the containment surface area will dictate the maximum available storage volume. The increases in dredged material surface height during the dredging phases and the decreases during settlement phases correspond to respective decreases and increases in remaining containment storage capacity, shown in Figure 8. Projecting the relationships for surface height

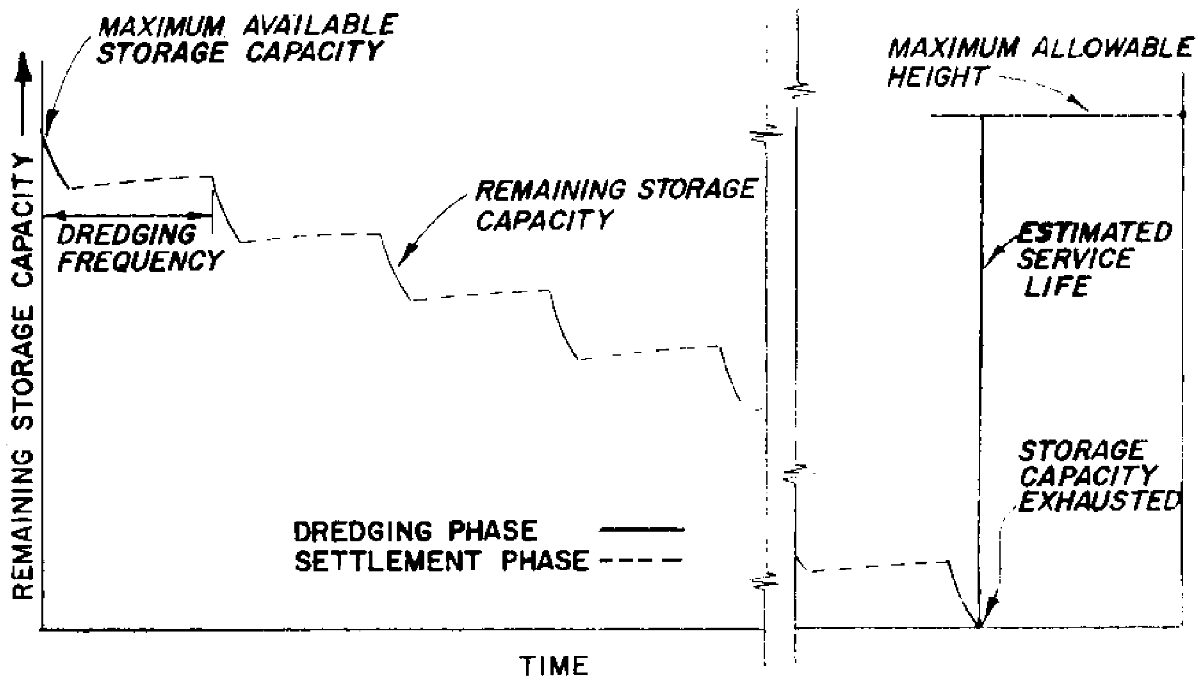

Figure 8. Projected storage capacity for determination of containment service life

(or remaining capacity) to the point of maximum allowable height (or exhaustion of storage capacity) will yield an estimate of the containment area service life. Gains in capacity due to anticipated dewatering or material removal should also be considered in making the projections.

The complex nature of the consolidation-time relationships for multiple lifts of compressible dredged material and the changing nature of the resulting loads imposed on compressible foundation soils will not allow accurate projections of remaining storage capacity over long time periods. For this reason such long-range projections should be used strictly for planning purposes. Accuracy can be greatly improved by periodically updating the estimates using data from newly collected samples and more recent laboratory tests. Observed field behavior should also be routinely recorded and used to refine the projections.

Mathematical Model

A computer model has been developed to assist in the design/analysis of dredged material containment areas.[2] The use of this computer model can greatly facilitate the estimation of storage capacity for containment areas. Although the computations for a simple case can be done relatively easily and quickly by hand, a typical analysis may require computations for a multi-year service life with variable disposal operations, and possibly material removal or dewatering operations, occurring intermittently throughout the service life. These complex computations can be done more efficiently using the computer model.

Model capabilities

This model is applicable to flooded containment areas for determination of the settlement resulting from primary consolidation of dredged material and foundation soils. This settlement is calculated by the model on the basis of the dissipation of excess pore water pressure according to the standard theory of one-dimensional (1-D) primary consolidation. A special explicit finite difference numerical technique was applied to solve the 1-D differential equation for primary consolidation. The numerical technique permits versatile boundary conditions which may be reasonably representative of actual field conditions for confinement of dredged material. The model is capable of computing the excess pore pressure distribution, average degree of consolidation, and settlement of dredged material and layered foundation soil strata of flooded containment areas. Time intervals for placement of dredged material during a single disposal operation and between various disposal operations may be varied. The consolidation parameters of the dredged material may be input as a function of the effective stress to permit improved simulation of actual field conditions; the consolidation parameters of the foundation soils are assumed constant.

A variety of dredging operations spaced at various time intervals may be handled by the code in a single computer run provided that the dredged material being placed is homogeneous with identical consolidation parameters. The consolidation behavior of dredged material and foundation soils may be calculated for a number of dredging operations involving different dredged material if (a) a single computer run of the code is used to solve for the consolidation behavior of the soil system for each disposal operation, and (b) the initial excess pore pressures are input from a data file.[2] The dredged material deposited during the previous disposal operation should be treated as the surface layer of the foundation soils (with constant consolidation parameters) for the current disposal operation.

Applications

The model can facilitate evaluation of existing containment areas or design of new sites by rapid determination of the settlement which will result from a specific loading condition. The method of application of the model to the design/analysis problem is dependent upon whether or not the surface area of the site under study is known. If the surface area is known, then the initial dredged material lift thickness and the resultant settlement for a given disposal operation can be determined directly. Otherwise, the optimum surface area must be determined by trial before these parameters can be determined.

In the evaluation of either an existing disposal site or a new containment area of predetermined volume and surface area, the model can be used to determine directly the settlement of the dredged material layer and foundation soils; analysis of these results will then yield the design service life of the containment area. The soil system analyzed for this application may be a simple one including placement of only one dredged material layer or it may involve placement of sequential lifts of dredged material in which the time required for the disposal operations and/or the time between operations may vary significantly.

The design of a new containment area for which the surface area of the site is not a fixed parameter requires a procedure of trials for which the computer model is invaluable. In this design situation, the known parameters such as required containment area service life, allowable dike height, frequency of dredging, and/or anticipated volume of dredged material should be held constant at the appropriate values for each computer run while the surface area is allowed to vary over the entire range of potential values. As the surface area is varied, the thickness of the dredged material layer will change as will the anticipated settlement resulting from the loading caused by the dredged material. This in turn will cause variation in the service life of the area for a given set of containment area parameters. By evaluating the results of several computer runs in which various surface areas are used, the optimum surface area for a given situation can be determined.

Conclusion

A methodology has been developed to estimate the long-term storage capacity of dredged material containment areas. This procedure for containment area design/analysis is based on evaluation of the engineering properties of dredged material and foundation soils. Conventional consolidation theory modified for dredged material behavior is used and permits consideration of the time-rate of consolidation. A computer model has been developed and its use is recommended to facilitate the design/analysis process. This methodology should allow design of containment areas for improved performance and utilization in an efficient and cost-effective manner.

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Notation

c_v coefficient of consolidation, in^2/min

c_{vf} coefficient of consolidation corresponding to average effective stress, in^2/min

e_f average void ratio at completion of primary consolidation

e_o average void ratio of dredged material at completion of dredging

e_1 void ratio of soil layer at pressure p_1

e_2 void ratio of soil layer at pressure $p_2 = p_1 + \Delta p$

G_s specific gravity of solids

H initial thickness of layer, ft

H_d effective drainage height, ft

H_{dm} thickness of dredged material layer at completion of the dredging operation, ft

p consolidation pressure or overburden pressure, lb/ft^2

$\bar{p}_f$ average effective stress acting at midheight of the dredged material layer, lb/ft^2

t_u time required to reach degree of consolidation U, min

T_u time factor for degree of ultimate consolidation U

U degree of ultimate consolidation, percent

γ_w unit weight of water, lb/ft^3

ΔH settlement (change in thickness) of the layer at completion of primary consolidation, ft

ΔH_{t_u} settlement of the layer at time t_u, ft

Δp increase in loading (change in consolidation pressure), lb/ft^2

STABILITY OF RETAINING DIKES FOR CONTAINMENT OF DREDGED MATERIAL

By D. P. Hammer[1]

ABSTRACT

Past experience with retaining dikes has indicated many problems exist in connection with the stability of such dikes. Foremost in the cause of these problems has been inadequate design with respect to dike stability. In 1973-1977, a study was made of past dike failures in the Corps of Engineers and guidelines were developed for future design which, if followed, will help minimize the chances of failure. This paper presents the results of that study, including recommendations for design to prevent dike failures from (1) inadequate shear resistance of embankment and/or foundation, (2) excessive uniform settlement, (3) differential settlement, (4) seepage, and (5) surface erosion (slope protection). Recommended minimum factors of safety for slope stability analysis are presented.

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INTRODUCTION

Retaining dikes used to form confined disposal facilities consist primarily of earth embankments constructed on lowland areas or near-shore islands with the principal objective of retaining solid particles within the disposal area while at the same time allowing the release of clean effluent back to natural waters. Retaining dikes are similar to flood protection levees in size and shape but differ in the following important respects: (a) a retaining dike will retain an essentially permanent pool, whereas most levees have water against them only for relatively short periods of time, and (b) the location of a retaining dike will usually be established by factors other than foundation conditions and available borrow material (i.e., proximity to dredge, only land available, etc.) from which there will be little deviation.

In their review of Corps of Engineers (CE) design and construction procedures for retaining dikes, Murphy and Zeigler (4) concluded that there is normally little effort expended in the design of most retaining dikes. It was found that, in most cases, no special effort was made to improve foundation conditions and that construction materials were normally borrowed from within the containment area, even though such materials often possessed very poor engineering properties. The method of construction generally was established through past practice and was not likely to be altered due to any particular foundation and/or dike material properties. Consequently, the selection of dike dimensions and construction methods was based largely on a review of previous dike construction experience. Dike heights, side slopes, and crown widths were chosen to match those of similarly constructed dikes that performed satisfactorily. In many cases a successful and stable dike was obtained; however, where foundation and/or dike materials were poor or dikes were constructed to appreciable heights, frequent failures occurred and continual maintenance was required.

For many containment facilities at unpopulated locations, there has been a tendency for less effort and expense to be applied to dike design and construction. Consequently, dike failures have been more frequent at these locations and resulted in the flow of dredged material onto tidal flats or marshes or into nearby rivers and streams. Not all failures have been confined to unpopulated or otherwise open areas, however. Damage to warehouses, a railroad embankment, a sewage treatment plant, and pastureland, and even flooding of a subdivision have been reported (4). In addition to property damage, there is usually the expense of redredging and repair of the dike.

Past experience indicates that the occurrence of dike instability can be related to the amount of design effort expended on the dike; i.e., as the dike design effort increased, the occurrence of dike failure decreased. Small dikes constructed in areas where design experience has been gained through actual dike construction will obviously require less design consideration than large dikes to be constructed in unfamiliar areas. Factors which will affect dike height and geometric configuration that should be considered during design are: (a) containment area capacity and operational requirements; (b) foundation conditions; (c) available construction materials; (d) construction methods; and (e) dike stability with respect to shear strength, seepage, settlement, and erosion. This paper is primarily concerned with the last item. A complete treatment of the design and construction of retaining dikes is given by Hammer and Blackburn (2).

FIELD AND LABORATORY INVESTIGATIONS

The importance of an adequate field and laboratory investigation to determine what types of materials exist and what their engineering properties are cannot be overemphasized. In fact, available material at a site to serve as a foundation and/or of which the embankment will be composed is probably the single most important factor that affects dike stability. This is because dike design must generally be adapted to the most economically available materials compatible with prevailing foundation conditions. Available disposal sites are normally lands not economically suited for private development, often being composed of soft clays and silts of varying organic content. In fact, many future confined disposal sites will undoubtedly have been used in the past for unconfined disposal, thereby forcing dikes to be constructed on previously deposited dredged material often consisting of soils having very poor engineering qualities.

Since dike construction requiring the use of material from inside the disposal area and/or immediately adjacent borrow areas is often an economic necessity, initial dike heights may be limited or the use of rather large embankment sections may result, expensive foundation treatment may be required, or expensive construction methods may be dictated. In some cases where more desirable borrow is available, its use can result in a lower construction cost if one or more of the above items can be eliminated (i.e., a smaller section, less expensive required foundation treatment, etc.). However, the use of select borrow does not alleviate instability problems to any great degree if the foundation is of poor quality and extends to depths that make simple foundation treatment such as excavation and replacement impracticable. In fact, poor foundation conditions are much more difficult to deal with than poor embankment materials.

In the past, many dike failures have been the direct result of subsurface conditions that were not discovered during design because of inadequate soils investigations. These failures were commonly characterized by embankment slides, excessive settlement, detrimental seepage, and other phenomena. Even though it is recognized that no matter how complete an exploration program may be, there is always a certain degree of uncertainty concerning the exact nature of subsurface conditions at a given site. An adequately designed exploration program can reduce this uncertainty significantly and place it within limits commensurate with sound engineering practices. It is, therefore, imperative that in order to attain an adequate dike design, a reasonably representative concept of the arrangement and physical properties of the foundation and embankment materials must be established. Detailed guidance on the conduct of field and laboratory investigation for retaining dike projects is contained in Reference 2.

EMBANKMENT AND FOUNDATION SHEARING RESISTANCE

Shear failures in retaining dikes are the result of overstressing the embankment and/or foundation materials. Low shear strengths in the dike and/or foundation (often coupled with seepage effects) are the cause of most dike failures. Failures from this cause are often the most catastrophic and damaging of all since they usually occur quickly and can result in the loss of an entire section of dike along with the contained dredged material. The photographs in Figure 1 show a dike failure initiated by inadequate shear strength and the resulting damage to a sewage treatment plant caused by escape of the previously confined dredged material.

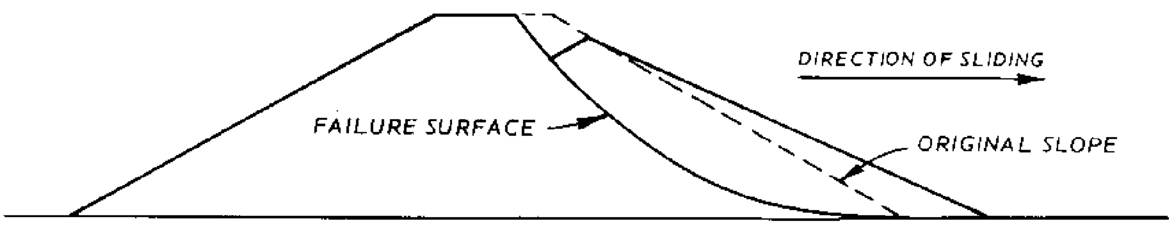
Dike failures from inadequate shear strength have occurred that involve the dike alone and that involve both the dike and the foundation. Failures within a dike alone result when the dike material possesses insufficient shear strength. Failures of this type generally take the form of rotational slides involving the dike slope as shown in Figure 2. However, if a weak plane or layer should exist at the contact between the dike fill and the foundation due to naturally existing weak surface material, inadequate foundation preparation, under-seepage effects, or construction techniques that allow soft material to be placed or trapped in the lower part of the fill, the failure could take the form of a wedge-type configuration characterized by horizontal sliding or translation near the base of the fill (see Figure 3). Rotational type slides as shown in Figure 4 also occur that involve the foundation as well as the embankment. This type of failure generally develops where the foundation is relatively homogeneous with insufficient foundation shear

a. A 150-ft-wide break in the 20-ft-high dike section

b. Flooded sewage treatment plant

Fig. 1. Retaining dike failure resulting in flooding of a nearby sewage treatment plant (Philadelphia District)

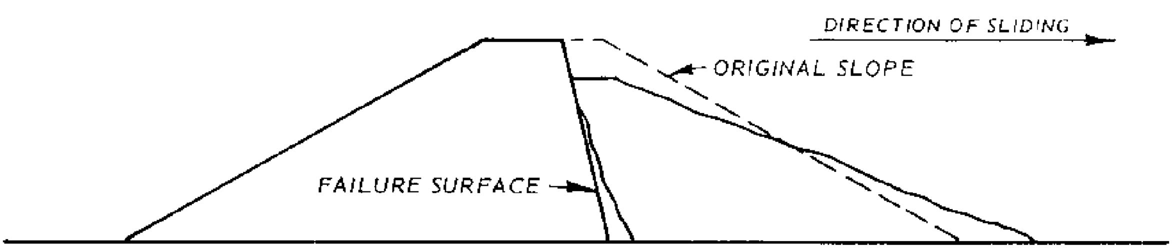
a. PHOTO OF FAILURE



b. CROSS SECTION OF FAILURE

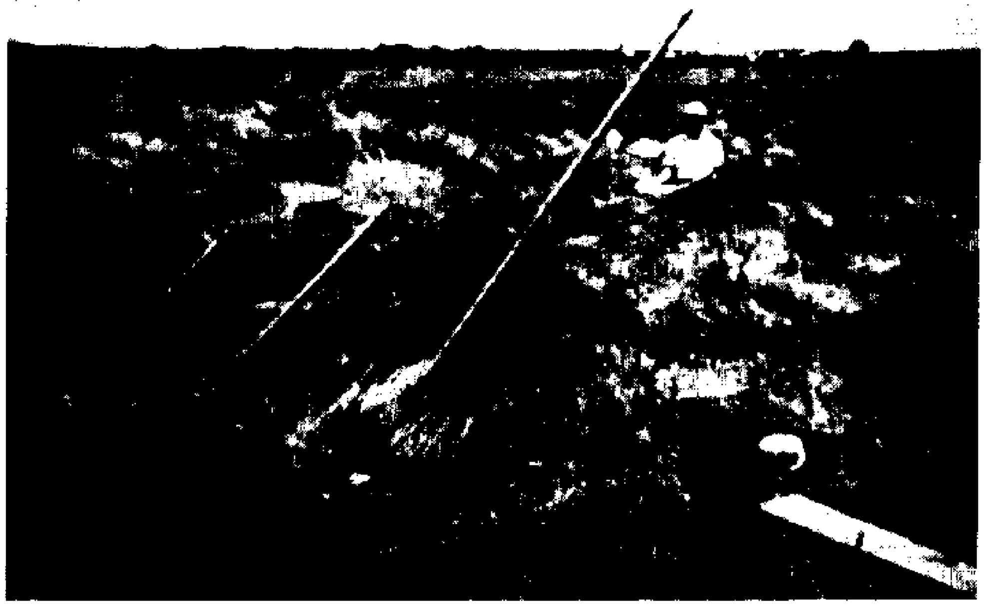
FIG. 2.--Rotational Failure in Dike

a. PHOTO OF FAILURE WHERE SLIDING TOOK PLACE AT EMBANKMENT/FOUNDATION CONTACT

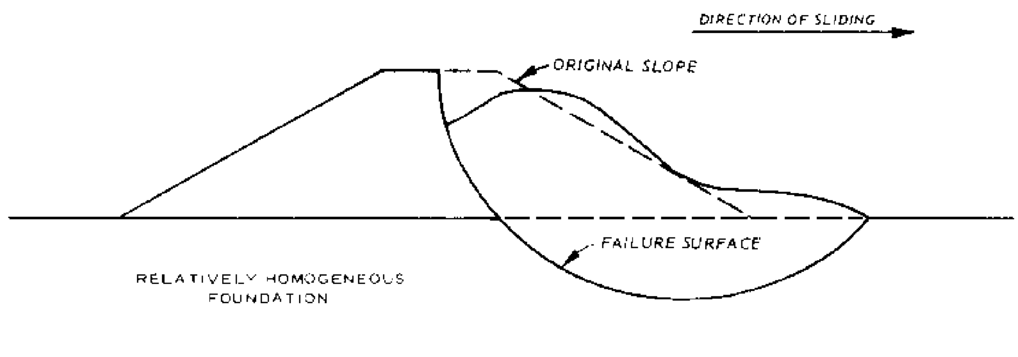


b. CROSS SECTION OF FAILURE

FIG. 3.--Translatory Failure in Dike



a. ROTATION OF MATERIAL BEYOND DIKE TOE



b. CROSS SECTION OF FAILURE

FIG. 4.--Rotational Failure Involving Both Dike and Foundation

strength being the usual cause of failure. A translatory or wedge-type failure can also occur in the foundation where the foundation consists of stratified strata of various soil types (see Figure 5). Horizontal sliding generally occurs in one of the weaker strata in the foundation.

METHODS OF ANALYSIS

The principal methods used to analyze dike embankments for stability with respect to shear failure are conventional limit equilibrium analyses that assume either a sliding surface having the shape of a circular arc or a composite failure surface composed of a long horizontal plane connecting with diagonal plane surfaces up through the embankment and foundation. These analyses simulate the types of shear failures shown in Figures 2 through 5 and are commonly referred to as the circular arc and wedge methods. Various computer programs are available to perform these analyses; therefore, the effort of making such analyses is greatly reduced and primary attention can be devoted to defining shear strengths, unit weights, geometry, and loading conditions. It is recommended that results of all computer analyses yielding minimum factors of safety be checked manually.

There are several methods of limit equilibrium analyses available that utilize a circular arc failure surface. For dike analysis any of these methods are suitable as long as the user is aware of assumptions and limitations involved in the method used. Johnson (3) and Wright (15) summarize and discuss several methods of stability analysis in some detail. Procedures for performing a wedge type of stability analysis can be found in Engineer Manual 1110-2-1902 (7) as well as most soil mechanics textbooks.

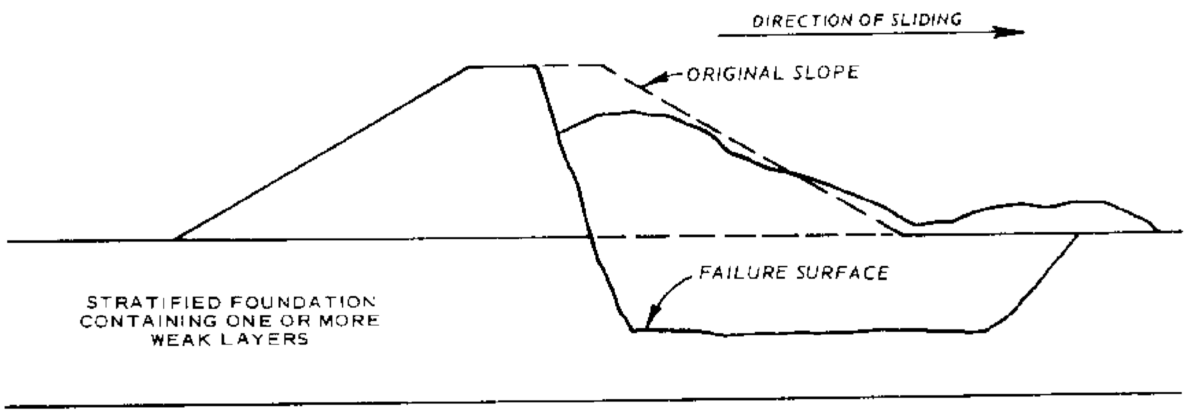


FIG. 5.--Translatory Failure in Dike and Foundation

The infinite slope method of analysis can be applied to dikes composed of cohesionless materials. For slopes without seepage, the factor of safety, FS, with respect to sliding in the cohesionless material is given by:

$$FS = \frac{\tan \phi}{\tan \beta} \tag{1}$$

where

ϕ = angle of internal friction of soil

β = slope angle

For dikes composed of cohesionless material subjected to a condition of steady seepage with the phreatic surface coincident with the outer slope, the factor of safety can be approximated by:

$$FS = \frac{\tan \phi/2}{\tan \beta} \tag{2}$$

Slope stability charts that provide solutions to certain slope stability problems are presented by Taylor (10). Although these solutions are applicable only to simple homogeneous embankments with finite slopes, they may also be used for rough approximations and preliminary solutions to more complex cases.

A quick assessment of the stability of dikes on soft clay without the use of more sophisticated stability analyses can be made by employing bearing capacity equations and an influence chart. Although approximate, this analysis can provide answers suitable for preliminary estimates of embankment heights. The derivation for and examples of use of the bearing capacity equations for preliminary analysis of dikes on soft clays is presented by Hammer and Blackburn (2), but basically the equation:

$$H = \frac{q_d}{\gamma} \tag{3}$$

where

H = dike height

γ = unit weight of embankment material

q_d = ultimate bearing capacity of soil

can be used to approximate maximum dike heights. It must be emphasized that approximations are just that and are subject to considerable error, therefore they should be used only for ballpark estimates and nothing more.

CONDITIONS OF ANALYSIS

There are three primary conditions for which dikes can be analyzed with respect to slope stability: end of construction, steady seepage, and a sudden drawdown. End of construction and steady seepage are the most commonly analyzed conditions with sudden drawdown being applicable to a lesser degree. The conditions for which any dike is analyzed must be those expected to occur under operating conditions, recognizing there may very well be variations from the aforementioned conditions that may be most applicable. In any case, it is imperative that the conditions analyzed be those that most nearly match actual field conditions. In other words, considerable judgment must be exercised in determining the most applicable conditions of loading to which a given dike will be subjected. The following paragraphs contain a discussion of each of the conditions mentioned.

For most dikes constructed on foundations of soft, weak materials or on foundations containing a weak stratum in an otherwise strong foundation, the most critical period involving failure due to inadequate shear strength is at the end of construction. This is because at this time the material is usually in its weakest state, not having had time to consolidate and gain strength under the imposed loading conditions. Consequently, all dikes should be checked for stability during the end of construction condition. Analysis for the end of construction condition is applicable to both interior and exterior slopes. The effects of underseepage and resulting hydrostatic uplift pressure acting in pervious foundation strata must be considered in this analysis.

A condition of steady seepage through the dike resulting from the maximum anticipated storage level in the containment area may be critical for stability of exterior dike slopes. A sketch depicting this condition is shown in Figure 6. All dikes should be analyzed for this condition if it is anticipated that saturation of the embankment will occur and a condition of steady seepage will develop within the dike and/or foundation. This condition is especially applicable to dikes composed of semipervious and pervious materials but should also be considered for dikes composed of any material. This is because it is very important that the dike be stable against failure resulting from steady seepage conditions since failure from this cause generally occurs

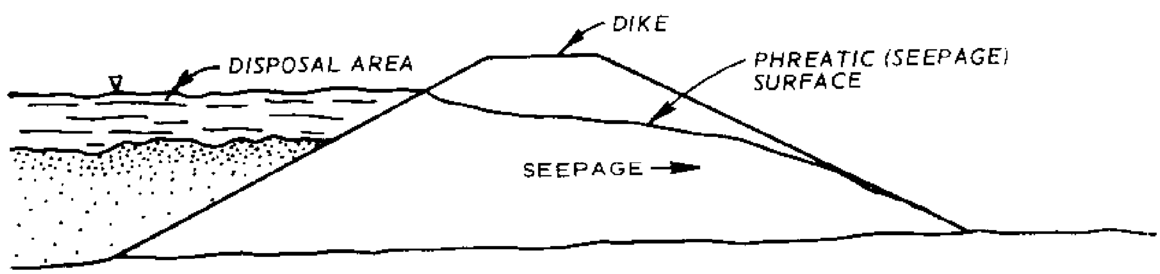


FIG. 6.--Dike Subjected to Steady-State Seepage Condition

with a considerable depth of dredged material in the disposal area and could therefore result in substantial damage due to loss of a high volume of dredged material.

Exterior dike slopes may become saturated during high water levels from adjacent streams or from high tides. If the water level then falls faster than the material can drain, excess pore water pressures and unbalanced seepage forces result. This phenomenon is termed sudden drawdown. In performing an analysis for the sudden drawdown condition, it is generally assumed that the water level drops instantaneously so that no pore pressure dissipation occurs. This condition is applicable to those dikes situated near large bodies of water or streams whose level may reach near the dike crest, remain there long enough to saturate the dike, and then fall fairly rapidly. It may also be applicable to dikes subject to the effects of substantial tidal fluctuations (Figure 7). Failure from sudden drawdown will usually be in the form of relatively shallow sloughing of the affected slope and thus is not considered as critical as failure from the end of construction or steady seepage conditions where an entire dike section may be lost. Loss of slope protection and a weakening of the dike are the usual consequences of failure from sudden drawdown. There are no recorded dike failures from sudden drawdown, but large dikes, especially those with substantial slope protection, subjected to the conditions previously described, should be analyzed for the effects of sudden drawdown.

