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DESIGN OF FINE-GRAINED DREDGED MATERIAL SEDIMENTATION BASINS

by

Raymond L. Montgomery, Ph.D., P.E.*

ABSTRACT

Sedimentation is an important process in the disposal of dredged material generated from navigable waterways in the United States. The sedimentation basins used in this disposal process are different from those used in water and wastewater treatment in that the dredged material basins must provide for sedimentation to achieve acceptable effluent water quality while providing storage volume for several years of material dredged from the local waterways. This paper presents a methodology for designing fine-grained dredged material sedimentation basins. This methodology can also be used for designing sedimentation basins for other slurries having high suspended solids levels.

Settling tests performed in an 8-in.-diam. column were found to be satisfactory for defining dredged material settling behavior. Settling behavior in the freshwater environment was best described by a flocculent settling test, while behavior in a saltwater environment was best described by a zone settling test. The same settling columns were used for both tests with only minor procedural changes.

Procedures are presented for designing new containment areas for suspended solids retention and for evaluating the suspended solids retention potential of existing containment areas during planned disposal activities. Dye tracer studies indicated that a correction factor of about 2.25 should be applied to design area and detention times to compensate for the deviation

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from ideal or plug flow conditions.

INTRODUCTION

Confinement of dredged material on land has been a disposal alternative used by the Corps of Engineers for a number of years. In more recent years this practice has increased, and added requirements have been placed on the solids retention capability of confined disposal areas. The confined disposal (containment) areas used for both retention and disposal of dredged material are simply sedimentation basins.

Sedimentation has been used far more widely than any other major process for the removal of suspended matter from water. No doubt it is the oldest process which has remained in continued use. The settling behavior of suspensions has always been the key to the design of effective sedimentation basins, and this factor has consequently captured the interest of researchers in a number of fields. Extensive literature is available on the subject.

Dredged material sedimentation basins are slightly different from those used in water and wastewater treatment in that the dredged material basins must provide for sedimentation to achieve acceptable effluent quality while providing storage volume for several years of material dredged from local waterways. In most cases, the amount of dredged material storage required is probably the controlling factor in sizing a conventional disposal area. Nevertheless, effluents from the large areas now in existence often have problems meeting the effluent requirements for suspended solids. This shortcoming can be attributed to the non-uniform lateral distribution of flows and short-circuiting currents that occur in most dredged material containment areas. As a result of short-circuiting currents, one section of flow is subjected to a different velocity from another. Since sufficient detention time is not provided in this section, the effluent has a higher solids level. This can be seen in aerial photographs of containment areas in operation. An example of a poorly designed containment area is shown in Figure 1. The flow in this containment area is essentially overland flow resulting in significant short-circuiting. In addition, the effective settling area appears to be reduced to about one half the diked area because of a build-up of the coarse-grained dredged material. These factors result in poor suspended solids removal as indicated by the turbidity plume from the weir.

The major problem is that very little is known about the actual sedimentation process in dredged material containment areas. The hydrodynamic problem of one particle falling through a fluid has been solved (Stoke's Law), and formulas have been developed by researchers to determine the fall speed when the density of a particle is very small and their distance apart is much greater than their size. In practice, dredged material is discharged into the sedimentation basin at concentrations averaging about 145 g/ℓ. Because of this high concentration, it is believed that sedimentation occurs under either flocculent or zone settling processes.

High density slurries have been observed near the surface of dredged material sedimentation basins indicating that hindered settling occurs in a significant portion of the water column. The velocity for hindered settling is less than that predicted by theories based on discrete settling because of the upward velocity of water displaced by the highly concentrated slurry. A review of present practices indicates that many dredging disposal operations cannot be undertaken on a continuous basis and still maintain acceptable suspended solids removal levels. Where strict effluent suspended solids limits are enforced, periods of interrupted dredging are common to reduce the loading rate and provide time for particle settling. These

Figure 1. Turbidity From Dredged Material Containment Area.

interrupted dredging operations usually result in increased overall operational costs.

This paper provides procedures for designing fine-grained dredged material sedimentation basins to provide adequate retention of suspended solids so that required effluent suspended solids levels can be met. The procedures described herein were developed by the author with funds from the Dredged Material Research Program, Environmental Laboratory, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

CONCEPTS OF CONTAINMENT AREA OPERATION

Diked containment areas are used to retain dredged material solids while allowing the carrier water to be released from the containment area. The two objectives inherent in the design and operation of a containment area are: (a) to provide adequate storage capacity to meet dredging requirements, and (b) to attain the highest possible efficiency in retaining solids during the dredging operation in order to meet effluent suspended solids requirements. These considerations are basically interrelated and depend upon effective design, operation, and management of the containment area.

The major components of a dredged material containment area are shown schematically in Figure 2. A tract of land is surrounded by dikes to form a confined surface area, and the dredged channel sediments are then pumped into this area hydraulically. The influent dredged material slurry can be characterized by suspended solids concentration, particle gradation, type of carrier water (fresh or saline), and rate of inflow.

In some dredging operations, especially in the case of new work dredging, sand, clay balls, and/or gravel may be present. This coarse material (more than half >No. 200 sieve) rapidly falls out of suspension

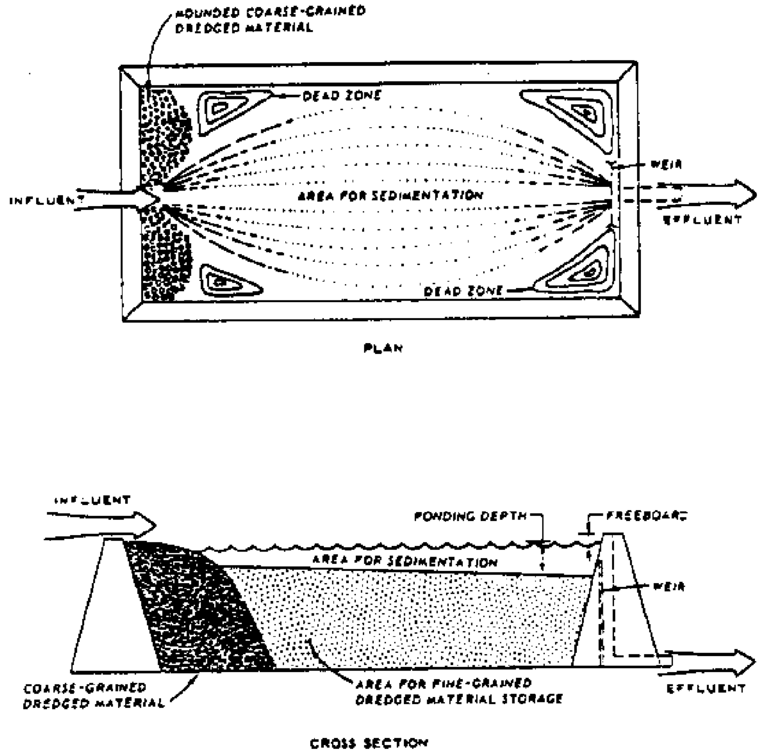

Figure 2. Example of Dredged Material Containment Area.

near the dredge inlet pipe forming a mound. The fine-grained material (more than half <No. 200 sieve) continues to flow through the containment area with most of the solids settling out of suspension, thereby occupying a given storage volume. The fine-grained dredged material is usually rather homogeneous and is easily characterized.

The clarified water is discharged from the containment area over a weir. This effluent may be characterized by its suspended solids concentration and rate of outflow. Effluent flow rate is approximately equal to influent flow rate for continuously operating disposal areas. Flow over the weir is controlled by the static head and the effective weir length provided. To promote effective sedimentation, ponded water is maintained in the area; the depth of water is controlled by the elevation of the weir crest. The thickness of the dredged material layer increases with time until the dredging operation is completed. Minimum freeboard requirements and mounding of coarse-grained material result in a ponded surface area smaller than the total surface area enclosed by the dikes. Dead spots in corners and other hydraulically inactive zones reduce the effective surface area, where sedimentation takes place, to considerably less than the ponded surface area. (Gallagher, 1978).

FIELD INVESTIGATIONS

Field investigations are necessary to provide data for containment area design. The channel must be surveyed to determine the volume of material to be dredged, and channel sediments must be sampled to obtain material for laboratory tests. This part of the paper describes field investigations required to obtain the necessary samples for laboratory testing. The methods in common use for determining volumes of channel sediment to be dredged are well known and do not warrant discussion here.

CHANNEL SEDIMENT SAMPLING

The level of effort required for channel sediment sampling is highly project-dependent. In the case of routine maintenance work, data from prior samplings and dredging activities can provide a basis for developing the scope of field investigations. Grab samples are considered adequate for sampling fine-grained sediments from maintenance dredging locations (Palermo, et al., 1978). Such samples are adequate for sediment characterization purposes and are relatively easy and inexpensive to obtain. An evaluation of dredged material after being removed by the hydraulic dredge indicated that grab samples were adequate to characterize the sediment properties (Montgomery 1978).

Research by Bartos (1977) summarized equipment available for sampling channel sediments. He concluded that there are two general classes of sampling equipment available for use in sampling channel sediments - grab samplers and tube samplers.

Petersen dredge. The Petersen dredge was found to be adequate for most sampling needs. An example of this type of grab sampler being used is shown in Figure 3. The Petersen dredge is a versatile sampler; it will

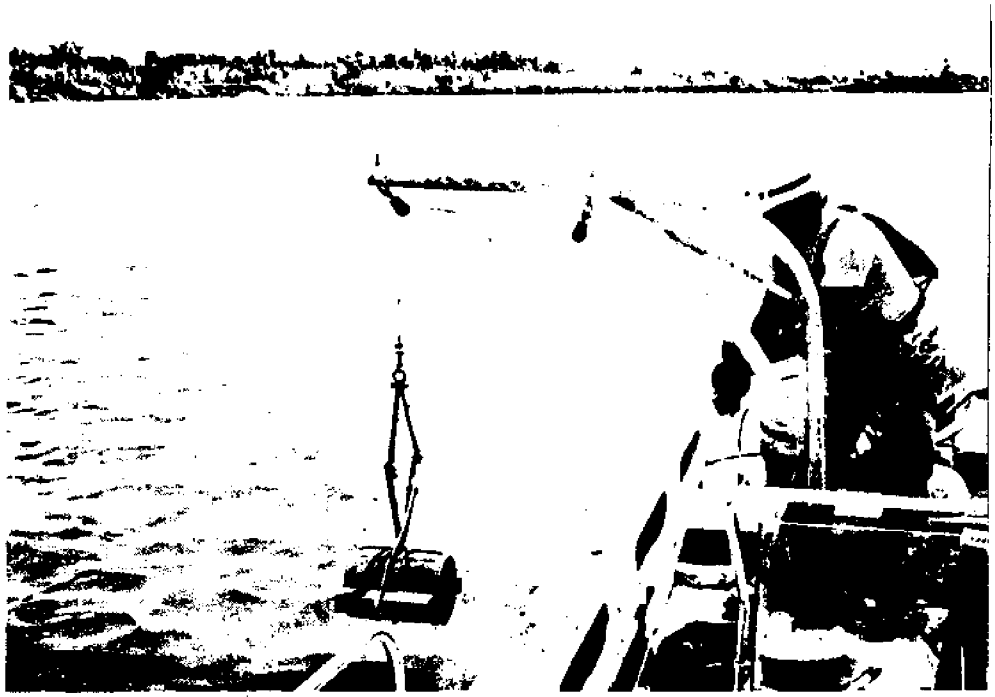

Figure 3. Petersen Dredge Being Used to Sample Channel Sediments.

sample a wide range of bottom textures, from fine-grained clays to sands. The Petersen dredge samples 144 sq in. to a depth of about 12 in., depending on the texture of the sediment. It can be seen in Figure 3 that the sampler closes tightly, minimizing the loss of sediment and water upon retrieval. The fine-grained sediment samples obtained with this type grab sampler are considered to be representative of in situ moisture contents.

Water samples. Water samples should be taken at the same time as channel sediment samples. The water samples should be taken near the water-sediment interface and used to determine the salinity of the sediment environment. As will be discussed later, salinity levels play an important role in the way sediments settle.

Quantity of sediment samples. The quantity of sediment required is based on the amount needed for the laboratory tests. Enough sediment to perform the necessary characterization tests and provide material for the column settling tests should be collected from each established sampling station. Five-gallon containers can be used to hold the sediment samples. These containers are about the largest that can be handled efficiently. Small samples of sediment should be collected and placed in 8-oz watertight jars for water content and specific gravity tests. Care must be taken to collect the small sediment samples that appear to be most representative of the entire sample.

After the characterization tests are performed on grab samples from each sampling station, the samples collected in the five-gallon containers can be combined to obtain sufficient material for the column settling tests.

Sample preservation. Samples should be placed in air- and watertight containers and then in a cold room (6 to 8°C) within 24 hours after sampling. The organic content should be determined for each sample, and, if less than about 10 percent, it is not considered necessary to have the samples remain in the cold room. Below this organic content level, it is assumed that little biological activity could occur that would affect subsequent testing.

LABORATORY TESTING

Laboratory tests are required primarily to provide data for sediment characterization and containment area design. A flow chart illustrating the complete laboratory testing program for sediment samples is shown in Figure 4. Sediment character and requirements for sedimentation data estimates will dictate which laboratory tests are required. Not all laboratory tests indicated in Figure 4 are required for every application. The required magnitude of the laboratory testing program is highly project-dependent. Fewer tests are usually required when dealing with a relatively homogeneous material and/or when data are available from previous tests and experience, as is frequently the case in maintenance work. For unusual maintenance projects where considerable variation in sediment properties is apparent from samples or for new work projects, more extensive laboratory testing programs are required. Laboratory tests should always be performed on representative samples selected using sound engineering judgement.

SEDIMENT CHARACTERIZATION TESTS

A number of sediment characterization tests are required before settling tests can be performed. Visual classification will establish whether the sediment sample is predominantly fine-grained. Tests required on fine-grained sediments include natural water content, Atterberg limits, organic content, and specific gravity. The coarse-grained sediments require only grain size analyses. Water samples taken from the channel should be tested for salinity to provide information for use in performing the column settling tests.

SEDIMENTATION TESTS

Sedimentation, as applied to dredged material disposal activities, refers to those operations in which the dredged material slurry is separated into more clarified water and a more concentrated slurry. Laboratory sedimentation tests must provide data for designing the containment area to meet effluent suspended solids criteria and to provide adequate storage

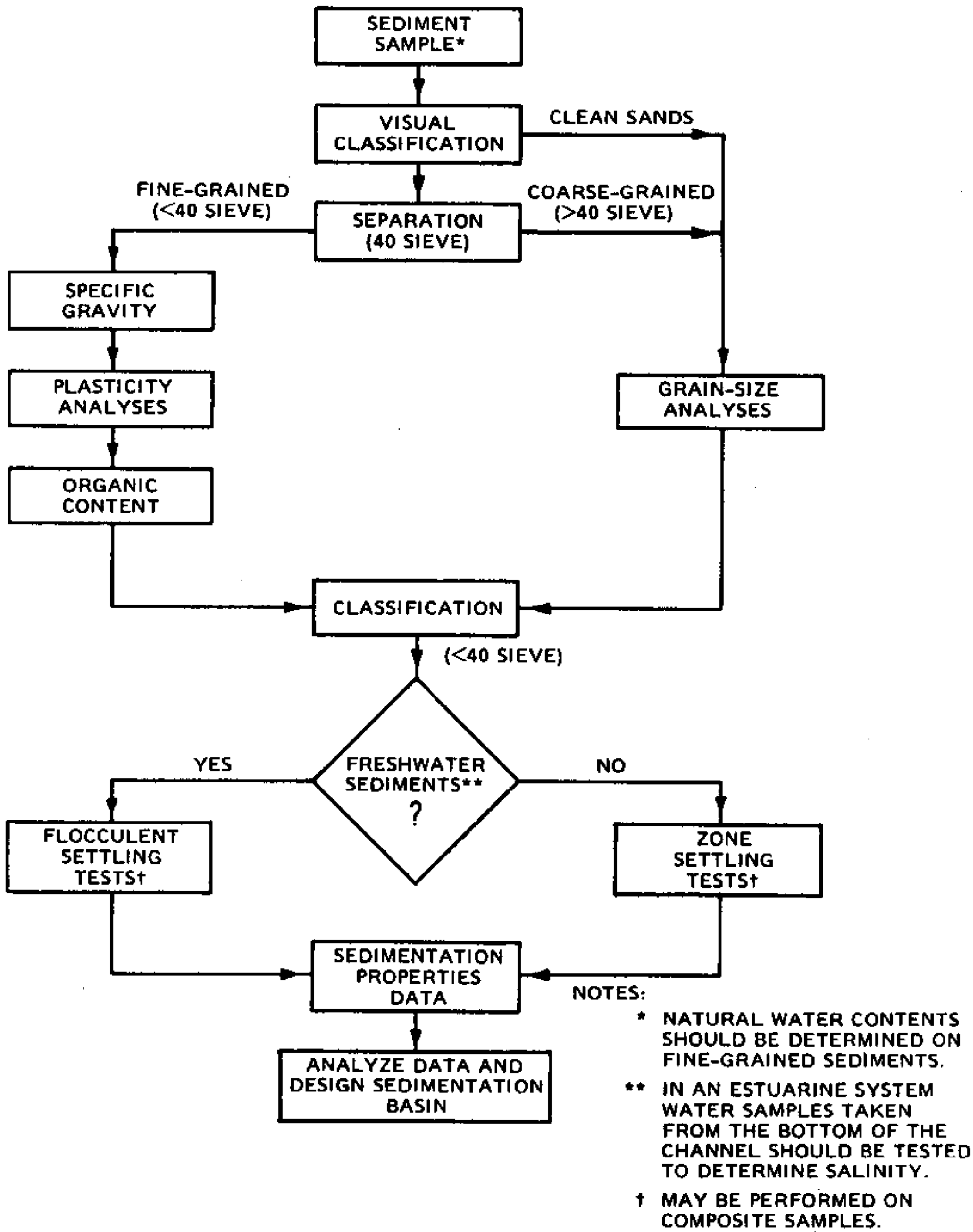

Figure 4. Flow Chart of Recommended Laboratory Testing Program.

capacity for the dredged solids. These tests are based on the gravity separation of solid particles from the transporting water.

The sedimentation process can be categorized according to three basic classifications: (a) discrete settling where the particle maintains its individuality and does not change in size, shape, or density during the settling process; (b) flocculent settling where particles agglomerate during the settling period with a change in physical properties and settling rate; (c) zone settling where the flocculent suspension forms a lattice structure and settles as a mass, exhibiting a distinct interface during the settling process.

The important factors governing the sedimentation of dredged material solids are initial concentration of the slurry and flocculating properties of the solid particles. Because of the high influent solids concentration and the tendency of dredged material fine-grained particles to flocculate, either flocculent or zone settling behavior governs sedimentation in containment areas (Montgomery, 1978). Discrete settling describes the sedimentation of sand particles and fine-grained sediments at concentrations much lower than those found in dredged material containment areas.

Laboratory tests are necessary to characterize the sediment and to provide data for containment area design. A flow chart of the laboratory testing program recommended for providing design data is shown in Figure 4. The recommended laboratory procedures discussed here are for characterization of the dredged material sedimentation processes. They are based on results from the extensive laboratory testing program (Montgomery, 1978, 1979).

The objective of running settling tests on sediments to be dredged is to define, on a batch basis, settling behavior in a large-scale, continuous-flow dredged material containment area. Results of tests must allow determination of numerical values for the design parameters which can be projected to the size and design of the containment area.

Sedimentation of freshwater sediments at alurry concentrations as high as 175 g/ℓ can be characterized by flocculent settling properties (Montgomery 1978). However, as slurry concentrations are increased, the sedimentation process may be characterized by zone settling properties. The settling column shown in Figure 5 can be used with procedural modifications for both flocculent and zone settling tests. Salinity enhances the agglomeration of dredged material particles (Migniot, 1968). The settling properties of all saltwater dredged material tested during the study by Montgomery (1979) could be characterized by zone settling tests.

Flocculent settling test. The flocculent settling test consists of measuring the concentration of suspended solids at various depths and time intervals in a settling column. If an interface forms near the top of the settling column during the first day of the test, sedimentation is governed by zone settling and that test procedure should be initiated. Information required to design a containment area in which flocculent settling governs can be obtained using the procedure described below.

1. A settling column such as that shown in Figure 5 is used. The test column depth should approximate the effective settling depth of the proposed containment area. A practical test depth is 6 ft. The column should be at least 8 in. in diameter with sample ports at 1-ft intervals. The column should have provisions to bubble air from the bottom to keep the slurry mixed during the column-filling period.

2. Mix the sediment slurry to the desired suspended solids concentration in a container with sufficient volume to fill the test column. At least two tests should be performed at the concentration selected to represent the concentration of influent dredged material C_i. Use the average detention time computed from these tests for design. Field studies indicate that for maintenance dredging in fine-grained material the disposal concentrations average about 145 g/ℓ.

3. Pump or pour the slurry into the test column using air to maintain a uniform concentration during the filling period.

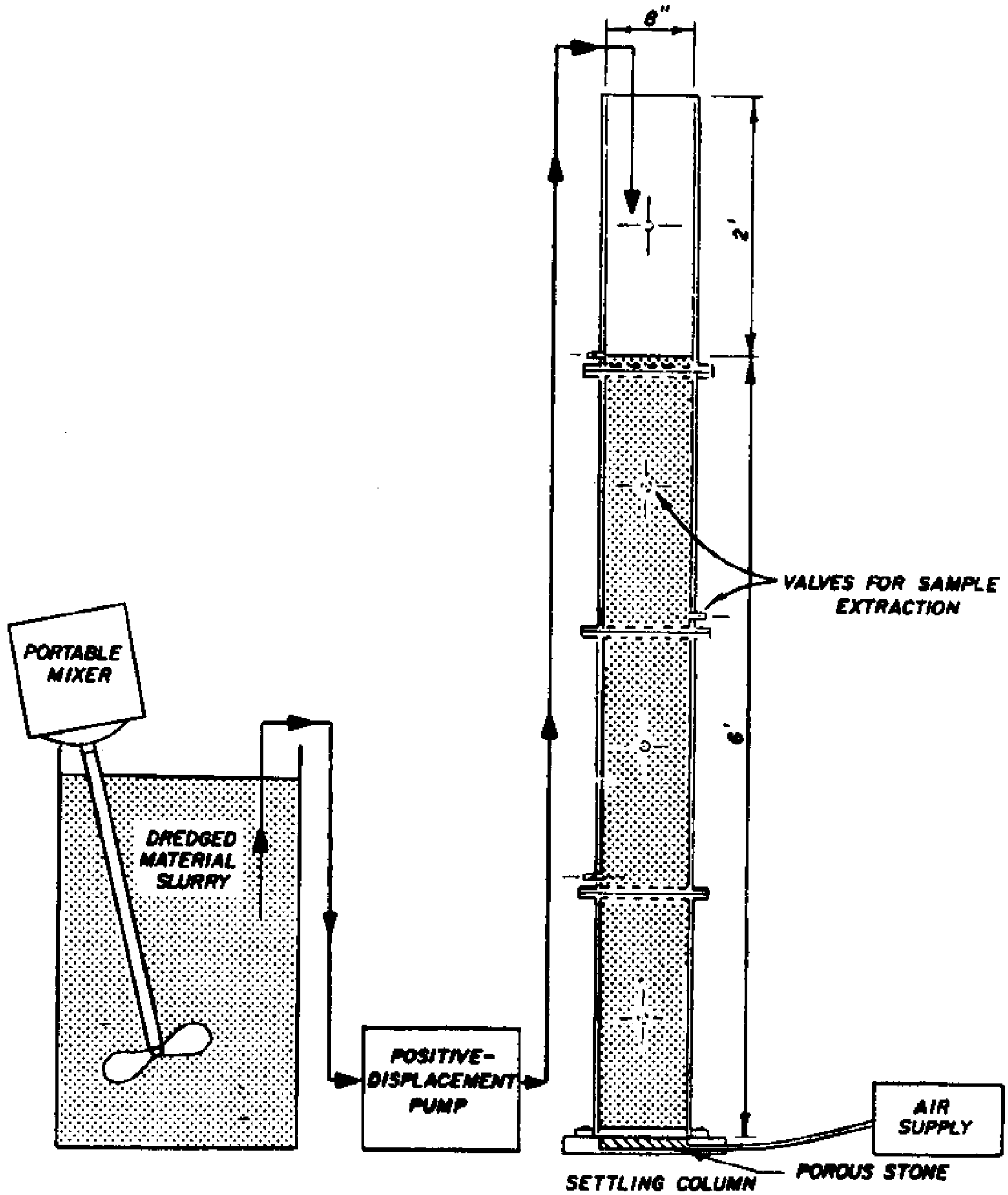

Figure 5. Schematic of Flocculent Settling Test Equipment.

4. While the column is completely mixed, draw off samples at each sample port and determine the suspended solids concentration. Average these values and use the results as the initial concentration at the start of the test. After the initial samples are taken, stop the air bubbling and begin the test.

5. Allow the slurry to settle, then withdraw samples from each sampling port at regular time intervals and determine suspended solids concentrations. Sampling intervals depend on the settling rate of the solids - usually at 30-min intervals for the first 3 hours and then at 4-hour intervals until the end of the test. The sampling times can be adjusted after the first complete test. Continue the test until the interface of solids can be seen near the bottom of the column and the suspended solids level in the fluid above the interface is <1 g/ℓ.

6. If an interface has not formed within the first day on any previous tests, run one additional test with a suspended solids concentration sufficiently high to induce zone settling behavior. This test should be carried out according to the procedures outlined below for zone settling tests. The exact concentration at which zone settling behavior occurs depends upon the sediment being used to estimate the volume required for dredged material storage.

Zone settling test. The zone settling test consists of placing a slurry in a sedimentation column and recording the fall of the liquid-solids interface with time. Plot the depth to the interface versus time. The slope of the constant settling zone of the curve is the zone settling velocity, which is a function of the initial test slurry concentration. Information required to design a containment area in which zone settling governs can be obtained by using the procedure described below.

1. Use a settling column such as that shown in Figure 5. It is important that the column diameter be sufficient to reduce wall effects and the test be performed at a slurry depth near that expected in the field. Therefore, a one-litre graduated cylinder should never be used to perform a zone settling test for sediment slurries representing dredging-disposal activities.

2. Mix the slurry to the desired concentration and pump or pour it into the test column. Test concentrations should range from about 60 to 200 g/ℓ. Air may not be necessary to keep the slurry mixed if the filling time is less than 1 minute.

3. Record the depth to the solid-liquid interface with respect to time. Observations must be made at regular intervals to gain data for plotting the depth to interface versus time curve. It is important to make enough observations to clearly define this curve for each test.

4. Continue the readings until sufficient data are available to define the maximum point of curvature of the depth to interface versus time curve for each test. The tests may require from 1 to 5 days to complete.

5. Perform a minimum of eight tests. Data from these tests are required to develop the curve of zone settling velocity versus concentration.

6. One of the above tests should be performed on sediment slurries at a concentration of about 145 g/ℓ. The test should be continued for a period of at least 15 days to provide data for estimating volume requirements.

DESIGN PROCEDURES

The flow chart shown in Figure 6 illustrates the design procedures recommended in the following paragraphs. The design procedures were adapted from procedures used in water and wastewater treatment and are based on field and laboratory investigations on sediments and dredged material (Montgomery 1978). Design methods for saltwater and freshwater sediments are presented. Essentially the method for saltwater sediments is based on zone settling properties, and the method for freshwater sediments is based on flocculent settling properties.

The design procedures presented here are for gravity sedimentation of dredged suspended solids. However, gravity sedimentation will not completely remove suspended solids from containment area effluent since wind and other factors can resuspend solids and increase effluent solids concentration. The sedimentation process, with proper design and operation, will normally provide removal of fine-grained sediments down to levels of 1 and 2 g/ℓ in the effluent for saltwater and freshwater sediments, respectively. If the required effluent standards are lower than this, the designer must provide for additional treatment of the effluent, e.g., flocculation or filtration.

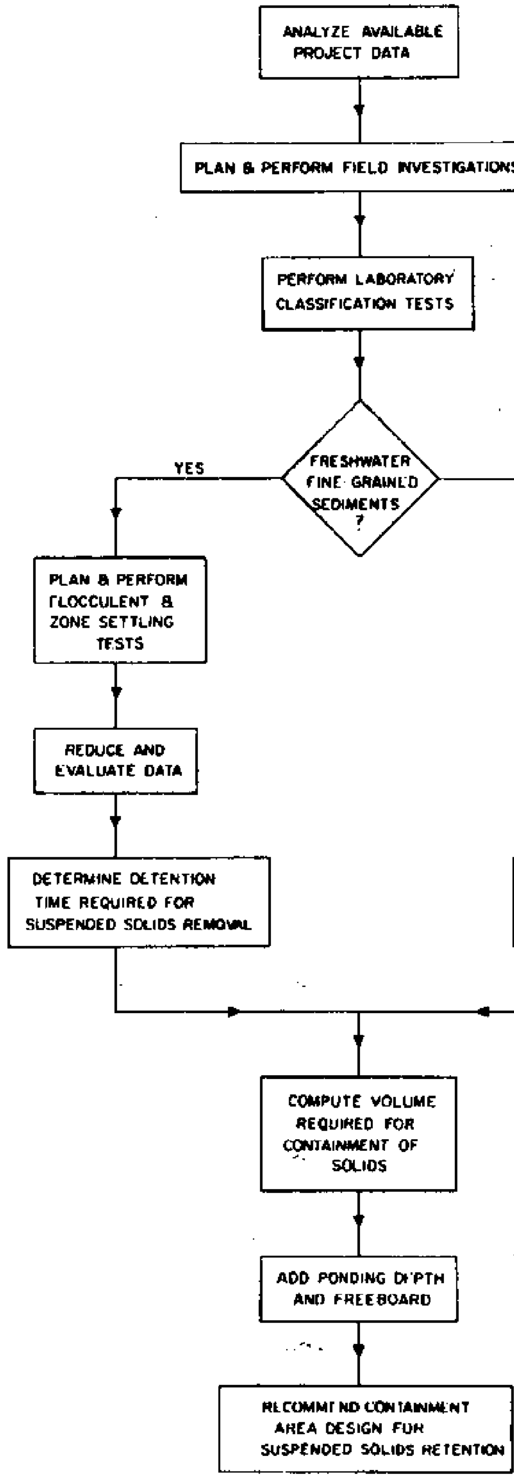

Figure 6. Flow Chart of Recommended Design Procedures for Fine-Grained Sediments.

DATA REQUIREMENTS

The data required to use the design procedures are obtained from field investigations, laboratory testing, dredging equipment designs, and past experience in dredging and disposal activities.

Estimate in situ sediment volume. The initial step in any dredging activity is to estimate the in situ volume of sediment to be dredged. Sediment quantities are usually determined from channel surveys on a routine basis.

Determine physical characteristics of sediments. Field sampling and laboratory testing should be accomplished according to the methods discussed by Palermo, et al. (1978). Adequate sample coverage is required to provide representative samples of the sediment. In situ water contents of the fine-grained sediments are also required. Care must be taken in sampling to ensure that the water contents are representative of the in situ conditions. The water content of representative samples, w, is used to determine the in situ void ratio e_i as follows:

$$e_i = \frac{(w/100)G_s}{S_d/100} \qquad (1)$$

where

w = water content in percent

G_s = specific gravity of sediment solids

S_d = degree of saturation (equal to 100 percent for sediment)

A representative value from in situ void ratios is used later to estimate volume for the containment area. Grain-size analyses must be performed to estimate the quantities of coarse- and fine-grained material in the sediment to be dredged.

Obtain and analyze proposed dredging and disposal data. The designer must obtain and analyze data concerning the dredged material disposal rate.

For hydraulic pipeline dredges, the type and size of dredge(s) to be used and the average solids concentration of the dredged material when discharged into the containment area must be considered. If the size of the dredge to be used is not known, the design must assume the largest dredge size that might be expected to perform the dredging.

Based on these data, the designer must estimate or determine containment area influent rate, influent suspended solids concentration, effluent rate (for weir sizing), effluent concentration allowed, and time required to complete the disposal activity. If no other data are available, for hydraulic pipeline dredges, an influent suspended solids concentration of 145 g/ℓ (13 percent by weight) can be used for design purposes.

Perform laboratory sedimentation tests. The procedures for sedimentation tests are given earlier under the part on laboratory testing. A designer must evaluate the results of salinity tests to determine whether the sediments to be dredged are freshwater or saltwater sediments. If the salinity is above 3 ppt, the sediments are classified as saltwater sediments for the purpose of selecting the laboratory sedimentation test.

DESIGN METHOD FOR SALTWATER SEDIMENTS

The following design method is recommended for sedimentation of dredged material from a saltwater environment. It can also be used for freshwater dredged material if the laboratory settling tests indicate zone settling properties. An example of this design method is presented by Montgomery (1978).

Analyze laboratory data. A series of zone settling tests must be conducted as detailed earlier. The results of the settling tests are analyzed to determine zone settling velocities at the various suspended solids concentrations. The procedure is as follows:

1. Develop a settling curve for each test. Plot depth to interface versus time.

2. Calculate the zone settling velocity, v_s, as the slope of the constant settling zone (straight line portion of curve). The velocity should be in ft per hour.

3. Plot v_s versus suspended solids concentration on a semi-log plot.

4. Use the plot developed in step 3 to develop a solids loading versus solids concentration curve as shown in Figure 7.

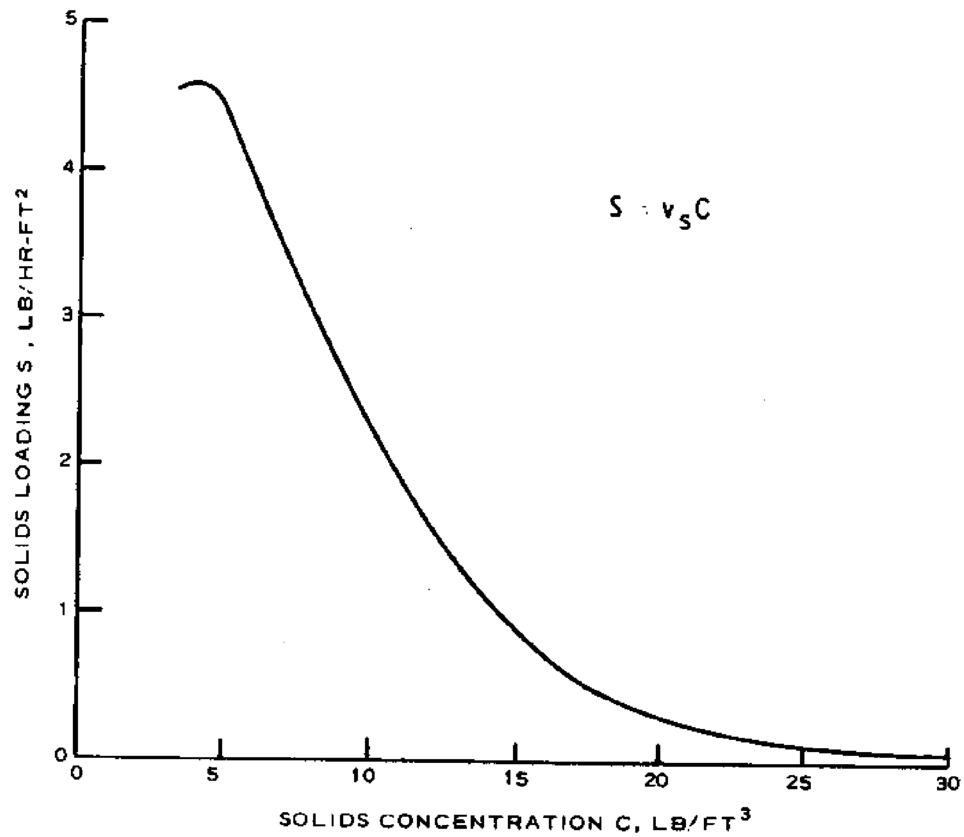

Figure 7. Typical Solids Loading Curve for Dredged Material.

Compute design concentration. The design concentration, C_D, is defined as the average concentration of the dredged material in the containment area at the end of the disposal activity and is estimated from data obtained from the 15-day column settling tests. The following steps can be used to estimate average containment area concentrations for each 15-day column settling test. It may be desirable to perform more than one 15-day test. If so, use an average of the values as the design concentration.

1. Compute concentration versus time for the 15-day settling test. Assume zero solids in the water above the solids interface to simplify calculation.

2. Plot concentrations versus time on log-log paper.

3. Draw a straight line through the data points. This line should be drawn through the points representing the consolidation zone.

4. Estimate the time of dredging by dividing the dredge production rate into volume of sediment to be dredged.

5. Estimate the concentration at time $t_{1/2}$ (one-half the time required for the disposal activity determined in step 4) using the figure developed in steps 2 and 3. This time is an approximation of the average time of residence for the dredged material in the containment area. Since concentration is a function of time, one-half the dredging time would represent a period during which one-half of the dredged material would have been in the area longer and the other half less than a time equal to one-half the dredging time.

6. Use the value computed in step 5 as the design solids concentration, C_D.

Compute area required for sedimentation. Containment areas designed according to the following steps should provide removal of fine-grained sediments well enough so that suspended solids levels in the effluent do not exceed 1 to 2 g/ℓ. The area required for the zone settling process to concentrate the dredged material to the design concentration is computed as follows, using the Yoshioka et al. (1957) graphic solution to the Coe and Clevenger procedure (1916).

1. Use the design concentration and construct an operating line from the design solids concentration tangent to the loading curve as shown in Figure 8. The design loading is obtained on the y-axis as S_L.