SELECTION OF SHEAR STRENGTHS

The selection of proper soil shear strength parameters for input into stability analyses is every bit as important as the method of analysis itself. In the past, soil strengths for dike design have

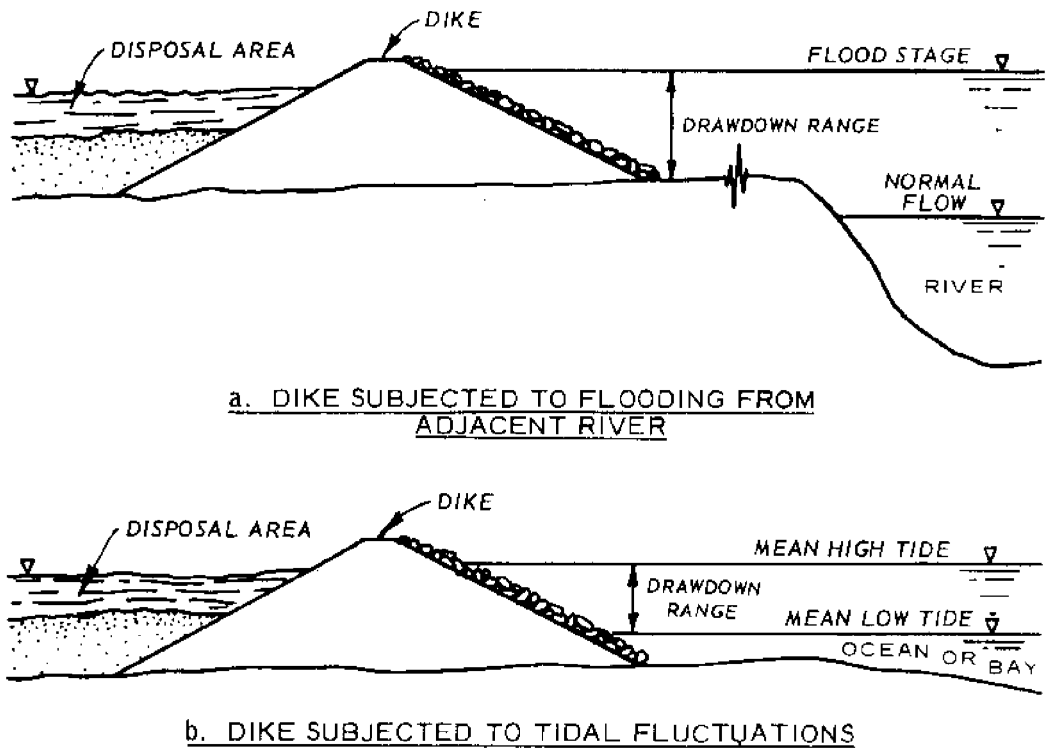


FIG. 7.--Situations Conducive to a Sudden Drawdown Condition

largely been assumed. However, as the need for more sophisticated analyses and design increases, it is imperative that shear strengths be determined from reliable laboratory and field test data. This by no means rules out the use of experience. Experience with respect to shear strengths should continue to play a vital role in dike design, but as a supplementary rather than a primary means of shear strength determination. Type strengths (i.e., unconsolidated-undrained, consolidated-undrained, and consolidated-drained) applicable for each condition of analysis previously discussed are given in Table 1. A comprehensive treatment of soil shear strength determination can be found in Reference 2.

RECOMMENDED MINIMUM FACTORS OF SAFETY

Recommended minimum factors of safety for slope stability analyses of retaining dikes designed by the CE are given in Table 1. These values are to be used where reliable subsurface data from exploration and testing are available for input into the stability analysis. The factors of safety given in Table 1 are applicable to dikes less than 30 ft in height where the consequences of failure are not extremely severe. For dikes greater than 30 ft in height and where the consequences of failure are severe, the criteria given in Table 1 of Engineer Manual 1110-2-1902 (7) should be used.

Table 1

Applicable Shear Strengths and Recommended

Minimum Factors of Safety*

Condition	Shear Strength: Impervious Soils**	Shear Strength: Free-Draining Soils	Slope Analyzed	Minimum Factor of Safety†: Main Dikes	Minimum Factor of Safety†: Appurtenant Dikes
End of construction	UU	CD	Exterior and interior	1.3‡	1.3
Steady seepage	UU, CU††	CD	Exterior	1.3	1.2
Sudden drawdown	UU, CU††	CD	Exterior	1.0	NA

* Criteria not applicable to dikes greater than 30 ft in height or where the consequences of failure are very severe. For such dikes use criteria given in Table 1 of Engineer Manual 1110-2-1902 (7).

** For low-plasticity silt where consolidation is expected to occur rather quickly, the CU strength may be used in lieu of the UU strength.

† To be applied where reliable subsurface data from exploration and testing are available; where assumed values are used, recommended minimum factors of safety should be increased by a minimum of 0.1.

†† Use UU strength where it is anticipated loading condition will occur prior to any significant consolidation taking place; otherwise use CU strength.

‡ Use 1.5 where considerable lateral deformation of foundation is expected to occur (usually where foundations consist of soft, high-plasticity clay).

IMPROVING FOUNDATION STABILITY

The condition of a dike foundation can be and often is the decisive factor in determining the feasibility of constructing a retaining dike. Since suitable areas for disposal of dredged material are usually limited, retaining dikes must be so aligned as to make optimum use of the disposal area, often without regard to foundation conditions. Thus, dike foundations must sometimes be improved in order that the dike may be built. Economically feasible methods of improving dike foundations are limited, but it should be recognized that the economic justification of a given method is not an absolute value but is directly related to the particular project.

The most positive method of dealing with excessively weak and/or compressible foundation soils is to remove them and backfill the excavation with more suitable material. This procedure is usually feasible only where deposits of unsuitable material are not excessively deep (i.e., up to about 20 ft in thickness), where suitable backfill material is available, and where a firm base exists upon which to found the backfill. The excavation and replacement can be accomplished by any practical means, but for most dikes in areas of high water tables (i.e., marshes, tidal flats, etc.) excavation is best accomplished with dredges, matted draglines, and barge-mounted draglines. Where backfilling is to be accomplished in the wet, only coarse-grained material should be considered for use as backfill. The amount of excavation need not always be under the entire section or to full depth of soft material, but can be partial if determined by stability analyses to be appropriate. Some sections successfully used in the past to prevent horizontal sliding of the embankment are shown in Figure 8.

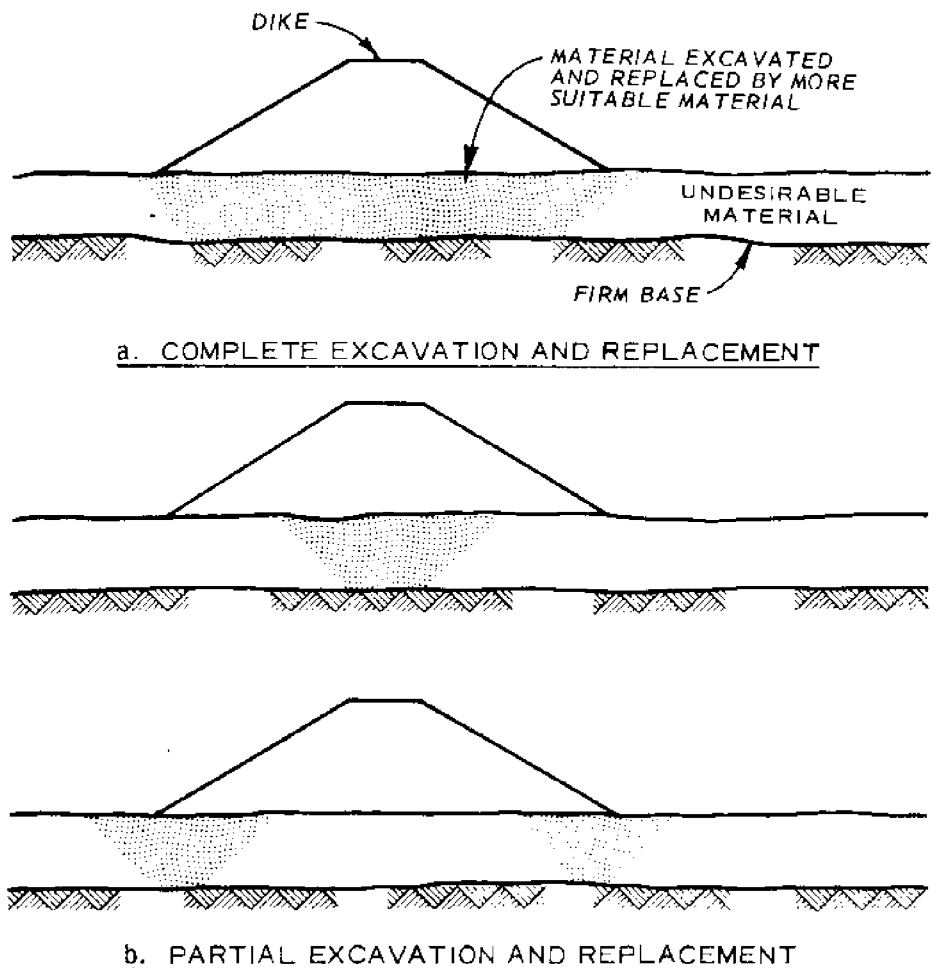


FIG. 8.--Typical Use of Excavation and Replacement Method to Improve Dike Stability

Dikes must frequently be built over areas consisting of very soft materials. Although the depths of these deposits may not be great, the cost of their removal may not be justified, and a dike having adequate stability can be constructed by end-dumping fill and utilizing its weight to displace the undesirable material. It is desirable to use this method where a firm bottom exists at a reasonably shallow depth; it has, however, been successfully employed in areas where no definite firm bottom existed, but the foundation material merely increased in strength with depth, in which case the depth of displacement is considered to be that necessary to stabilize the embankment at the desired height (Figure 9). However, use of the displacement method in the latter case does increase the likelihood of post-construction settlement. Due to the construction techniques required to successfully use this method, it is highly desirable to place fill by end-dumping methods rather than by hydraulic means. It is also desirable that the material to be displaced exhibit some sensitivity and have average in situ shear strength of less than about 150 to 200 psf. The greater the sensitivity of the material and the lower its in situ strength, the easier it is to displace.

Basically, the displacement technique consists of advancing the fill along the desired alignment by end-dumping and pushing fill over onto the soft material with dozers, thus continually building up the fill until its weight displaces the foundation soils to the sides and in front of the fill. By continuing this operation, the dike can finally be brought to grade. Since this method involves the encouragement of foundation displacement, the section should be as steep sloped as possible and built as high as possible as it advances across the foundation. The

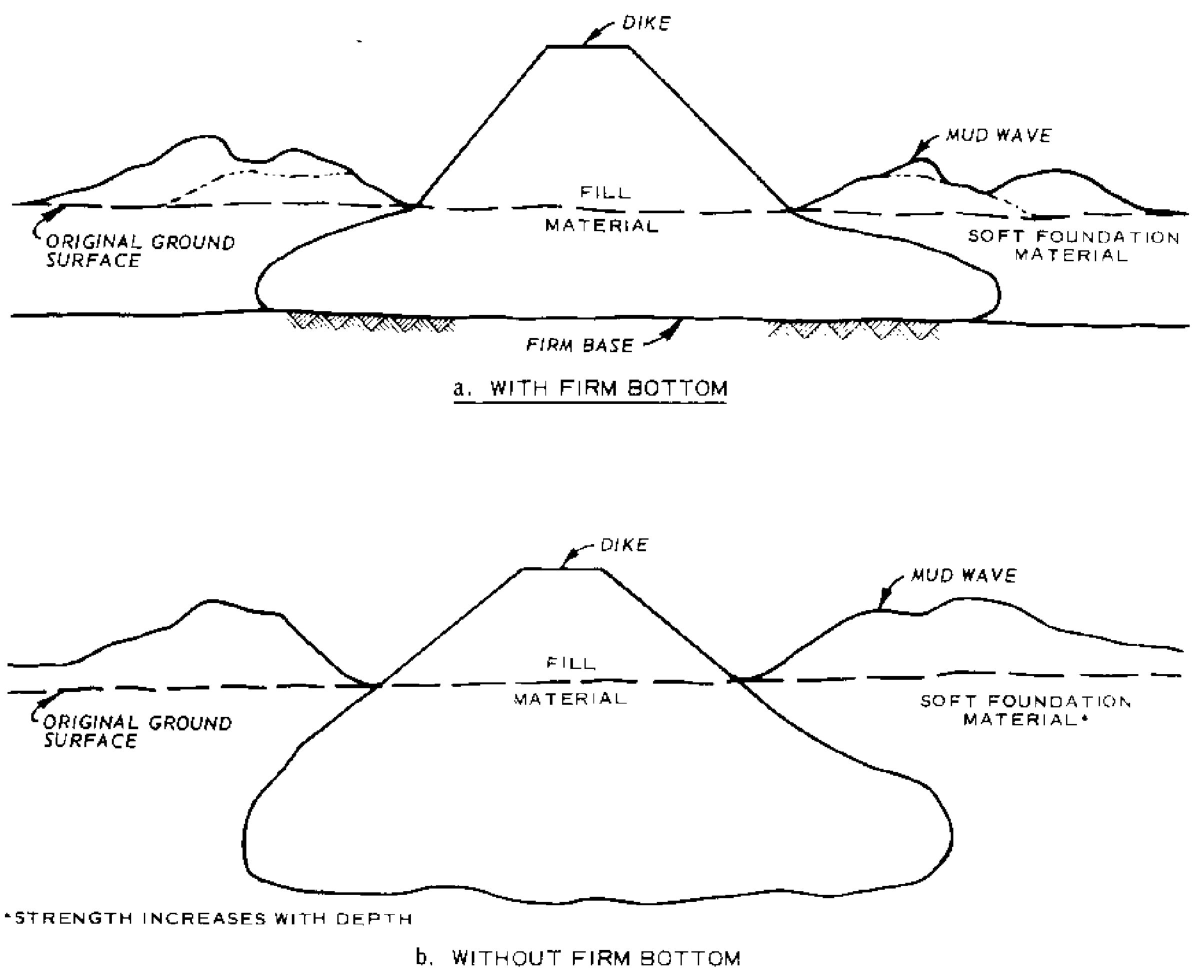
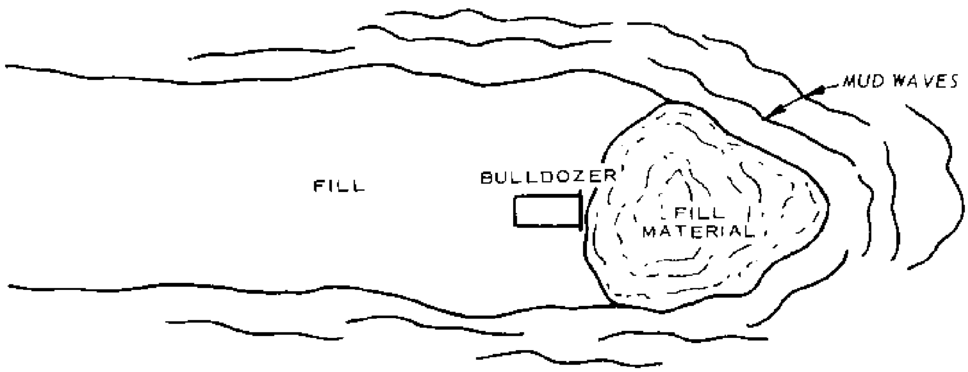


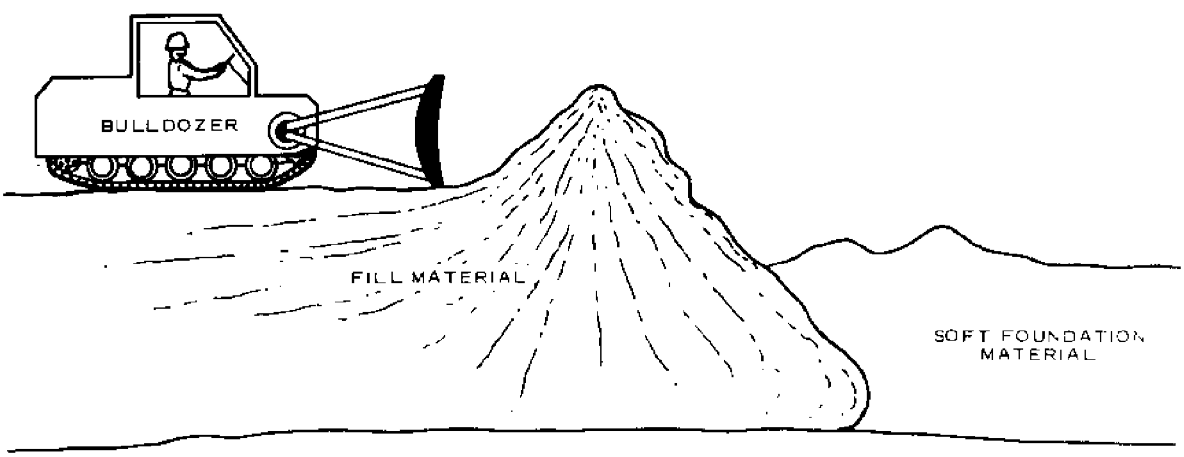
FIG. 9.--Final Dike Sections After Displacement of Soft Foundation Material

fill should be advanced with a V-shaped leading edge so that the center of the fill is always the most advanced, thereby displacing the soft material to both sides (Figure 10). This will greatly lessen the chances of trapping soft material beneath the fill. A wave of displaced material will develop along the sides of the fill. These mud waves have been known to be as high as the top of the fill; however, they should not be removed.

Improving dike stability by stage construction refers to the building of an embankment in increments or stages of time. This method is used when the strength of the foundation material is inadequate to support the entire dike if built at one time. Using stage construction, the dike is built to intermediate grades and allowed to rest for a time before placing more fill. Such rest periods permit dissipation of pore water pressures and consolidation that result in a gain in strength so that higher dikes can be supported. Obviously, this method is most appropriate for foundations that consolidate rather rapidly and works best for clay deposits interspersed with continuous seams of pervious silt or sand. However, lack of speed of consolidation may not be a drawback if the filling rate of the disposal area is slow enough to allow considerable time between construction of the various dike stages. In fact, stage construction appears to be a promising method of constructing retaining dikes as the intervals of construction can, in many cases, coincide with the filling of the disposal area; i.e., full dike height may not be needed until many years after initial construction. In using stage construction, estimates of strength gain with time should be made as described in Reference 2. Also, it is highly desirable to



a. PLAN



b. PROFILE

FIG. 10--Advancement of Fill Using End-dumping and Displacement Technique

install piezometers to monitor the dissipation of pore water pressures. Disadvantages of this method include the need for separate construction contracts and uncertainties with respect to the gain in strength with time.

In seismically active areas, the possibility of liquefaction of loose sand deposits in dike foundations may have to be considered. Since methods for densifying sands such as vibroflotation, blasting, etc., are costly, they are generally not considered except for dikes where the consequences of failure are very severe or at locations of important structures in the diking system. However, less costly defensive design features may be provided, such as additional freeboard, wider dike crest, and flatter slopes.

Flattening embankment slopes will usually increase the stability of an embankment against a shallow foundation failure or a failure that takes place entirely within the embankment. Flattening embankment slopes reduces unbalanced gravity forces that tend to cause failure and increases the length of potential failure surfaces, thus increasing resistance to sliding.

Stability berms provide essentially the same effect as flattening embankment slopes but are generally more effective since they concentrate additional weight where it is needed most and force a substantial increase in the potential failure surface. Thus, berms can be an effective means of stabilization, not only for preventing shallow foundation and embankment failures, but for preventing deep-seated foundation failures as well. Berm thickness and width should be determined from stability analyses and the length should be great enough to encompass the entire problem area, the extent of which is determined from the soil profile. Foundation failures are normally preceded by lateral displacement of material beneath the embankment toe and by noticeable heave of material just beyond the toe. When such a condition is noticed, berms are often used as an emergency measure to stabilize the dike and prevent further movement. The main disadvantages of berms are the increase in area occupied by the embankment and the amount of material required for their construction.

STABILIZATION JUST PRIOR TO AND AFTER FAILURE

With the use of proper observational techniques, impending stability failures may be detected and measures taken to improve the stability of the section prior to failure. Lateral movement of slopes, slight sinking of the crest, or heave near the toe, as well as development of tension cracks, can give advance warning of failure. Since most failures begin slowly, early detection and immediate corrective action can often prevent complete failure. Flattening dike slopes and adding berms have often been effective as stop-gap measures for increasing stability.

Once failure has occurred in a soft clay foundation, the process of rebuilding is often more difficult than initial construction because many soft clays are sensitive and their remolded strengths are often much less than their initial shear strengths. It is good practice after a failure to allow time for some consolidation and resulting gain in shear strength before attempting to rebuild. This will give the remolded clay time at least to partially overcome the effects of strength reduction due to remolding. When remedial construction is started, care should be taken not to load the foundation too quickly. Reconstruction should be done as slowly as possible with the entire area brought up together rather than building to full height in sections.

SEEPAGE

Uncontrolled seepage will occur through earth dikes and foundations consisting of pervious or semipervious material unless prevented by positive means such as impervious linings, blankets, or cutoffs. Seepage effects can create instability through internal erosion (piping) of dike or foundation materials or may lead to a shear failure by causing a reduction in the available shear strength of the dike and/or foundation through increased pore pressure or by the introduction of seepage forces. A dike failure caused by uncontrolled seepage is shown in Figure 11. The conditions given in the following paragraphs may create or contribute to seepage problems in retention dikes.

Dikes with steep slopes composed of coarse-grained pervious materials or fine-grained silt. In this case the seepage line through the embankment may exit on the outer slope above the dike toe resulting in raveling of the slope. If the dike contains alternating layers of pervious and impervious materials, the seepage surface may even approach a horizontal line at the ponding surface elevation, thus creating an even more severe stability problem (Figure 12).

Dikes built on pervious foundation materials or where pervious materials are near the surface or exposed as a result of nearby excavation (Figure 13). This is a common condition where dikes are constructed by dragline using an adjacent borrow ditch. In this case surface or near-surface peat and other fibrous materials are included as pervious foundation materials.

a. Washout at sluice structure

b. Debris on tidal flats downstream of failed sluice structure

FIG. 11--Dike Failure Caused By Uncontrolled Seepage

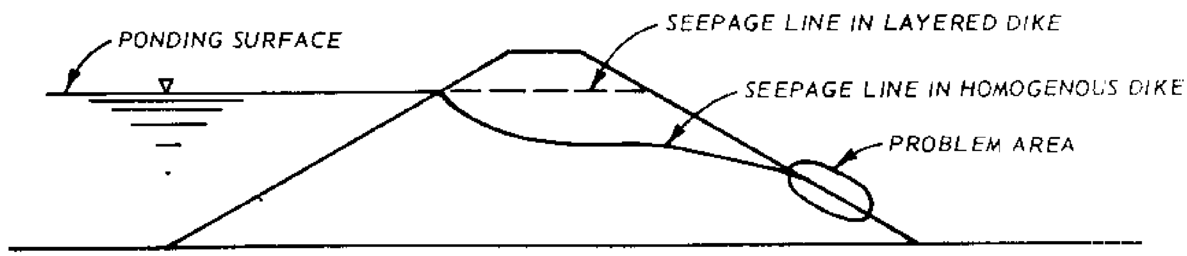


FIG. 12.--Seepage Lines Through Dike

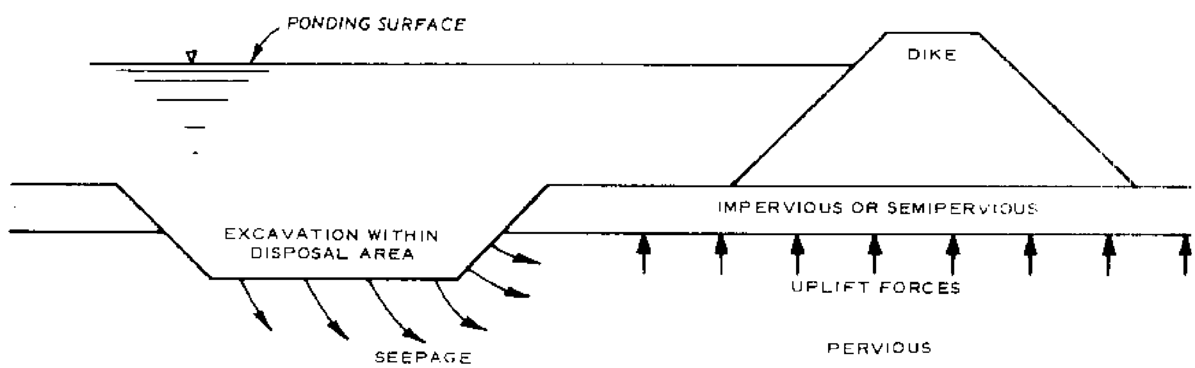


FIG. 13.--Seepage Entrance Through Area Excavated Within Disposal Area

This condition may lead to the development of large uplift pressures beneath and at the outer toe of the dike, causing overall instability from inadequate shear strength or may result in piping near the embankment base.

Dikes constructed by casting methods with little or no compaction. This method of construction may leave voids within the dike through which water can freely flow, resulting in piping of dike material.

The existence of seepage paths along the plane between the foundation and the dike. This can occur when the dike base and foundation surface are not properly bonded together. Seepage occurring at this point can result in piping of the embankment material along the base of the dike or the development of high uplift pressures, either of which can eventually cause failure of the embankment.

The existence of seepage paths along the contact between structures within the dike and the dike. This condition can be caused by inadequate compaction of dike materials against structures, shrinkage of material adjacent to structures, or differential settlement. As in previous cases, piping of the dike material usually results and normally leads to breaching of the dike.

METHODS OF ANALYSIS

Seepage analyses for dikes will primarily consist of determination of the position of the seepage line (or phreatic surface) within the dike itself, determination of uplift pressures resulting from foundation underseepage, and, to a lesser degree, determination of the quantity of flow. Several mathematical and graphical methods are available for these determinations. Reference 14 and Cedegren (1) contain guidance in the analysis of seepage problems and their control. A graphical solution for estimating the position of the seepage surface developed by L. Casagrande is given on p. 184 of Reference 10. A chart for estimating the time required for the development of the seepage line of an embankment is given by Cedegren (1, p. 253). Once the position of the seepage line is determined, it should be compared with the location of the outer slope line to determine if measures are needed to avoid the emergence of seepage on the outer slope. Uplift pressures should be applied in the stability analyses and either the design made to take such pressures into account or steps taken to reduce the uplift pressures to acceptable values. Flow quantities are needed to design and size exterior ditches to handle the water. This is often required where the dike or parts of the dike are designed as filtration devices for the dredged material. The references previously given also contain guidance on the design of filters to avoid piping. The phenomenon of piping cannot be analyzed theoretically, but conditions conducive to it, such as high gradients, can be determined by theoretical means.

SEEPAGE CONTROL

Seepage through retention dikes constructed of pervious or semipervious materials may be controlled by placement of an impervious barrier on the interior dike slope to restrict flow. This barrier may consist of a layer of impervious soil or polyethylene sheeting. Impervious soil barriers should be a minimum of 3 ft in thickness and thoroughly comapcted. Sheeting placed for this purpose should have a minimum overlap of 2 ft at joints, and provisions should be made to ensure that the joints are sealed. Recent developments in the area of chemical spray-on plastics have also shown possibilities in the control of through-seepage. Experience in the Philadelphia District, CE, has shown that for pervious dikes in low hazard areas, a policy of compaction of the dike material plus increasing the section width by slope flattening or by increasing the top width has proven adequate against failure, although through-seepage in the dike does develop. Seepage problems resulting from the presence of voids in dikes constructed by casting can best be controlled by requiring the dikes to be compacted to some degree in order to eliminate open voids. Adequate compaction for this purpose can usually be attained by extra tracking by the dozer during shaping. In performing this operation, it is necessary that the dike be cast up in lifts rather than built to grade as the section advances across the foundation.

Where pervious foundation materials are encountered, the seepage path can be blocked by constructing an impervious cutoff through the pervious materials, the dike section can be increased in weight to counteract the seepage pressures, or the dike section may be increased in length in order to reduce exit gradients to within tolerable limits. Cutoffs are feasible only for relatively shallow and thin pervious deposits as they should fully cut off the pervious stratum. Partial cutoffs have been shown to be relatively ineffective. If a cutoff is considered to reduce seepage through a surface root mat or peat deposit, its effect on the overall stability of the section should be considered. In many cases these surface deposits have been shown to be beneficial from a slope stability standpoint, but they must be fairly continuous in order to be of benefit. It is therefore recommended that if such a cutoff is considered, it should be placed at or near the interior dike toe rather than under the dike center line.

To prevent piping of foundation materials, it is recommended that the exit gradient have a safety factor of at least 1.5 when compared with the critical exit gradient of the material through which flow is occurring. A factor of safety of about 1.5 based on net uplift forces is also recommended for failure due to uplift of semipervious or impervious top strata (Figure 13). Larger safety factors may be required where the consequences of dike failure are great. The seepage path may be lengthened by berms, impervious blankets, and/or flattening of exterior dike slopes.

Seepage problems at the contact between a sluice and the dike may be avoided by ensuring that adequate compaction of the dike material is obtained at the contact. Also, it is desirable to use material on the wet side of optimum to increase its plasticity, thereby increasing its resistance to cracking and the formation of seepage paths. It is also desirable to install impervious seepage fins extending from the structure into the dike. An additional degree of security may be obtained by increasing the dike cross section at these locations. Prevention of seepage at the dike-structure contact is further discussed in Reference 2.

Proper clearing and preparing of the dike foundation to receive the newly constructed dike can prevent problems caused by seepage paths between the ground surface and dike. In areas with very soft foundations where marsh grass and root mats are to be left in place for stability, measures previously discussed should be taken to reduce or block seepage through this material. Also where these materials are to be left in place, if the dike crosses a hard spot such as an old dike or road, the hard spot should be completely denuded of all vegetative growth. The Mobile District, CE, reported a failure in a retaining dike because this material was not stripped where the new dike crossed an old dike resulting in seepage and piping of the dike material in this area.