2. Compute the required area as

$$A = \frac{Q_i C_i}{S_L} \tag{2}$$

where

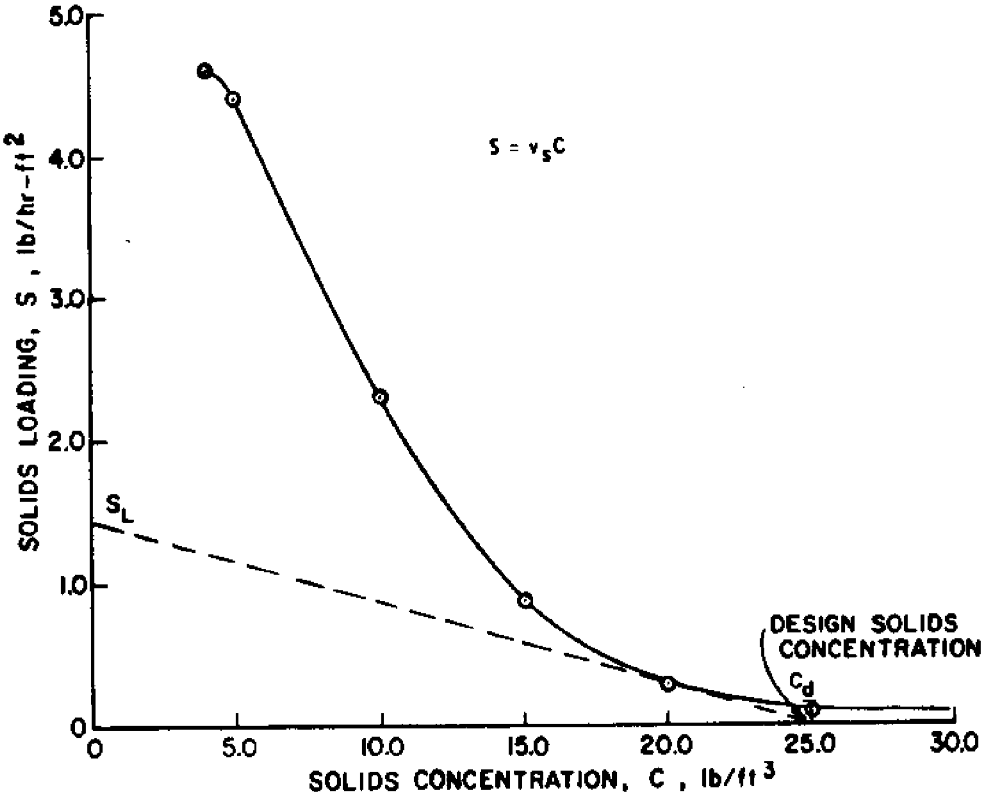

Figure 8. Solids Loading Curve Showing Design Line.

A = containment area surface requirement, ft^2

Q_i = influent rate, ft^3/hr
($Q_i = A_pV_d$; assume V_d = 15 ft/sec in absence of data and convert Q_i calculated in ft^3/sec to ft^3/hr)

A_p = cross-sectional area of dredge pipeline, ft^2

V_d = velocity of dredge discharge, ft/sec

C_i = influent solids concentration, lb/ft^3
(use 145 g/ℓ or 9.2 lb/ft^3 if no data are available)

S_L = design solids loading, $lb/hr\text{-}ft^2$

3. Increase area by a factor of 2.25 to compensate for containment area short-circuiting and dispersion.

$$A_d = 2.25 A \quad (3)$$

where

A_d = design basin surface area, ft^2

A = area determined from Equation 2, ft^2

DESIGN METHOD FOR FRESHWATER SEDIMENTS

Sediments in a dredged material sedimentation basin are comprised of a broad range of particle flocs of different sizes and surface characteristics. In the sedimentation basin under flocculent settling conditions the larger particle flocs settle at faster rates, thus overtaking finer flocs in their descent. This contact increases the floc sizes and enhances settling rates. The greater the ponding depth in the containment area, the greater is the opportunity for contact among sediments and flocs. Therefore, sedimentation of freshwater dredged sediments is dependent on the ponding depth as well as the properties of the particles.

Analyze laboratory data. Evaluation of the sedimentation characteristics of a freshwater sediment slurry is accomplished as discussed earlier. The design steps are as follows: (Refer to Montgomery (1978) for example problem.)

1. Arrange data from laboratory tests illustrated in Table 1 into the form shown in Table 2.

2. Plot these data as shown in Figure 9. The percent by weight of initial concentration for each depth and time is given in Table 2. The solid curved lines represent the concentration depth profile at various times during settling. (Refer to Figure 9 for more details.) Numbers appearing along the horizontal depth lines are used to indicate area boundaries.

3. Compute a design concentration using data from the 15-day zone settling test. Follow the procedure outlined in the design method for saltwater sediments.

Compute detention time required for sedimentation. The detention time is computed as follows:

TABLE 1

OBSERVED FLOCCULENT SETTLING CONCENTRATIONS WITH DEPTH, IN GRAMS PER LITRE

Time, min	Depth from top of Settling Column, ft						
	1	2	3	4	5	6	7
0	132	132	132	132	132	132	132
30	46	99	115	125	128	135	146
60	25	49	72	96	115	128	186
120	14	20	22	55	78	122	227
180	11	14	16	29	75	119	
240	6.8	10.2	12	18	64	117	
360	3.6	5.8	7.5	10	37	115	
600	2.8	2.9	3.9	4.4	14	114	
720	1.01	1.6	1.9	3.1	4.5	110	
1020	.90	1.4	1.7	2.4	3.2	106	
1260	0.83	1.14	1.2	1.4	1.7	105	
1500	0.74	0.96	0.99	1.1	1.2	92	
1740	0.63	0.73	0.81	0.85	0.94	90	

Note: Data from actual test on freshwater sediments. Although a 6-ft test depth is recommended, an 8-ft depth was used in this test.

TABLE 2

PERCENT OF INITIAL CONCENTRATION WITH TIME

Time, T, min	Depth From Top of Settling Column		
	1 ft	2 ft	3 ft
0	100	100	100
30	35	75	87
60	19	37	55
120	11	15	17
180	8	11	12
240	5	8	9
360	3	4	6
600	2.0	2.2	3.0
720	1.0	1.2	1.4

Note: Initial suspended solids concentration = 132 g/ℓ

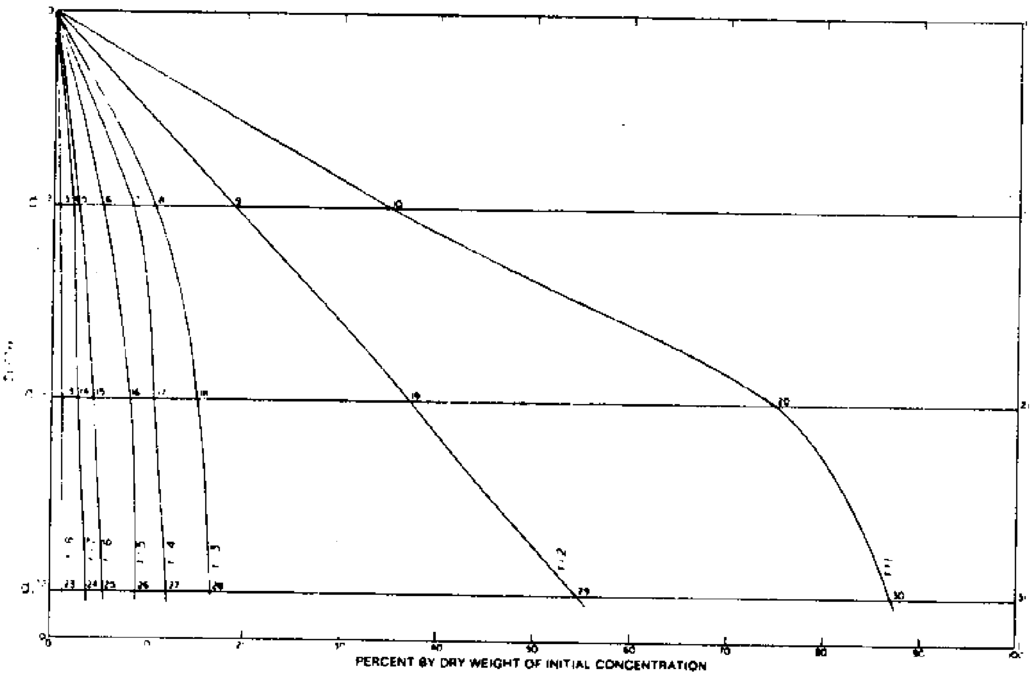

Figure 9. Removal of Flocculating Dredged Material Particles.

1. Calculate removal percentage at depths of 1, 2, and 3 ft for various times using the plot illustrated in Figure 9. The removal percentage for depth d_1 and t = 1 is computed as follows:

$$R = \frac{\text{Area } 0,\ 10,\ 11,\ 1^*}{\text{Area } 0,\ 2,\ 11,\ 1} \times 100 \qquad (4)$$

where

R = removal percentage

Determine these areas by either planimetering the plot or by direct graphic measurements and calculations. This approach is used to calculate removal percentages for each depth as a function of time. The depths used should cover the range of ponding depths expected in the containment area. This report recommends at least 2 ft of ponding depth at the end of the dredging project.

* These numbers correspond to the numbers used in Figure 9 to indicate the area boundaries for the total area down to depth d, (0,2,11,1) and the area to the right of the t = 1 time line (0,10,11,1).

2. Plot the solids removal percentages versus time as shown in Figure 10.

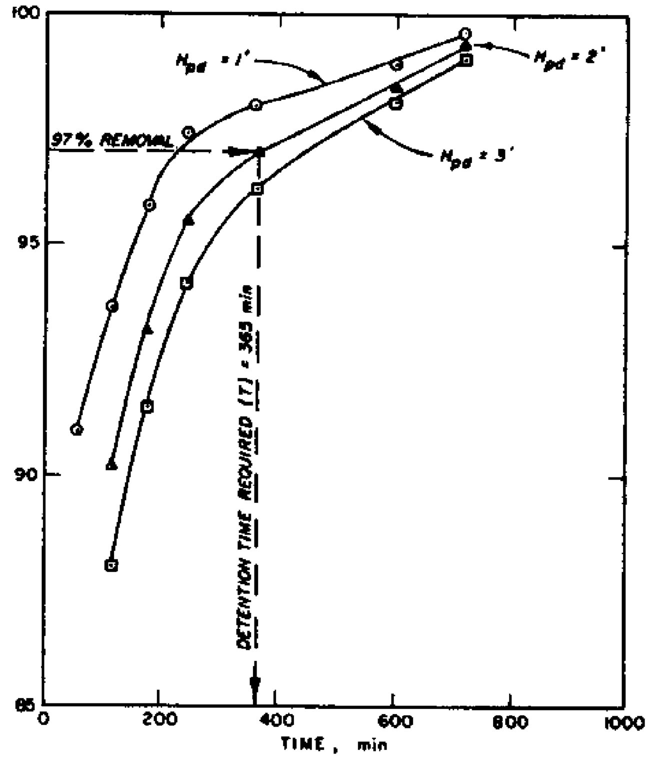

Figure 10. Solids Removal Versus Time as a Function of Depth.

3. Theoretical detention times can be selected from Figure 10 for various solids removal percentages. Select the detention time, T, that gives the desired removal percentage for the design ponding depth.

4. The theoretical detention time, T, should be increased by a factor of 2.25 to compensate for the fact that sedimentation basins, because of short-circuiting and dispersion, have average detention times less than volumetric detention times:

$$T_d = 2.25\ T \tag{5}$$

where

T_d = design detention time

VOLUME REQUIREMENTS FOR CONTAINMENT OF SOLIDS

The procedures outlined in the above paragraphs are aimed at providing sedimentation basins with sufficient areas and detention times to accommodate continuous disposal activities while providing sufficient suspended solids removal to meet effluent suspended solids requirements. Sedimentation basins must also be designed to meet volume requirements for a particular disposal activity. The total volume required of a sedimentation basin includes volume for storage of dredged material, volume for sedimentation (ponding depths), and freeboard volume (volume above water surface). Volume required for storage of the coarse-grained material (>200 sieve) must be determined separately, because this behaves independently of the fine-grained (<200 sieve) material.

ESTIMATE VOLUME OCCUPIED BY DREDGED MATERIAL IN SEDIMENTATION BASIN

The volume occupied by dredged material in the sedimentation basin after the completion of a particular disposal activity is computed as follows. The volume is not an estimate of the long-term needs for multiple-disposal activities. The procedures given below can be used to design for volume required for one disposal activity, or used to evaluate the adequacy of the volume provided by an existing sedimentation basin.

1. Compute the average void ratio of fine-grained dredged material in the sedimentation basin at the completion of the dredging operation using the design concentration determined in earlier steps as dry density of solids. (Note that design concentration is determined for both the flocculent and the zone sedimentation design procedure.) Use the following equations to determine void ratio:

$$e_o = \frac{G_s \gamma_w}{\gamma_d} - 1 \qquad (6)$$

where

e_o = average void ratio of dredged material in the sedimentation basin at the completion of the dredging operation

G_s = specific gravity

γ_w = density of water, g/ℓ

γ_d = dry density of solids at design concentration, ($C_D = \gamma_d$) g/ℓ

2. Compute the change in volume of fine-grained channel sediments after disposal in the sedimentation basin from

$$\Delta V = V_i \frac{e_o - e_i}{1 + e_i} \qquad (7)$$

where

ΔV = change in volume of fine-grained channel sediments after disposal in the sedimentation basin, ft^3

e_i = average void ratio of in situ channel sediments

V_i = volume of fine-grained channel sediments, ft^3

3. Compute the volume required by dredged material in the sedimentation basin from

$$V = V_i + \Delta V + V_{sd} \qquad (8)$$

where

V = volume of dredged material in the sedimentation basin at the end of the dredging operation, ft^3

V_{sd} = volume of sand (compute using 1:1 ratio), ft^3

ESTIMATE OF BASIN DEPTH

Previous calculations have provided a design area A_d and design detention time T_d required for fine-grained dredged material sedimentation. Equations 6-8 are used to estimate the volume and corresponding depth requirements for the storage of solids in the containment area. Throughout the design process, the existing topography of the containment area must be considered, since it can have a significant effect on the average depth of the containment area.

Saltwater sediments (zone settling). The following procedure should be used for saltwater sediments:

1. Estimate the thickness of dredged material at the end of the disposal operation from

$$H_{dm} = \frac{V}{A_d} \qquad (9)$$

where

H_{dm} = thickness of the dredged material layer at the end of dredging operations, ft

V = volume of dredged material in the basin, ft^3 (from Equation 8)

A_d = design area, ft^2 (as determined from Equation 3 or known surface area for existing sites)

2. Consult with soils design engineers to determine maximum height allowed for confining dikes. Anticipated settlement of the dikes should also be considered.

3. Add ponding depth and freeboard depth to H_{dm} to determine the required containment area depth (dike height).

$$D = H_{dm} + H_{pd} + H_{fb} \qquad (10)$$

where

D = dike height, ft

H_{pd} = average ponding depth over the area, ft (a minimum of 2 ft is recommended)

H_{fb} = freeboard above the basin water surface to prevent wave overtopping and subsequent damage to confining earth dikes, ft (a minimum of 2 ft is recommended to account for fetch and wind)

4. Compare with allowable dike height

Freshwater sediments (flocculent settling). The following procedure should be used for freshwater sediments:

1. Compute the volume required for sedimentation from

$$V_B = Q_i T_d \qquad (11)$$

where

V_B = sedimentation basin volume required for meeting suspended solids effluent requirements, ft^3

T_d = required detention time from Equation 5

2. Consult with soils design engineers to determine maximum height allowable for confining dikes D. In some cases, it might be desirable to use less than the maximum allowed dike height.

3. Compute the required design area as a minimum required surface area for solids storage from

$$A_d = \frac{V}{H_{dm(max)}} \tag{12}$$

where

$$H_{dm(max)} = D - H_{pd} - H_{fb}$$

or set the design area A_d equal to the known surface area for existing sites.

4. Evaluate volume available for sedimentation near the end of the disposal operation from

$$V^* = H_{pd}A_d \tag{13}$$

where

V^* = volume available for sedimentation near the end of disposal operation, ft^3

5. Compare V^* and V_B. If the volume required for sedimentation is larger than V^*, the sedimentation basin will not meet the suspended solids effluent requirements for the entire disposal operation. The following three measures can be considered to ensure that effluent requirements are met: (1) increase the design area A_d, (2) operate the dredge on an intermittent basis when V^* becomes less than V_B or use smaller size dredge, and (3) provide for post-treatment of effluent to remove solids.

6. Estimate the thickness of dredged material at the end of disposal operation using Equation 9. A_d is determined using step 3 above.

7. Determine the required sedimentation basin depth using Equation 10.

8. Compare with maximum allowable dike height. (See paragraph below.)

At most sedimentation basins, the foundation soils are soft. Such foundations limit the heights of confining earth dikes that can be economically constructed. Therefore, soils design engineers must be consulted to determine the maximum dike heights that can be constructed. If the maximum dike height allowed by foundation conditions is less than the sedimentation basin depth requirement, the design area A_d must be increased until the depth requirement can be accommodated by the allowable dike height; the thickness of the dredged material layer must also be decreased.

SUMMARY

The field verification work performed by Montgomery (1979) indicated that conservative values could be estimated from laboratory tests for solids concentrations expected in the dredged material sedimentation basin. The laboratory test data were reasonably close and should be adequate for design purposes. The column sedimentation tests can be improved with further experience in dredged material sedimentation basin design and with more laboratory testing.

The flocculent settling tests and design procedures recommended for freshwater sediments were found to provide design estimates that agreed closely with actual field values. However, additional cases should be evaluated before full substantiation of these procedures is proclaimed.

A significant amount of work is required on the hydraulic efficiency of dredged material sedimentation basins before firm design correction factors can be established to account for scale-up and flow-through problems. Five dye tracer tests were evaluated by Montgomery (1979). The correction factors determined from these tests varied from 2.13 to 2.72. Based on the work accomplished during this research, a correction factor of 2.25 appears reasonable. This factor is higher than those recommended in the sanitary engineering literature. However, the conditions experienced at a dredged material sedimentation basin are more complex than those of wastewater treatment facilities.

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DREDGED MATERIAL SEDIMENTATION BASINS

EXAMPLE DESIGN CALCULATIONS*

* Taken from report entitled "Methodology for Design of Fine-Grained Dredged Material Containment Areas for Solids Retention," by R. L. Montgomery.

EXAMPLE DESIGN CALCULATIONS

This Appendix presents example calculations for sedimentation basin designs. The examples are developed to illustrate the use of field and laboratory data and include designs for sedimentation and weir design. Weir design is based on the research reported by Walski and Schroeder (19). Separate examples are developed for saltwater and freshwater sedimentation designs as described in Chapter VIII of the main text. Only those calculations necessary to illustrate the procedure are included in the examples.

Example I: Sedimentation Basin Design Method for Freshwater Sediments

Project information

Each year an average of 300,000 yd^3 of fine-grained channel sediment is dredged from a harbor on Lake Michigan. A new in-water containment area is being constructed to accommodate the long-term dredged material disposal needs in this harbor. However, the new containment area will not be ready for approximately 2 years. One containment area in the harbor has some remaining storage capacity, but it is not known whether the remaining capacity is sufficient to accommodate the immediate disposal requirements. Design procedures must be followed to determine the detention time needed to meet effluent requirements of 4 g/ℓ and the storage volume required for the 300,000 yd^3 of channel sediment. These data will be used to determine if the existing containment area storage capacity is sufficient for the planned dredged material disposal activity. The existing containment area is about 3 miles from the dredging activity.

Records indicate that, for the last three dredgings, an 18-in. pipeline dredge was contracted to do the work. The average working time was 17 hours per day, and the dredging rate was 600 yd^3 per hour of in situ channel sediment. The project depth in the harbor is 50 ft.

Results of containment area survey

The existing containment area has the following dimensions:

1. Size: 96 acres.
2. Shape: length-to-width ratio is about 3.
3. Volume: 1,600,000 yd^3 (Average depth, from surveys, is 10 ft.)
4. Weir length: 24 ft (rectangular weir).

Results of laboratory tests and analysis of data

The following data were obtained from laboratory tests as described in the main text:

1. Salinity: <1 ppt.
2. In situ water content of channel sediment, w: 86 percent.
3. Specific gravity, G_s: 2.69.
4. Observed flocculent settling concentrations as a function of depth (See Table 6.)
5. Variation of solids concentration with time (as percent of original concentration) (See Table 7.) This is determined as follows:

   Column concentration at beginning of tests is 132 g/ℓ, concentration at 1-ft level at time = 30 min is 46 g/ℓ (Table 6). Percent of initial concentration = 46 ÷ 132 = 0.35 = 35 percent. These calculations are repeated for each time and depth to

TABLE 6

OBSERVED FLOCCULENT SETTLING CONCENTRATIONS
WITH DEPTH, IN GRAMS PER LITRE

Time, min	Depth from top of Settling Column, ft						
	1	2	3	4	5	6	7
0	132	132	132	132	132	132	132
30	46	99	115	125	128	135	146
60	25	49	72	96	115	128	186
120	14	20	22	55	78	122	227
180	11	14	16	29	75	119	
240	6.8	10.2	12	18	64	117	
360	3.6	5.8	7.5	10	37	115	
600	2.8	2.9	3.9	4.4	14	114	
720	1.01	1.6	1.9	3.1	4.5	110	
1020	0.90	1.4	1.7	2.4	3.2	106	
1260	0.83	1.14	1.2	1.4	1.7	105	
1500	0.74	0.96	0.99	1.1	1.2	92	
1740	0.63	0.73	0.81	0.85	0.94	90	

Note: Data from actual test on freshwater sediments. Although a 6-ft test depth is recommended, an 8-ft depth was used in this test.

TABLE 7

PERCENT OF INITIAL CONCENTRATION WITH TIME

Time, T, min	Depth From Top of Settling Column		
	1 ft	2 ft	3 ft
0	100	100	100
30	35	75	87
60	19	37	55
120	11	15	17
180	8	11	12
240	5	8	9
360	3	4	6
600	2.0	2.2	3.0
720	1.0	1.2	1.4

Note: Initial suspended solids concentration = 132 g/ℓ

TABLE 8

AVERAGE CONCENTRATION OF THE FRESHWATER SEDIMENTS FROM LONG-TERM COLUMN TEST AS A FUNCTION OF TIME

Time, days	Concentration g/ℓ
1	190
2	217
3	230
4	237
5	240
6	242
7	244
9	249
10	247
15	256

develop Table 7.

6. Plot the percent of initial concentration versus depth profile for each time interval from data given in Table 7. (See Figure 72.)

7. Average concentration from long-term column test as a function of time (15-day settling column data) (see Table 8).

8. Plot concentration versus time from data in Table 8 as shown in Figure 73.

9. Laboratory tests indicate that 20 percent of the sediment is coarse-grained material (>40 sieve); therefore, the volume of coarse-grained material V_{sd} is:

$$V_{sd} = 300{,}000\ (0.20) = 60{,}000\ yd^3$$

and the volume of fine-grained material V_i is:

$$V_i = 300{,}000 - 60{,}000 = 240{,}000\ yd^3$$

Compute detention time required for sedimentation

The design detention time required for sedimentation in the containment area is determined by the computations illustrated in the following steps. The laboratory data are used to develop Figures 72 and 73. Figure 74 is developed from computations of suspended solids removal percentages using Equation 22. The required suspended solids removal is used to determine an allowable theoretical detention time. This time is increased by a factor of 2.25 to account for short-circuiting and dispersion in the containment area.

1. Calculate removal percentages for depths of 1, 2, and 3 ft.

$t = 30$ min

$d = 1$ ft

$C_1 = 132\ g/\ell$

$H_{pd} = 2$ ft

$C_e = 4\ g/\ell$

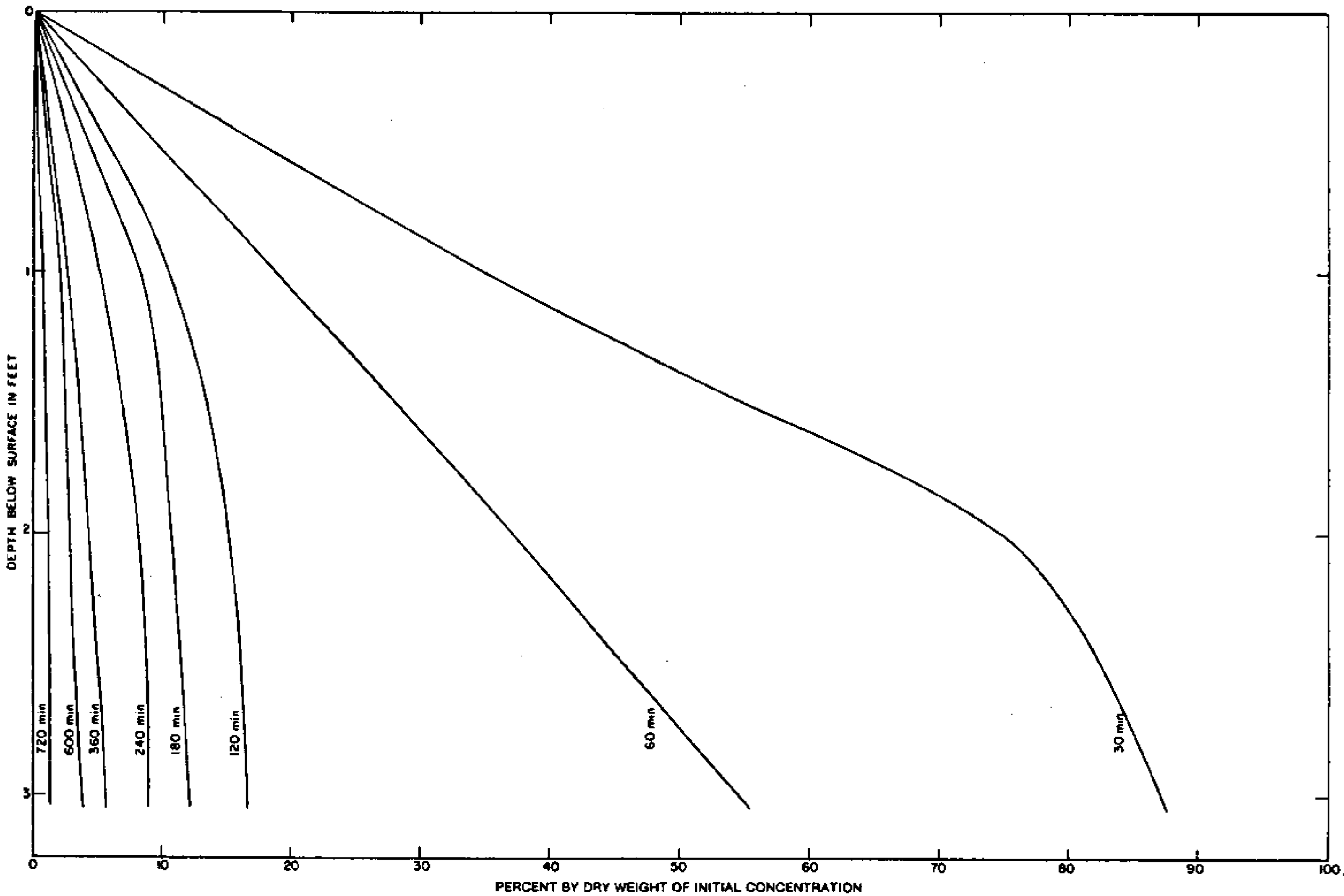

Fig. 72. Percent of initial concentration versus depth profile

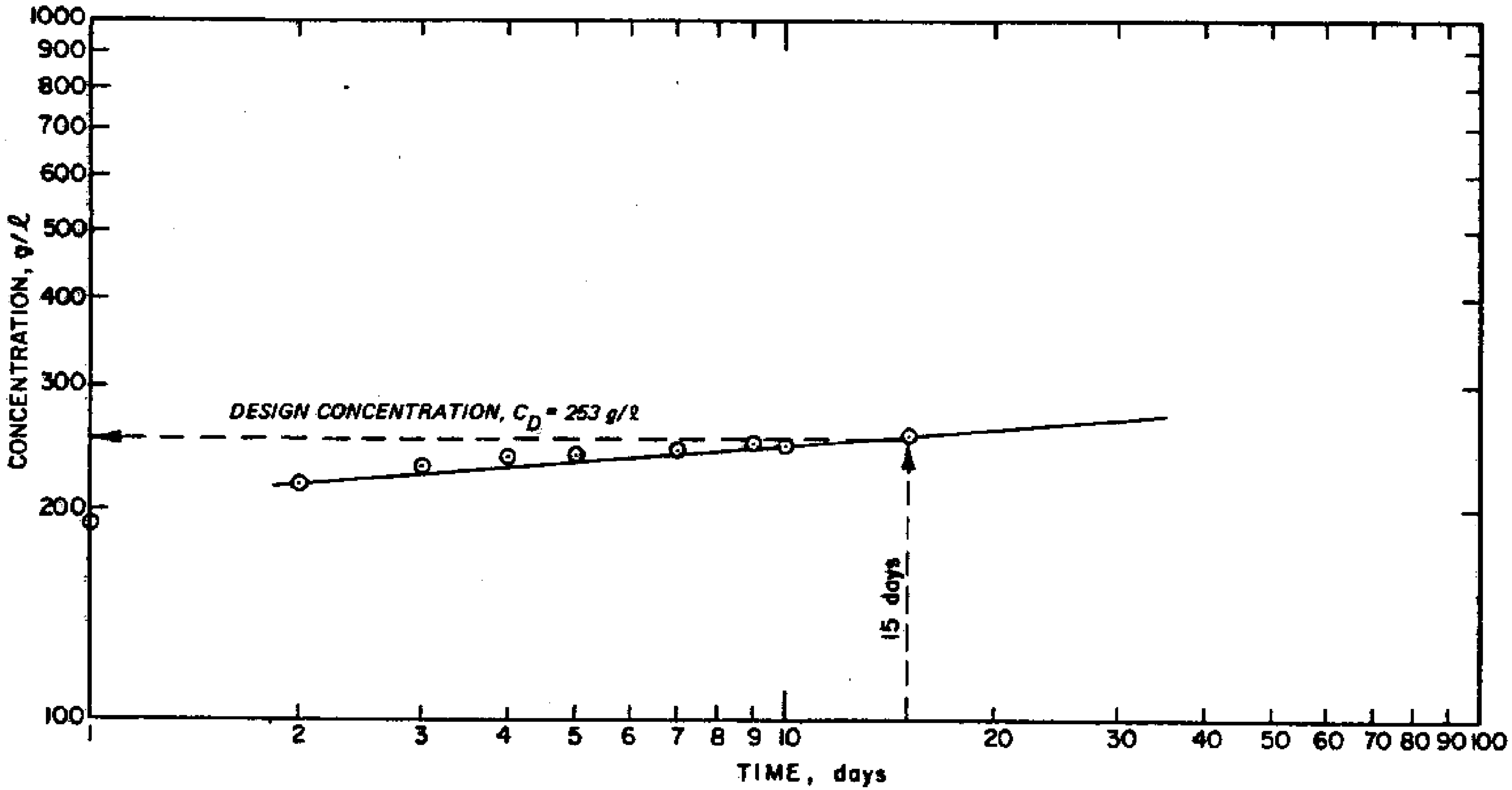

Fig. 73. Concentration determined from column test versus time

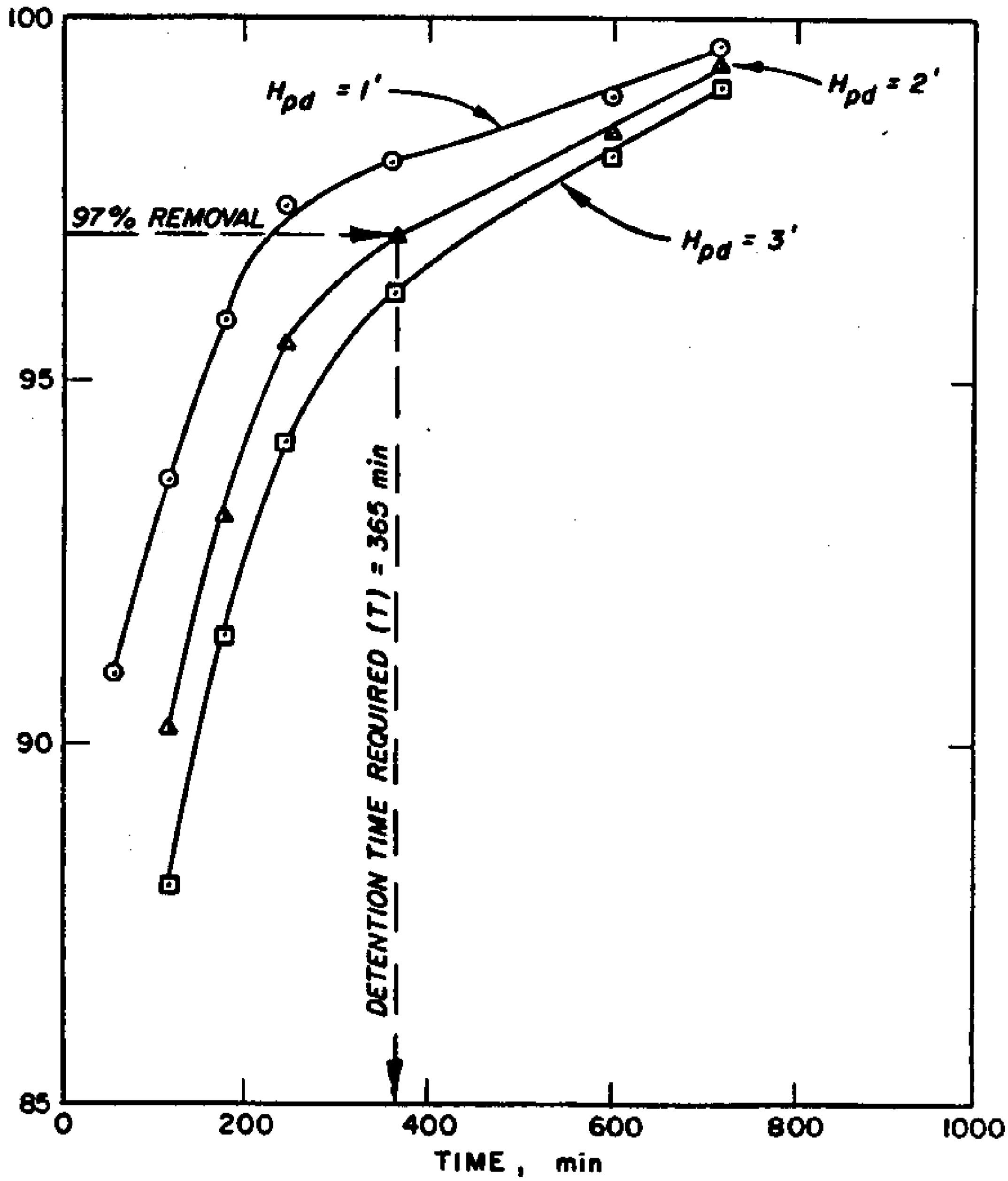

Fig. 74. Solids removal versus time as a function of depth

Calculating the total area down to a depth of 1 ft from Figure 72 gives an area of 100 (scale units). Calculating the area to the right of the 30-min time line down to a depth of 1 ft gives 82.5 (scale units). These areas could also have been determined by planimetering the plot. Compute removal percentages as follows (See Equation 22 in the main text.)

$$R = \frac{82.5}{100} \times 100 = 82.5$$

For a settling time of 30 min, 82.5 percent of the suspended solids are removed from the water column above the 1-ft depth.

2. The calculations illustrated in step 1 are repeated for each depth as a function of time and the results are tabulated in Table 9.

3. Plot the data in Table 9 as shown in Figure 74.

4. Since the average ponding depth H_{pd} is 2 ft, use the 2-ft depth curve shown in Figure 74 and determine the theoretical detention time required to meet the 4-g/ℓ effluent suspended solids requirement:

$$\text{Required Solids Removal} = \frac{C_i - C_e}{C_i}$$

$$\text{Required Solids Removal} = \frac{132 - 4}{132} = 0.97 \text{ or } 97 \text{ percent}$$

5. From Figure 74: T = 365 min.

6. Increase the theoretical detention time T by a factor of 2.25:

$$T_d = 2.25\ T \qquad (23)$$

$$T_d = 2.25\ (365)$$

design detention time, T_d = 822 min.

Compute volume required for sedimentation

$$V_B = Q_i T_d \qquad (29)$$

$$Q_i = \frac{\left(\frac{18 \text{ in.}}{12}\right)^2 \pi}{4} \times 15 \text{ ft/sec}$$

$$Q_i = 26.5 \text{ ft}^3/\text{sec}$$

$$Q_i = 1590.4 \text{ ft}^3/\text{min}$$

$$V_B = 1590.4 \text{ ft}^3/\text{min } (822 \text{ min}) \approx 1{,}300{,}000 \text{ ft}^3$$

TABLE 9

REMOVAL PERCENTAGES AS A FUNCTION OF SETTLING TIME

Time, min	Depth From Top of Settling Column		
	1 ft	2 ft	3 ft
30	82.5	62.0	47.0
60	91.0	81.0	73.0
120	93.7	90.2	88.1
180	95.8	93.1	91.5
240	97.4	95.5	94.2
360	98.0	97.0	96.2
600	98.9	98.4	98.1
720	99.6	99.3	99.1

Compute design concentration and estimate volume required

The design concentration is determined from the long-term column test and used to estimate the volume that will be occupied by the dredged material in the containment area at the end of the dredged material disposal process.

1. Project information:

   (a) Dredge size: 18 in.

   (b) Volume to be dredged: 300,000 yd^3

   (c) Average operating time: 17 hr/day

   (d) Production: 600 yd^3 per hour

2. Estimate time of dredging activity:

$$T = \frac{300{,}000\ yd^3}{600\ yd^3/hr} = 500\ \text{hours}$$

$$\frac{500\ \text{hours}}{17\ \text{hr/day}} = 29.4\ \text{or}\ 30\ \text{days}$$

3. Average time for dredged material consolidation:

$$\frac{30\ \text{days}}{2} = 15\ \text{days}$$

4. Design solids concentration C_D is the concentration shown in Figure 73 at 15 days.

$$C_D = 253\ g/\ell$$

5. Compute the average void ratio e_o using Equation 24.

$$e_o = \frac{G_s \gamma_w}{\gamma_d} - 1 \qquad (24)$$

$$G_s = 2.69$$

$$\gamma_w \approx 1000\ g/\ell$$

$$\gamma_d = 253\ g/\ell$$

$$e_o = \frac{2.69(1000)}{253} - 1$$

$$e_o = 9.63$$

6. Compute the change in volume of fine-grained channel sediments after disposal in containment area using Equation 25:

$$\Delta V = V_i \frac{e_o - e_i}{1 + e_i} \tag{25}$$

$$e_i = \frac{(w/100)\ G_s}{S_d/100} \tag{19}$$

$$e_i = \frac{(85/100)(2.69)}{1.00}$$

$$e_i = 2.29$$

$$V_i = 240{,}000 \text{ yd}^3$$

$$\Delta V = \frac{9.63 - 2.29}{1 + 2.29} \times 240{,}000 = 535{,}440 \text{ yd}^3$$

7. Estimate volume required by dredged material in containment area using Equation 26:

$$V = V_i + \Delta V + V_{sd} \tag{26}$$

$$V_i = 240{,}000 \text{ yd}^3$$

$$\Delta V = 535{,}440 \text{ yd}^3$$

$$V_{sd} = 60{,}000 \text{ yd}^3$$

$$V = 835{,}440 \text{ yd}^3$$

<u>Estimate thickness of dredged material layer</u>

1. Design area is equal to existing surface area given in preliminary data:

$$A_d = 96 \text{ acres} \times 43{,}560 \text{ ft}^2/\text{acre}$$

$$A_d \simeq 4{,}200{,}000 \text{ ft}^2$$

2. Compute the thickness of dredged material using Equation 27:

$$H_{dm} = \frac{V}{A_d} \tag{27}$$

$$H_{dm} = \frac{835{,}440 \text{ yd}^3 \times 27}{4{,}200{,}000 \text{ ft}^2} = 5.4 \text{ ft}$$

Determine required containment area depth

The containment area depth can be computed using Equation 28:

$$D = H_{dm} + H_{pd} + H_{fb} \tag{28}$$

$$D = 5.4 + 2 + 2$$

$$D = 9.4 \text{ ft}$$

It is recommended that a value of 2 ft be required for both ponding depth (H_{pd}) and freeboard (H_{fb}). Because of the foundation conditions, dike heights are limited to 10 ft. However, the depth required for the containment area is 9.4 ft. Sufficient volume is available for the project because the dikes can be constructed to 10 ft.