SETTLEMENT

Settlement of dikes can result from consolidation of embankment and/or foundation materials, shrinkage of embankment materials, or lateral deformation of foundation materials. Like uncontrolled seepage, settlement of dikes can result in failure of the dike, but more likely will serve to precipitate failure by another mode such as seepage or shear failure. Distress from settlement usually takes some time to develop as consolidation, shrinkage, and lateral deformation are time-dependent, directly related to the soil permeability and loading. Some lateral deformation can occur quickly, however, such as during construction (particularly in relation to the displacement method of construction). Settlement problems in dikes are almost always related to fine-grained soil because settlement of coarse-grained permeable soil is generally much less, occurs relatively quickly, and is compensated for during construction. Specific forms of settlement that commonly cause problems with dikes include: (a) excessive uniform settlement, (b) differential settlement, (c) shrinkage of uncompacted embankment materials, and (d) settlement resulting from lateral deformation (sometimes referred to as creep) of soft foundation soils. Excessive uniform settlement can cause a loss in containment area capacity due to loss of dike height (Figure 14). Differential settlement can result in cracking of the

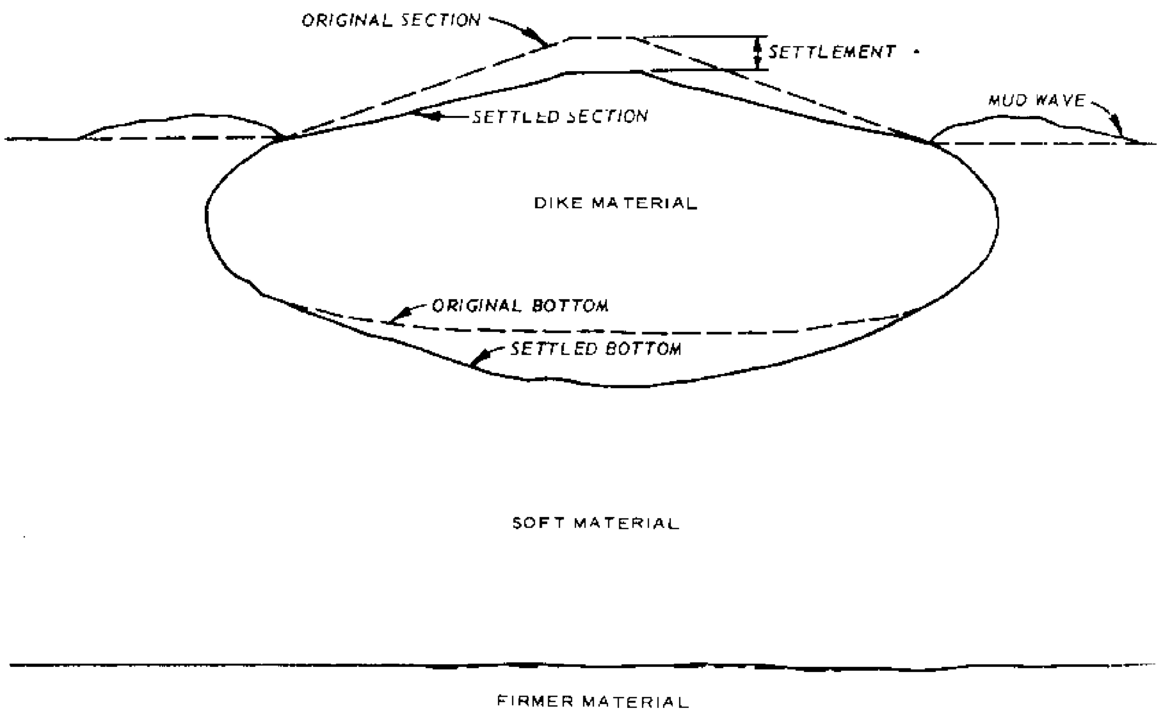


FIG. 14.--Example of Excessive Uniform Settlement

dike, which can lead to a shear or piping failure. This is an especially acute problem at junctions between dikes and structures in dikes. Differential settlement is caused by the foundation being subjected to varying loads over a relatively short distance (as in the case of a structure within a dike), or by a foundation consisting of materials of different compressibility, usually of varying thicknesses (as in the case of a foundation containing an old slough filled with soft compressible material or noncompressible material). Examples of differential settlement resulting from these different causes are shown in Figures 15 and 16. Both excessive uniform and differential settlement can cause distortion and/or rupture of weir discharge pipes located under or through dikes and can cause distortion of the weir box itself. Embankment shrinkage in dikes built with fine-grained cohesive material by hydraulic or cast methods can result in volume reductions as high as 30 percent (11). Shrinkage of loosely placed cohesive materials is differentiated from consolidation in that it occurs from evaporation of water in the soil rather than a squeezing out of water, as occurs with consolidation, although both result in a loss of volume.

SETTLEMENT ANALYSES

Where estimates of amount of time and total settlement are needed, a conventional analysis such as that contained in Engineer Manual 1110-2-1904 (6) or in various textbooks on soil mechanics is recommended. NAVFAC DM-7 (5) is also recommended for guidance in performing settlement analyses. In order for an estimate of settlement by theoretical means to be valid, the materials analyzed must be fairly uniform and capable of being represented by a laboratory consolidation test, and the drainage conditions must be well defined. Unfortunately, the above conditions are often not satisfied with respect to dike materials or dredged material. However, theoretical analyses are still applicable, even though in many cases they are somewhat inexact.

UNIFORM SETTLEMENT

For most earth structures on compressible foundations, uniform settlement resulting from consolidation of the foundation can cause a loss of design grade and must be compensated for in the initial design. However, for retaining structures a unique situation exists with respect to the effects of uniform dike settlement: the containment area will also be loaded and should also undergo settlement that may compensate for the dike settlement, resulting in little or no loss in capacity of the retaining area. For dikes on compressible foundations, this fact should be verified, however. This can be done by performing settlement analyses for both the dike foundation and the containment area (using projected filling rates) and comparing the amount and rate of settlement of each. If such an analysis shows a net loss of dike height (as is often the case when a considerable period of time elapses between the time of dike construction and filling of the disposal area), it should be compensated for by overbuilding the dike or by making provisions to raise the dike back to the original design grade at a later date (i.e., use stage construction).

Overbuilding dikes by the amount of anticipated loss of grade due to settlement often appears the easiest and cheapest solution to the problem, but is really not practical in many cases, as it can significantly affect stability of the dike against shear failure (i.e., can require higher dike sections), as well as cause additional settlement. This is not to say that use of overbuilding to compensate for anticipated settlement should be ruled out, but it should be closely studied before being specified as a compensating procedure.

The use of stage construction (i.e., raising dikes as necessary after settlements occur) is somewhat more troublesome and expensive than overbuilding, but is often the only practical solution, especially for dikes on highly compressible foundations where overbuilding can create more problems than it solves, as previously discussed. The use of stage construction to compensate for dike settlements has often been successful in the past on many dike projects.

EMBANKMENT CONSOLIDATION AND SHRINKAGE

Consolidation and shrinkage of embankment materials will vary considerably, being dependent not only on material type but on method of placement. Generally, methods for theoretical settlement analyses of embankment materials are only applicable to dikes composed of compacted uniform materials. (These materials will usually exhibit the least amount of consolidation and shrinkage.) The amount of embankment consolidation and shrinkage usually must be estimated.

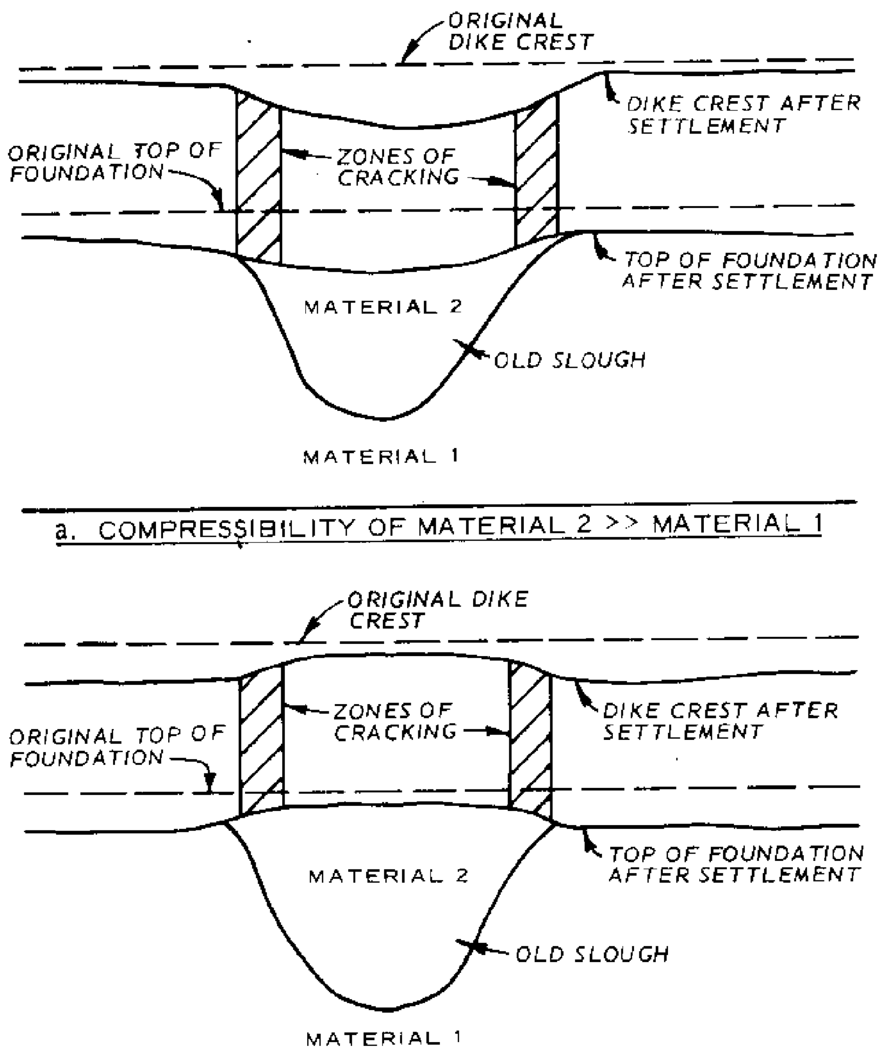
As a general rule, dikes built of semicompacted fill will experience a reduction in volume on the order of 10 to 15 percent. Usually, this small amount of volume decrease can be compensated for by overbuilding.

Estimating the reduction in volume of uncompacted fill (i.e., fill placed by casting) is a difficult task, as it will depend greatly upon the consistency and water content of the material being placed and the construction procedures used (i.e., the amount of equipment coverage during shaping, etc.). Estimates of reduction in volume of uncompacted fill should generally be based on knowledge of the previously mentioned factors and experience with fills built of similar materials and by similar construction procedures. In the absence of any supporting data, a reduction in volume of 15 to 20 percent should be applied for uncompacted fill.

The compressibility of hydraulic fill containing stiff cohesive soil results primarily from deformation of the clay lumps, while the rate of consolidation is determined by the characteristics of the matrix surrounding the clay lumps. Hydraulic fills containing soft cohesive soil are highly compressible, but again, the rate of consolidation is dependent on the matrix material. Consolidation of cohesive materials with a sandy matrix may be essentially complete within a few weeks, while consolidation of cohesive materials with a clay matrix may continue for years.

DIFFERENTIAL SETTLEMENT

Where the possibility of differential settlement (as shown in Figures 15 and 16) exists, an analysis should be made to determine the total differential settlement across the area under concern. Although there are no specific criteria that set forth how much differential settlement a particular soil can withstand before cracking, measures can be taken to reduce the magnitude of the differential settlement so that



b. COMPRESSIBILITY OF MATERIAL 2 << MATERIAL 1

FIG. 15.--Differential Settlement From Foundation Containing Materials of Different Compressibility

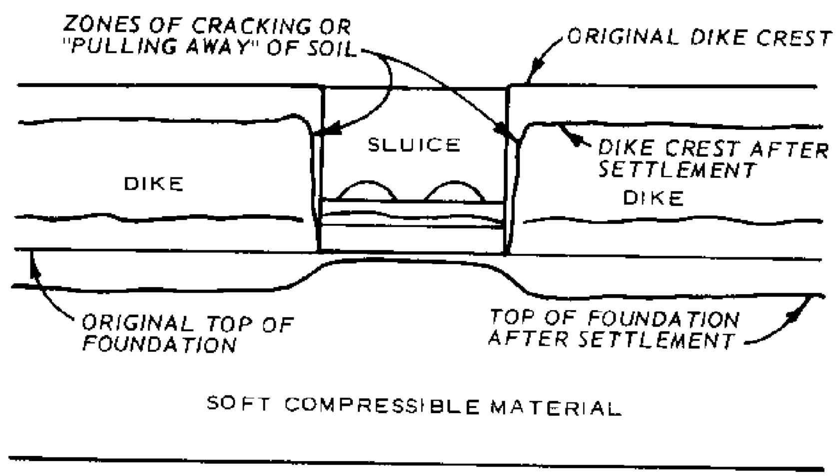


FIG. 16.--Cracking at Dike-Structure Junction Caused By Differential Settlement Due to Dike Load on Foundation Being Greater Than Sluice Load

the chances of distortion and cracking are lessened. These measures include (a) removing all or part of the compressible material and replacing with more suitable material; (b) using flatter excavation slopes (1V on 4H minimum) where excavations (usually for structures) are involved; and (c) specifying good compaction procedures and more plastic embankment materials adjacent to structures.

LATERAL MOVEMENT

In some cases where extremely poor foundation conditions are encountered, settlement due to lateral movement of foundation materials may also warrant consideration. Experience with instrumented test sections in the Atchafalaya Basin, Louisiana, in the New Orleans District, CE, has shown that more than 30 percent of observed settlement induced by the addition of an 11-ft height of fill was due to lateral movement of foundation materials. This was observed in an area where the foundation consisted of peat and soft organic clay with very high water contents underlain by soft and medium clays of high plasticity and where the sections were constructed with safety factors of about 1.3 against shear failure. Other sections constructed with safety factors of about 1.1 indicated as much as 50 percent of observed settlement was due to lateral movement of foundation materials. Experience from the Atchafalaya Basin Floodway has shown that overbuilding should not be considered as a solution for lateral spreading as the additional load from overbuilding will generally tend to aggravate the problem rather than help solve it. This same experience has also shown that vertical settlement due to lateral movement will be minimized by designing a section with a higher minimum factor of safety with respect to shear failure (on the order of 1.5).

SURFACE EROSION

Retaining dike failures can be initiated by the effects of wind, rain, waves, and currents that can cause deterioration of interior and exterior dike slopes. The exterior slopes of dikes subject to constant or intermittent wave and/or current action of tidal or flood waters are generally exposed to the most severe erosion. However, interior dike slopes may also be subjected to this type of erosion, particularly in large confinement areas during periods of high discharges from disposal operations. Dikes adjacent to navigable rivers and harbors are also subject to erosion from wake waves of passing vessels.

Erosion of dike slopes due to the effects of wind, rain, and ice is a continuing process. While these forces are not as immediately damaging as wave and current action, they can gradually cause extensive damage to the dike section, particularly dikes composed of coarse-grained cohesionless materials.

Normal disposal operations can cause erosion of interior dike slopes from pipeline discharge and to exterior slopes at outlet structures. Improper and/or poorly supervised operations of this type can cause dike failure. The pipeline discharge of dredged material is a powerful eroding agent, particularly if the flow is not dispersed. When straight discharge is employed, a depression as shown in Figure 17 is formed at the point of impact, which, as it enlarges, can undermine the pipe foundation and, if too close to the dike, deteriorate the section. Discharge from weir and spillway outlets can damage exterior dike slopes if the discharge is located too close to the dike (Figure 18). Likewise, location of weir inlets too close to the dike can cause erosion of interior dike slopes. Also, disposal areas are occasionally negligently

FIG. 17.--Depression at Discharge Point Formed By Impact of Pumped Material

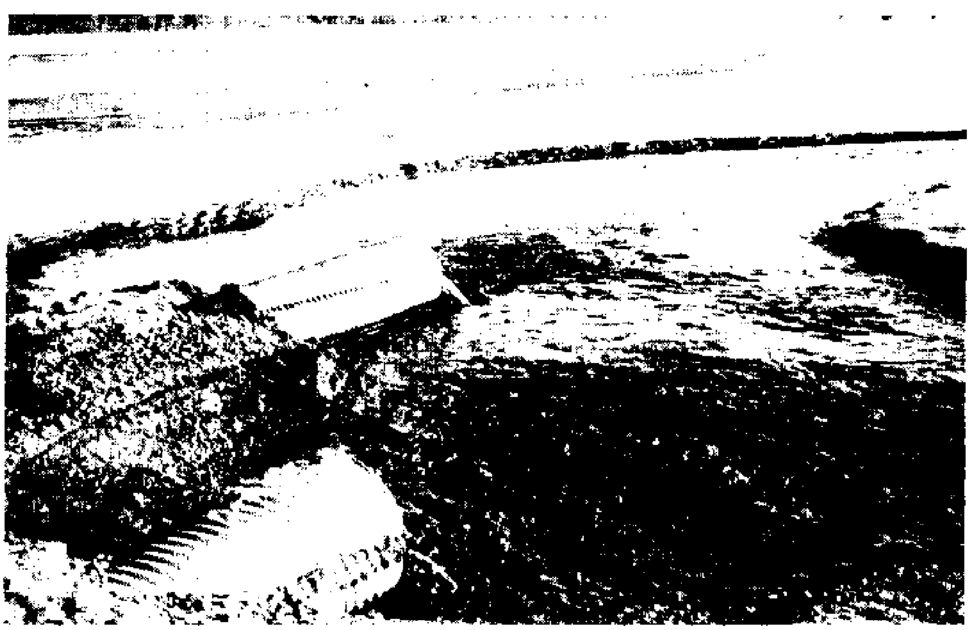


FIG. 18.--Erosion From Outfall Discharge (Note Loss of One Section of Pipe

overfilled to the point of overtopping the dike. When this occurs, severe damage to the dike can result from erosion of the crest and exterior slopes. Figure 19 shows damage to dike crest caused by overtopping.

Almost all dikes will require some sort of protection against failure due to erosion of their exterior slopes and possibly their interior slopes. For dikes where the consequences of failure would be so severe as to be intolerable, slope protection must be designed to prevent failure under the worst foreseeable conditions. Where failures can be tolerated, the expense and degree of protection must be weighed against the expense and frequency of repairing failures. Generally, it will be more desirable to provide adequate protection rather than suffer the economic and environmental damages of failure.

There are many methods of slope protection. These methods vary from minimal, such as grassing to prevent damage from weathering, to substantial, such as massive stone or concrete revetments to prevent damage from storm waves such as that shown in Figure 20. Since the conditions affecting design of retaining dikes are widely varied, the design of slope protection for each structure must be considered on an individual basis. The following paragraphs discuss some of the methods commonly used for slope protection.

FLAT BEACHES

Where material quantities and real estate are available, a gently sloping beach, as shown in Figure 21, may be used to protect the dike against wave action. Gently sloping beaches are effective since wave

FIG. 19.--Damage to Dike Caused By Overtopping

FIG. 20.--Displacement and Loss of Stone Protection on Dike Due to Overtopping

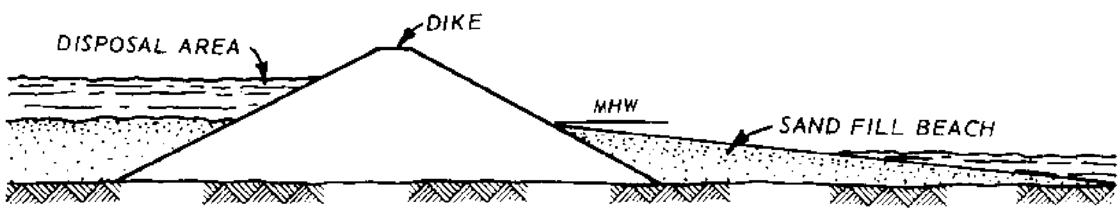


FIG. 21.--Use of Sand Beach for Dike Slope Protection

energy is dissipated by runup on the flat slope. This type of protection is of particular interest for use as protection on exterior slopes of dikes that are adjacent to large bodies of water and continuously experience wave action. Where the material and space are available, flat beaches are often far more economical than riprap, particularly if long haul distances are involved for transportation of the riprap. Another consideration in favor of flat beaches is that for dikes constructed of pumped hydraulic fill, flat slopes normally result anyway. Design of flat beaches should be based on a study of nearby existing beaches with similar controlling conditions. A slope of 1V on 10H should be suitable for preliminary design. It should be recognized that partial or complete replacement by riprap or other means may be necessary in certain areas such as at structures within the embankment or areas subjected to particularly severe wave or current action. Guidance for use in the design of flat beaches may be obtained from the Coastal Engineering Research Center (CERC) publication, "Shore Protection Manual" (12).

RIPRAP

Quarry-run riprap or graded stone riprap placed over a crushed stone bedding material (filter) or filter cloth is the most commonly used method of substantial slope protection against wave and current erosion. The widespread use of riprap is due to several reasons, some of which are (a) quarried stone is readily available in most areas; (b) common construction equipment and techniques are utilized in placement; (c) the performance history of riprap is good; and (d) riprap is usually the most economical method to achieve the protection desired. A typical riprap protected dike is shown in Figure 22.

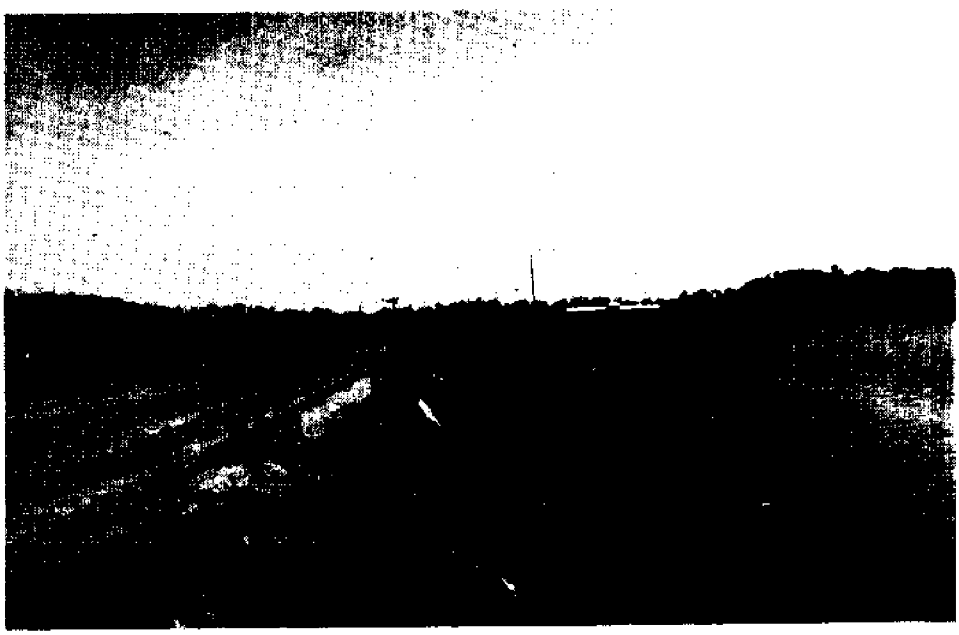


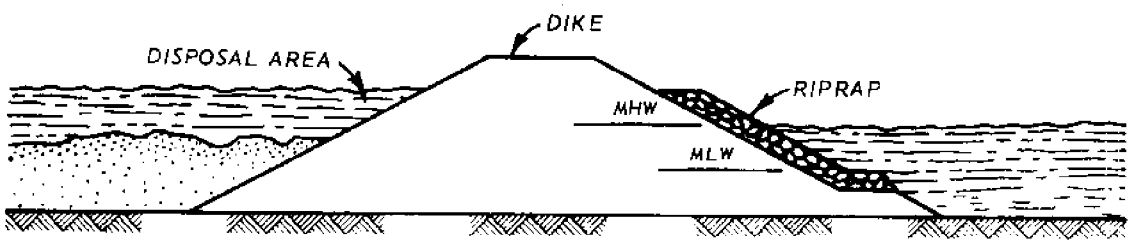
FIG. 22.--Typical Riprap Slope Protection

Design procedures using riprap to protect against wind-driven or ship-generated waves are presented in Engineer Manual 1110-2-2300, "Earth and Rockfill Dams, General Design Considerations" (9). Engineer Manual 1110-2-1601, "Hydraulic Design of Flood Control Channels" (8) contains guidance on riprap design for protection against current or flow velocities. Guidance for coastal installations is contained in Reference 12.

The upper limit (or maximum height) of riprap protection should provide adequate freeboard above the maximum water level (usually high tide, highest expected interior water level, or design flood stage) plus design wave height; the lower limit should provide a toe or key below minimum water level (low tide or minimum flow). In any event, riprap protection should extend well above and below design high and low water levels. Often this will be the dike crest and a minimum of 2 to 3 ft below water, respectively (Figure 23).

OTHER METHODS

Although riprap is the most common method of substantial slope protection, other methods should be considered to determine which is the most feasible and economical. Factors such as site access, high transportation cost, availability of suitable stone, or other considerations peculiar to a particular site can make alternative methods of slope protection more feasible. Other available methods of slope proteciton include (a) grout-filled nylon revetments (FABRIFORM, VSL, HYDROMAT, etc.), (b) interlocking concrete blocks (LOK-GRAD), (c) concrete paving, (d) sacked concrete, (e) stone-filled wire mesh baskets (GABIONS),



NOTE: MHW = MEAN HIGH WATER
MLW = MEAN LOW WATER.

FIG. 23.--Cross Section of Dike With Exterior Slope Protected by Riprap

(f) soil-cement, and (g) precast concrete forms (Tribars, Tetrapods, etc.). Specifications and design criteria for most newly developed slope protection systems can be obtained from manufacturers' literature.

A small amount of cohesion in dike embankment materials greatly increases resistance to erosion caused by wind and rain. On the other hand, where frost heave is common, dikes of cohesionless material will be less susceptible to damage than those of cohesive materials. Cohesionless material subject only to effects of weathering may best be protected by establishing a vegetative cover. Often a layer of topsoil is necessary to establish such growth, along with a light cover of emulsified asphalt or mulch to prevent erosion until such time as the vegetation is established. The Mobile District, CE, has successfully protected sand dikes from erosion caused by rain by cupping the dike crest to catch rainwater and providing drains at certain locations along the alignment. This method of protection is shown in Figure 24.

Polyethylene sheeting, if properly placed and overlapped, can be effective in preventing erosion of interior dike slopes from wave and current action and heavy discharge flow. Polyethylene sheeting can also be used on exterior slopes on a short-term basis where erosive forces are not too severe. Disadvantages from the use of polyethylene sheeting are deterioration from sunlight, damage from burrowing animals, and removal due to wind action and vandalism.

To prevent direct washout and erosion of interior dike slopes from the pipeline discharge of dredged material, the discharge pipe should extend at least 50 to 100 ft beyond the dike toe. In addition, a diffuser should be used to dissipate as much energy as possible. Also, a trench

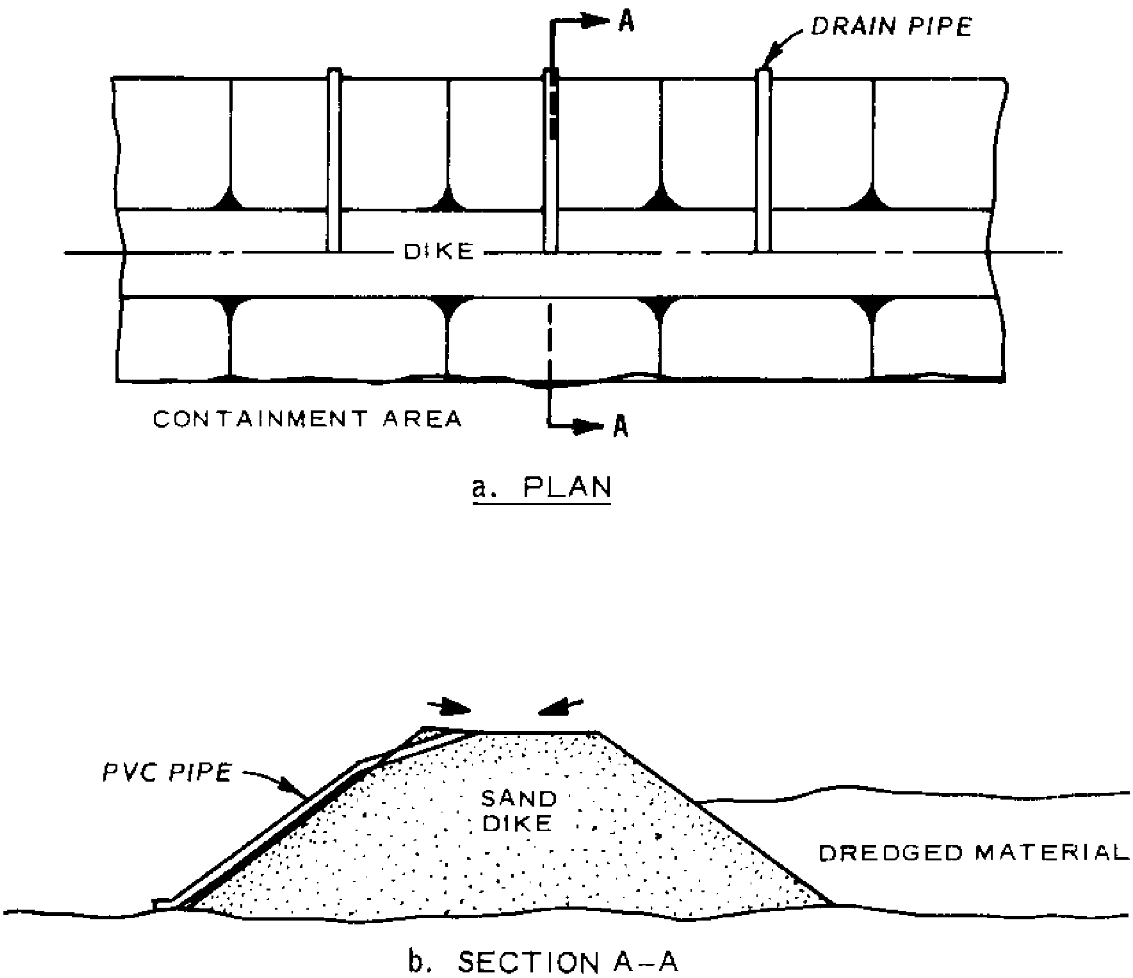


FIG. 24.--Protection of Sand Slopes From Slopewash Due to Rainfall

100 to 200 ft long should extend from the discharge point toward the center of the disposal area to prevent the discharge from flowing along the dike toe in the vicinity of the discharge pipe (Figure 25). If, due to the topography of the area, channelization develops along the toe of the dike or through other undesirable areas, spur dikes or cross dikes should be constructed.

Outfall pipes for sluice discharges should extend at least 10 to 15 ft from the exterior dike slope. Also, a ditch should be cut to allow ready escape of discharge water away from the dike toe. Where spillway outlets are used, special consideration should be given to protection of the dike in the area of discharge. Included in these considerations should be riprapping or concreting of the dike slope in the area.

Prevention of erosion due to overtopping caused by overfilling the disposal area can only be controlled by eliminating negligence on the part of personnel in charge of disposal operations. The fact that failures such as this occur indicates the need for constant inspection of disposal operations by qualified personnel.

FIG. 25.--Channelization Along Dike Toe

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APPENDIX II-NOTATION

The following symbols are used in this paper:

CD = consolidated-drained

CU = consolidated-undrained

FS = factor of safety

H = dike height

q_d = ultimate bearing capacity

UU = unconsolidated-undrained

β = slope angle

γ = unit weight of embankment material

ϕ = angle of internal friction of soil

LABORATORY DETERMINATION OF BULKING FACTORS

Frank DiGeorge[1], J. B. Herbich[2], and W. A. Dunlap[3]

ABSTRACT

This paper presents the results of a laboratory determination of bulking factors for 27 different soil samples representing a variety of consolidated sandy and silty clays typical to the Texas coastal area. The laboratory methodology is similar to that recommended by Lacasse, et al., 1977, but includes the use of a special impeller to simulate the dredging process in preparing soil slurries. Sedimentation tests are conducted in 1000 ml graduated cylinders and observed for periods ranging from 5 to 30 days. The effects of varying cylinder size and water salinity are also investigated. The results indicate that bulking factors decrease with increasing water salinity, and that significant flocculation of soil particles occurs at water salinities as low as 13%. The results also indicated that bulking factors tend to increase with increasing h/d ratios, where h is the height of slurry initially placed in the cylinder, and d is the cylinder diameter. Equations derived from the data are presented, and relate bulking factors to such soil variables as containment area average void ratio, percent silt and clay, in situ water content, and Atterberg limits. Use of these equations to predict the laboratory observed bulking factors results in an accuracy ranging from ± 18% to ± 30%.

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I. INTRODUCTION

Background

The maintenance of minimum water depth for safe navigation in U.S. harbors and navigable waters is a multi-million dollar endeavor involving the removal by dredging of approximately 380,000,000 cubic yards of sediment annually (1). Disposal of the dredged material has in recent years proven to be one of the major problems facing the U.S. Army Corps of Engineers in meeting its responsibility of maintaining and improving U.S. navigable waters. One of the factors is the steadily rising concern for environmental protection and the realization that open water disposal of polluted dredged material may release harmful pollutants into the water column, adversely affecting marine life and water quality. In the past, as much as 70% of dredged material was disposed of in open water, or otherwise unconfined areas. The alternative to this method is disposal in confined areas ashore to minimize the effect of polluted dredged material on the environment. Accordingly, it has been said, "the basic purposes of confining dredged material are to prevent the spread of pollutants into the environment, reduce the level of biologically harmful or asthetically unpleasant constituents in the effluent, and decrease the unrestricted spread of dredged materials into the adjacent environment" (1). Other advantages of this method of disposal include the potential use of the area for industrial or recreational development, or to create a habitat for supporting wildlife. On the other hand, cost of disposal in confinement areas may vary from five to fifteen times as much as open-water disposal. The method may also destroy valuable wetlands, and the land surrounding the highly industrialized environment of U.S. ports may be far too expensive to use for dredged material containment (2, 3). These factors combine to pose a highly complex problem for which no complete solution is easily achieved.

Problem Statement

If the choice of dredged material disposal in containment areas ashore is made over the possible choice of disposal at sea, the efficient long range use of the selected areas becomes critical, and a new array of problems comes into focus. One of these problems, and the one with which this investigation deals, is the change in volume, or "bulking" of a sedimented material when disturbed by dredging. The mechanical disturbance of the soil by the dredging process and removal of the overburden pressure caused by overlying material causes an expansion of the soil (4). This increase in volume is accompanied by an increase in the void ratio and water content of the soil (5, 6). In the case of dense sands this expansion may be minimal, while for loose sands the dredging process may consolidate the material through placement of additional weight above it in a containment area. However, in the case of silts and clay soils, the bulking of the soil may be quite substantial, particularly for consolidated clays. This phenomenon is partially due to the aforementioned factors, and partially due to the absorption of water by the clay. The "bulking factor" of a particular soil is the dimensionless factor expressed by the ratio of the volume of the soil in a containment area after dredging to that volume of the soil in situ. The term "sizing factor" as used in this report and in the literature implies that such variables as dredge system efficiency and long term consolidation are included in the factor. The following equations are presented from reference 7:

$$B = \frac{V_c}{V_i} \qquad (1\text{-a})$$

$$B = \frac{\gamma_{d_i}}{\gamma_{d_c}} \qquad (1\text{-b})$$

$$B = \frac{w_c G_s + 100}{w_i G_s + 100} \qquad (1\text{-c})$$

where B = bulking factor; V_c = volume in containment area; V_i = volume in situ; γ_{d_i} = dry density in situ; γ_{d_c} = dry density in containment area; w_c = water content in the containment area; w_i = water content in situ; and G_s = specific gravity of the solids. Equation 1-c is valid only for a soil under saturated conditions. In order to calculate the unit volume of a soil in situ, the water content and specific gravity must be determined. In order to predict the volume occupied by the dredged material in the containment area, the bulking factor for the soil type must be accurately known. If any long term prediction is expected, the settlement and consolidation characteristics of the soil type must also be considered (8, 9). In the majority of cases cited in the literature, government agencies and private contractors involved in dredging operations rely heavily on practical experience to predict bulking or sizing factors. There has been much dissatisfaction or uncertainty expressed in regard to those factors commonly employed for clay or silty-clay soils. These factors have historically resulted in undersizing areas by as much as 50% or oversizing them by as much as 100% (10). This problem is partially related to the wide range of clay soil characteristics, and the close relationship between the behavior of clay and the dredging system utilized to move it. Once dredged material is in suspension, its settlement characteristics are a function of water salinity, turbulence, and solids concentration as well as the properties of the soil. Increasing the salinity tends to intensify flocculation up to a limiting concentration, above which increased concentration has little effect. Those clay particles suspended in fresh water tend to remain in suspension until all water motion ceases, and then settle very slowly to the bottom where they accumulate as sediments. As water salinity approaches 14 $^{o}/oo$, the clay particles flocculate and settle out of suspension much faster than in fresh water. This phenomenon is due to the fact that the abundant number of positively charged ions in salt water tend to change the surface charge of some of the clay particles from negative to positive. These clay particles with positive surface charges tend to aggregate with clay particles having negative surface charges, forming flocculants which rapidly settle out of suspension (11).

As turbulence increases, so does flocculation due to the increased opportunity for collisions between particles; turbulence also has a limiting value above which further increases tend to break up the flocculated soil particles. For solids concentrations less than 2.7% by weight an increase in concentration tends to cause an increase in flocculation. On the other hand, for typical dredge slurries which range from 10% to 30% solids by weight, increases in concentration tend to reduce particle movement, increase excessive pore-water pressure, and reduce flocculation (12). Prediction of bulking factors for clay is further complicated by the fact that some clay particles tend to remain in clods, depending on the dredging method. These clods are transported as a "bed load" in the pipeline, exiting the line as clay balls. These clay balls do not contribute significantly to the bulking characteristics of clay, thus the percentage of clay so transported directly influences the bulking factor (13, 7, 14).

Another problem associated with the use of containment areas is that a horizontal sorting of the dredged material by particle size results. The larger, heavier particles tend to settle near the discharge line in a fan-shaped distribution, while the fine silts and clays tend to remain in suspension longer and settle nearer the discharge wier. This horizontal sorting appears to be limited to an area with a radius of 90 to 200 meters from the discharge line, depending on particle size distribution, discharge velocity, and containment area topography (15, 16).

State of The Art

The previous section implies that the sizing factor which must be employed to accurately predict the volume required for the containment of a given volume of in situ material after dredging is a highly complex function of numerous variables, only some of which are directly related to the soil characteristics. Some of the soil related variables include in situ density, water content, void ratio, plasticity, cohesiveness, compressibility, permeability, soil particle distribution, and organic material content (14). Variables not directly related to the soil include the method of dredging, dredge efficiency, soil losses within the dredging and containment systems, water salinity, and long range climatic conditions in the area. In addition, the height of material to be placed in a containment area is directly related to the long-term settlement of the dredged material as well as consolidation of the underlying soil.

The sizing factors currently in use for particular soil types seem to vary with geographical location; these factors were mostly developed through practical experience and field observations on the part of those individuals and organizations involved in dredging operations. The results of an excellent and apparently exhaustive search for the bulking and sizing factors in current use may be found in references 6 and 7. These and other references as noted were used to compile the data presented in Tables I and II. The wide range of values recommended or used for sand, or silt, or clay, by various organizations clearly implies that the use of a single factor for all silts or all clays has proven unsatisfactory. The need for sizing factors related to soil characteristics more descriptive than merely sand, clay, or silt is apparent.

Previous Investigations

Baltimore Harbor, 1961.— Prior to the placement of dredged material in a Baltimore Harbor containment area in 1962, the Baltimore District Office of the Corps of Engineers conducted a series of sedimentation tests in transparent vertical cylinders in an attempt to predict the containment volume required for placement of the material. It had been decided that the rule of thumb that the enclosed area should be two to three times the in situ volume of material was insufficiently accurate to ensure compliance of the final elevation with contract specifications. Transparent tubes of four inches in diameter and 6,

TABLE I

Range of Sizing and Bulking Factor Values (5, 6, 7)

Source	Soil Type	Sizing Factors*	Bulking Factors+
Various U.S. Army Corps of Engineer Districts (based on experience) (12,17)	Sand	.56-1.3	1.0-1.3
	Silt	.5 -1.35	1.0-2.0
	Clay	1.0 -2.0	1.8-2.0
John Huston, Hydraulic Dredging (based on experience); use a weighted average for mixed soil types (11)	Sand	1.0	
	Silt	1.45	
	Clay	2.0	
	Sandy Clay	1.25	
	Rock and Gravel	1.75	
Japan Dredging & Reclamation Engineering Association, Tokyo (based on experience and laboratory tests) (12,17)	Sand	1.0	
	Silt	1.3 -1.6	
	Clay	2.0	
Port & Harbor Technical Research Institute, Tokyo (based on laboratory tests) (12,17)	Sand and Silt	0.7 -0.9	1.3

*Factors such as settlement and dredging efficiency taken into account.

+Bulking factors only - no consideration given to long-term settlement.

TABLE II

Bulking Factors Based on Soil Types and Equation 1-b (16, 17)

Material (100% mod. AASHO)	Unit Weight γ_{d_i} (lbs/ft^3)(10)	γ_{d_c} (lbs/ft^3)(19)	Bulking Factors (Equation 1-b)
Clay	105	30-78	1.3-3.5
Sandy Clay	60-135		
Silt (inorganic)	80-118	65-82	1.2-1.4
Sand	83-118	93	.9-1.3
Gravel	110		
Quartz Rock	165	93	1.8

12, and 24 feet in height were used for the tests. These heights were used in an attempt to simulate the plans for placement of material in the disposal area. The full-scale tests were conducted in addition to earlier tests in 1000 cc graduates. A slurry mix of 1300 grams per liter was selected based on experience, and the material was placed in the 12- and 24-foot-high cylinders in six-foot lifts. The supernatant liquid was drawn off after four days and a second lift of material placed in the cylinders. On the 27th day, the supernatent liquid was again removed and a third lift placed in the 24 foot-high tube. The cylinders were observed for a period of 500 days. The material tested had a void ratio of 3.413 to 6.063, a medium diameter of 0.0022 mm, and a specific gravity of solids from 2.70 to 2.79. The material was primarily a fine silt. Table III presents the resulting data from these tests.

Tests 1 and 2 were conducted in the six-foot-high cylinders, and tests 3 and 4 in the 12-foot cylinders. The 24-foot-tall cylinder developed a leak during the test and the results are not presented. The conclusions of these tests were that the 12-foot cylinder tests yielded higher bulking factors than the six-foot tests. The bulking factors were accepted as reliable, but were reduced to allow for the production of clay balls in the actual dredging operation. A final bulking factor of 1.7 was selected; this value is slightly higher than those currently recommended for silt (18).

Potomac River Marsh, 1974.— In this case of laboratory sedimentation testing specially designed cylinders were utilized. They were 7.9

TABLE III

Bulking Factors From Full-Scale Sedimentation Tests (18)

Percent of Job Complete	Bulking Factors			
	Test 1	Test 2	Test 3	Test 4
100%	1.50	1.57	1.59	1.65
70%	1.80	1.66	1.60	1.70
40%	1.66	1.73	1.78	1.75
20%	1.82	1.80	1.89	1.79
10%	1.93	1.84	2.06	1.83
5%	2.10	1.93	2.29	1.92
Final Bulking Factors	1.63	1.62	1.72	1.72

inches in diameter and were constructed of plexiglass in two-foot high sections; the sections were connected with o-ring seals. The tubes were erected to a height of ten feet and easily dismantled after testing to allow access to the various layers of sedimented soil. A slurry of 13% solids by weight and tap water was placed in the cylinders in seven lifts. Prior to adding a lift of material, the time differential settlement curve was allowed to reach a near-linear state, and the supernatant liquid was drawn off the soil. A new lift of material was then placed to a total height of 9.5 feet. The following equations were presented:

$$se = \frac{wG_s}{100} \qquad (2)$$

$$e = \frac{V_v}{V_s} = \frac{\gamma_{sat} - \gamma_d}{\gamma_d} G_s \qquad (3)$$

where s = saturation (%); e = void ratio; w = water content (%); V_v = volume of the voids; V_s = volume of the solids; and γ_{sat} = saturated density of the soil. The cylinders were observed for a period of one week beyond placement of the final lift. The supernatant liquid was then drawn off, the cylinders dismantled, and the sedimented soil divided into six-inch layers. Water content, grain size distribution, and solids specific gravity were obtained for each layer and the void ratio (e) was calculated from equation (2), assuming that saturation was 100%. The submerged unit weight of the material was calculated from the following equation:

$$\text{submerged unit weight} = \gamma_{sub} = \frac{\gamma_w(e + G_s)}{1 + e} - \gamma_w \qquad (4)$$

This method yielded a plot of void ratio versus effective vertical stress. Conventional floating ring consolidation tests were run on the samples and the bulking factors were calculated from the equation:

$$B = \frac{V_v}{V_i} \text{ (same as equation 1-a)}$$

$$\text{where, } V_c = V_s(1 + e_{c\ ave.}) \quad \ldots \quad (5)$$

$$\text{and, } V_i = V_s(1 + e_{i\ ave.}) \quad \ldots \quad (6)$$

Another equation for determining the bulking factor was presented. This equation can be derived from equation 1-c of the previous section for soils with 100% saturation.

$$B = \frac{1 + e_{c\ ave.}}{1 + e_{i\ ave.}} \quad \ldots \quad (7)$$

where $e_{c\ ave.}$ and $e_{i\ ave.}$ are the average void ratio in the containment area, and the average void ratio in situ, respectively.

For the material tested, $G_s = 2.68$. The average in situ void ratio was 1.70, the average void ratio for the sediment in the cylinders was 3.26, and the bulking factor thus determined was 1.58. An equation was also presented for determining settlement from the laboratory test results as a prediction of settlement in the containment area.

$$\text{settlement} = \frac{e_{c_1} - e_{c_2}}{1 + e_{c_1}} H \quad \ldots \quad (8)$$

where e_{c_1} = average void ratio after the sedimentation test; e_{c_2} = average void ratio after consolidation test; and H = layer thickness. The bulking factor selected resulted in correct sizing of the containment area, and a factor of 1.58 does fall within the range of those values currently recommended for silty clay (19).

Toledo, Ohio, 1974.—Krizek and Giger conducted a very extensive study of several containment areas in the port of Toledo, Ohio from 1972 through 1974. The work included detailed topographic surveys of a containment area prior to the placement of dredged material and after the site was filled. Laboratory compaction tests were also performed on undisturbed samples of dredged material from the containment area to evaluate the merits of dewatering and compaction efforts. The dredging was completed by hopper dredge and the bin volumes were provided for the study by the Corps of Engineers. The in situ volume was determined by dividing the bin volume by .82, a factor commonly used by the Corps of Engineers. The ratio of the containment area volume to the bin volume as determined by a topographic survey 1.5 years after placement of the dredged material was .62, comparing favorable with the Corps of Engineers' commonly employed factor of .65. The resulting bulking factor calculated by equation 1-a was 0.8. The study further revealed combined settlement and consolidation within the containment area at the rate of about 4% of the original volume per year. It was noted that a limiting density would be achieved after some period of time(t) in excess of ten years. The following empirical equation was developed relating in situ volume to disposal site volume:

$$V_c = B(1 - 0.04t)V_i \quad \text{. . . . . (9)}$$

or $V_c = 0.08(1 - 0.04t)V_i$

With the exception of containment area dry densities, no soil data were provided in the paper, but it is theorized that the soil was fine and polluted material commonly associated with maintenance dredging operations (20).

Sizing of Containment Areas for Dredged Material, 1977.— This technical report was prepared under contract to the U.S. Army Engineers and presents a complete methodology for determining the required volume of a containment area. A laboratory determination of bulking factors through sedimentation tests is suggested, and an empirical equation considering the in situ void ratio of dredged material as well as the efficiencies of various dredging system components is introduced. This empirical equation involves the theory of material balance and its development is presented here. A determination of the volume of dredged solids is calculated from equation 10.

$$V_s = \frac{V_i}{1 + e_i} \quad (10)$$

The basic material balance equation is:

$$V_{s_c} = V_{s_i}(1 + Fo)Fe\ Fp\ Fc \quad (11)$$

where V_{s_c} = volume of solids retained in the containment area; V_{s_i} = volume of solids in situ; Fo = overdredging factor; Fe = dredge efficiency; Fp = transport system efficiency; and Fc = containment area efficiency.

The required containment volume (V_c) is also a function of the containment area average void ratio, therefore

$$V_c = V_{s_c}(1 + e_{c\ ave.}) \quad (12)$$

Substituting equations 10 and 11 into 12 yields

$$V_c = \frac{V_i(1 + Fo)\ Fe\ Fp\ Fc\ (1 + e_{c\ ave.})}{1 + e_i} \quad (13)$$

Table IV presents commonly employed values of Fo, Fe, Fp, and Fc. The next step is to determine the in situ void ratio and the average void ratio of the sedimented material. The in situ void ratio can be determined from undisturbed samples in routine laboratory tests. "The void ratio of the dredged material in a containment area represents one of the most important parameters in the sizing method and can be determined from laboratory tests or field measurements" (10). In this study, sedimentation tests were conducted in 20 centimeter and 30 centimeter diameter cylinders two meters high, using a slurry of 15% solids by weight. The material was placed in a single lift and allowed to settle for about two weeks, or until settlement was less than .1 centimeter per day. The water content (w) was then determined for as many layers of the sediment as possible and the void ratio calculated from equation (2) assuming s = 100%. Bulking factors were then calculated from equation (7). G_s was determined to be 2.66 for all samples with the exception of one whose G_s was 2.70. A variety of soil samples were tested in fresh and salt water, resulting in the following conclusions:

(1) Soils with a low plasticity in fresh water (plasticity index < 20%) will result in a bulking factor $\leq$ 1.1.

(2) Soils with a high plasticity in salt water (plasticity index > 50%) will result in a bulking factor $\geq$ 1.3.

(3) Soil void ratio increases with water salinity and plasticity.

(4) Various gains and losses in material during the dredging process can be expected to alter the containment volume by about $\pm$ 6% (6, 10).

TABLE IV

Range of Values for Dredge System Efficiency Factors (10)

Factor	Range of Values
Fo	32%-78%
Fe	80%-100% (usually 97%-100%)
Fp	81%-100% (usually 98%-100%)
Fc	86%-100% (usually 95%-100%)

Research Scope and Objectives

It has been demonstrated by the documented research outlined in the previous section that laboratory sedimentation tests in transparent cylinders can successfully be utilized to predict the sedimentation characteristics of dredged material in containment areas. There was also an indication that scale effects may be a problem in conducting such tests. The close relationship between bulking factors and such variables as in situ and containment area average void ratios of the soil, water contents, grain size distributions, and Atterberg limits has been revealed. This investigation represents an attempt to develop significant graphical correlations between these variables and bulking factors through laboratory testing of a variety of undisturbed samples of Texas coastal fine-grained materials. The testing included sedimentation tests of 1200 grams per liter slurries (by weight) in 1000 cc graduated cylinders, as well as determination of sedimented material void ratios (6, 18, 21). The effects of varying water salinity and cylinder size were investigated, and the results of 43 sedimentation tests on 27 different samples were presented. The primary objective of this research was to correlate bulking factors with the aforementioned soil characteristics in an attempt to develop design curves for the prediction of bulking factors for soils similar to those tested. Since prediction of sizing factors based on long range settlement and consolidation was beyond the scope of the intended research, determination of bulking factors based on the placement of a single lift of material in a cylinder was deemed sufficient.

II. LABORATORY PROCEDURES

The following is a step by step chronological presentation of the laboratory tests performed on those samples listed in Table V:

1. The wax was trimmed from the undisturbed samples as each was prepared for testing. The sample itself was then trimmed into a uniform cylindrical shape through the use of a knife and a small carpenter's square (Fig. 1).

2. The sample was carefully measured with a triangular metric scale, and its volume in cubic centimeters calculated from the measurements. This volume was later used in bulking factor calculations (Fig. 2).

3. The weight of the sample to the nearest decigram was obtained through the use of a metric balance and its specific volume obtained by dividing the volume from step 2 by this weight.

4. The sample was then chopped and mixed into a homogeneous mass using a knife and a glass cutting plate (Fig. 3).

5. For those samples tested in fresh water, 13.1 ^{o}/oo salt water, and 28.6 ^{o}/oo salt water, three 200 gram portions were taken from the sample and placed aside in separate, covered 1000 ml beakers. For those samples subjected to sedimentation tests only in 23.6% ^{o}/oo salt water, a single 200 gram portion was placed aside in a covered 1000 ml beaker. Portions of each sample were resealed in parrafin and retained for Atterberg limit and sieve analysis testing.

6. The in situ volume of each 200 gram sample was calculated by multiplying its weight by the specific volume obtained in step 3.

7. The portions of each sample were then allowed to slake in their

TABLE V

Physical Classification and Description of Soil Samples (7, 21, 30)

Sample Number	Symbol (1)	Consistency (2)	Depth (ft)	Source (3)	Physical Description
1631	CH	H	1.0- 2.5	A.E.	Gray clay with random calcareous nodules
1636	CL	VST	12.5-15.0	A.E.	Light brown clay with random calcareous nodules
1646	CH	VST	15.0-22.5	A.E.	Gray clay
1650	CL	M	27.5-30.0	A.E.	Gray sandy clay with random weathered limestone pockets
1651	CH	M	0.0- 2.5	A.E.	Gray sandy clay
1663	CH	M	0.0- 2.5	A.E.	Gray sandy clay with random sand seams
1685	SC	ST	5.0- 7.5	A.E.	Dark gray clayey sand with random sand pockets
1689	CH	VS	15.0-20.0	A.E.	Gray sandy clay
1692	CL	H	22.5-25.0	A.E.	Light brownish gray sandy clay with random calcareous nodules
1693	CL	H	25.0-27.5	A.E.	(Same as #1692; slightly harder)
1694	CL	VST	27.5-30.0	A.E.	(Same as #1692; slightly softer)
1704	CH	VS	22.5-25.0	A.E.	Gray sandy clay with random shell fragments
1726	CH	ST	7.5-10.0	A.E.	Tan sandy clay with random shell fragments
1732	CH	M	0.0- 2.5	A.E.	Tan sandy clay with random sand pockets and weathered limestone pockets
1742	CL	ST	2.5- 5.0	A.E.	Light grayish brown sandy clay with random calcareous nodules and sand layers
1757	CH	VST	5.0- 7.5	A.E.	Gray clay with random calcareous nodules
1761	CH	VST	15.0-17.5	A.E.	Yellowish brown caly with random calcareous nodules
1762	CH	H	17.5-20.0	A.E.	(Same as #1761; slightly harder)
1764	CH	VST	0.0- 2.5	A.E.	Dark gray clay with numerous shell fragments in the top four inches
1765	CH	VST	2.5- 5.0	A.E.	Gray clay with random sand seams
1766	CH	VST	5.0- 7.5	A.E.	Gray to yellowish brown clay with gypsum
1768	CH	VST	10.0-12.5	A.E.	Yellowish brown clay with gypsum
1769	CL	VST	12.5-15.0	A.E.	Yellowish brown sandy clay
1771	CL	VST	17.0-20.0	A.E.	Light grayish brown clay with gypsum
B# 38	CH	ST	24.5-25.5	H.	Light gray and brown silty clay
B# 1	OH	VS	25.0	M.E.	Very soft olive gray clay with light brown sand seams
B# 8	OH	S	214.0	M.E.	Soft olive gray clay

(1) Symbol: SC - sandy clay mixture; CL - inorganic clay of low to medium plasticity; CH - inorganic clay of high plasticity; OH - organic clay of high plasticity

(2) Consistency: VS - very soft; S - soft; M - medium; ST - stiff; VST - very stiff; H - hard

(3) Source: A.E. - Army Engineers; H. - personal communication - J.B. Herbich; M.E. - McClelland Engineers

separate beakers with 500 ml of the water in which they were to be tested. This slaking continued for a period of approximately 24 hours (Fig. 4).

8. After slaking, a variable-speed mixer and a special four-bladed plastic impeller-shaped blade were used to mix the soil particles and water into a homogeneous slurry. Water was added during the mixing process, increasing the slurry volume to about 975 ml. A slurry density of 1200 grams per liter by weight was desired to simulate that density typically found in dredge slurries (18, 19, 21, 23). The impeller blade (Fig. 5) provided a strong vortex within the beaker (Fig. 6) and served to raise the soil particles into suspension (Fig. 7, 8). By observing through the walls of the beaker, it was easily determined that a mixing time of five to six minutes was required to ensure all soil particles were in suspension.

9. The slurry was then poured into a 1000 ml graduated cylinder. About 25 ml of the proper salinity water was used to rinse the adhering soil particles from the impeller blade and beaker into the graduated cylinder, raising the volume within the cylinder to exactly 1000 ml. This procedure reduced the loss of soil particles through the mixing process to a negligible amount.