Evaluate volume available for sedimentation near the end of the disposal operation

$$V* = H_{pd}A_d \tag{31}$$

$$V* = 2 \text{ ft } (4,200,000 \text{ ft}^2)$$

$$V* = 8,400,000 \text{ ft}^3$$

Compare V* and V_B

Since $V* > V_B$, a 96-acre containment area will meet the suspended solids effluent requirement of 4 g/ℓ for the entire disposal operation.

Check weir length

Existing effective weir length L_e = weir crest length L for rectangular weirs:

$$L_e = 24 \text{ ft}$$

$$C_e = 4 \text{ g/ℓ}$$

$$Q_i = 26.5 \text{ ft}^2\text{/sec}$$

$H_{pd} = 2$ ft

With an average ponding depth within the containment area, H_{pd}, of 2 ft, the ponding depth at the weir D_p is estimated to be in excess of 3 ft, accounting for a dredged material surface which slopes toward the weir. Using Figure 75, a 3-ft ponding depth at the weir required an effective weir length of approximately 13 ft. The existing 24-ft weir length should therefore be adequate, but effluent suspended solids should be monitored periodically.

The remaining volume of 1,600,000 yd^3 in the existing containment area is sufficient to accommodate disposal of the 300,000 yd^3 of maintenance channel sediment into the basin under a continuous disposal operation. Since the required basin depth is less than existing depth, no upgrading will be necessary to accommodate the first dredging operation.

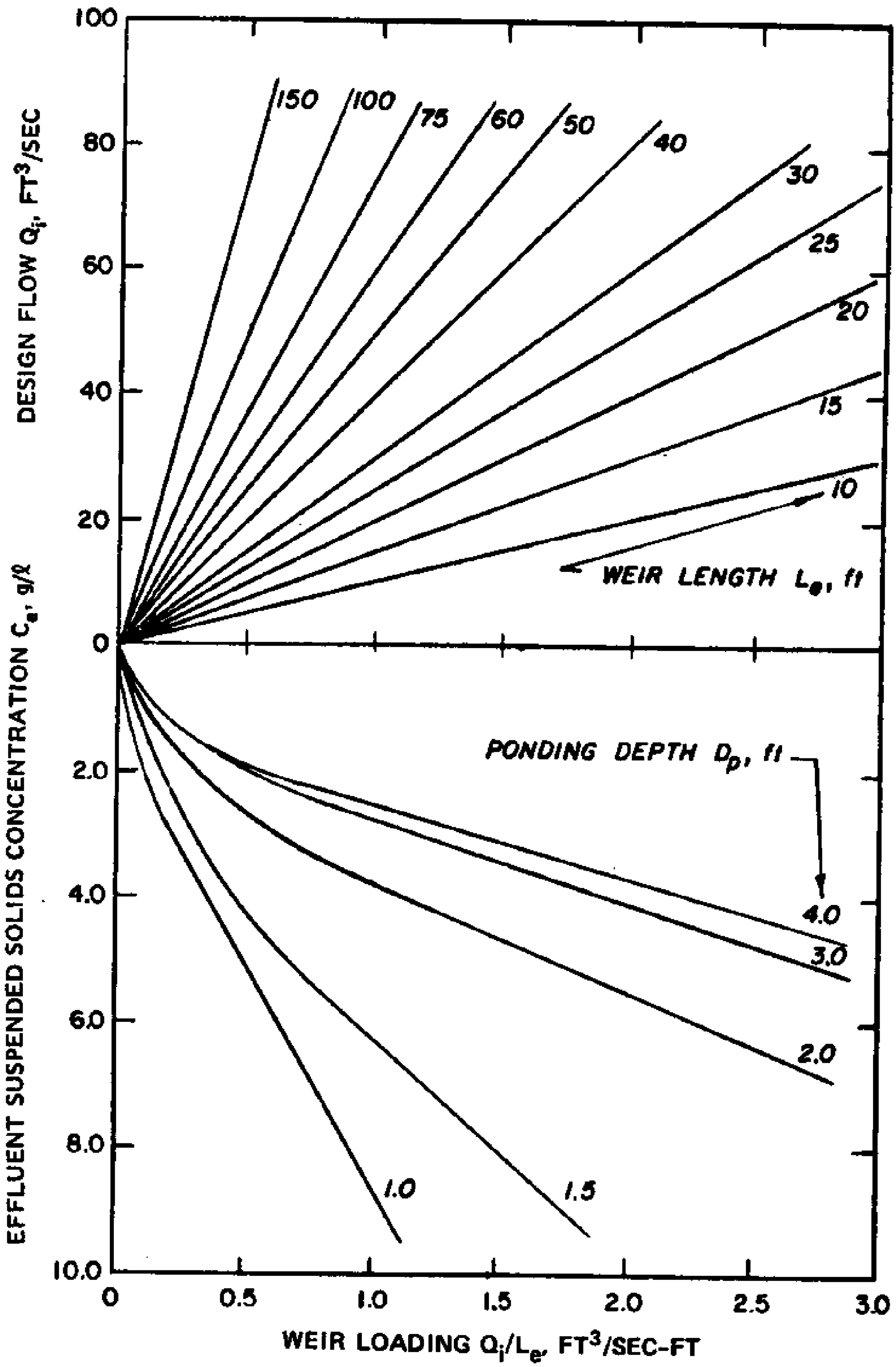

Fig. 75. Weir design nomogram for freshwater clays (19)

Example II: Sedimentation Basin Design Method for Saltwater Sediments

The steps in the design method for saltwater sediments involve the computation of a design concentration from long-term settling column test data, the computation of an area required for sedimentation, and an estimation of volume required for dredged material storage. The data necessary to complete these steps are obtained primarily from laboratory settling tests.

Project information

Fine-grained material is scheduled to be dredged from a harbor maintained to a project depth of 50 ft. Channel surveys indicated that 500,000 yd^3 of channel sediment must be dredged. All available disposal areas are filled near the dredging activity, but land is available for a new site 2 miles from the dredging project. The sedimentation basin must be designed to accommodate continuous disposal operations while meeting effluent suspended solids levels of 4 g/ℓ. In the past, the largest dredge contracted for the maintenance dredging has been a 24-in. pipeline dredge. This is the largest size dredge located in the area.

Results of laboratory tests

The following data were obtained from laboratory tests:

1. Salinity: 15 ppt.
2. In situ water content of channel sediment, w: 92.3 percent.
3. Specific gravity, G_s: 2.71.

4. Depth to solids interface as a function of time (settling column data). (See Table 10.)

5. Zone settling velocity as a function of concentration. (See Table 11.)

6. Curve of zone settling velocity versus concentration. (See Figure 76.)

7. Calculations of solids loading values: Use data given in Figure 76 to develop Table 12.

8. Solids loading versus solids concentration: Use data in Table 12 to develop Figure 77.

9. Data of concentration from the long-term column test as a function of time data (15-day settling column data). (See Table 13.)

10. Curve of concentration versus time. (See Figure 78.)

11. Representative samples of channel sediments tested in the laboratory indicate that 15 percent of the sediment is coarse-grained material (>40 sieve).

$$V_{sd} = 500{,}000\ (0.15) = 75{,}000\ yd^3$$

$$V_i = 500{,}000 - 75{,}000 = 425{,}000\ yd^3$$

Compute design concentration

The design concentration is defined as the average concentration of the dredged material in the containment area at the end of the dredging disposal project. It is estimated from data obtained from the long-term column settling tests described in Chapter VII.

1. Project information:

   Dredge size: 24 in.
   Volume to be dredged: 500,000 yd^3

2. Good records are available from past years of maintenance dredging in this harbor. They show that each time a 24-in. dredge was used, the dredge averaged operating 12 hours per day and dredged an average of 900 yd^3 per hour.

TABLE 10

DEPTH TO SOLIDS INTERFACE (FEET) AS A FUNCTION OF SETTLING TIME (HOURS)

Time Hours	Initial Suspended Solids Concentration, g/ℓ							
	55	73	120	143	163	215	243	310
0	0	0	0	0	0	0	0	0
0.25	0.230	0.145	0.065	0.050	0.065	0.026	0.010	--
0.50	0.390	0.290	0.165	0.090	0.138	0.050	0.020	0.005
0.75	0.530	0.435	0.270	0.170	0.210	0.075	0.030	--
1.0	0.620	0.535	0.360	0.230	0.276	0.100	0.040	0.009
2.0	0.690	0.635	0.490	0.420	0.430	0.225	0.080	0.020
3.0	0.740	0.680	0.535	0.475	0.467	0.340	0.100	0.025
4.0	0.770	0.700	0.555	0.505	0.495	0.365	0.122	0.035
5.0	0.805	0.710	0.580	0.530	0.510	0.390	0.140	0.050
6.0	0.820	0.730	0.585	0.553	0.515	0.410	0.160	0.070
7.0	0.830	--	--	0.565	--	0.430	0.175	--
8.0	0.840	--	--	0.575	--	0.440	0.188	--
10.0	--	--	--	0.595	--	0.459	0.212	--
20.0	--	--	--	0.655	--	0.522	0.259	0.190
30.0	--	--	--	0.690	--	0.564	0.292	0.250

TABLE 11

ZONE SETTLING VELOCITY AS A FUNCTION
OF SUSPENDED SOLIDS CONCENTRATION

Concentration			Zone Settling Velocity
g/ℓ	%	lb/ft^3	ft/hr
55	5.2	3.4	1.238
73	6.8	4.5	0.571
120	10.8	7.5	0.410
143	12.7	9.0	0.245
163	14.3	10.2	0.282
215	18.5	13.5	0.133
243	20.7	15.2	0.041
310	25.8	19.5	0.015

TABLE 12

CALCULATIONS OF SOLIDS LOADING VALUES*

Concentration Suspended Solids C			Settling Velocity v_s	Solids Loading $S = v_s(C)$
%	g/ℓ	lb/ft^3	ft/hr	$lb/hr\text{-}ft^2$
6.1	65	4	1.15	4.60
7.4	80	5	0.88	4.40
14.2	160	10	0.23	2.30
20.4	240	15	0.05	0.87
26.0	320	20	0.02	0.29
31.2	400	25	0.004	0.09

* Developed from curve shown in Figure 76

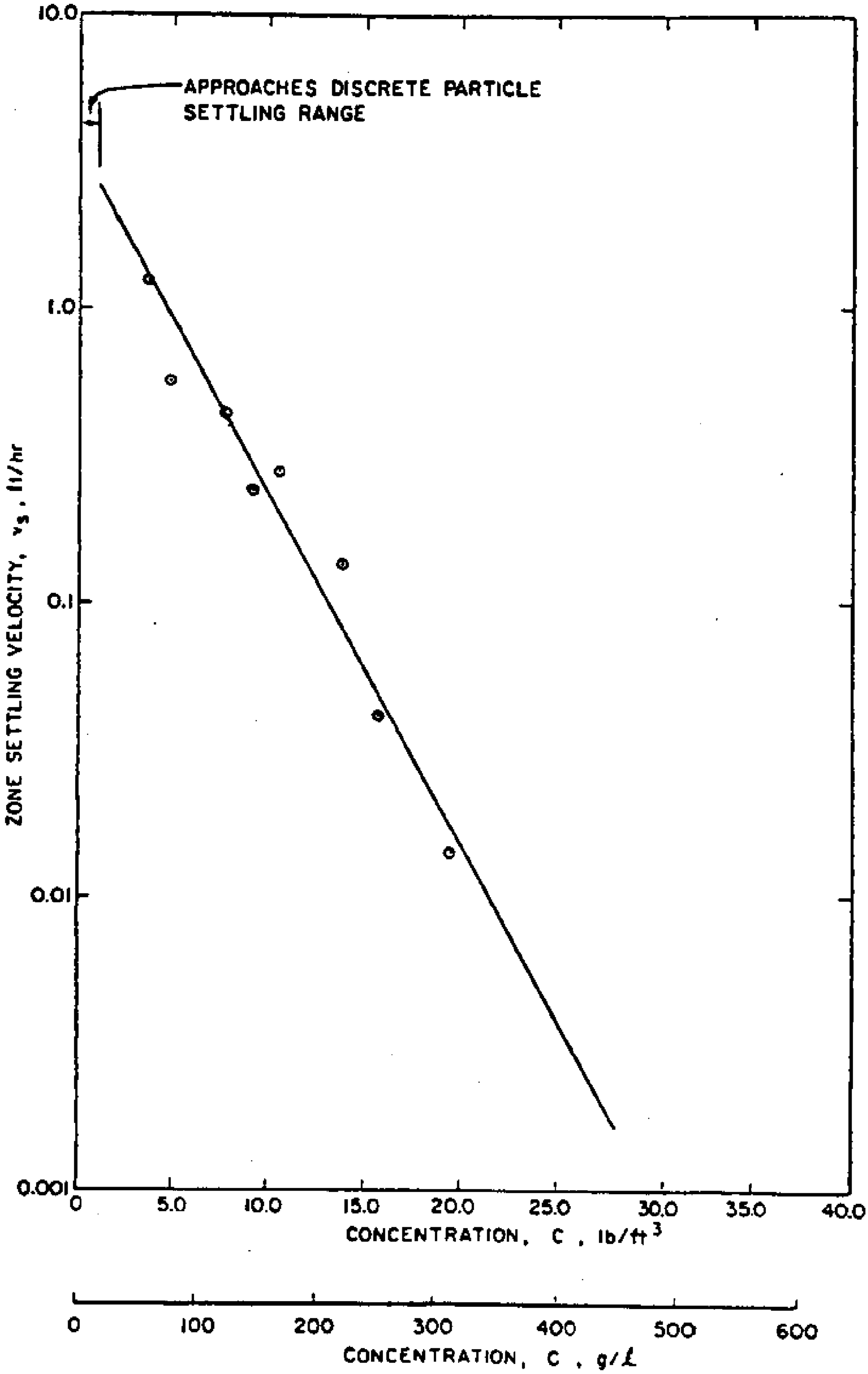

Fig. 76. Zone settling velocity versus concentration

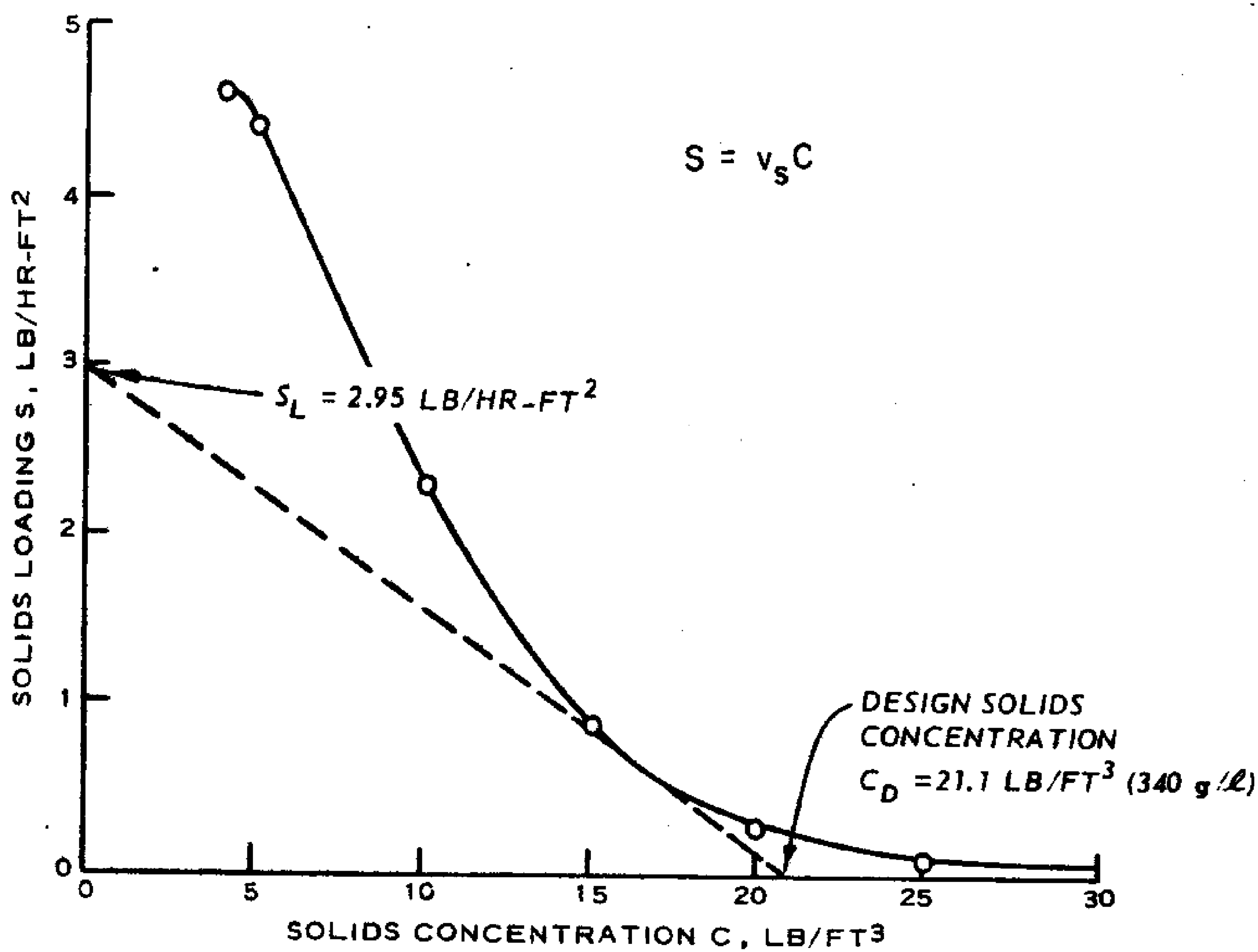

Fig. 77. Solids loading versus concentration

TABLE 13

AVERAGE CONCENTRATION OF THE SALTWATER SEDIMENTS FROM LONG-TERM COLUMN TEST AS A FUNCTION OF TIME

Time days	Concentration g/ℓ
1	192
2	215
3	219
4	240
5	251
6	272
8	280
10	290
15	320

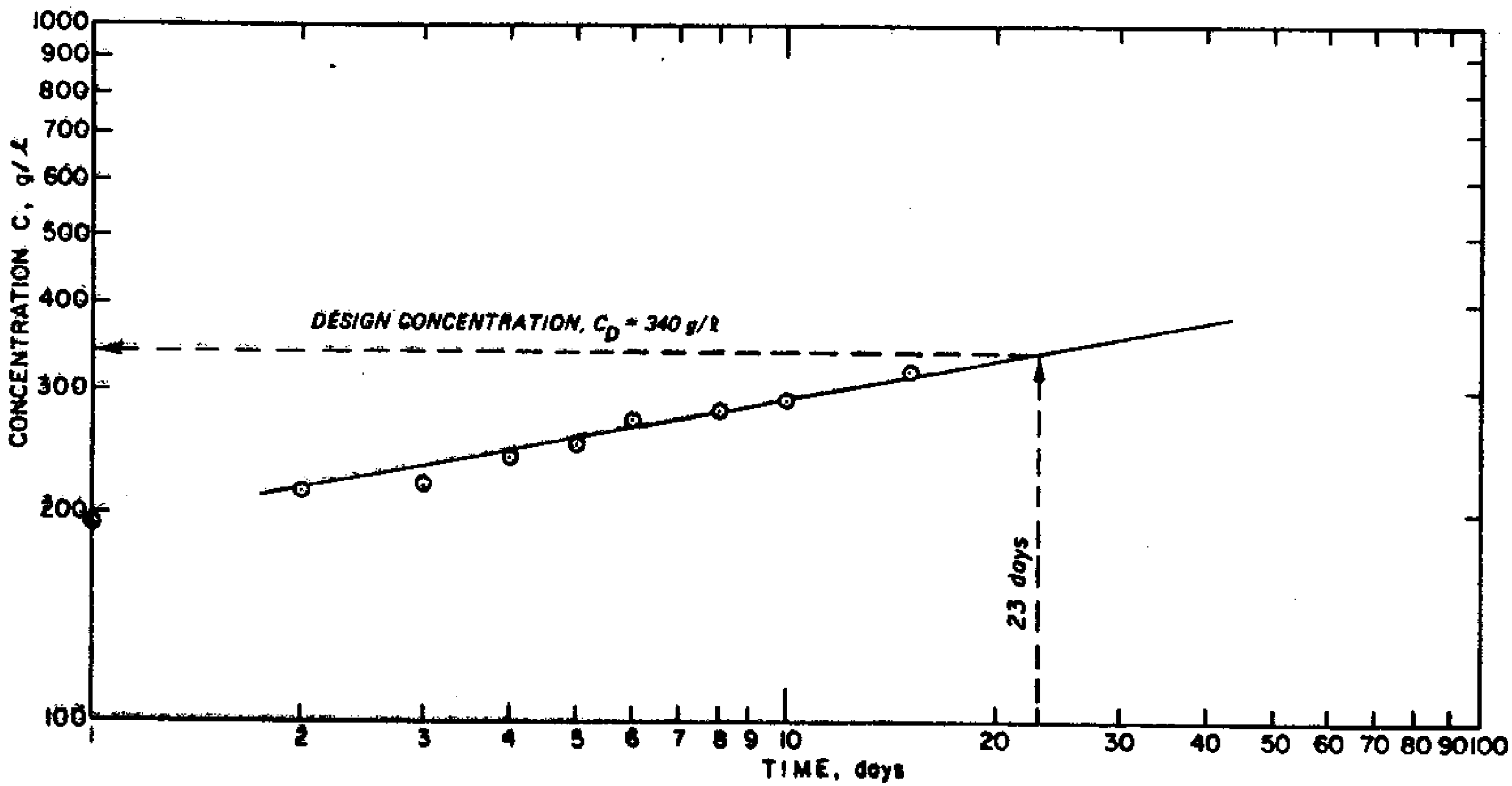

Fig. 78. Concentration determined from column test versus time

3. Estimate time of dredging activity:

$$\frac{500{,}000\ yd^3}{900\ yd^3/hr} = 556\ hours$$

operating time per day = 12 hours

$$\frac{556\ hours}{12\ hours/day} \approx 46\ days$$

4. Average time for dredged material consolidation:

$$\frac{46\ days}{2} = 23\ days$$

5. Design concentration is the concentration shown in Figure 78 at 23 days.

C_D = 340 g/ℓ or 21.1 lb/ft^3

Compute area required for sedimentation

In the saltwater method, the size of the containment area is based on the area required for the zone settling process to concentrate the dredged material to the design concentration. This area is determined using the Yoshioka et al. (11) graphical procedure and increased by a factor of 2.25 to account for short-circuiting and dispersion during disposal operations.

1. Construct an operating line from design concentration (21.1 lb/ft^3) tangent to the loading curve (Figure 77):

   Design solids loading S_L = 2.95 $lb/hr\text{-}ft^2$

2. Compute the area requirement using Equation 20:

$$A = \frac{Q_i C_i}{S_L} \qquad (20)$$

$Q_i = A_p V_d$

V_d = 15 ft/sec

$$C_i = 9.2 \text{ lb/ft}^3$$

$$S_L = 2.95 \text{ lb/hr-ft}^2$$

$$Q_i = \frac{\left(\frac{24 \text{ in.}}{12}\right)^2 \pi}{4} \times 15 \text{ ft/sec}$$

$$Q_i = 47.12 \text{ ft}^3\text{/sec}$$

$$Q_i = 169{,}632 \text{ ft}^3\text{/hr}$$

$$A = \frac{169{,}632\ (9.2)}{2.95}$$

$$A = 529{,}022 \text{ ft}^2$$

$$A = \frac{529{,}022}{43{,}560} = 12.14 \text{ acres}$$

3. Increase the area by a factor of 2.25:

$$A_d = 2.25\ (12.14 \text{ acres})$$

$$A_d = 27.3 \text{ acres}$$

Thus, the area required for sedimentation is 27 acres.

Estimate volume required for dredged material and depth of containment area

After the containment area has been sized to provide for sedimentation, the area must then be sized to accommodate the volume of dredged material to be placed into the area. The total volume required includes volume for storage of dredged material, volume for sedimentation (ponding depth), and freeboard volume to account for wind effects (volume above water surface). The procedures given below and in Chapter VIII can be used to design for volume required for one disposal activity, or used to evaluate the adequacy of the volume provided by an existing containment area.

1. Compute average void ratio using:

$$e_o = \frac{G_s \gamma_w}{\gamma_d} - 1 \quad (24)$$

$G_s = 2.71$

$\gamma_w = 1000$ g/ℓ

$\gamma_d = 340$ g/ℓ = design concentration, C_D (Figure 78)

$$e_o = \frac{2.71(1000)}{340} - 1$$

$$e_o = 6.97$$

2. Compute the change in volume of fine-grained channel sediments after disposal in the containment area using:

$$\Delta V = \frac{e_o - e_i}{1 + e_i} \times V_i \quad (25)$$

Using equation 19 in the main text,

$$e_i = \frac{(w/100)G_s}{S_d/100}$$

$$e_i = \frac{(92.3/100)(2.71)}{1.00}$$

$$e_i = 2.5$$

$$V_i = 425{,}000 \text{ yd}^3$$

$$\Delta V = \frac{6.97 - 2.50}{1 + 2.50} \times 425{,}000$$

$$\Delta V = 542{,}785 \text{ yd}^3 = 543{,}000 \text{ yd}^3$$

3. Estimate the volume required by the dredged material in the sedimentation basin using:

$$V = V_i + \Delta V + V_{sd} \quad (26)$$

$V_i = 425{,}000$ yd^3

$\Delta V = 542{,}785$ yd^3

$$V_{sd} = 75,000 \text{ yd}^3$$

$$V = 425,000 + 543,000 + 75,000$$

$$V = 1,043,000 \text{ yd}^3$$

4. Estimate thickness of dredged material at end of disposal operation:

$$H_{dm} = \frac{V}{A_d} \qquad (27)$$

$$H_{dm} = \frac{1,043,000 \text{ yd}^3 \ (27)}{27 \text{ acres } (43,560)}$$

$$H_{dm} = 23.9 \text{ ft}$$

Because of foundation problems, dike heights are limited to 15 ft. Therefore, the area of the disposal area must be increased to accommodate the storage requirements. Use Equation 28 to determine the allowable dredged material height:

$$D = H_{dm} + H_{pd} + H_{fb} \qquad (28)$$

$$D = 15 \text{ ft}$$

$$H_{pd} = 2 \text{ ft}$$

$$H_{fb} = 2 \text{ ft}$$

$$H_{dm} = D - H_{pd} - H_{fb}$$

$$H_{dm} = 15 - 2 - 2$$

$$H_{dm} = 11 \text{ ft}$$

5. Compute new area requirement:

$$H_{dm} = \frac{V}{A_d} \qquad (27)$$

$$A_d = \frac{1,043,000 \text{ yd}^3 \ (27)}{11}$$

$$A_d = 2,560,090 \text{ ft}^2$$

A_d = 59 acres

Design for weir

Q_i = 47.12 ft^3/sec

C_e = 4 g/ℓ

Using Figure 79, operating lines constructed at Q_i = 47.12 ft^3/sec and C_e = 4 g/ℓ indicate possible combinations of ponding depth and effective weir length required. Assuming that a 1-ft ponding depth at the weir is the minimum that could be allowed, a weir length of 35 ft is required. However, a ponding depth of 2 ft is recommended during the operation to provide a margin of safety. Note that 59 acres is the minimum area required for storage of one dredging of 500,000 yd^3 and will not meet the long-term storage capacity requirement.

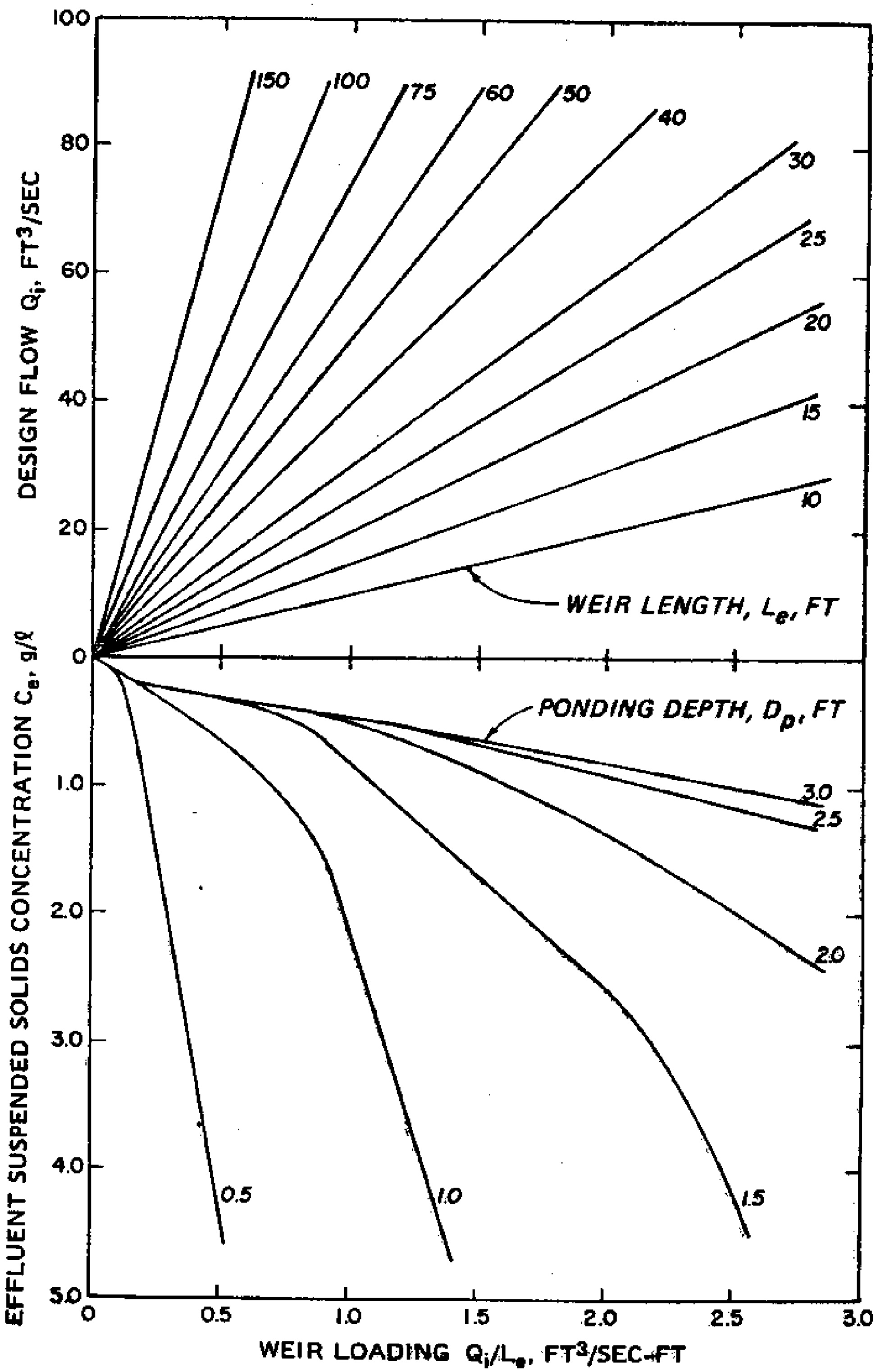

Fig. 79. Weir design nomogram for all silts and saltwater clays (19)

MEASUREMENT AND PREDICTION OF

CONSOLIDATION OF DREDGED MATERIAL

by

W. David Carrier, III[1]

and Berg Keshian, Jr.[1]

ABSTRACT

In recent years, it has become more and more common in the United States to dispose of dredged materials in confined impoundment areas. To estimate the useful life of a disposal area and the time required to achieve reclamation, it is essential that the rate of consolidation and the final density of the dredged material be estimated. Usually this is done on the basis of rules of thumb and local experience. A new method of predicting consolidation is proposed, based on sound geotechnical principles. The method utilized field and laboratory data to determine the compressibility and permeability vs. void ratio of the dredged material. Virtually any sequence of filling and settling in the disposal area can then be analyzed by a specially written computer program, which uses a sophisticated formulation of large strain, non-linear consolidation theory.

The proposed prediction method has been used successfully on a recent dredging project for the U.S. Navy. Two confined disposal areas were intensively studied during and after the initial filling period. A total of 8 million cubic yards of sediment were dredged over a 10-month period. The field investigation included borings, sampling, pore pressure measurements, and settlement measurements.

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The laboratory tests included the usual geotechnical index properties, plus slurry consolidometer tests.

The dredged material naturally separated into two distinct material types: a coarse fraction and a fine fraction. The dredging contractor also assisted this natural process in an attempt to maximize the production of material that would be suitable as fill. The coarse fraction consists of poorly graded silty sands and sands (SM, SM-SP, and SP). On the average, 95% is finer than No. 4 sieve and 13% is finer than a No. 200 sieve. Shortly after deposition, the coarse fraction settles to a void ratio of 0.8 and consolidation is essentially complete almost immediately. In one of the disposal areas, the coarse fraction represents 78% of the total dredged material by weight, but only 46% by volume. The remainder is fine fraction, which consists of sandy silts, sandy clays, and silty clays (ML-MH and CH). Computer simulations of the fine fraction, based on measured material properties, predict the average initial void ratio to be 7.5. This has been confirmed by extensive field sampling. The simulations also predict that approximately one year will be required for complete consolidation and that the final void ratio will be 4. This represents an approximate volume reduction in the fine fraction of 40%. The additional volume will be used to dispose of maintenance dredged material in future seasons. This material is anticipated to be similar to the fine fraction, and consequently, the filling and settling periods for the disposal areas have been projected several years in advance.

SUMMARY

An improved technique for measuring and predicting the consolidation (dewatering) of dredged material is proposed and examples of its application are presented. The method is based on determining the compressibility and permeability of the dredged material by means of a series of field and laboratory tests. Once the soil properties have been determined, finite difference computer programs are used to analyze consolidation under field conditions. These computer programs are based on large-strain, non-linear consolidation theory. Finally, design guidelines are presented which provide a simple and rapid means of estimating the amount and rate of consolidation of dredged material for various disposal situations.

INTRODUCTION

In recent years, it has become more and more common in the United States to dispose of dredged materials in confined impoundment areas. To estimate the useful life of a disposal area and the time required to achieve reclamation, it is essential that the rate of consolidation and the final density of the dredged material be estimated. Usually this is done on the basis of rules-of-thumb and local experience. Previous studies by Salem and Krizek (1973, 1976) and Lacasse, et al. (1977) attempt to apply soil mechanics principles and tests to predict final dredged material densities.

A new method of predicting the consolidation of the fine-grained fraction of dredged material is described in this paper. The method is directly applicable to any fine-grained mineral slurry. It was originally developed by Bromwell Engineering, Inc., to predict the consolidation of waste clay produced by the Florida phosphate industry (Bromwell and Carrier, 1979).

To predict the consolidation of dredged material at a specific site, it is necessary to perform an integrated program of field testing and sampling, laboratory testing, and computer analysis. The purpose of the field and laboratory testing is to measure the compressibility and permeability of the dredged material, such that the constants in the following expressions can be determined:

$$e = A\,\bar{\sigma}^{B}$$

$$k = C\,e^{D}$$

where e = void ratio

$\bar{\sigma}$ = vertical effective stress

k = permeability

A,B,C,D = constants for a specific dredged material.

The first expression determines how much consolidation will occur; the second determines how fast it will occur. A computer analysis can then be performed and virtually any sequence of filling and quiescent settling can be evaluated.

For illustrative purposes, examples of field testing, laboratory testing, and computer analysis have been taken from a dredging project which is part of the construction of a channel and docking facilities near Cumberland Sound in southeast Georgia. Approximately 9 million cubic yards of sediment have been dredged over a 10-month period and deposited in a salt water environment into two confined disposal areas. A map indicating the location of the project is presented in Fig. 1. Bromwell Engineering has investigated the dredged material properties and has designed new disposal areas for future project and maintenance dredging.

The dredged material under study has naturally separated into two distinct material types: a coarse fraction and a fine fraction. The dredging contractor also assisted this natural process in an attempt to maximize the production of material that would be suitable for fill. The coarse fraction consists of poorly graded silty sands and sands. On the average, 95% is finer than a No. 4 sieve and 13% is finer than a No. 200 sieve. During deposition, the coarse fraction sediments to a void ratio of about 0.8 and negligible additional consolidation occurs. In the mainside disposal area, shown in Fig. 1, the coarse fraction represents 78% of the total dredged material by dry weight, but only 46% of the initial volume. The remainder is fine fraction which consists primarily of silty clays (MH-CH), with some sandy silts and sandy clays. All of the examples described in this paper are related to this fine fraction material.

It should be noted that testing and sampling of the dredged material in this project was begun after dredging ha already begun. Thus, the method could not be applied to the design of the first two disposal areas. In fact, these two areas were designed by another organization based on experience and previous practice. However, in principle, it is possible to sample an area to be dredged in advance and perform similar tests and analyses before dredging commences. It is anticipated that this new prediction method will be refined and improved as it is applied to additional dredging projects and greater experience is obtained.