10. The slurry was then allowed to stand undisturbed (Fig. 9) while the soil particles settled out of suspension. The level of the interface between the suspended material and the supernatant liquid was observed and recorded one-half hour after beginning the test, and hourly thereafter until the rate of settlement had decreased to 10 ml per hour or less. The level of suspended

FIG. 1.—Trimming of soil sample into cylindrical shape

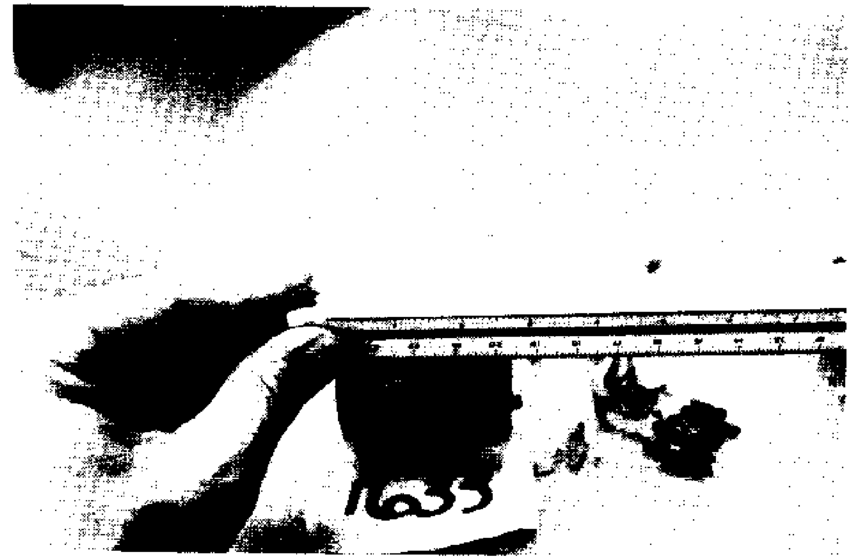

FIG. 2.—Obtaining measurements for volumetric calculations

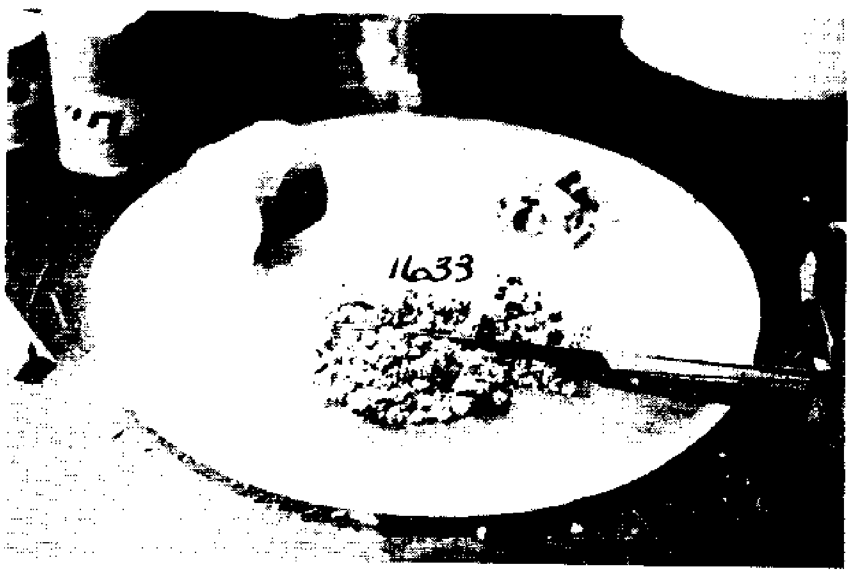

FIG. 3.— Preparation of soil sample for slaking

FIG. 4.— Miscellaneous soil samples slaking in salt water

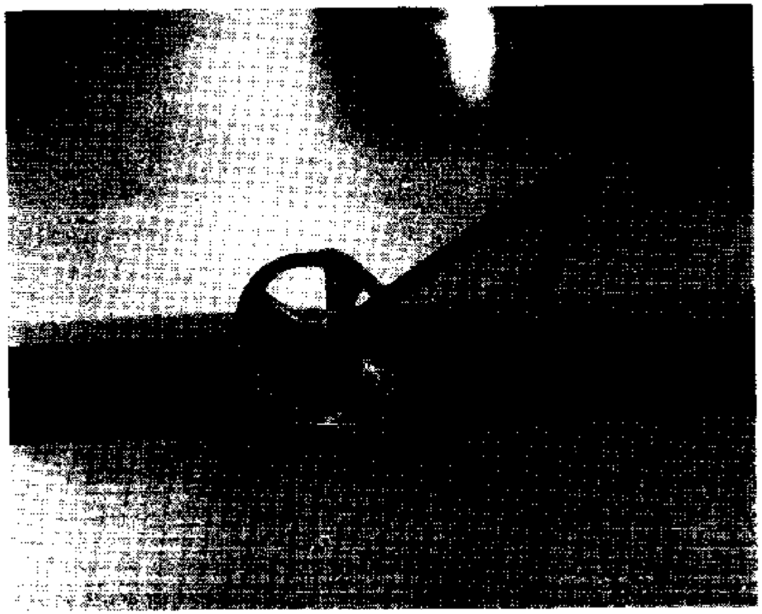

FIG. 5.—Impeller blade used to simulate dredging process

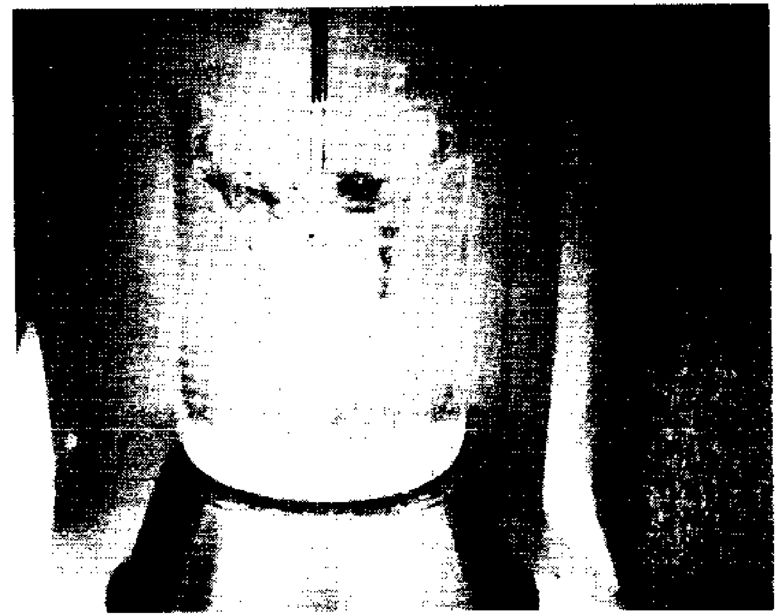

FIG. 6.—Vortex created by impeller blade

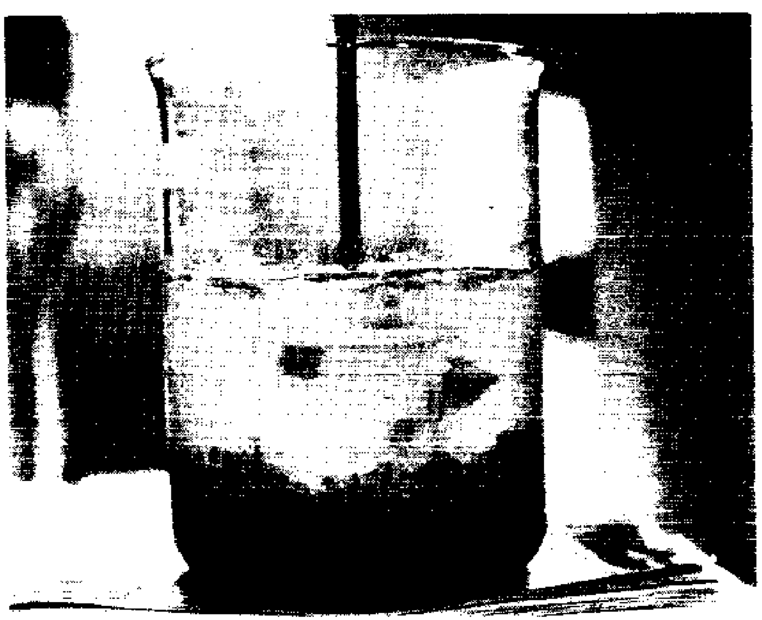

FIG. 7. — Soil particles being raised into suspension at slow impeller speed

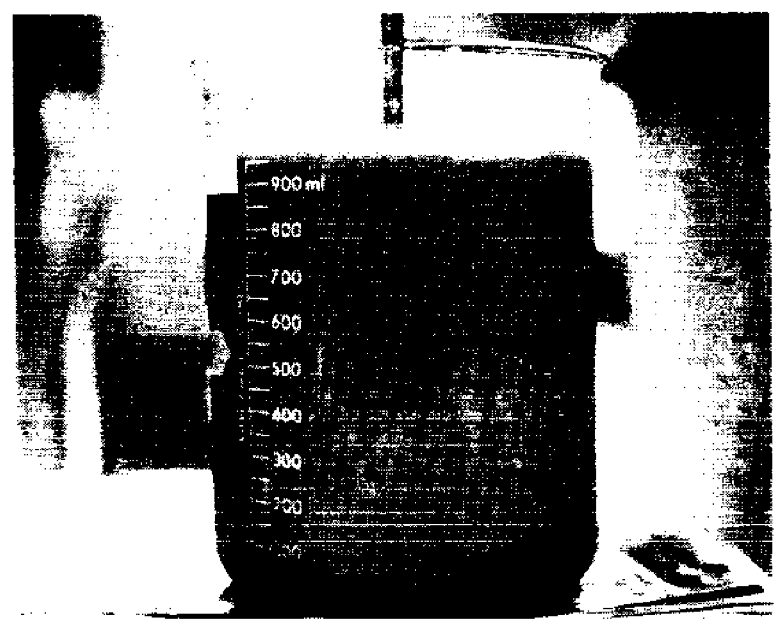

FIG. 8. — Slurry at completion of mixing, and prior to placement in cylinder

material was recorded thereafter at 24-hour intervals from the start of the test until the amount of settlement was undetectable for a forty-eight hour period (6, 10, 21).

11. When the sedimentation test for a particular sample was completed, the supernatant liquid was drawn off and its salinity determined through the use of the "Hydrolab" conductivity meter shown in Fig. 10. The sample and cylinder were weighed together, then placed in an oven to dry at 140^{0} F (24).

12. The cylinders were removed from the oven and weighed at 24-hour intervals until two consecutive weights were the same. By compensating for the weight of the small amount of salt water left in the cylinder prior to placement in the oven, and dried salt on top of the soil after removal from the oven, and the cylinder weight, the dry weight of the sample and moisture content of the sedimented material were determined. This moisture content was later used to calculate the void ratio of the sedimented material.

Applicability of the employed test procedures to field conditions is based on the following assumptions:

1. The 200 gram portion selected for testing from each sample was representative of the total sample.

2. The artificial sea water of 28.6 $^{o}/oo$ salinity was a reasonable approximation of field conditions.

3. The blending process employed to mix the slurry was a reasonable approximation of the dredging process (21). In most cases, the vortex motion of the slurry within the beaker caused small portions of the sample to roll on the bottom, forming clay balls of 1/16 inch to 1/4 inch in diameter. No attempt was made to break these

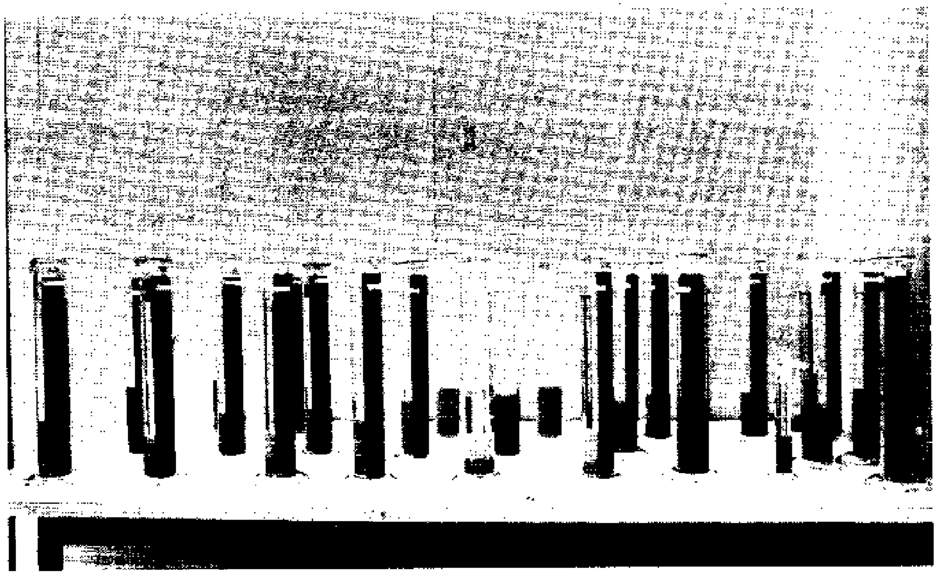

FIG. 9. — Sedimentation tests in progress in 1000 ml graduated cylinders

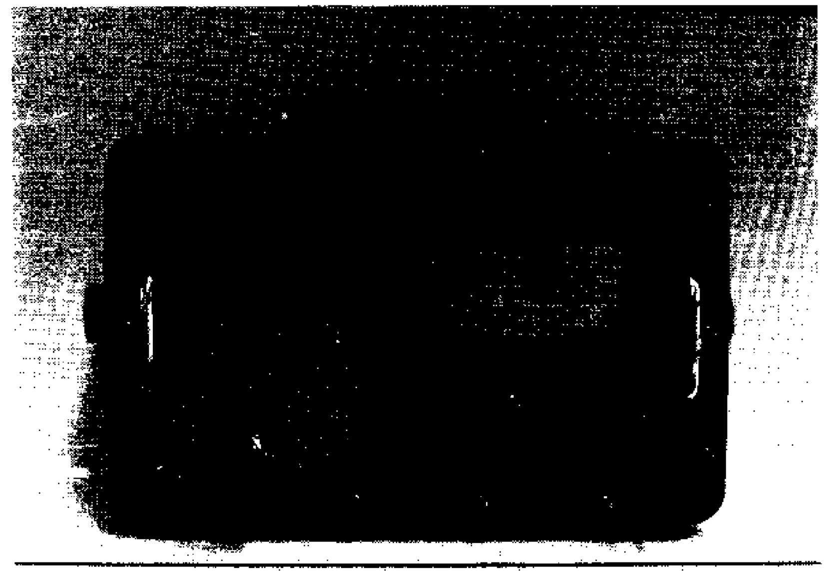

FIG. 10. — "Hydrolab" conductivity meter

FIG. 11. — Clay balls formed during the mixing process

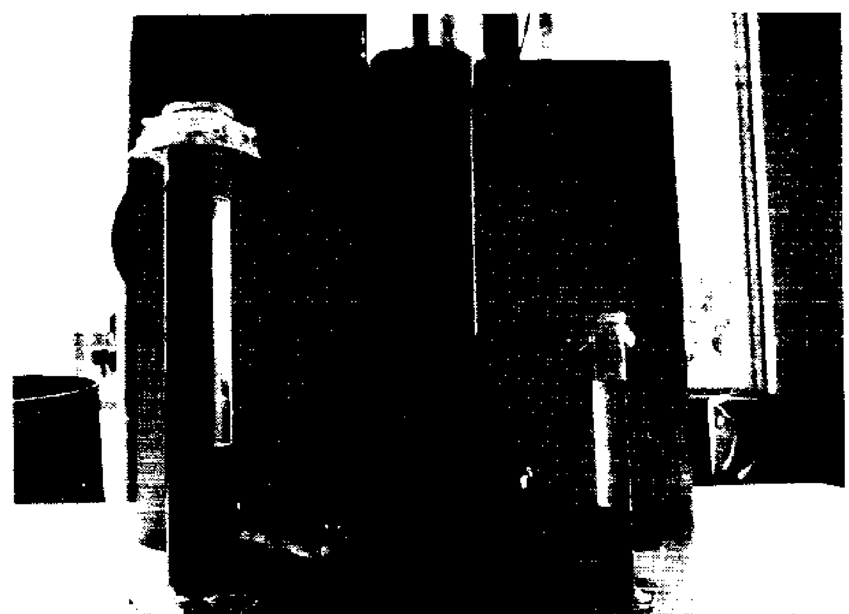

FIG. 12. Scale effects testing in progress (left to right) 1000 ml, 10 liter, and 1000 ml capacity cylinders

spheres down and force them into suspension. Doing so would have resulted in larger sizing factors, and it was felt that their formation was an indicator of reasonably accurate simulation of the dredging process. The clay balls, shown in Fig. 11, immediately settled to the bottom when the slurry was poured into a cylinder.

4. The loss of soil particles through adhesion to laboratory mixing apparatus was negligible.
5. The effects of compaction due to placement of successive lifts of dredged material was not simulated.
6. The effects of surface drying through mechanical dewatering and natural evaporation were not simulated.
7. The effects of long-term settlement and sub-grade consolidation were not simulated, nor were the projected effects of shrinkage from these causes reflected in the plotted bulking factors.

Salinity Variation Testing

Eight of the samples were used to determine the effect of water salinity on sedimentation. For six of these samples, sedimentation tests were conducted in fresh tap water, 13 $^{o}/oo$ salt water, and 28.6 $^{o}/oo$ salt water. This range of salinities was deemed sufficient to represent the range of salinities commonly found in Texas coastal estuaries where dredging operations typically occur (21). The results are presented in Chapter III.

Scale Effects Testing

Kolessar, 1962 (18), determined that scale effects might well be a problem in sedimentation tests conducted in small cylinders. In order to determine the magnitude of scale effects, a slurry of 1200 grams per liter by weight was placed in 100 ml and 1000 ml graduated cylinders. Ten liters of the slurry were also placed in a plexiglass cylinder, 10 centimeters in diameter and 2.44 meters tall (Fig. 12). The settlement rates of the solids in each of the cylinders as well as the final volume of sedimented solids and resulting bulking factors were compared. The results are presented in Chapter III.

Drying and Shrinkage

The resulting volume of material in a sedimentation test or in a dredged material containment area contains a very large percentage of water. Drying the material will of course reduce the moisture content and the volume of material. However, drying of the material presents a serious problem in dredged material containment areas and complete drying cannot be achieved. However, drying a small sample in a laboratory test is easily accomplished. The results can provide data for calculating the saturated void ratio of the material, and provide, at least, the extreme potential for consolidation of the soil through drying alone (29). This potential was investigated for 21 soil samples by completely drying them in an oven at 140^{0} F. The data were used to calculate saturated void ratios and sizing factors of the sedimented materials. The calculation of these quantities is discussed in the final two sections of this chapter, and the results are presented in Chapter III.

Void Ratio Calculations

In calculating the in situ void ratio (e_i) equation 2 was employec. It was assumed that the soil was saturated in situ, and the specific gravity of the solids (G_s) was assumed to be 2.68 (9, 12, 13, 17, 18).

The in situ void ratio was also calculated by the following equation (2):

$$\text{void ratio} = e = \frac{w\,\gamma_d}{\gamma_w - w\,\gamma_d} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

where γ_w = specific weight of water. Thus, knowledge of the in situ water content and dry specific weight of the soil permits calculation of the void ratio.

The void ratio in the containment area (cylinder) was also calculated using equation 2. The saturated water content of the sedimented material was determined by weighing the soil prior to and after drying. The specific gravity of the solids was again assumed to be 2.68, and saturation was assumed to be 100%. Thus, the average void ratio of the sedimented material in the graduated cylinder was obtained for each of 21 samples. These values are presented in Table VI of Chapter III.

Bulking Factor Calculations

Bulking factors were initially calculated from equation 1-a; knowledge of the in situ volumes as determined in step 6 of the laboratory procedures, and the final sedimented volumes in the graduated cylinders permitted this simple calculation. Bulking factors were also calculated by equation 7 using the in situ and sedimented average void ratios. Table VII in Chapter III presents a tabular comparison of bulking factors from various sources for those soil samples tested. Those factors as calculated by equation 1-a were used in the data plots presented in Chapter III.

III. EXPERIMENTAL RESULTS

A total of 27 different soil samples were tested in this program, and 43 sedimentation tests were performed. The samples were obtained from three sources: a) the Corps of Engineers soils laboratory at Galveston, Texas, b) McClelland Engineers, Geotechnical Consultants, Houston, Texas, and c) reference samples in the Texas A&M University Geotechnical Laboratory. Soil types were selected to be representative of Recent and Pleistocene Gulf Coast soils which are likely to be dredged in new dredging projects. Material typical of maintenance dredging was not included. Physical properties are presented in Table VI. For the most part, the samples were sandy and silty clays classed as CH (high plasticity clays) and CL (low plasticity clays) according to the Unified Soil Classification System. A few organic clays (OH) and one SC (sandy clay) were included. Liquid limits ranged from 28 to 80, with plasticity indices from 12 to 57. Sand content ranged from 0 to 48%, and thus the combined silt and clay content ranged from 52 to 100%.

The results of the sedimentation tests are generally expressed in terms of bulking factor versus time where bulking factor is defined earlier by Eq. 1. In viewing the test results, it is important to understand the phenomenon involved. Sands, silts and even coarse clay particles will settle out of an aqueous suspension by gravity according to Stokes' Law. However, the smaller clay particles begin to approach the size of individual water molecules, and when bombarded by the water molecules, they move randomly in response to the momentary hydrostatic pressure difference over their surface. As a result of this movement, termed Brownian movement, these small particles tend to stay in suspension almost indefinitely. If

TABLE VI

Physical Properties of Soil Samples (2, 7, 21, 30)

Boring Number	γ_{d_i} - dry density in situ (gm/ml)	w_i - water content in situ (%)	w_c - water content in cylinder (%)	e_i - void ratio in situ $\left(\frac{\text{Eq. 2 + Eq. 14}}{2}\right)$	e_c - void ratio in cylinder	PL - plastic limit (%)	LL - liquid limit (%)	PI - plasticity index	V_i - volume in situ (ml) (tested)	V_c - volume in cylinder (sedimented) (ml)	Particle Size Distribution**		
											Gravel (%)	Sand (%)	Fines (%)
1631	1.458	29.	+	0.755	+	*	*	*	107.48	320.	0	8.	92.
1636	*	18.	+	.482	+	12.0	40.5	28.5	99.03	239.	0	24.	76.
1646	1.394	37.	142.6	1.030	3.82	26.7	64.5	37.8	100.82	297.	0	9.	91.
1650	1.218	20.	79.5	.536	2.13	*	*	*	97.50	210.	8.	34.	58.
1651	1.137	50.	232.6	1.330	6.23	27.0	70.0	43.0	112.74	393.	0	14.	86.
1663	*	33.	203.8	.880	5.46	26.3	62.8	36.5	113.20	380.	0	32.	68.
1685	1.394	35.	+	.938	+	19.2	45.5	26.3	103.20	287.	0	50.	50.
1689	1.025	65.	184.4	1.865	4.94	24.0	79.0	55.0	123.42	330.	4.	28.	68.
1692	1.794	17.	+	.456	+	16.3	41.0	24.7	95.50	293.	0	46.	54.
1693	1.794	19.	78.1	.515	2.09	16.2	28.0	11.8	92.84	212.	0	47.	53.
1694	1.187	17.	88.5	.455	2.37	14.0	36.0	22.0	91.34	252.	0	48.	52.
1704	.977	66.	264.0	1.790	7.07	25.5	75.0	49.5	117.44	426.	1.	16.	83
1726	1.217	47.	173.1	1.295	4.64	22.0	79.0	57.0	110.42	329.	4.	18.	78.
1732	1.602	28.	98.9	.780	2.65	15.0	66.0	51.0	93.76	250.	0	40.	60.
1742	1.516	18.	94.5	.430	2.53	16.9	61.4	44.5	108.16	241.	2.	32.	66.
1757	1.826	17.	+	.453	+	*	*	*	92.65	255.	1.	5.	94.
1761	1.698	22.	122.3	.590	3.28	20.0	57.8	37.8	96.92	274.	0	10.	90.
1762	1.714	22.	+	.597	+	14.0	54.0	40.0	94.85	329.	0	10.	90.
1764	1.346	39.	129.8	1.070	3.48	24.0	84.0	60.0	110.18	291.	0	8.	92.
1765	1.330	37.	165.9	.980	4.45	29.9	79.4	49.5	109.80	321.	0	16.	84.
1766	1.362	38.	173.3	1.045	4.64	27.3	75.5	48.2	104.18	341.	1.	13.	86.
1768	1.554	28.	89.2	0.760	2.39	21.5	61.5	40.0	99.78	311.	0	0	100.
1769	1.682	20.	120.8	.525	3.23	18.0	47.0	29.0	97.18	290.	0	16.	84.
1771	1.634	24.	148.1	.640	3.97	22.1	65.4	43.3	99.78	327.	0	5.	95.
B-38	*	*	162.1	*	4.35	17.2	40.0	22.8	99.70	338.	0	14.	86.
B-1	.909	70.	139.6	1.750	3.74	38.0	92.0	54.0	103.03	267.	0	14.	86.
B-8	.781	55.	264.4	.750	7.08	35.0	85.0	50.0	115.24	410.	0	2.	98.

* Insufficient sample to permit analysis, or insufficient data

** Gravel-diameter 2mm, retained on No. 8 sieve; sand - 2 mm > diameter >.05 mm, retained on the No. 200 sieve; fines - diameter < .05 mm, passes the No. 200 sieve.

\+ Tests not performed

the suspension is concentrated, it is inevitable that collisions between particles will occur. Depending on the characteristics of the suspension, the particles may repel each other, or they may be attracted together. Since clay particles have a negative charge which is manifested on their surface, one would ordinarily expect the particles to repel each other. Water molecules, which are bipolar, are attracted to these negatively charged surfaces to build up an "adsorbed" water layer around the clay particles. This layer or film of water will also contain positively charged ions which may be available in the pore water, with the net result that there is a positively charged layer adjacent to, and somewhat counter-acting, the negatively charged clay surface. This will greatly reduce the repulsive forces between the clay particles. This effect is even more pronounced if the predominant ion in the adsorbed water film and the predominant ion in the free water are the same.

The reduction of the repulsive forces between the particles will allow the small attractive forces which also exist between particles to predominate. Thus, as the random movement of the particles brings them into contact, they will join together to form groups of particles, or floccules. These will then have enough mass to settle under gravity.

Influence of Water Salinity on Bulking Factors

The conditions for flocculation, as described above, are ideal in a salt water environment with its high concentration of sodium ions, and the degree of flocculation should affect the bulking factor. Thus, one of the first experiments performed was to examine the effects of water salinity on the bulking factor. Eight marine sediment samples were selected for this experiment - six from Corpus Christi Bay and two from offshore Gulf of Mexico. The samples were mixed with fresh water and with salt water of 13.1% and 28.6

% salinity and allowed to settle. As expected, the soils in the fresh water tended to remain in suspension while flocculation and relatively rapid sedimentation occurred in the salt water suspensions.

Fig. 13 shows an extreme example of the effect of salinity. The upper curve, for the fresh water suspension, had a continual and gradual decrease of bulking factor to a value of 7.44 after 768 hours when the test was terminated owing to time constraints. The two salinities of 13.1% and 28.6% had nearly identical curves and nearly identical bulking factors of 2.81 (13.1%) and 2.75 (28.6%). Also, both reached a constant value of bulking factor in a very short period of time. Similar results were obtained with other samples, although in some of the soils the fresh water bulking factor was much closer to the bulking factors obtained in the saline suspensions (Fig. b, for example). Also, the slope of the bulking factor versus time curve for some of the fresh water samples indicates that the bulking factor may eventually approach that of the saline suspensions. This does not appear to be a function of soil type, and it is possible that this was a function of the original salinity of the pore water in the sample.

The results are summarized in Table VII, which shows the final bulking factors and test time. These results show that a high degree of flocculation can be expected at salinities as low as 13.1%, and since there was little change between 13.1% and 28.6%, it appears that even lower salinities will still cause flocculation. Of course, salinities much higher than 13.1% are normally found in Texas bays and estuaries, and thus rapid flocculation should be expected in these areas during dredging operations.

Since bulking factors are a function of water salinity, it is obvious that any attempts at predicting bulking factors for a particular site should include testing in water of the same salinity as at the site.