FIELD TESTING

Field testing of a dredged material disposal area presents special problems not ordinarily encountered in typical geotechnical investigations. The sampling and testing operations in a disposal area are generally controlled by the water level. If there are a few inches or more of clear water above the material, then flat-hulled boats are used for access. For areas having very little free water and/or extensive herbaceous vegetation, airboats are especially practical, providing rapid mobility and a convenient work platform. If the water has been drained off, but the surface is still soft, then tracked-amphibious vehicles have been found to be useful. If a crust has formed, access can be accomplished on foot or with any convenient low-pressure vehicle. In most cases, the dredged material is very soft and can be sampled and tested with manually operated equipment. Even if a crust has formed, it is generally feasible to excavate a small hole with a shovel and then sample the underlying soft sediment. Consequently, it is usually not necessary to mobilize a heavy drill rig and barge.

Sampling and testing is performed with equipment that has been specifically developed for very soft, fine-grained soils and slurries, such as dredged material. These are:

- Piston Tube Sampler
- Pore Pressure Probe
- Piezometer
- Nuclear Density Probe

Each of these is described in more detail in the following sections.

Piston Tube Sampler

A schematic of the piston tube sampler is shown in Fig. 2. In operation, the sampler is manually pushed into the soil to a specified depth and a sample is taken by raising the piston. The sample is then extruded into a jar, sealed, and returned to the laboratory for testing. The sample thus recovered is not undisturbed, but the water content is representative of the *in situ* condition. Hence, the total weight (saturated), dry unit weight, and void ratio can be determined with reasonable accuracy. A series of samples are usually taken at the same location at different depths so that a profile of unit weight and void ratio can be determined; an example from the project is shown in Fig. 3. This procedure is repeated at various locations so that representative samples are recovered from the entire disposal area. A typical sampling grid is shown in Fig. 4. In this case, more than 250 samples were taken at about 50 locations, which required two field technicians working for nearly a week.

With the sampling data, it is then possible to calculate the total volume, average depth, mass, and weighted average void ratio of the dredged material. By repeating the sampling procedure at regular time intervals, it is possible to estimate the rate of consolidation. An example is shown in Fig. 5. It can be seen that deposition of fine fraction in the Mainside disposal area began in early October 1978 and continued through mid-March 1979. Initially, the material was placed very rapidly and consequently had a high void ratio. The mass filling rate then decreased for slightly more than a month and the volume actually remained constant as the void ratio decreased. After that, the filling rate picked up again and the volume and void ratio increased.

As will be discussed in a later section, it is possible to perform computer simulations of virtually any sequence of filling and settling. Thus, with some judgment, it is possible to "back-calculate" soil compressibility and permeability values which best fit the data presented in Fig. 5. As more sampling data are gathered over a longer period of time, this is, in fact, the most accurate method of estimating the soil properties.

Pore Pressure Probe

The pore pressure probe is used to obtain rapid profiles of pore pressure with depth. A schematic of the probe is shown in Fig. 6. To obtain pore pressure measurements, the probe is inserted into the dredged material to a specified depth. The fluid pressure is transmitted through the porous tip and acts against the diaphragm of the electrical transducer. The output from the transducer in millivolts is then converted to pressure by means of a calibration curve. It is necessary to hold the probe in position until the electrical reading indicates equilibrium has been achieved, which may take 60 minutes or longer, depending upon the soil characteristics. After a reading has been completed, the probe is pushed deeper into the dredged material to measure the pore pressure at the next specified depth. A typical pore pressure profile is shown in Fig. 7.

Piston tube samples of the dredged material are then taken immediately adjacent to the pore pressure probe location at the same depths. By measuring the total unit weight of the samples, it is possible to calculate the total stress vs. depth, which is also shown in Fig. 7. Then, of course, the effective stress vs. depth is calculated by subtracting the pore pressure from the total stress. Finally, the void ratio of each sample is correlated with the effective stress in the soil at the same depth. Thus, by performing a pore pressure probe and recovering samples, it is possible to directly evaluate the compressibility of the dredged material. This procedure is normally repeated at different locations in the disposal area in order to investigate the variations in material properties.

An important limitation of this method is that if very little consolidation has occurred, then the effective stress is very low. To be able to define the compressibility curve over a greater range of effective stress, it is necessary to repeat the testing procedure at selected time intervals as the degree of consolidation increases. On the other hand, by making repetitive tests, the rate of consolidation can also be determined and, again with the aid of the computer programs, the permeability can be "back-calculated."

Piezometer

Many types of piezometers have been developed for soil mechanics applications. The simple Casagrande-type, consisting of a porous polyethylene cylindrical element attached to a PVC riser pipe, has proven satisfactory for measurements in dredged material. The piezometer will tend to float in the dredged material and generally will not remain at a specified depth. Thus, it is necessary to drive one or two steel rods into the firm foundation soil beneath the disposal area and fix the piezometer at the desired depth.

The piezometers are used to monitor the rate of pore pressure dissipation as well as to check the pore pressure probe results. Falling head tests can also be performed to measure permeability *in situ*. Small gradients are generally employed in these tests in order to avoid hydraulic fracturing of the low density dredged material.

Nuclear Density Probe

A special nuclear density probe employing a low-level Cessium 137 gamma ray source has been developed for *in situ* density determination with depth. The nuclear probe has proven useful in a variety of loose sediments as a supplement to direct sampling.

LABORATORY TESTING

Extensive field testing and sampling of dredged material is not always possible. Furthermore, predictions of consolidation performance are needed before actual field behavior can be determined. Therefore, laboratory techniques have been developed to provide a predictive capability.

Standard geotechnical classification tests are regularly performed on dredged material: water content, Atterberg limits, specific gravity, and grain size. Index properties for the Mainside disposal area is presented in Table 1. The purpose of running these tests is the same as in any geotechnical investigation: to characterize the nature of the material. In addition, scanning electron microscope and X-ray diffraction analyses are usually run on samples from a new disposal area to determine the major minerals. As is well known, the behavior of fine-grained soils can be significantly influenced by mineral composition (c.f., Bromwell and Oxford, 1977).

Two additional tests have been specifically developed for measuring the compressibility and permeability of soft, fine-grained sediments. These are:

- 30-day Settling Test
- Slurry Consolidometer Test

30-Day Settling Test

In a 30-day settling test, a sample of dilute dredged material is simply allowed to settle under self-weight consolidation in a 2000-cm^3 graduated cylinder. At the end of 30 days, when consolidation is essentially complete, the average void ratio of the sample is measured and the effective stress at the mid-height of the column of soil is calculated, thereby establishing a single point on a compressibility curve. By running several tests at different initial conditions, a series of points are determined and a compressibility curve can be defined which is valid for low effective stress.

Slurry Consolidometer Test

A schematic of the slurry consolidometer is shown in Fig. 8. It is similar to an oedometer, except that it has a greater height and diameter. Slurry consolidometers have also been designed by other investigators (Salem and Krizek, 1973; Roma, 1976; Barvenik, 1977) and have been found to be an extremely valuable tool.

To begin a test, a sample of the dredged material slurry is carefully poured into the consolidometer cylinder to a height of approximately 18 inches. The initial void ratio of the sample is usually adjusted in advance to be similar to the void ratio of freshly deposited dredged material in the field (typically, a void ratio of about 25). The sample is then allowed to settle under self-weight consolidation. The height of the sample is periodically measured so that the average void ratio can be calculated versus time. Depending on the type of test being run, either the pore pressure is measured at the base of the sample, or bottom drainage is allowed to occur. The former requires more time to run the test, but allows the permeability of the sample to be estimated during consolidation.

After self-weight consolidation is completed, additional load is applied to the top of the sample by means of the loading piston. Each succeeding increment of load is twice that of the preceding load, similar to an oedometer test. Measurements of the height and permeability of the sample continue to be made with each increment. This is repeated until the compressibility and permeability of the sample have been determined up to the maximum stress level necessary to simulate field conditions. In many cases, the maximum stress is less than 2000 psf, which is significantly less than the usual oedometer test. On the other hand, each slurry consolidometer test may require months to perform, due to the slow rate of consolidation.

COMPRESSIBILITY AND PERMEABILITY

As discussed in the Introduction, the primary purpose of the field and laboratory testing is to develop functional expressions for the compressibility and permeability of the dredged material in the form:

$$e = A\ \bar{\sigma}^{B}$$

$$k = C\ e^{D}$$

where e = void ratio

$\bar{\sigma}$ = vertical effective stress

k = permeability

A, B, C, D = constants for the particular dredged materials.

The objective is to find values for the constants which best fit the data from the various sources. As an example, compressibility and permeability data from sampling, pore pressure probes, 30-day settling tests, and slurry consolidometer tests from the project are presented in Figs. 9 and 10. Best-fit curves through the data points have been determined and are also indicated in the figures. It can be seen that the data are generally very consistent and there is not too much scatter from the best-fit curves.

Based on these best-fit curves, the compressibility and permeability for the dredged material under consideration are given by:

$$e = 10.21\ \bar{\sigma}^{-0.22}$$

$$k = 5.01 \times 10^{-11}\ e^{5.40}$$

when $\bar{\sigma}$ is in g/cm^2 and k is in cm/sec.

COMPUTER ANALYSIS

As dredged material consolidates, the void ratio typically spans an enormous range; e.g., an initial void ratio of 25 to a final void ratio of 4 or less. The classic Terzaghi consolidation theory cannot be used to analyze the behavior of this material, because the theory assumes infinitesimal strain (constant void ratio). Consequently, finite-strain consolidation theory must be utilized. In recent years, many large-strain solutions have been obtained (Gibson, et al., 1967; Olson and Ladd, 1979). Currently, the most useful solutions for predicting the consolidation of dredged materials is a family of finite difference computer programs written by Prof. Frank Somogyi of Wayne State University, under contract to Bromwell Engineering.

These computer programs are based on sound geotechnical principles, and they use a sophisticated formulation of large-strain, non-linear consolidation theory. There are presently eight programs, which have been named:

FLINITS	FLINITD
FLCONTS	FLCONTD
QSNONUS	QSNONUD
QSUNIFS	QSUNIFD

The "FL" programs analyze consolidation during filling for a given period of time, or until a specified height is reached. The "QS" programs analyze consolidation during quiescent settling for a specified period of time. The final letter "D" denotes those programs which permit double drainage through both the top and bottom surfaces of the dredged material. The final letter "S" denotes those programs which are restricted to single drainage through the top surface of the dredged material. All of the programs are one-dimensional and assume that the dredged material is always submerged; desiccation as a function of time cannot be simulated at this time.

The FLINIT programs are used to analyze consolidation during an initial filling period.

The FLCONT programs are used to analyze consolidation during filling over an existing deposit. The FLCONT programs must be used whenever a change in the filling rate occurs, or when filling is resumed after a period of quiescent settling.

The QSNONU programs are used to analyze consolidation during quiescent settling of dredged material in which the void ratio varies with depth. The QSNONU programs are utilized when filling is stopped, either permanently or temporarily.

The QSUNIF programs are used to analyze consolidation during quiescent settling of dredged material in which the initial void ratio is uniform with depth.

In addition to the compressibility and permeability of the dredged material, the computer programs require the following input parameters:

- Area of disposal site
- Maximum height of fill
- Fill rate of dry solids
- Initial void ratio
- Filling period
- Surcharge.

The output includes predicted settlement, void ratio, and pore pressure versus depth and time. The output of one program becomes the input for the next program and virtually any sequence of filling and quiescent settling can be analyzed. A typical analysis may require first a FLINITS run, followed by QSNONUS, FLCONTS, QSNONUS, FLCONTS, etc.

An example is presented in Fig. 11. In this case, the purpose of the analysis was to determine how much dredged material could be placed in the Mainside disposal area. The starting point for the analysis was January 1979; the dredged material had already been consolidating for some time and the surface had settled to an elevation of slightly less than 17 feet. Using the computer program QSNONUS, it was predicted that most of the consolidation would be completed within 200 days. The program FLCONTS was then used to analyze the effect of adding 250,000 tons of maintenance dredged material. These analyses were repeated until it was found that the additional increment of dredged material that could be placed in the disposal area was declining rapidly. This is the effective life of the disposal area, which was predicted to be August 1981. The final height after consolidation was also predicted to be approximately elevation 17.5 feet and the average final void ratio will be 3.6.

DESIGN GUIDELINES

The computer programs described in the previous section are extremely powerful analytical tools. However, they are not particularly convenient for use in design, where a number of variables must be considered, balanced, and traded off. This is common in most fields of engineering: the design for a project is based on experience, local practice, codes, and rules-of-thumb; a sophisticated analysis is then performed to confirm the design. For example, earth dams are frequently designed on the basis of experience; then, the slope stability is analyzed to confirm that the embankment has an adequate factor of safety.

Hence, design guidelines have also been developed for sizing dredged material disposal areas. While not as accurate as the analytical methods, the design guidelines provide a simple and rapid means of estimating the amount and rate of consolidation of dredged material. Once a design has been selected, the analytical methods are used to check and refine the predicted consolidation behavior.

It should be pointed out that these design guidelines have been developed specifically for the conditions at this project. They are not necessarily directly applicable to other dredged material disposal projects. However, as more experience is obtained with these testing and analytical techniques, it is anticipated that the guidelines can be generalized.

There are generally two types of dredged material disposal that must be considered:

- Project dredging, in which a large amount of material is deposited in a short period of time, usually less than a year.
- Maintenance dredging, in which small amounts of material are periodically deposited over the life of the project, typically 30 years or more.

Design guidelines for each of these are discussed in the following sections.

Project Dredging

A typical filling schedule for project dredging is shown in Fig. 12. During the filling period, material is added to the disposal area at a high rate and only limited consolidation can occur. High pore pressures remain at the end of filling and considerable consolidation occurs during the quiescent period. In the example in Fig. 12, the ultimate height of the deposit is less than one-third of the initial height after filling.

Computer analyses of project dredging have been performed over a wide range of conditions, and it has been found that the filling schedule can be approximated as follows:

$$\begin{cases} h_i = 6.07 \times 10^{-5}\, Q^{1.45}\, \Delta t^{0.85} \\ e_i = 0.21\, Q^{0.45}\, \Delta t^{0.15} - 1 \end{cases}$$

$$\begin{cases} h_u = 1.25 \times 10^{-2} (Q\Delta t)^{0.72} \\ e_u = 44.18 (Q\Delta t)^{-0.28} - 1 \end{cases}$$

$$\begin{cases} h_{90} = 3.76 \times 10^{-3}(Q\Delta t)^{0.88} \\ e_{90} = 13.29(Q\Delta t)^{-0.12} - 1 \\ t_{90} \approx \Delta t - 0.15 + 4.40 \times 10^{-6} Q^{1.45} \Delta t^{0.88} \end{cases}$$

where the subscript

i denotes initial conditions after filling

u denotes ultimate conditions after complete consolidation

90 denotes a point when 90% of the consolidation has occurred ($h_{90} = h_i - 0.9(h_i - h_u)$)

and

h = height of the deposit (ft)

e = average void ratio of the dredged material

Q = instantaneous fill rate (tons/yr/ac)

Δt = filling interval (yr)

For example, say the project dredging requires the disposal of 8000 tons of dry solids per acre per year for a period of one year. Then,

$$h_i = 6.07 \times 10^{-5}(8000)^{1.45} (1)^{0.85} = 27.7 \text{ ft}$$

$$e_i = 0.21 (8000)^{0.45} (1)^{0.15} - 1 = 11.0$$

$$h_u = 0.0125(8000 \cdot 1)^{0.72} = 8.1 \text{ ft}$$

and so on.

Thus, it is possible to estimate the initial fill height (which controls the design elevation of the surrounding embankment) and the ultimate height (which controls the eventual land-use/reclamation options) by simple hand calculations.

It is important to note the influence of the exponents in these expressions. Thus, if the filling interval in the previous example is extended to two years, the instantaneous fill rate would be reduced to 4000 t/yr/ac and the initial height would be 18.3 ft. Therefore, the same mass of dredged material could be stored in two-thirds of the initial volume. The ultimate volume, of course, would be precisely the same. Alternatively, if the area of the disposal site is doubled and the filling interval maintained at one year, the instantaneous fill rate would be halved, and the initial height would be just 10.1 ft; the ultimate height would be 4.9 ft.

In this way, various alternatives for operation of the disposal area can be considered and traded off against other factors, such as foundation conditions, pumping distance, availability of embankment construction materials, reclamation alternatives, land-use priorities, etc. After an optimal design has been developed, a more detailed analysis is then performed to check the design and make minor corrections.

Maintenance Dredging

A typical filling schedule for maintenance dredging is shown in Fig. 13. Usually, most of the consolidation of the previously deposited dredged material occurs prior to the addition of the next increment of material. This is what distinguishes maintenance dredging from project dredging.

Again, computer analyses of maintenance dredging have been performed over a wide range of conditions, and it has been found that the filling schedule can be approximated as follows:

$$\begin{cases} h|_t = 2.2\times10^{-3} \frac{Q\Delta t}{T} t^{0.72} \\ e|_t = \frac{7.78}{Tt^{0.28}} - 1 \end{cases}$$

$$\begin{cases} h_i|_{t+\Delta t} = h|_t + 1.57 \times 10^{-4} \quad Q^{1.34} \quad \Delta t^{0.95} \\ e_i|_{t+\Delta t} = \frac{1}{t+1} \left[\frac{7.78t^{0.72}}{T} + \frac{0.55Q^{0.34}}{\Delta t^{0.05}}\right] - 1 \end{cases}$$

where,

t = total elapsed time (yr)

Δt = filling cycle (yr)

h = height of dredged material at end of each filling cycle

h_i = height of dredged material at end of each filling interval

For example, if the instantaneous fill rate is 2000 tons/yr/ac, the filling interval is 0.25 yr, and the filling cycle is 1 yr, then

$$h_i\Big|_{t+\Delta t} = \frac{(0.0022)(2000)(0.25)}{(1)} t^{0.72} +$$

$$1.57 \times 10^{-4}(2000)^{1.34}(0.25)^{0.95} = 1.1t^{0.72} + 1.1$$

thus, at $t+\Delta t = 30.25$ yr,

$$h_i\Big|_{t+\Delta t} = 1.1(30)^{0.72} + 1.1 = 13.8 \text{ ft}$$

and $e_i\Big|_{t+\Delta t} = 2.2$

As before, various alternatives can be considered for operations of the disposal area, and an optimal design can be developed.

CONCLUSIONS

An integrated program has been developed for the measurement and prediction of consolidation of dredged material. The program consists of field testing and sampling, laboratory testing, and computer analysis. Geotechnical principles and test procedures have been used to determine the compressibility and permeability of soft sediments and slurries. The primary objective of the program is to express the void ratio, e, and permeability, k, as follows:

$$e = A \bar{\sigma}^B$$

$$k = C e^D$$

The first expression determines how much consolidation will occur; the second determines how fast it will occur.

In the field, techniques for sampling and testing extremely soft sediments have been developed; these include the piston tube sampler, pressure probe, and nuclear density probe.

In the laboratory, slurry consolidometer testing has been extensively utilized and is now becoming routine. It is expected that many more laboratories will have this equipment in the future.

Finally, sophisticated computer programs have been written which can be used to simulate virtually any sequence of filling and settling in a disposal area. In addition, design guidelines have been developed which simplify the evaluation of a disposal area and significantly reduce the number of computer analyses required. Presently, these design guidelines are only applicable to the project described in this paper; additional research is required to expand and generalize these guidelines for other projects.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance of Dr. R. T. Martin of M.I.T. in the development of field and laboratory equipment, and of Prof. Frank Somogyi of Wayne State University in the development of the computer programs. The authors also thank Dr. L. G. Bromwell for his comments and criticisms of this paper.