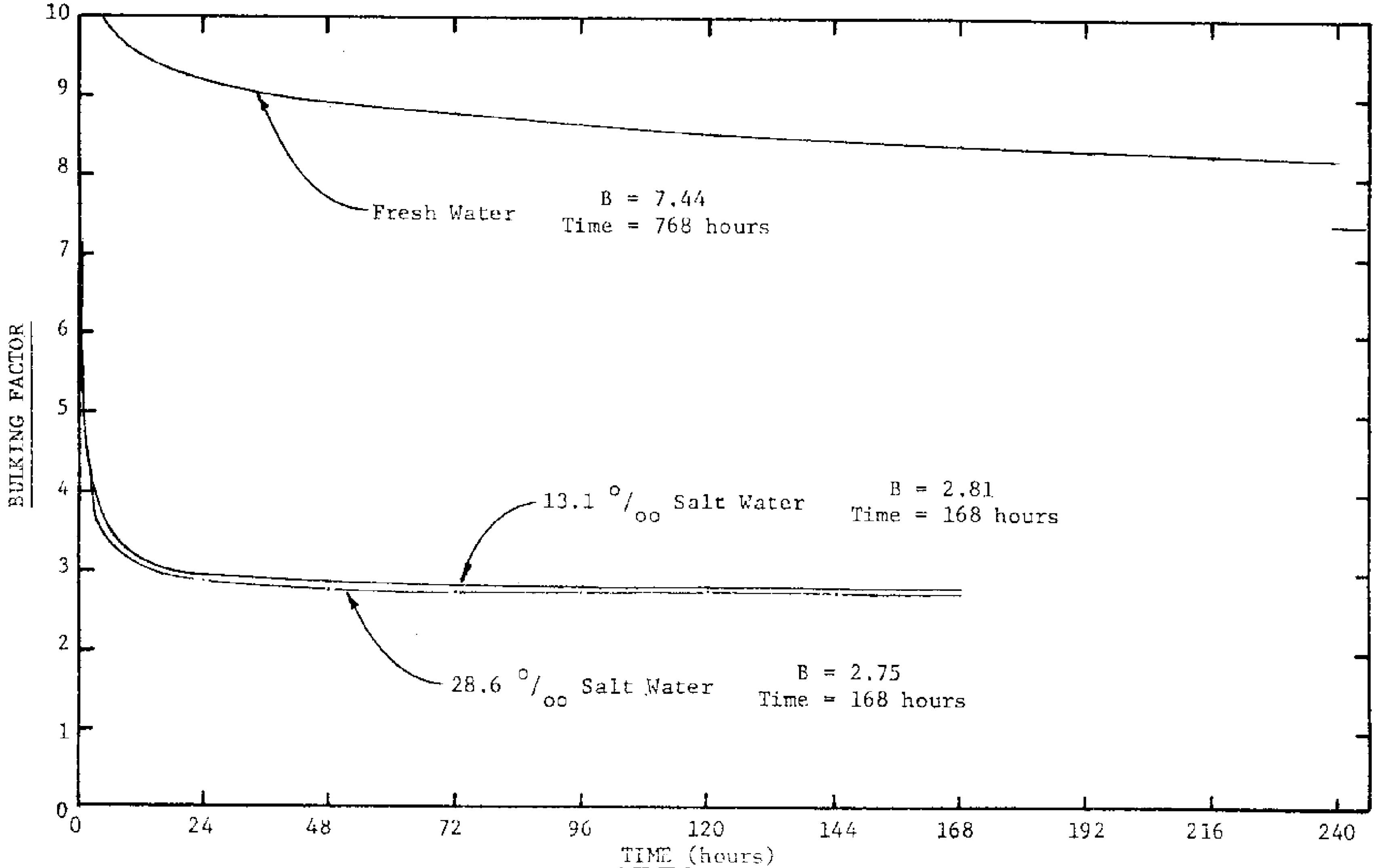

Fig. 13 Bulking Factor as a Function of Time Salinity for Sample No. 1757

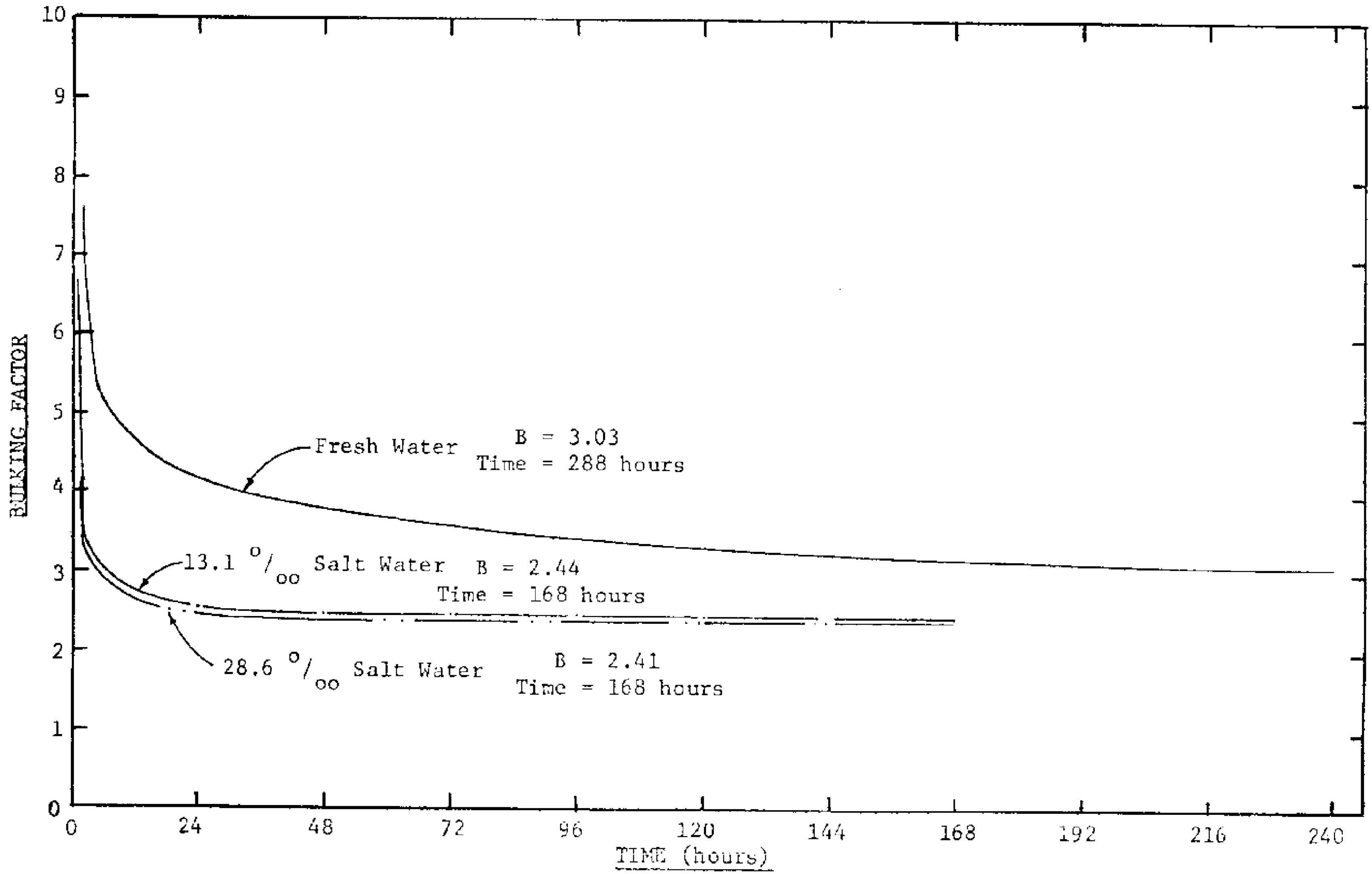

Fig. 14. Bulking Factor as a Function of Time and Salinity for Sample No. 1636

TABLE VII

Bulking Factors as a Function of Water Salinity

Sample Number	+ Soil Type	Bulking Factor/Time Fresh Water	Salt Water 13.1 $^{o}/oo$	Salt Water 28.6 $^{o}/oo$
1631	CH	3.13/360 hours *	2.89/168 hours	2.98/168 hours
1636	CL	3.03/288 hours *	2.44/168 hours	2.41/168 hours
1685	SC	3.04/360 hours *	2.83/264 hours	2.78/240 hours
1692	CL	4.29/360 hours	3.01/240 hours	3.07/240 hours
1757	CH	7.44/768 hours	2.81/168 hours	2.75/168 hours
1762	CH	4.81/360 hours *	3.50/240 hours	3.47/216 hours
B-1	OH	(not tested)	2.83/384 hours	2.59/576 hours
B-8	OH	(not tested)	3.30/576 hours	3.56/552 hours

* Tests were terminated prior to achieving settlement rates of less than 1 ml per 48-hour period due to apparent linear settlement rates and time constraints.

+ Soil Classification: CH - inorganic clay of high plasticity; CL - inorganic clay of low to medium plasticity; OH - organic clay of high plasticity; SC - sandy clay mixture

One factor which these laboratory tests do not consider is the influence of turbulence on the bulking factor since the tests are obviously performed in quiet water. Turbulence acts to bring the small particles into contact more rapidly so they can floc and to bring flocs together to form larger flocs. Although some turbulence enhances flocculation and settlement rates, a high degree of turbulence will break up the flocs and cause dispersion.

Influence of Time of Settlement on Bulking Factors

Eighteen additional samples were tested to examine the relationship between time and bulking factors. These tests were continued until virtually no change in bulking factor occurred over a 48-hour period. Figs. 15-17 show typical results. The tests showed that bulking factor was a nearly linear function of time beyond 200 hours for all samples tested, but most samples reached this point much quicker. The final bulking factors and the time to attain these bulking factors are shown in Table VIII. The bulking factors of these samples all tested at ranges from a low of 2.15 to a high of 3.81, and the times required to reach these bulking factors ranged from 120 to 455 hours.

Scale Effects

The possibility that the size of the laboratory equipment influenced the results was examined on one sample. This sample was tested in three cylinders - 0.1, 1.0 and 10 liters in size. Kolessar (18), in the pioneering work on this method, used cylinders which varied in height from 6-12 feet, but in our study, both height and diameter were varied.

As shown in Fig. 18, the sediment initially settled fastest in the 100 ml and slowest in the 10 liter cylinders, but after approximately 96 hours the situation reversed, and the greatest settlement occurred in the

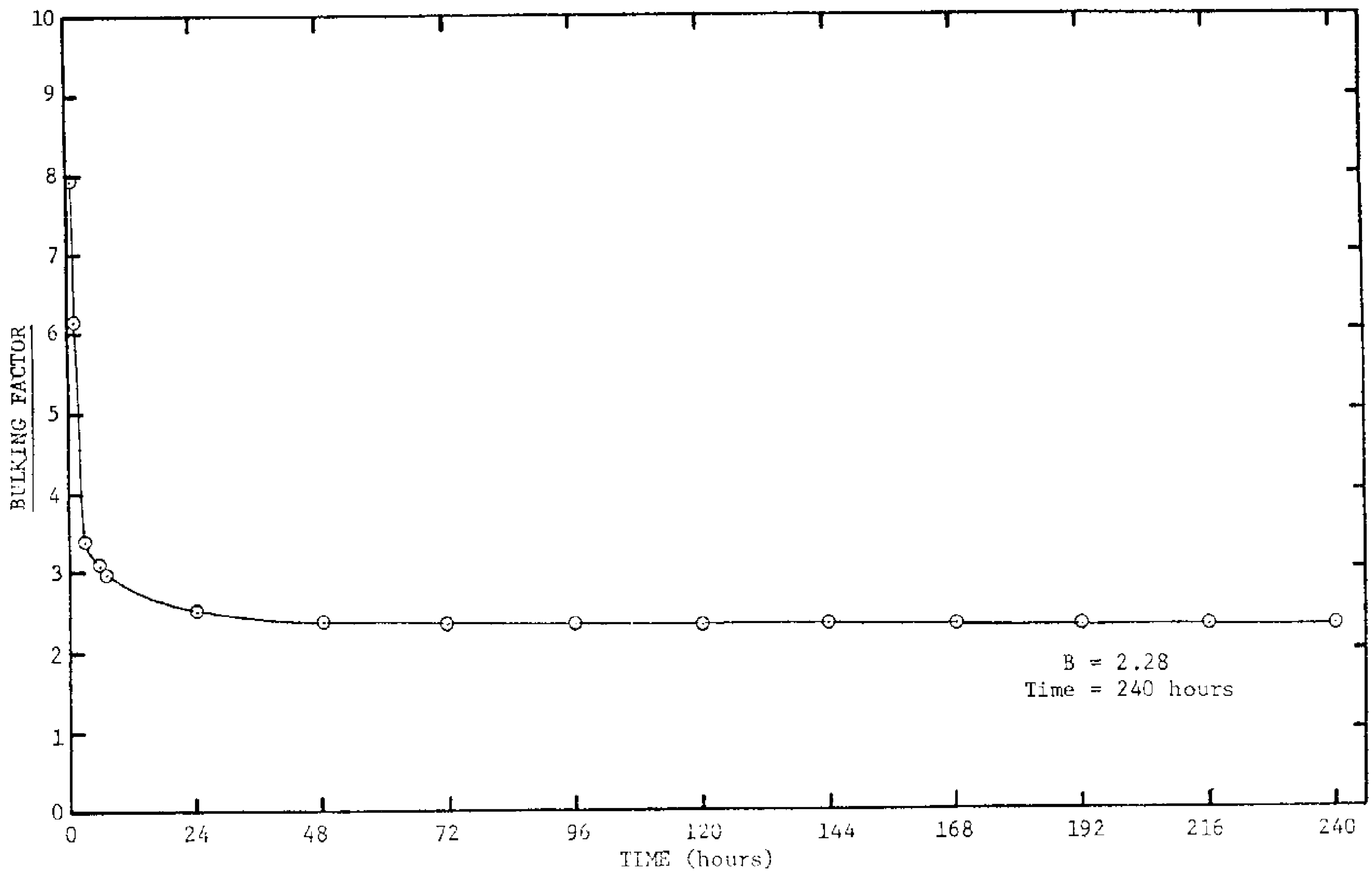

Fig. 15. Bulking Factor as a Function of Time for Sample No. 1693

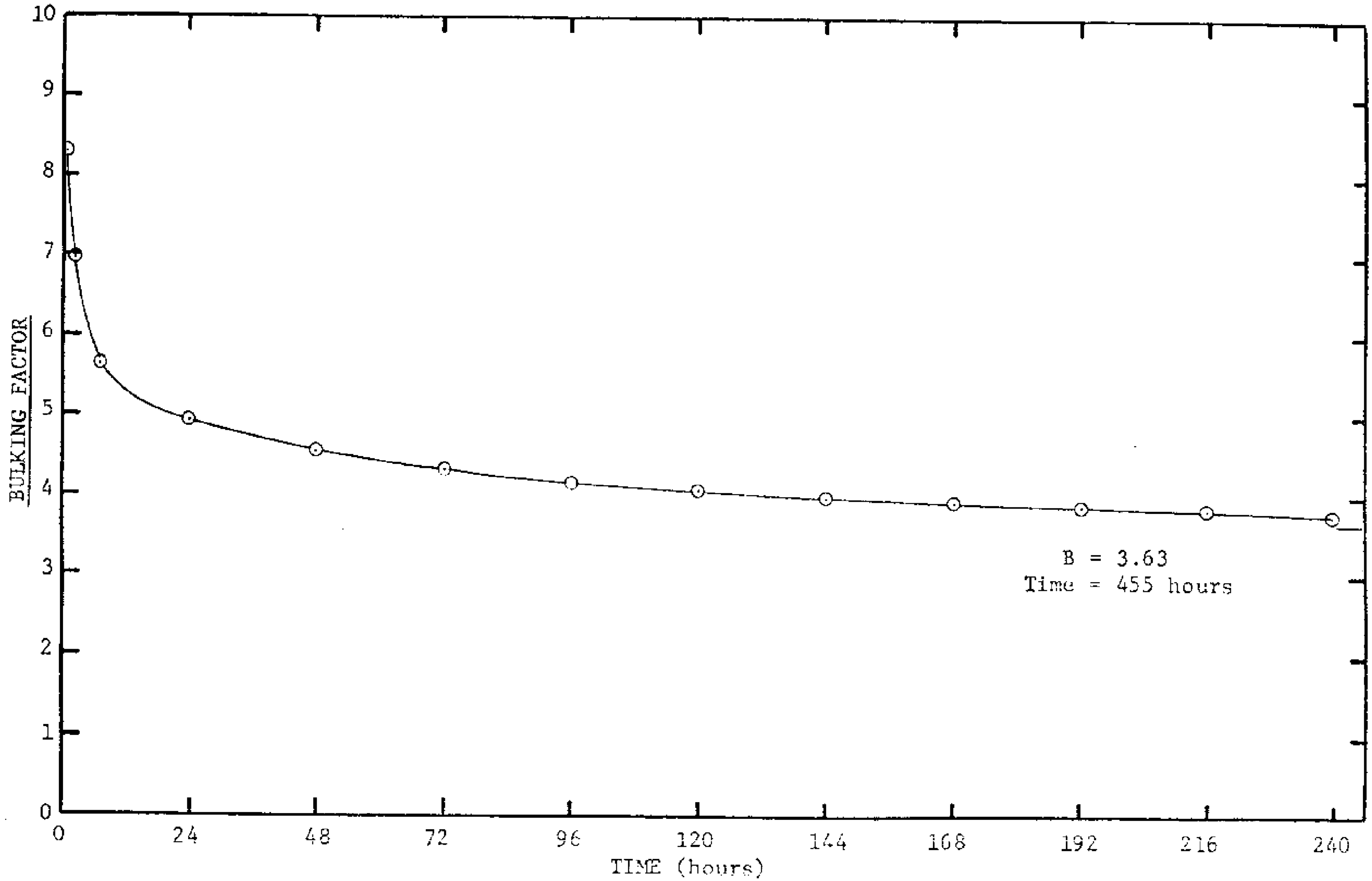

Fig. 16. Bulking Factor as a Function of Time for Sample No. 1704

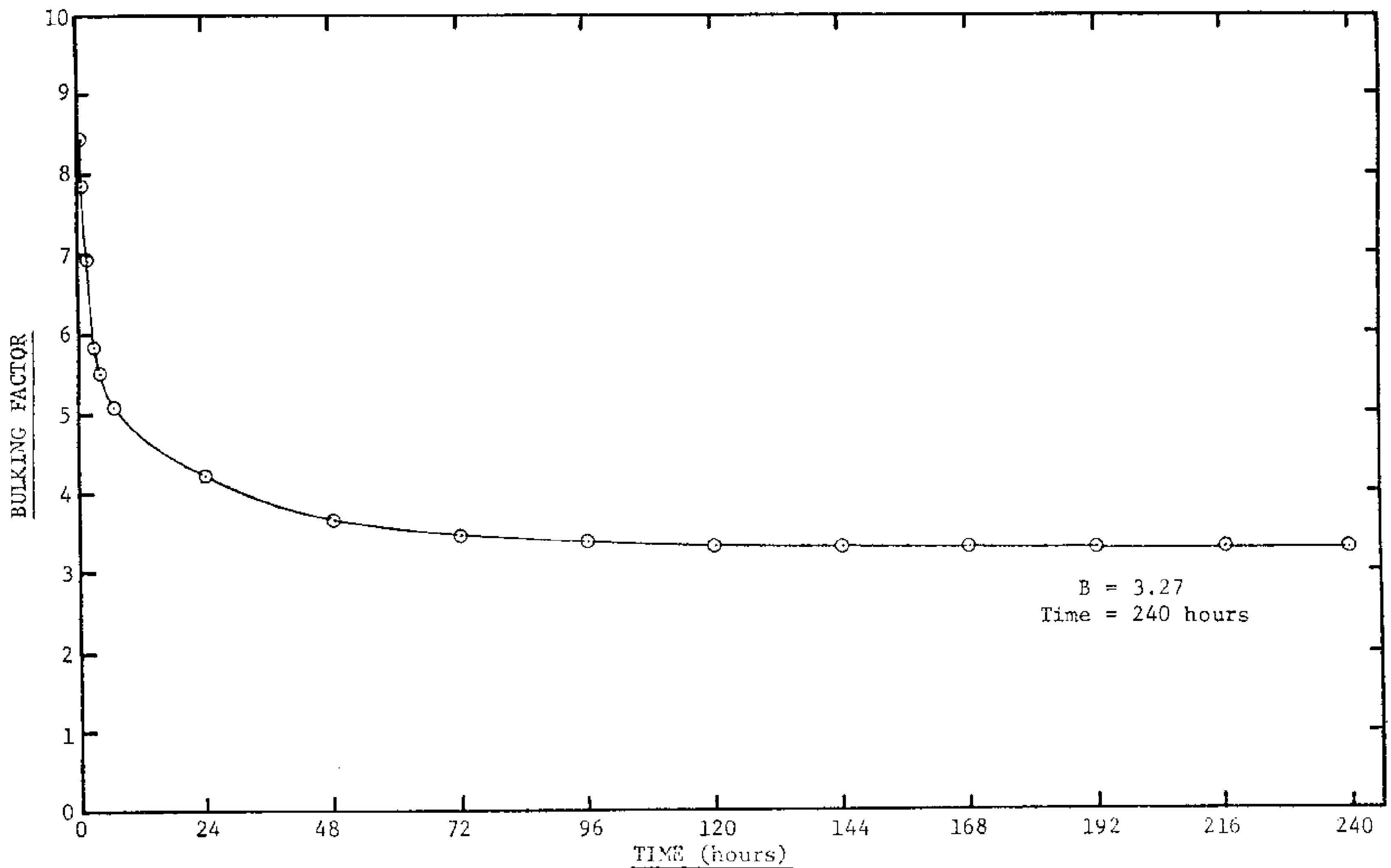

Fig. 17. Bulking Factor as a Function of Time for Sample No. 1766

TABLE VIII

Bulking Factors as a Function of Time

Sample Number	Soil Type+	Bulking Factors (V_c/V_i)	Time++ (hours)
1646	CH	2.94	240
1650	CL	2.15	144
1651	CH	3.48	432
1663	CH	3.36	312
1689	CH	2.67	240
1693	CL	2.28	240
1694	CL	2.76	288
1704	CH	3.63	455
1726	CH	2.98	240
1732	CH	2.67	168
1742	CL	2.23	168
1761	CH	2.83	144
1764	CH	2.64	216
1765	CH	2.92	240
1766	CH	3.27	240
1768	CH	3.12	240
1769	CL	2.98	120
1771	CL	3.32	240
B-38*	CH	3.02	408
B-38**	CH	3.39	408
B-38***	CH	3.81	240

+ Soil Classification: CH - inorganic clay of high plasticity; CL - inorganic clay of low to medium plasticity

++ Time based on settlement rate of 1 ml or less per 48 hour period

* Test conducted in 10 liter cylinder; cylinder diameter = 100 mm

** Test conducted in 1000 ml cylinder; cylinder diameter = 60.5 mm

*** Test conducted in 100 ml cylinder; cylinder diameter = 27.0 mm

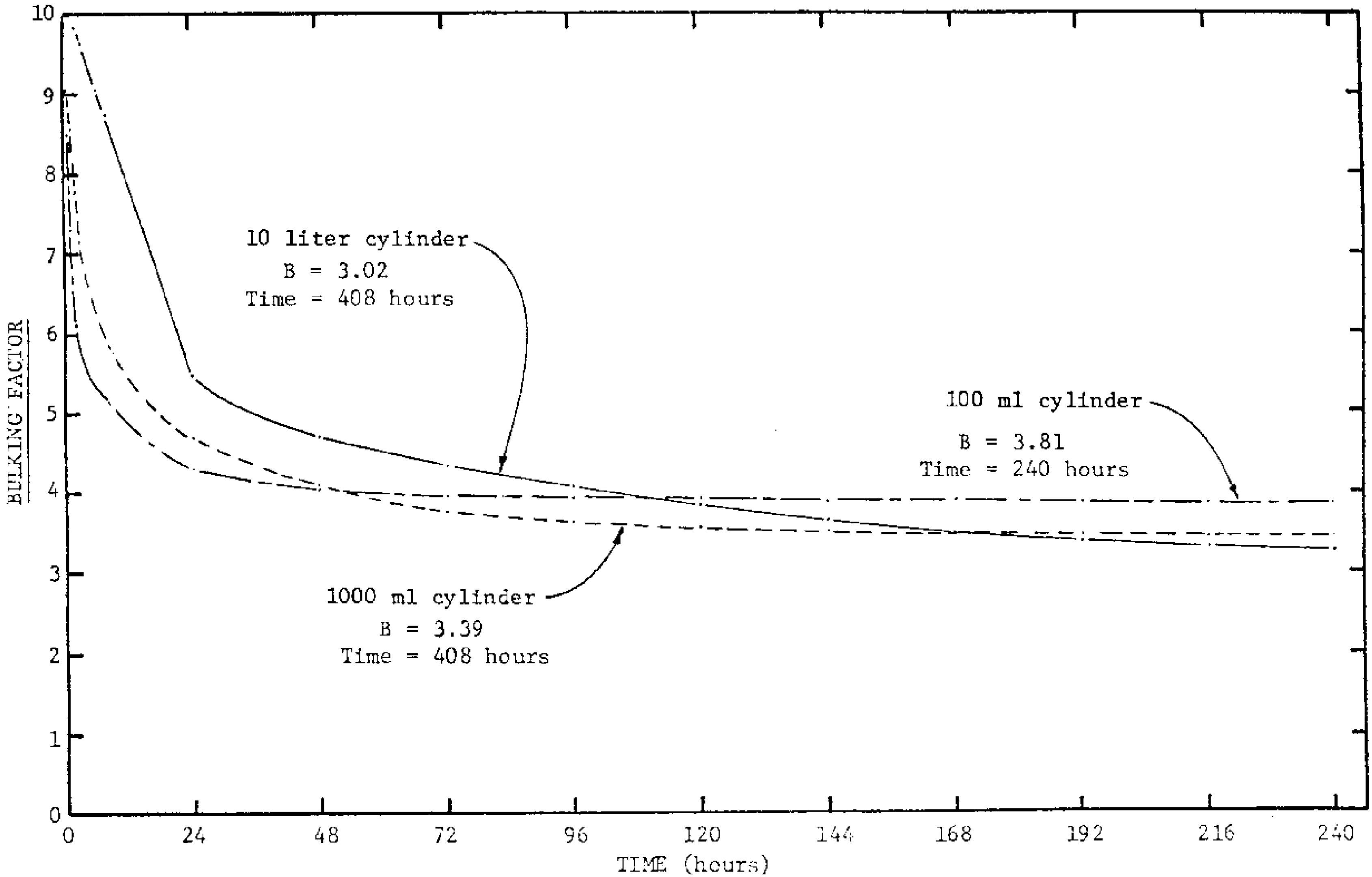

Fig. 18. Influence of Cylinder Size on Bulking Factor for Sample B-38

10 liter cylinder. Although Fig. 18 shows data only to 240 hours of settling time, the tests were actually continued until the bulking factor became constant with time; the final bulking factors are shown in Table VIII. The smallest bulking factor of 3.02 occurred in the 10 liter cylinder, with bulking factors of 3.81 occurring in the 100 ml and 3.39 in the 1000 ml cylinders.

Thus, there is an indication from this sparse data that the height/diameter ratio influences the laboratory determined bulking factor and this substantiates earlier conclusions by Kolessar (18).

An interesting facet of this examination was the establishment of drainage paths in the samples approximately 48 hours after the test started (Fig. 19). The strongest paths were observed at the walls of the containers, although the phenomenon occurred throughout the sample. Fine soil particles were physically carried to the surface by the water flow in the drainage paths, and these were deposited on the surface in the form of mud volcanoes. These mud volcanoes have been observed on a much larger scale by Coleman (1976) in areas of rapid deposition off the mouth of the Mississippi River on side scan sonar and high resolution profile records. They usually occur as fields of mud volcanoes. Apparently, these represent weak spots in the sediment where the excess pore water pressure can vent to the surface. This rapid venting of the pore water will undoubtedly enhance consolidation. If this also occurs in dredged material, then it seems very unlikely that simple one-dimensional consolidation theory can be used to calculate subsequent settlement of dredged spoil. The enhanced drainage will certainly have an effect on the bulking factor and/or sizing index.

Figure 19 - Photograph showing Drainage paths

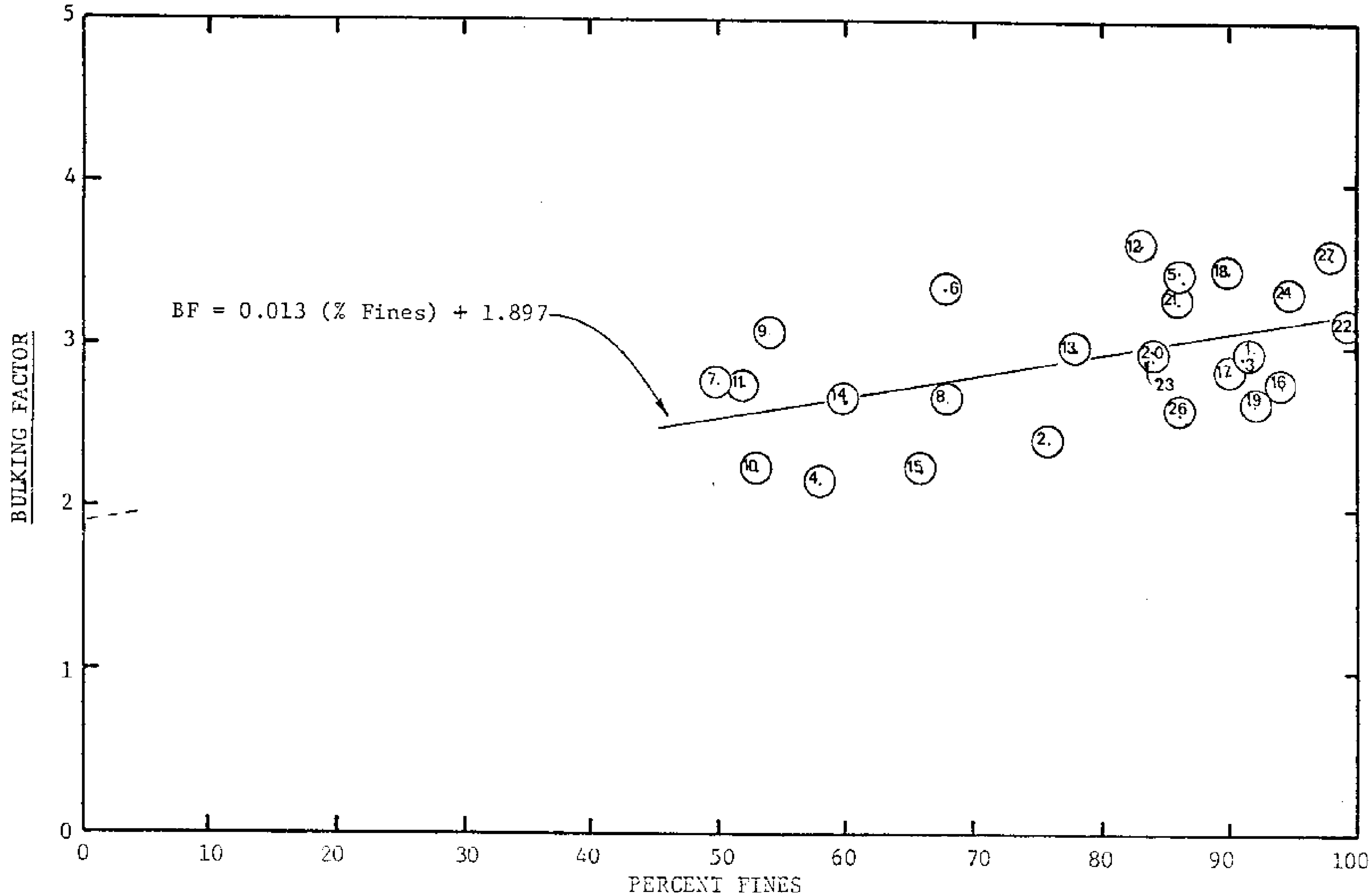

Fig. 20. Bulking Factor as a Function of Percent Soil Particles Passing No. 200 Sieve

Effect of Physical Properties of Soil on Bulking Factor

The data were also examined to determine the relationship between the physical properties of the soil and the bulking factor. The effect of the fines (silt and clay content) is shown in Fig. 20. While there is considerable scatter, a relationship does seem to exist. A linear regression equation was developed which resulted in:

$$\text{Bulking Factor} = 1.897 + 0.013\ (\%\ \text{fines})$$

The correlation coefficient was quite low ($r^2 = 0.25$), due in part to the flatness of the line, but the average error in using this equation is only about 11%, well within any state-of-the-art predictions of sizing factor.

The plasticity of the soil could be expected to influence the bulking factor and this was examined in the same fashion as for the percent fines. Fig. 21 shows the relationship between the liquid limit (LL) and bulking factor. Again, using linear regression, the following equation was obtained:

$$\text{Bulking Factor} = 0.005\ (\text{LL}) + 2.66$$

The correlation coefficient (r^2) for this equation was only 0.05.

A similar attempt was made to relate the plasticity index (PI) to the bulking factor. A plot of the data is shown in Fig. 22. The following equation was obtained from linear regression:

$$\text{Bulking Factor} = 0.0043\ (\text{PI}) + 2.80$$

A correlation coefficient of 0.03 was obtained for this relationship.

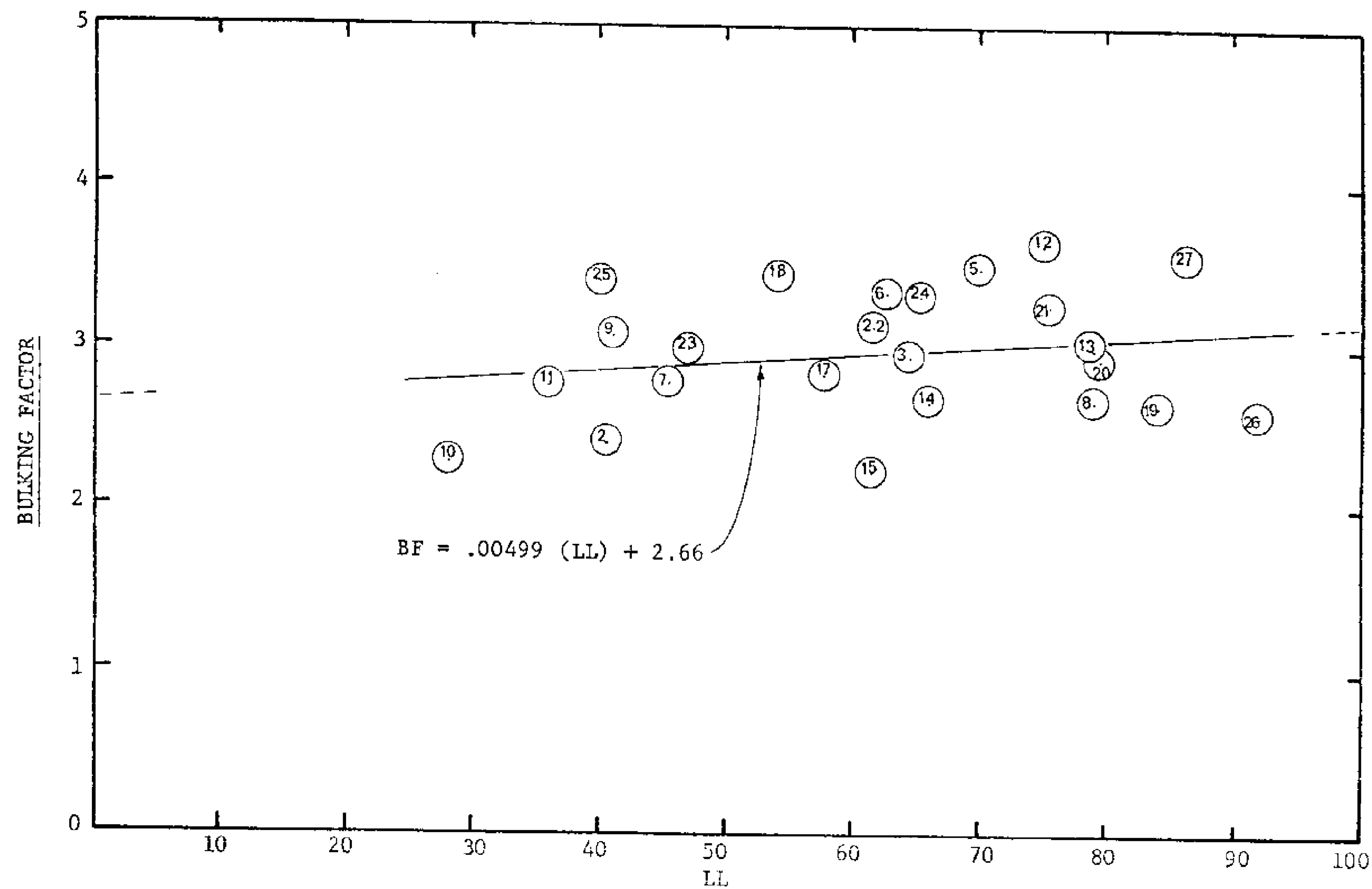

Fig. 21. Bulking Factor as a Function of Liquid Limit

The Liquidity Index, which is defined as:

$$LI = \frac{\text{natural water content - plastic limit}}{\text{plasticity index}},$$

is a measure of the in situ water content with respect to the Atterberg limits. It has been found that liquidity index often correlates well with other geotechnical properties of soils, but a plot of bulking factor versus liquidity index (Fig. 23) indicated only a weak relationship between the two. In this case, the liquidity indices were calculated using the natural water content of the core samples before they were broken down for the test. The relationship found was:

$$\text{Bulking Factor} = 0.31\ (LI) + 2.87$$

with a correlation coefficient of 0.03.

Previous research by Lacasse (6, 10) and Skempton (25) substantiates the thought that bulking factor increases as the Atterberg limits increase, but the results of this investigation can only be described as barely supportive of this.

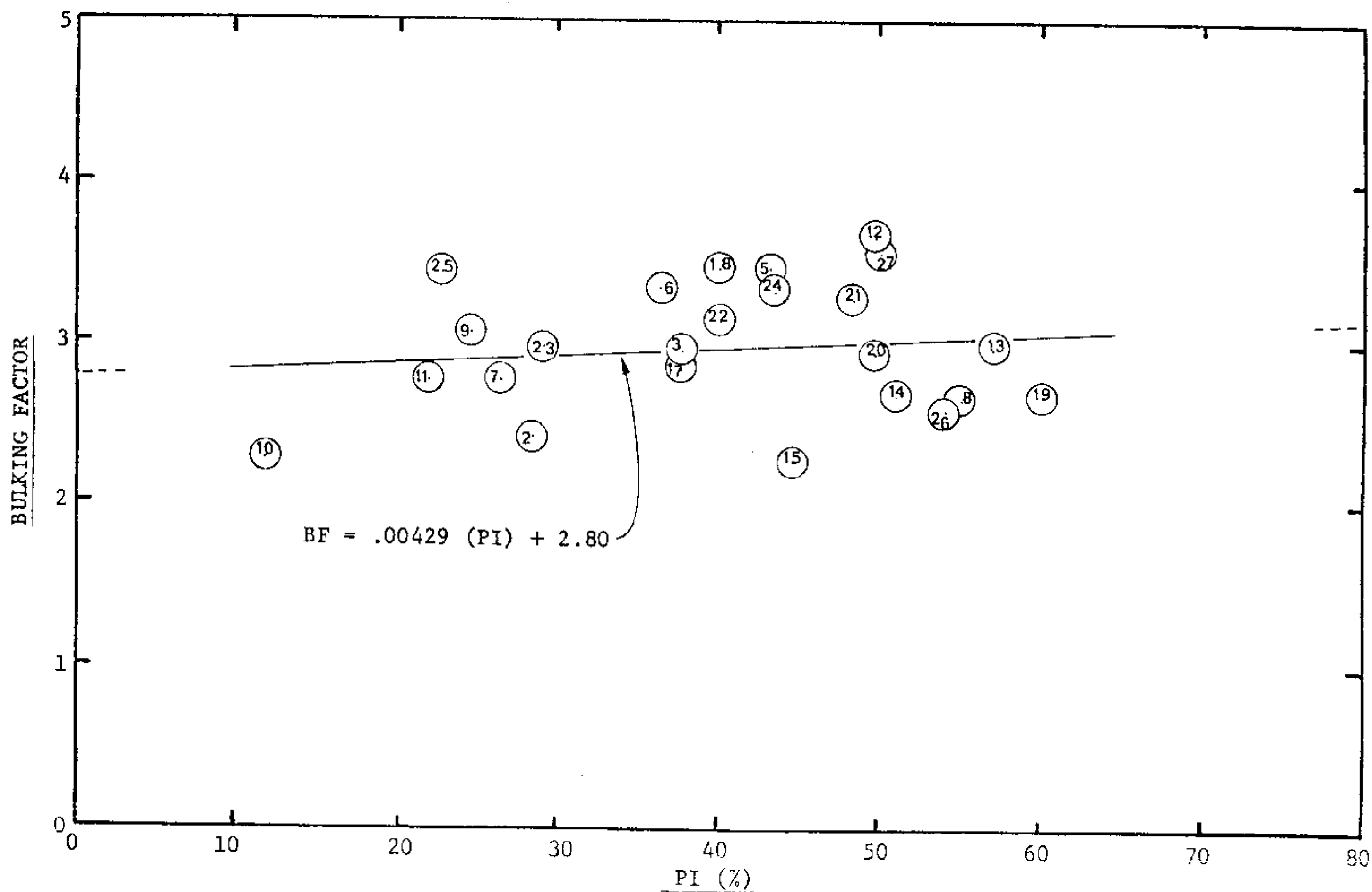

Fig. 22. Bulking Factor as a Function of Plasticity Index

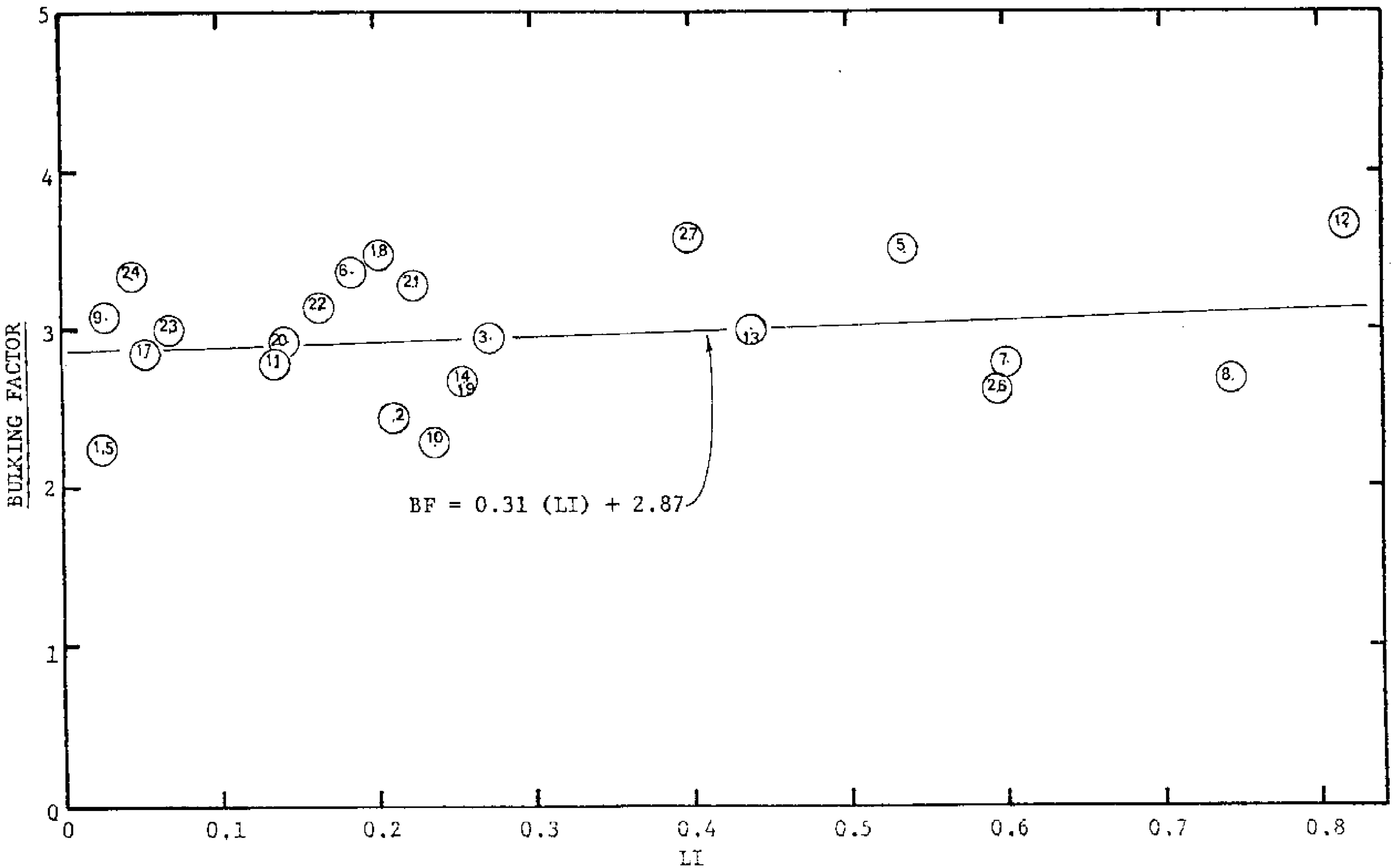

Fig. 23. Bulking Factor as a Function of Liquidity Index

Conclusions

A procedure has been described, and test results presented, for the determination of the bulking factor. It is important to distinguish between the bulking factor and the sizing index. The latter will include long-term settlement, consolidation, dredging efficiency, and other factors which are also needed to size a dredge spoil containment area. Based on the test precedure and results, the following conclusions are warranted:

a. Salinity has a definite effect on the bulking factor. A threshold salinity below which flocculation will not occur was not determined from these tests; however, it seems apparent that in performing bulking factor tests, water of the same salinity as expected in situ should be utilized.

b. The bulking factors obtained by this method appear to be a function of cylinder geometry, but the relationships are not yet developed. Possibly the amount of drainage along the walls of the cylinder is an important geometrical effect.

c. The bulking factors determined were larger than usually experienced. Placement of a large single lift of material rather than smaller multiple lifts may have contributed to the large bulking factors.

d. The bulking factors increased as the fines content (silt and clay) increased.

e. The bulking factors seemed to generally increase as the Atterberg limits increased. The relationship found between Atterberg limits and bulking factor was, however, very weak.

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APPENDIX A.— List of Symbols

The following symbols are used in this paper:

B or BF = bulking factor;

d = cylinder inside diameter;

e = soil void ratio;

e_c = soil void ratio in test cylinder;

e_i = soil void ratio in situ;

Fc = containment area efficiency factor;

Fe = dredge system efficiency factor;

Fo - overdredging factor;

Fp = dredge transport system efficiency factor;

G_s = specific gravity of solids;

H = thickness of sediment layer;

h = initial height of slurry mix when placed in test cylinder;

LI = liquidity index;

LL = liquid limit;

PI = plasticity index;

PL = plastic limit;

S = sizing factor;

s = percent saturation;

t = time;

V_c = soil volume in containment area (cylinder);

V_i = soil volume in situ;

V_s = volume of solids;

V_v = volume of voids;

w_i = percent water content of soil in situ;

w_c = percent water content of soil in containment area (cylinder);

γ_d = dry specific weight of soil;

γ_{sat} = saturated specific weight of soil;

γ_{sub} = submerged specific weight of soil;

γ_w = specific weight of water;

i and c are used as subscripts throughout the paper to indicate in situ and containment area/cylinder quantities, respectively.

APPENDIX B.— Glossary

Bin Volume - the volume of material held in the bin of a hopper dredge during dredging operations.

Bulking Factor - a dimensionless factor expressed by the ratio of the volume of soil in a containment area soon after dredging to that volume of the soil in situ; such variables as dredge system efficiency, long-term settlement and consolidation, drying, and shrinkage are not included in the factor.

Containment Area Efficiency Factor - a dimensionless factor expressed by the ratio of the weight of solids retained in a containment area to the weight of solids initially placed in the containment area through dredging operations. The factor is dependent upon losses of solids over the discharge wier and losses through over-topping of the levees (17).

Dredge System Efficiency Factor - the ratio of the weight of solids taken into the dredge system to that weight of solids removed from in situ. The factor depends on the type of dredge, rate of advance, type of material, and tidal velocities (17).

Flocculation - the process by which soil particles suspended in salt water aggregate and settle out of suspension. This phenomenon is the result of positively charged salt ions changing the surface charge of some of the soil particles from negative to positive. These positively charged particles then aggregate with soil particles retaining negative surface charges, forming soil clusters which rapidly settle out of suspension (6).

Hopper Dredge - a dredge which raises soil particles through use of a dredge pump, and retains the material in hoppers or bins onboard the dredging vessel for eventual discharge at a disposal site (8).

Multiple Lift - the process through which a slurry of predetermined density is placed in a cylinder and allowed to settle. The supernatant liquid is then drawn off and another "lift" of the same density slurry placed on top of the settled material. The process can be repeated, resulting in consolidation of that soil initially placed through the application of pressure caused by the overlying material.

Overdredging Factor - the percentage of material removed in a dredging operation which exceeds the amount prescribed by contract specifications, and for which no payment is made. The factor is dependent upon material properties, dredge operator experience, side slope stability, and other local conditions (17).

Single Lift - the process through which a slurry of predetermined density is placed in a cylinder and allowed to settle. No further material is added, and the calculation of settled volume is based only on the single placement of material.

Sizing Factor - a dimensionless factor expressed by the ratio of the volume of a given soil in a containment area to that volume of the soil in situ. The volume in the containment area is based on the inclusion of such variables as long-term settlement and consolidation, dredge system efficiency, drying and shrinkage, and losses of soil from the containment area (17).

Supernatant Liquid - the relatively clear liquid which remains above the soil-water interface after settlement of the soil particles from a soil-water slurry.

Transport Efficiency Factor - the dimensionless factor expressed by the weight of solids leaving a dredged material transport system (pipeline) divided by the weight of solids entering the system. The factor is dependent upon soil properties and losses through leaks in the system.

HABITAT DEVELOPMENT ALTERNATIVES

TO DREDGED MATERIAL DISPOSAL: A REVIEW

by Hanley K. Smith[1]

ABSTRACT

This paper presents a review of research conducted under the Habitat Development Project (HDP) of the Dredged Material Research Program (DMRP). The DMRP was a five-year, multidisciplinary program, conducted by the U.S. Army Corps of Engineers Waterways Experiment Station. One aspect of that program, the HDP, evaluated the feasibility of using dredged material as a substrate for the development of productive biological communities, and determined the environmental impact of dredged material disposal on wetlands. These studies demonstrated that four basic habitat types are particularly suited for development on dredged material: marsh, islands, upland, and aquatic. Of these four alternatives, marsh, island, and upland habitat development are considered feasible and have been demonstrated successfully at several sites. Aquatic habitat development, although promising, has not yet been fully tested.

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WEIR DESIGN FOR DREDGINGS CONTAINMENT AREAS

by

D. K. Atmatzidis[1] , G. M. Karadi[2] , and B. J. Gallagher[3]

ABSTRACT

The vast majority of dredged material containment areas are equipped with some kind of sluicing device through which supernatants are discharged. The sedimentation regime that exists in a containment area has a significant effect on the amount of suspended solids in the discharged supernatants. Thus, any type of discharge device or structure should serve the dual purpose of (a) controlling flow and (b) improving, or at least maintaining, the settling effectiveness of the area and the quality of the discharged supernatants. However, the design of containment area overflow weirs is highly empirical, and pertinent guidelines are virtually non-existent.

Adequate weir design for a dredgings disposal area necessitates the selection of the type and location of the weir and the computation of the minimum allowable ponding depth of water in the vicinity of the weir, the length of the crest, and the head of water over the weir. Through a comprehensive literature review, available design formulations and methodologies were identified, which are directly applicable to conditions prevailing in containment areas. Based on these formulations and methods, a simple set of recommendations and guidelines was developed which can be used with confidence for the design of weirs as components of dredged material containment areas.

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CURRENT PRACTICE

Ten Corps of Engineers Districts, with active dredgings disposal operations were visited, and discussions were held with appropriate personnel on the problems and needs of present and future disposal operations (Gallagher et al.,1978). The information obtained on current practices for the design of outlet structures (weirs) is summarized in Table 1. A large variety of weirs are presently in operation, including standard Armco type weirs, usually 6-foot wide, rectangular drop inlet structures of various sizes, and large polygonal weirs. Flashboards are frequently used to control or establish a crest elevation, and a specified maximum head of water over the weir is used as control parameter for the design and operation of weirs by two Districts.

Weirs are mainly designed as flow discharge (control) structures, and no consideration is given to their effect on the settling and solids retention effectiveness of the disposal area. Weirs are frequently located as close to the receiving water body as possible. Since economic considerations dictate the utilization of the shortest possible inflow pipe, both the inlet and outlet points of many disposal areas are located on the side of the area closest to the free water body (river, lake, or sea). Under such conditions, flow concentrations and short circuiting occur, and the surface area of the settling basin is not utilized effectively.

The ponding depth of water in containment areas in the vicinity of a weir is often kept at a minimum in order to reduce the load on containment dikes, which are not designed properly as earth and water retaining structures. This practice, however, can cause scour and resuspension of bottom sediments, which may have a detrimental effect on the quality of the effluents.

Table 1

Weirs in Dredgings Containment Areas

District	Weir Description
Philadelphia	Large, 16 feet by 20 feet, steel polygonal weirs perpendicular to the dikes are now used to replace old, wooden, box weirs.
Mobile	Rectangular steel "boxes" 8 feet long and 4 ft wide with removable flashboards; located in dikes
Galveston	Large drop inlet box structures located about 30 feet from the dike inside the disposal area.
Portland	Large variety; from small, Armco-type, flashboard weirs to large steel frame weirs; 3 inches maximum head over weir crest.
Seattle	Large variety; from small box structures to very large square drop inlets located inside the disposal area; 4 inches maximum head over weir crest.
Norfolk	Prefabricated steel weirs, crest length 14 feet to 28 feet; some large E-shaped weirs with very long crest (about 300 feet).
Baltimore	Three-sided, concrete wall weirs with a total crest length of 12 feet.
Charleston	Standard 6-foot wide Armco-type weirs arranged in pairs.
Savannah	Standard 6-foot wide weirs with two sets of 3-foot wide flashboards, more than one per disposal area
Vicksburg	Standard 100-foot wide, rectangular, internal weirs with fixed crest elevation; drop inlet structures in final ponds.

WEIR PARAMETERS AND SETTLING EFFECTIVENESS

In developing guidelines for the design of weirs, the qualitative and quantitative relationships between weir parameters and factors controlling settling effectiveness must be considered. In general, the ideal settling conditions in a sedimentation basin are hampered by a number of factors which include (a) the physical and chemical characteristics of the suspension, (b) short circuiting, (c) resuspension of sediment, and (d) nonuniform deposition of sediment. With the exception of the first factor, all others are influenced to a variable extent by the outlet structure (overflow weir).

The effect of short circuiting becomes increasingly dominant as the inlet velocities increase; under such conditions density currents occur, the material mixes within the pond, and concentration of flow develops. Experiments performed on settling tanks of various shapes (Marske and Boyle, 1973) indicate a close relationship between tank shape and settling effectiveness. If the outlet weir is contracted, the flow approaching the weir will concentrate, and depending on the degree of contraction (type and physical dimensions of weir), dead zones of considerable extent will develop; this situation will, in turn, increase short-circuiting and decrease settling effectiveness. Location of the outlet weir with respect to the inlet structure has an even more significant impact on short-circuiting, as well as settling effectiveness, and constitutes an important design criterion.

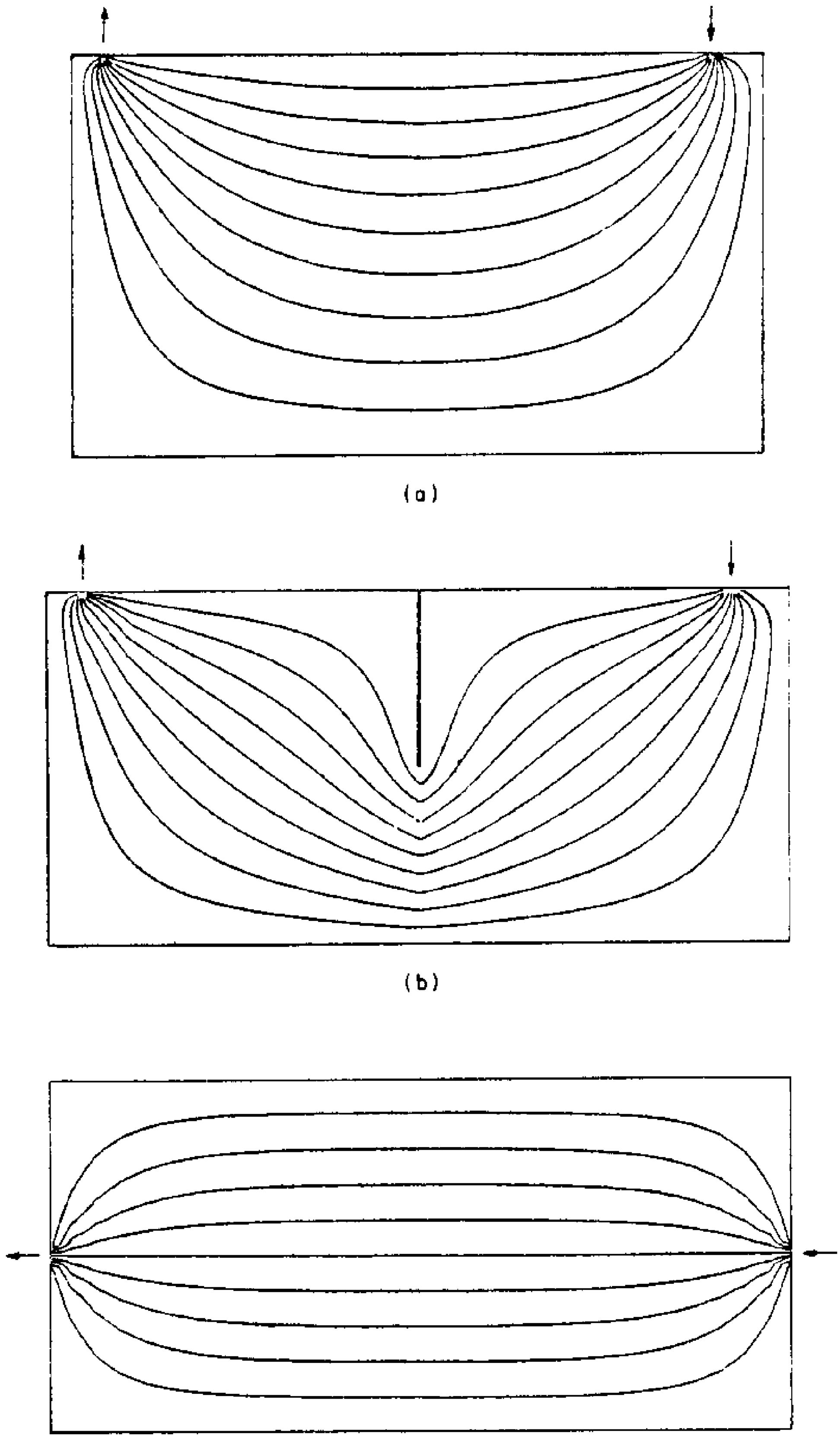
Resuspension of sediment is a major factor resulting in reduced settling effectiveness. Presently available approaches to determine the conditions favorable for resuspension are not conducive to the development of quantitative criteria (Gallagher et al., 1978). However, qualitative analyses indicate that areas of flow concentration will result in bottom scouring (i.e., resuspension of sediment). Since the outlet structures used in dredged material disposal sites are usually contracted weirs, flow concentration of varying degree should be expected. Unless flow concentration is held to a practical minimum, resuspension of sediment by flow approaching the weir will occur, with an undesirable deterioration in effluent quality. A similar situation arises if extensive short-circuiting develops (this is strongly influenced by the physical size and location of the outlet weir).

The size and location of a weir have an indirect influence on the uniformity of sediment deposition. Insufficient weir size and/or improper weir location give rise to short-circuiting and flow channelization and, as a result, to undesirable non-uniform sediment deposition.

WEIR LOCATION

Since dredgings containment areas operate as crude settling basins, their effectiveness is directly influenced by the prevailing flow pattern. Accordingly, the proper design of a disposal area requires, among other things, a knowledge of the flow pattern between inflow and outflow points. Proper location of the overflow weir or weirs with respect to the inflow pipe can reduce short-circuiting of the flow and increase the effective surface area of the settling basin. A hydrodynamical model was developed (Gallagher et al.,1978) to predict the flow field and retention time in a sedimentation basin and was applied to the study of hydraulic efficiency of dredgings containment areas. Pertinent conclusions, obtained from this study, are presented and discussed herein.

Confined disposal frequently takes place with both the inflow pipe and the outflow weir located on the same side of the disposal area. Such a configuration and the associated flow pattern are shown in Figure 1a. It can



(a)

(b)

(c)

Figure 1. Flow Patterns for Containment Areas with Different Configurations.

be observed that the density of streamlines is higher near the side of the area where the inlet and outlet are located. At the opposite side, and especially near the corners, waters are nearly stagnant; consequently, the effective surface area of the basin is substantially reduced with respect to ideal plug flow. These adverse effects are even more pronounced when both inlet and outlet points are located on one of the shorter sides of the area. Consideration has recently been given to the use of spur dikes to improve hydraulic conditions in a disposal area. For a single spur dike (Figure 1b) it has been determined that (a) shortcircuiting is generally reduced, (b) there is no noticeable improvement in the active surface area, and (c) location of a spur dike close to the overflow weir has an adverse effect on the solids removal efficiency of the basin because the conditions for sediment resuspension in the vicinity of the weir are enhanced. Multiple spur dikes can serve to increase retention time and minimize short-circuiting.

Location of the inlet and outlet on the opposite, shorter sides of the disposal area (Figure 1c) unavoidably results in some short-circuiting of the flow and reduction of the effective surface area of the basin. Variation of the weir length has some effect on the flow pattern, but this is pronounced only in the vicinity of the weir. This effect would be significant for areas with a high width-to-length ratio. The disadvantages of short weir lengths are that (a) inactive surface area develops at the corners of the basin on the side of the weir and (b) flow velocities in the vicinity of the weir are high, and this may give rise to resuspension of bottom sediments. The effect of multiple weirs on the flow pattern is not significantly stronger than the effect of a single weir of the same total crest length. Multiple weirs would be preferable in the case of areas with high width-to-length ratios.

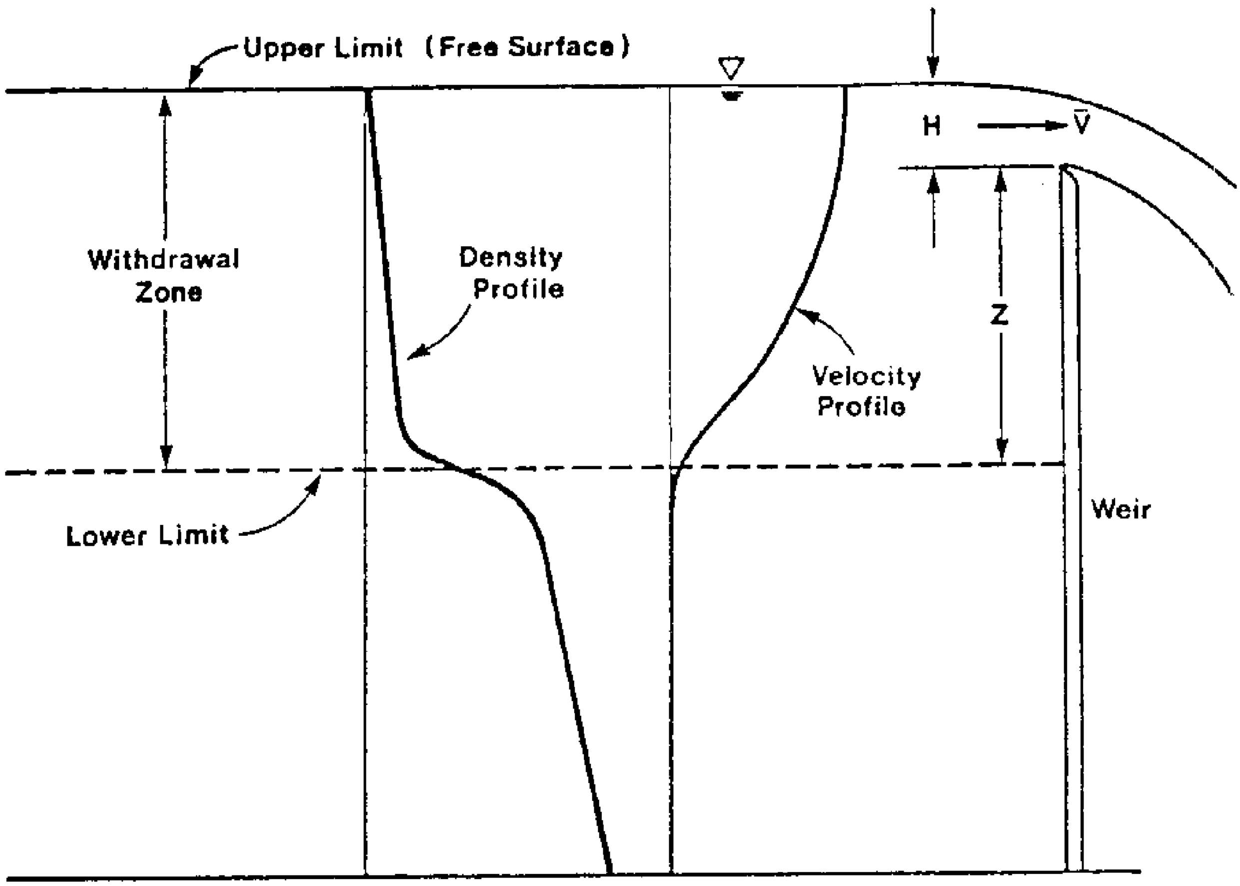


Figure 2. Withdrawal Zone and Fluid Density Profile

An extensive literature review (Gallagher et al.,1978) indicates that the withdrawal zone characteristics under the conditions and assumptions stated above can best be described by the following relationship (Bohan and Grace, 1973):

$$V = 0.32 \left(\frac{Z + H}{H}\right) \sqrt{\frac{\Delta\rho_w}{\rho_w} g\ Z} \qquad (1)$$

where V is the average velocity over the weir (in ft/sec), Z is the vertical distance from the elevation of the weir crest to the lower limit of the zone of withdrawal (in ft), H is the head on the weir for free flow (in ft), $\Delta\rho_w$ is the density difference of the fluid between the elevations of the weir crest and the lower limit of the zone of withdrawal (in g/cm^3), ρ_w is the density of the fluid at the elevation of the weir crest (in g/cm^3), and g is the acceleration of gravity (in ft/sec^2). The head over the weir

PONDING DEPTH

Scour and resuspension of bottom sediments have a detrimental effect on the performance of any sedimentation basin and on the quality of the effluents. In the vicinity of a weir, flow usually contracts considerably, and approach velocities increase to levels much higher than those existing in the rest of the flow domain. Since the possibility of bottom sediment resuspension is enhanced in the vicinity of a disposal area weir, a sufficient ponding depth should be maintained to avoid resuspension. However, no simple methodology is presently available which can be applied to determine the safe ponding depth in the vicinity of a disposal area weir.

To prevent resuspension of bottom sediments and achieve effluent quality control in dredgings containment areas, selective withdrawal principles may be applied and available formulations may be adapted to fit the conditions of withdrawal over a weir. Furthermore, since the density variation in the vertical direction is unknown, a simplifying assumption can be introduced. The fluid is assumed to consist of two layers, the upper layer being acceptable for discharge while the lower layer is not; the boundary between these layers can be defined as the level where the fluid density increases (due primarily to suspended solids) to levels higher than those dictated by the acceptable effluent quality standards. Alternatively, this boundary can be considered as the level below which the flow velocities are minimal, if not zero, so that scouring or resuspension of bottom deposits due to turbulent eddies does not occur. Shown schematically in Figure 2 are the widthdrawal zone and flow characteristics in the vicinity of a free-flow, sharp-crested, rectangular weir.

should be measured at a sufficient distance upstream from the weir crest to assure that the measurement is beyond the zone of appreciable surface curvature (drawdown). For an outlet weir, however, it is more practical to measure the water depth above the weir crest (i.e., in the zone of surface curvature). Hence, the measured depth is smaller than the value of the head to be substituted in Equation 1; for the practical range of values for H, this effect is insignificant. According to the original studies by Rehbock (1929), which were subsequently supported by several investigators, the relationship between the head, H, and the depth of water over the weir, h, is

$$H = 1.18\ h \qquad (2)$$

and the measured value of h has to be adjusted to obtain H.

To obtain an understanding of the relations between the parameters involved in Equation 1, the case where the density difference decreases by an order of magnitude can be considered. Since the velocity and head over the weir are interrelated parameters, the depth of the withdrawal zone would increase by a factor of about two for constant velocity and head values, and this could result in a change in the effluent quality. Although the density difference enters Equation 1 as a square root and its variations do not cause equal-magnitude variations of other parameters, it becomes apparent from the above simple presentation that the density profile and its variations should be well known for successful application of Equation 1.

To apply the principle of selective withdrawal to dredged material disposal areas, the density profile of the waters in the vicinity of the weir should be known. However, specific density information is usually not available and values can only be approximated from available data and experience. The density of predominantly fine-grained dredged material in disposal areas is reported to range from about 1.40 g/cm^3 to 1.65 g/cm^3 (Krizek and Salem, 1974; Lacasse, 1977). Krizek, Roderick, and Jin (1974) report densities of about 1.20 g/cm^3 for freshly deposited dredged material during laboratory quiescent settling tests; these tests were conducted on samples of dredged bottom sediments which were not fractionated to separate the clay and silt portion from the coarser material. However, waters approaching a weir carry only the finer portion of the dredged material slurry, and this material would be expected to have an even lower density when freshly deposited. Over such a freshly deposited layer in a sedimentation basin, there exists another layer where grains are still settling from suspension, but the density (or the concentration of suspended solids) of this layer would be higher than the average density of the overlying water. The removal efficiency of disposal areas acting as sedimentation basins ranges from very poor to excellent, but values lower than 90 percent should be seldom encountered when sites are properly designed and managed. Considering that the concentration of suspended solids in the influent slurry does not exceed 25 percent by weight, then the amount of suspended solids in the waters approaching a weir should not exceed 2.5 percent by weight (about 25 g/l or a density of 1.016 g/cm^3) and would often be much lower. Recent samples collected 3 to 5 feet below water surface near the weir of an active disposal site indicated densities ranging from about 1.005 to 1.05 g/cm^3 (Gallagher et al., 1978).

According to the limited information presented above, the density profile shown in Figure 3 appears to be characteristic of an average disposal site. On the basis of this density profile, selective withdrawal principles can be applied (a) to withdraw waters with acceptable quality and (b) to avoid resuspension or scour of bottom sediments. To achieve this effect, the withdrawal zone should be located well within the upper layer of water and flow velocities should be minimal, if not zero, in the second water layer. The following approach is suggested to accomplish this objective,

Water Layer of Variable Solids Concentration	ρ = 1.00 to 1.02 g/cm^3
Water Layer of High Solids Concentration	ρ = 1.02 to 1.10 g/cm^3
Fresh Sediments	ρ = 1.10 to 1.20 g/cm^3
Old Sediments	ρ = 1.20 to 1.65 g/cm^3

Figure 3. Typical Density Profile in the Vicinity of a Weir

and Figure 4 has been prepared to aid in applying this procedure:

a. Determine the flow velocity, V, and head, h, over the weir according to an accepted design procedure; note that there is only one value of the velocity, V, corresponding to a given head, h.

b. Use a density difference, $\Delta\rho_w$, and determine the depth, Z, of the withdrawal zone according to Equation 1.

RECTANGULAR WEIRS

Rectangular weirs are the most common outlet structures and are characterized as (a) sharp-crested or broad-crested, depending on the thickness of their cross-section, (b) with or without side contractions, depending on the ratio of weir length to channel width, and (c) free or suppressed weir depending on the level of the downstream water body.

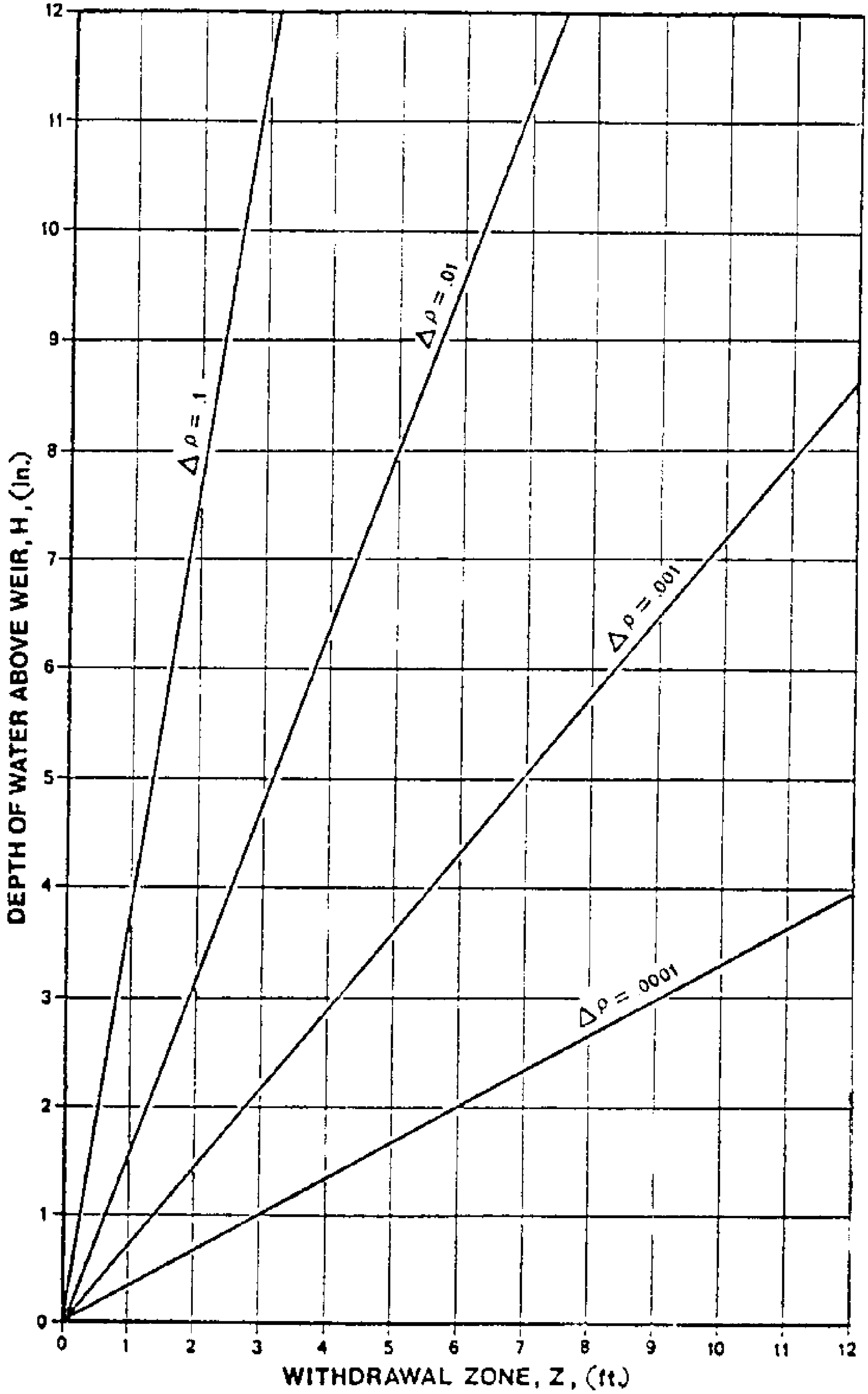


Figure 4. Relationship between Depth of Withdrawal Zone and Head over Weir for Various Density Differences

Sharp-crested weirs have the cross-section of a thin plate; weirs without contraction have lengths equal to the channel width; and for free-flow weirs the downstream water level is lower than the weir crest elevation and does not affect flow rates.

The most widely used relationship for the determination of flow discharge over a sharp-crested rectangular weir is based on the application of Bernoulli's law and assumes free-flow conditions. After accounting for local head losses, this relationship takes the form

$$Q = \frac{2}{3} \sqrt{2g}\ C_c\ L \left[\left(H + \frac{V^2}{2g} \right)^{3/2} - \left(\frac{V^2}{2g} \right)^{3/2} \right] \qquad (3)$$

where Q is the flow rate, L is the length of the weir, H is the head over the crest, g is the acceleration of gravity, V is the approach velocity, and C_c is the coefficient of contraction.

The effect of both the approach velocity, V and the contraction coefficient, C_c, may be represented by a single coefficient, C_D, such that

$$Q = \frac{2}{3} \sqrt{2g}\ C_D\ L\ H^{3/2} \qquad (4)$$

which, by incorporating the constant $\frac{2}{3} \sqrt{2g}$, becomes the coefficient C_D'

$$Q = C_D'\ L\ H^{3/2} \qquad (5)$$

where $C_D' = \frac{2}{3} \sqrt{2g}\ C_D$ is the overflow coefficient. The measurement of head, H, must be made a sufficient distance upstream from the crest to be beyond the zone of appreciable drawdown. If it is more practical to measure the head above the crest, h, adjustment has to be made for the effect of drawdown according to Equation 2.

The overflow and discharge coefficients, C_D' and C_D are dependent on (a) the relative dimensions (geometry) of the weir, (b) the height of the weir crest above the bottom, (c) the side contraction, (d) the approach velocity, (e) the direction of approach, and (f) the head over the weir crest. One of the best known formulae for calculating C_D' was advanced by Rehbock (1929) (in British units):

$$C_D' = \left(3.24 + 0.43\,\frac{H}{P} + \frac{0.018}{H}\right) \qquad (6)$$

where H is the head over the crest and P is the height of the weir or the ponding depth in the vicinity of the weir. More recently an extensive study was undertaken by Kindsvater and Carter (1959) who introduced the concepts of effective weir length, L_e, and effective head, H_e, and proposed the following general formula to calculate the flow rate, Q:

$$Q = C_D'\, L_e\, H_e^{1.5} \qquad (7)$$

where $H_e = H + 0.003$ ft and $L_e = L + k_b$. Nomographs were provided for the determination of values for K_b and C_D'.

It is evident that for small values of H/P, the various formulae show very small disagreement but for H/P > 1 the deviations become increasingly more significant. Furthermore, the extensive studies (Rouse, 1949) have shown that the effect of side contraction on the discharge coefficient is insignificant if the head over the crest, H, does not exceed one-third of the crest length, L.

The discharge coefficients of sharp-crested weirs are affected by the angle that the weir makes with the vertical (Starasolszky, 1970). If the weir tilts toward the downstream side, the effect is favorable for angles up to 70°. However, the flow rate over the weir increases only by about 10 percent for the most favorable conditions (40° to 60°

angle with the vertical). The quantity of flow over a weir is also affected by the direction of the approach velocity. This effect becomes significant only for angles smaller than 30° between the weir crest and the direction of the approach velocity (Kiselev, 1950).

To simplify the design of rectangular, sharp-crested weirs for dredgings containment areas the following assumptions can be introduced: (a) the direction of the approach velocity is perpendicular or nearly perpendicular to the direction of the weir crest; (b) weirs have a vertical upstream face; (c) the head over the weir, H, is not more than a few inches; (d) the height of the weir or the ponding depth, P, is not less than one foot. With the exception of overflow structures with very long crests, weirs in disposal areas should be considered contracted. The coefficient of contraction is a function of the ratio of the head over the weir, H, to the height of weir, P, and the ratio of the weir crest length to the width of the flow channel. When realistic values are assigned to the ratio H/P (not more than 0.5 and often less than 0.1) the required correction for the flow rate is negligible for all practical purposes. To aid in the design of weirs for dredgings containment areas, Figure 5 was prepared using the Kindsvater-Carter (1959) formulations.

Most rectangular weirs used in sedimentation ponds can be considered as sharp-crested weirs, despite the fact that they are, in effect, narrow-crested weirs. This assumption causes some error in the calculations, but it is insignificant, because outlet weirs are used as flow control rather than flow-measuring devices. For this reason, Figure 5 can be used also for narrow-crested weirs. The majority of dredged material confinement areas are expected to have either sharp-crested or narrow-crested weirs,

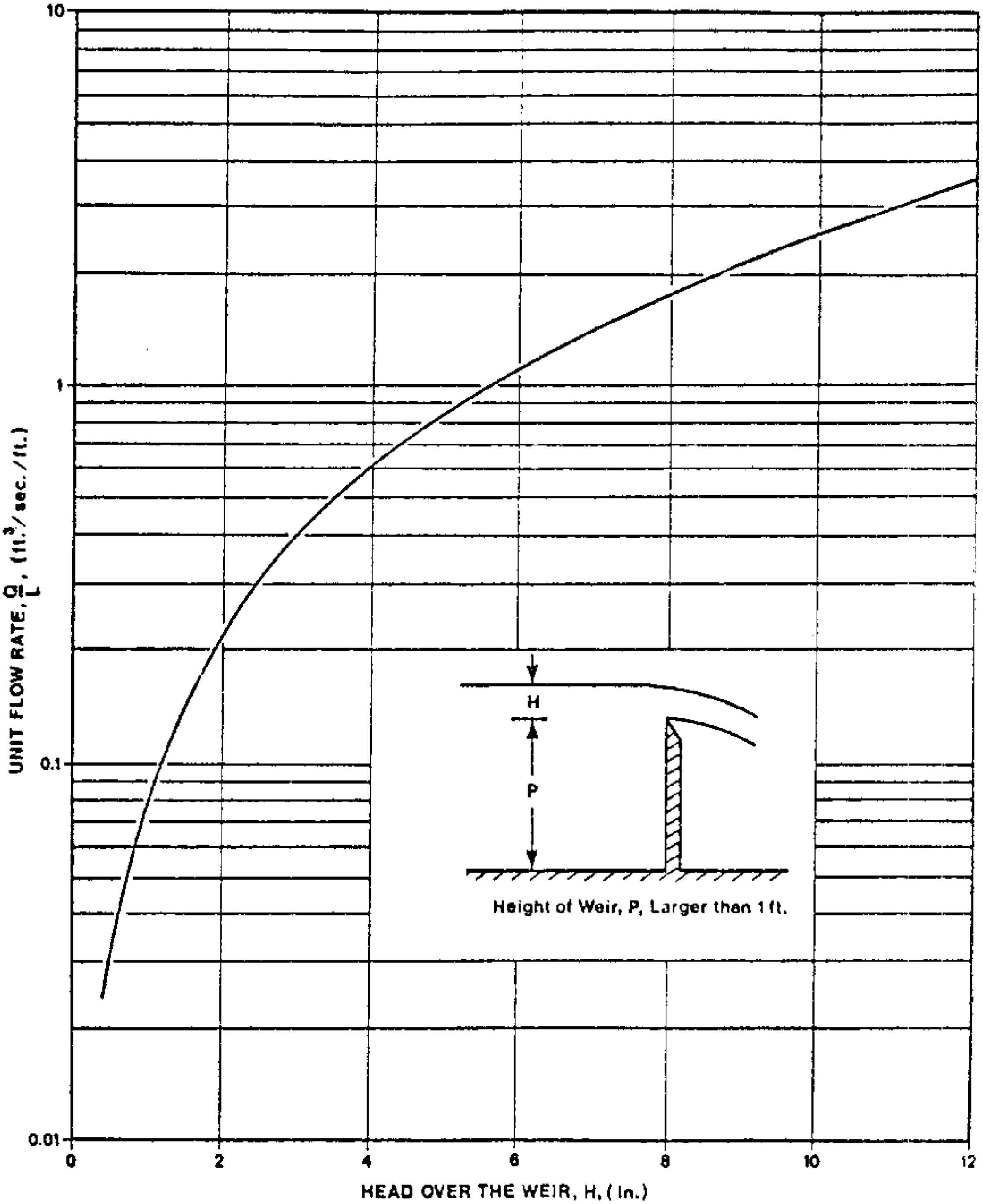


Figure 5. Relationship between Head and Height of Weir and Unit Flow Rate

and this figure can be directly applied for their design.

Observation of Figure 5 indicates that the head, H, over the weir affects substantially the flow rate and care should be exercised when selecting a value for the head, H, during the stages of a design. If, for instance, the head over the weir is limited to two inches, the required weir length will be equal to the ratio of the total flow rate divided by the weir loading (flow rate per unit length), obtained from Figure 5. The relative simplicity of this approach explains its popularity, but it has serious limitations. The weir loading itself does not have a direct effect on the settling effectiveness. Furthermore, the height of the weir (ponding depth) is also a significant factor that influences the flow rate. If this height is very small, the approach velocity to the weir will be high, and resuspension of sediment near the weir will occur. Thus, a two-inch head limit may be insufficient if the height of the weir is too small, while a higher head can be accepted (with a corresponding decrease in weir length) if the height of the weir is sufficient.

POLYGONAL WEIRS

Weirs of polygonal shape include square intake towers, labyrinth weirs, duck-bill over-falls, etc., and are characterized by a broken axis (crest) in plan. The purpose of polygonal weirs is to increase the active weir length (length of crest), thus making it possible to increase the discharge per unit length of structure for a given head. Such an arrangement is advantageous if the available width and the head over the weir are limited. Several authors (Aichel, 1907; Escande and Sabathe, 1937; Gentilini, 1941; Kozák and Sváb, 1961; Hay and Taylor, 1970; Darvas, 1971; Indlekofer and Rouve, 1975) have studied the capacity of various polygonal overflow structures. The most comprehensive analysis was performed by

Hay and Taylor (1970), and recently Indlekofer and Rouve (1975) studied the effect of corners on the discharge capacity of weirs.

Hay and Taylor (1969) developed a computer program for the analysis of labyrinth weirs and substantiated the validity of this program with laboratory tests. The performance of labyrinth weirs was evaluated by direct comparison of labyrinth weir flows, Q_L, with the corresponding sharp-crested linear weir flows, Q_N. This method of analysis is dependent on the accurate knowledge of Q_N. Hay and Taylor (1969) used the formula proposed by Kindvater and Carter (1959) (see Equation 7) and used a C_D' value of $3.22 + 0.4 \frac{H}{P}$, where H and P are the measured head and weir crest height corresponding to the labyrinth weir discharge, Q_L. Their results clearly indicate that, for small $\frac{H}{P}$ ratios, the increase of weir length by applying a polygonal arrangement will result in an almost proportional increase in the flow rate, as compared to a regular sharp-crested weir. For instance, for a crest-length magnification of 8, the value of Q_L/Q_N decreased from its initial value of 8 to 4 as H/P increases from 0 to 0.5.

While the concept of polygonal weirs may be useful in the design of large spillways, it can not be considered advantageous for dredged material disposal areas. A decrease in the weir loading by utilizing polygonal shapes can not and does not improve settling effectiveness. Consider, for example, the case where the rectangular weir of a disposal area is replaced by a polygonal weir of the same effective crest length but contracted to 1/3 of the actual width of the rectangular weir. Assuming that the head in both cases is small relative to the height of the weir, both outlet structures will carry the same total discharge at the same head over the weir. The cross-section of the flow toward the weir is significantly smaller for the polygonal structure than for the rectangular structure, and therefore, the approach velocity for the former will be three times that of the latter. The higher velocity associated with the polygonal structure would then create favorable conditions for sediment resuspension or for the development of short-circuiting. Hence, polygonal weirs (as compared to rectangular weirs) have an adverse effect on the settling effectiveness of sedimentation basins. This implies that a limitation on the head over a weir may not be an effective criterion to guarantee maximum settling effectiveness and acceptable effluent quality. It is clear that, under identical hydraulic conditions and sediment load, a long rectangular weir will result in a significantly higher efficiency than a polygonal weir of the same effective length.

SHAFT TYPE WEIRS

In shaft type weirs, the water flows over a circular or rectangular crest and discharges down a shaft. Calculation of the discharge capacity of a shaft spillway is based on the same principles used for sharp-crested rectangular weirs. The flow rate may be calculated from Equation 7, where $L_e = 2\pi r$ for a circular shaft (r is the radius) and $L_e = 2a + 2b$ for a rectangular shaft (a and b are side lengths of the rectangular cross section.) Values for the overflow coefficient, C_D', can be obtained from charts which were specifically developed for the design of shaft-type weirs (Davis and Sorensen, 1969) or can be assumed to be those given for sharp-crested weirs.

Shaft-type weirs, such as box weirs and riser pipes are used frequently in dredged material containment area operations. Standard box weirs do not appear to be very effective in improving settling effectiveness for reasons identical to those advanced for polygonal weirs. Box weirs function as point sinks and force flow lines to concentrate in the area of the weir; the approach velocities are considerably increased and favorable conditions for short-circuiting and sediment resuspension are developed. Hence, compared to rectangular weirs, shaft-type weirs appear to be inferior as far as settling effectiveness is concerned. This problem can be overcome by using perforated riser pipes with plastic filter cloth wrapped around the riser. The technique has been utilized and proven effective in containment basins for small dredgings operations. The disadvantage of this technique is that the filter cloth clogs with fine silty material, the outflow riser acts as a simple nonperforated riser (shaft), the water level in the pond rises above the top of the pipe, and the safety of the dikes may be jeopardized. Shaft-type sluices can be very effective when used in a final catch basin separated from the main basin by a long rectangular weir of fixed crest height. This configuration will provide an additional settling area if heavy solids are released over the rectangular weir, as might occur when the main basin is almost filled.

DESIGN PROCEDURES AND GUIDELINES

The most suitable outflow structure for dredged material containment areas appears to be a rectangular, sharp-crested, free-flow weir. To maximize settling effectiveness, weir crests should be long and the head over the weir should be small. In the vicinity of the weir, the ponding depth of the water should be such that resuspension of bottom sediments is avoided and withdrawal of waters of acceptable quality is facilitated. The overflow weir should be strategically located with respect to the inflow pipe in order to decrease short-circuiting and maximize the length of flow paths and the effective surface area of the sedimentation basin. On the basis of the foregoing information and discussions the following procedures and guidelines can be advanced for the design of rectangular sharp-crested weirs for dredged material containment areas.

Inflow and outflow location

a. If the area is an elongated rectangle and no spur dikes are used, locate the inflow and outflow structures in the middle of the opposite, shorter, sides of the area, or along a diagonal between corners, if possible.

b. If the inflow and outflow must be located on the same side of the containment area, the distance between them should be maximized and one or more odd numbered spur dikes should be constructed between them.

c. It is preferable to utilize a single sharp-crested weir with sufficient length to prevent concentration of flow and increased approach velocities. If this is not possible, then a number of smaller weirs with the same total crest length can be used as an alternate.

Overflow weir

a. The overflow structure should be a rectangular, sharp-crested, free-flow weir, constructed in or near the dike, with its length parallel to the dike.

b. Preferably, the slope of the dike below the weir should be as steep as possible to prevent resuspension of sediments from weir overflow currents.

c. Select a head, H, of water over the weir, preferably between 1 inch and 6 inches.

d. Determine the unit flow rate, Q/L, over the weir in accordance with Figure 5.

e. Compute the required length, L, of weir crest according to the influent discharge rate, Q, and the unit flow rate over the weir, Q/L.

f. Increase the computed length of the weir crest, L, by up to 10 percent to account for the effects of flow contraction.

Ponding depth

a. Use the established value of the head, H, of water over the weir to determine the required ponding depth, Z, of water in the vicinity of the weir, according to Figure 4.

b. It is recommended that a density different, $\Delta\rho_w$, of 0.01 g/cm^3 be used in this procedure.

c. Increase the calculated ponding depth, Z, by up to 1 foot and specify this value as the minimum allowable ponding depth in the containment area.

CONCLUSIONS

Based on the foregoing information, the following conclusions can be advanced:

1. The design of weirs for dredged material containment areas is highly empirical; pertinent guidelines are presently practically non-existent.

2. The operating conditions of an overflow weir can substantially affect the sedimentation effectiveness of a dredged material confinement area and the quality of the discharged supernatants.

3. Weirs should be strategically located to minimize flow contraction, reduce short-circuiting, and maximize the length of flow lines.

4. Long, sharp-crested, rectangular weirs appear to be the most promising candidates to improve the sedimentation effectiveness of

disposal areas.

5. A simple and reasonably accurate procedure has been advanced to design a rectangular weir for a disposal area.

6. Selective withdrawal principles should be applied to estimate the necessary ponding depth in the vicinity of a weir to avoid resuspension of bottom sediments.

7. Large ponding depths in the vicinity of the weir and low water heads at the crest of the weir improve the settling effectiveness of a disposal area.

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