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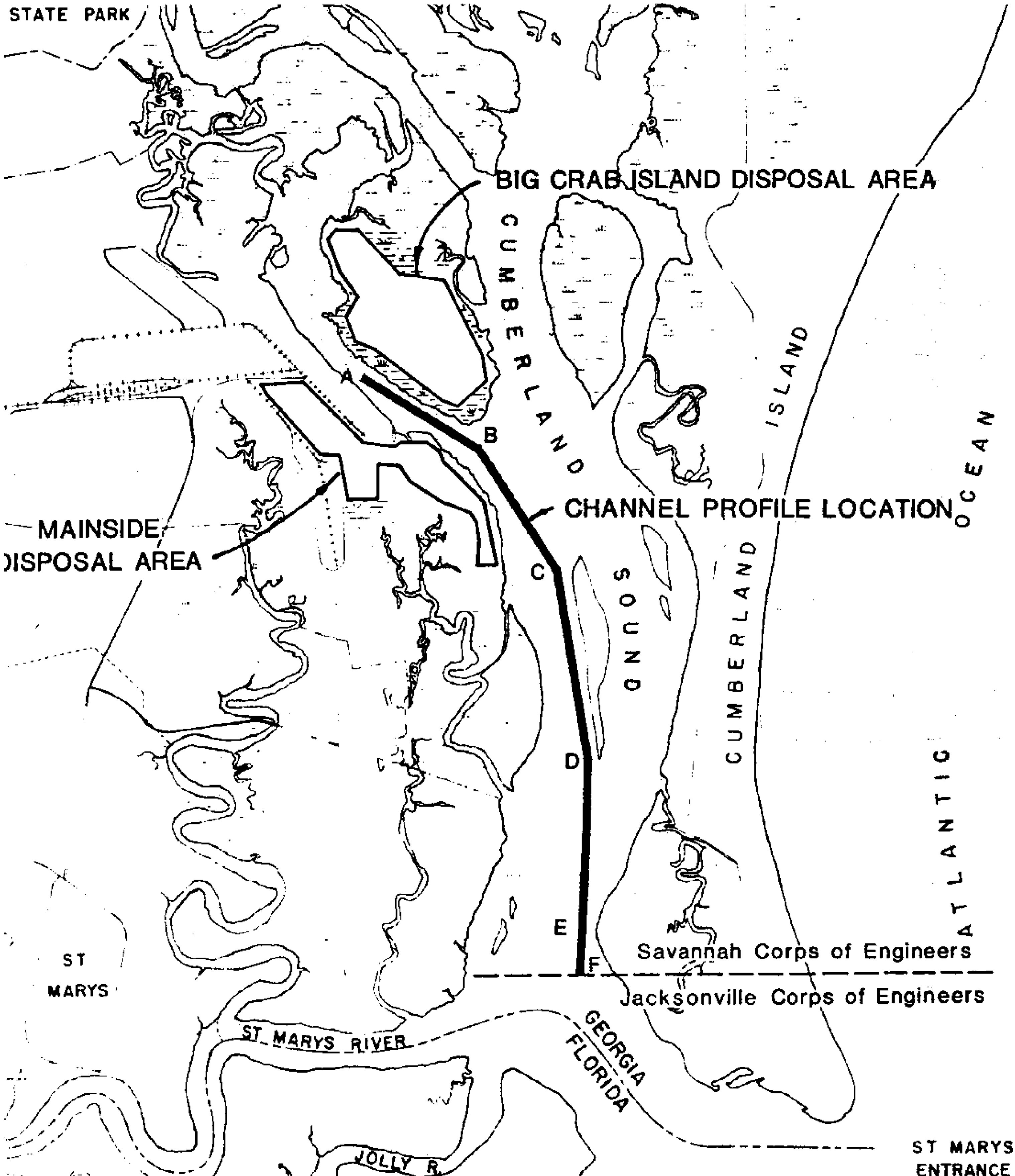
TABLE 1

INDEX PROPERTIES

FINE FRACTION OF KINGS BAY DREDGED MATERIAL

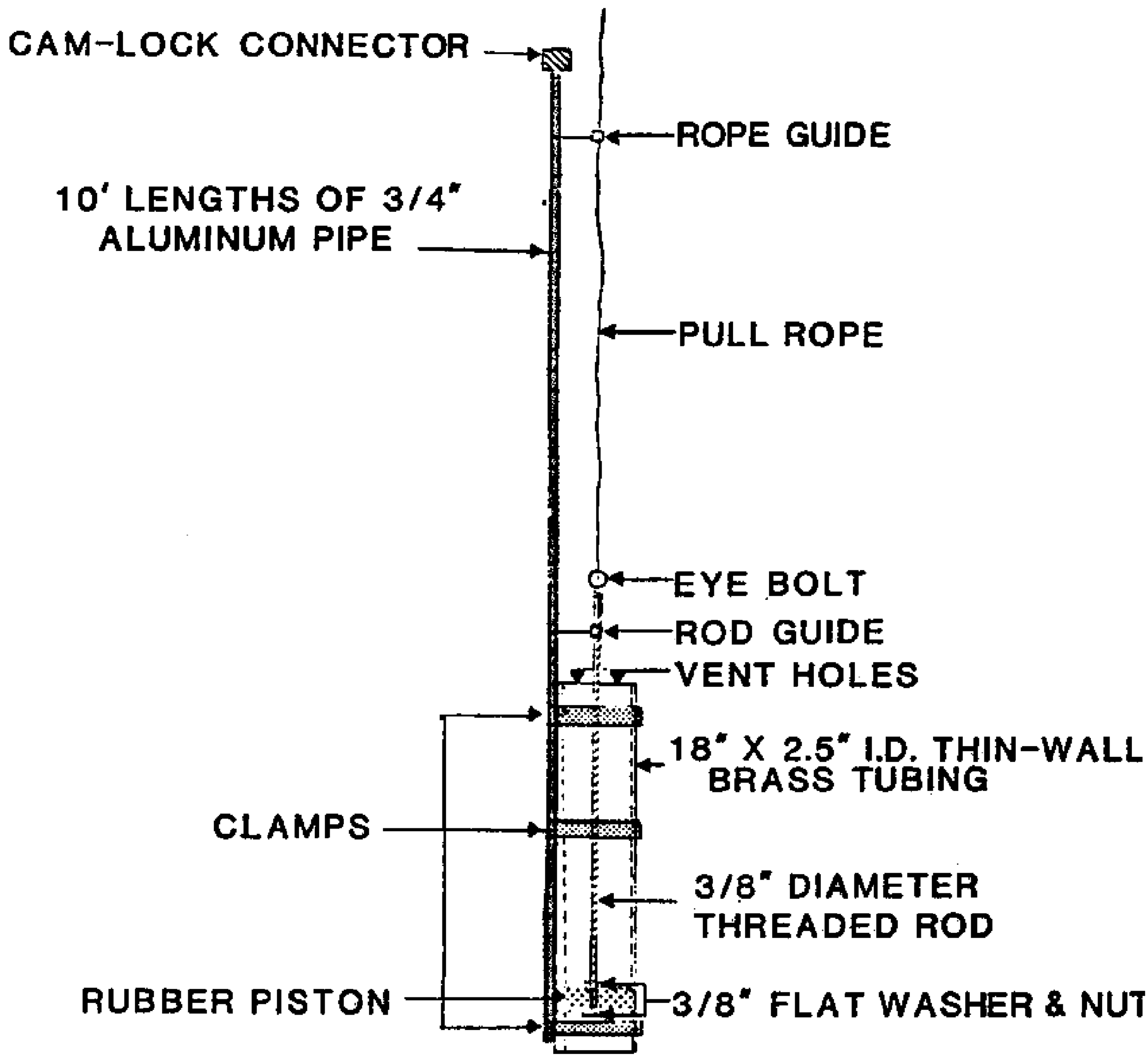
Classification	MH, CH	
Grain Size	79% finer than a No. 200 sieve	
Plasticity	LL = 40 - 200 PL = 20 - 60 PI = 20 - 140	
Specific Gravity	2.6	
Dry Density	19 pcf	(Initial)
Water Content	290 %	(Initial)
Void Ratio	7.5	(Initial)

LIST OF FIGURES



Plan View of Dredging Project

FIGURE 1



Schematic of Piston Tube Sampler

FIGURE 2

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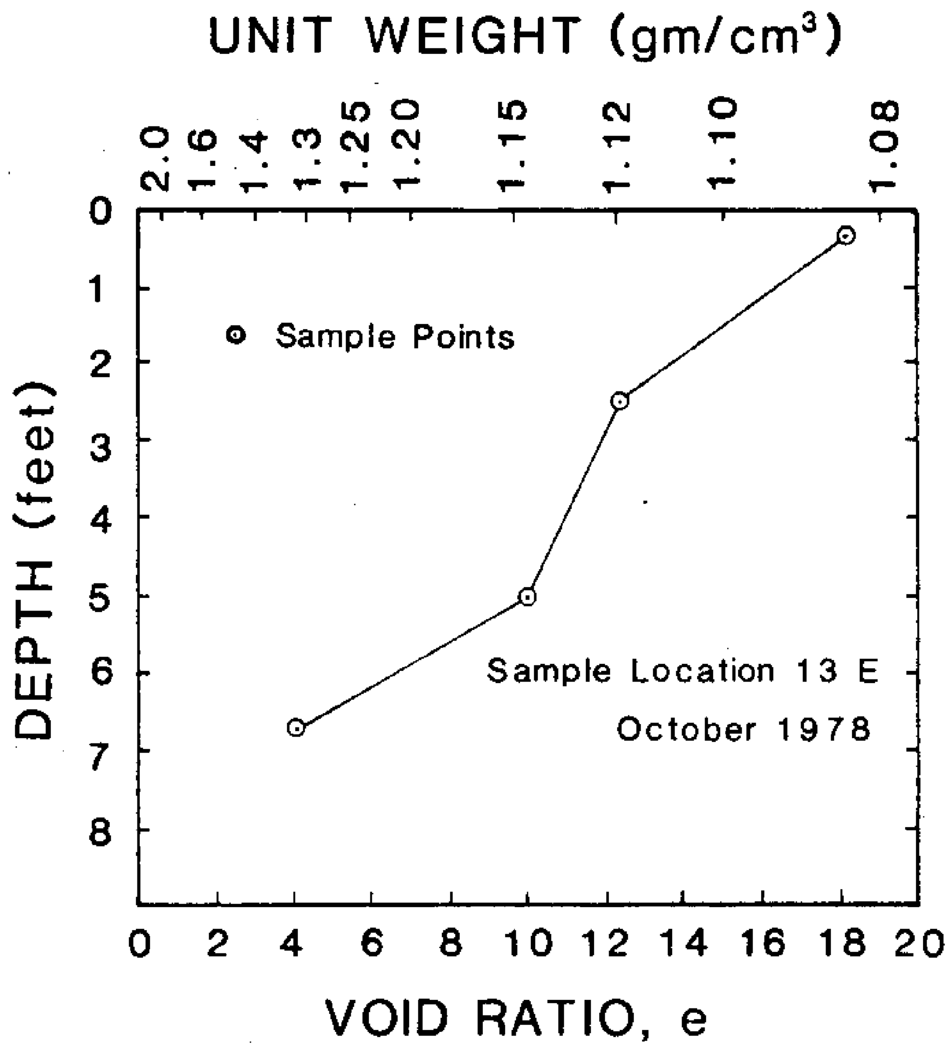
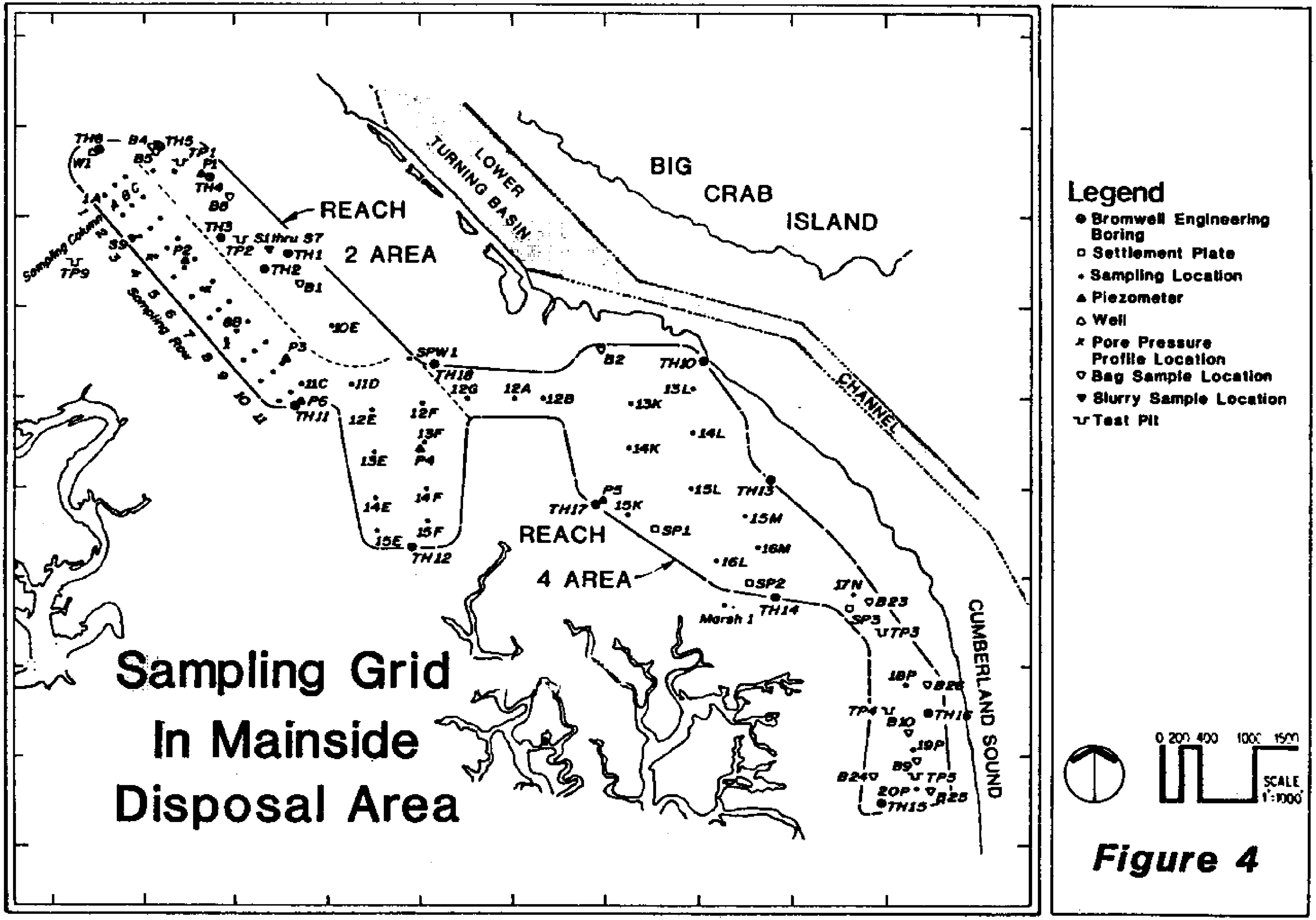


Profile of Unit Weight and Void Ratio in Dredged Material

Figure 3



BIG
CRAB
ISLAND
LOWER
TURNING BASIN
CHANNEL
CUMBERLAND SOUND
REACH
2 AREA
REACH
4 AREA
Sampling Column
Sampling Row
Marsh 1
Sampling Grid
In Mainside
Disposal Area
Legend
Bromwell Engineering Boring
Settlement Plate
Sampling Location
Piezometer
Well
Pore Pressure Profile Location
Bag Sample Location
Slurry Sample Location
Test Pit
0 200 400 1000 1500
SCALE 1":1000'

Figure 4

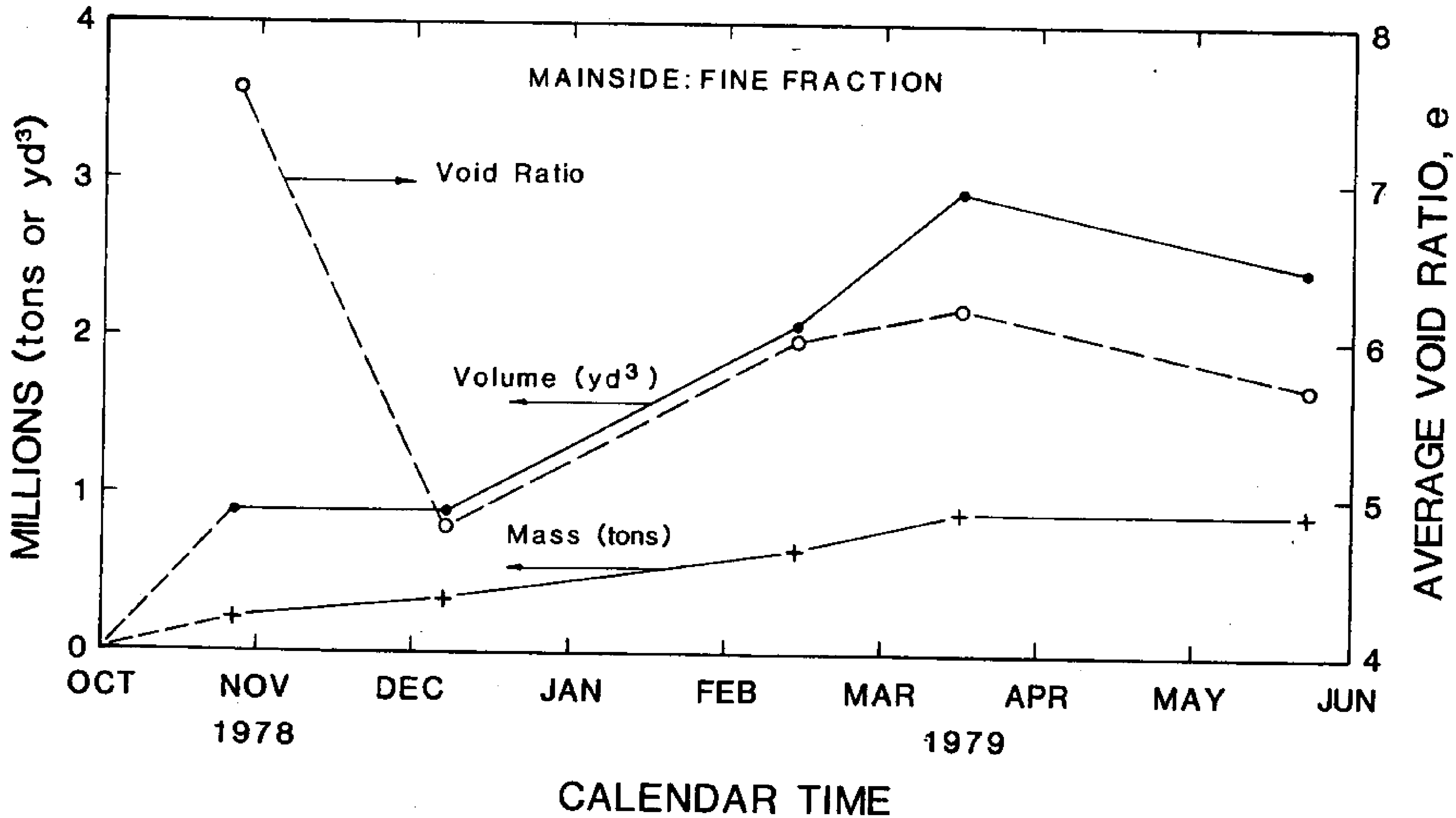
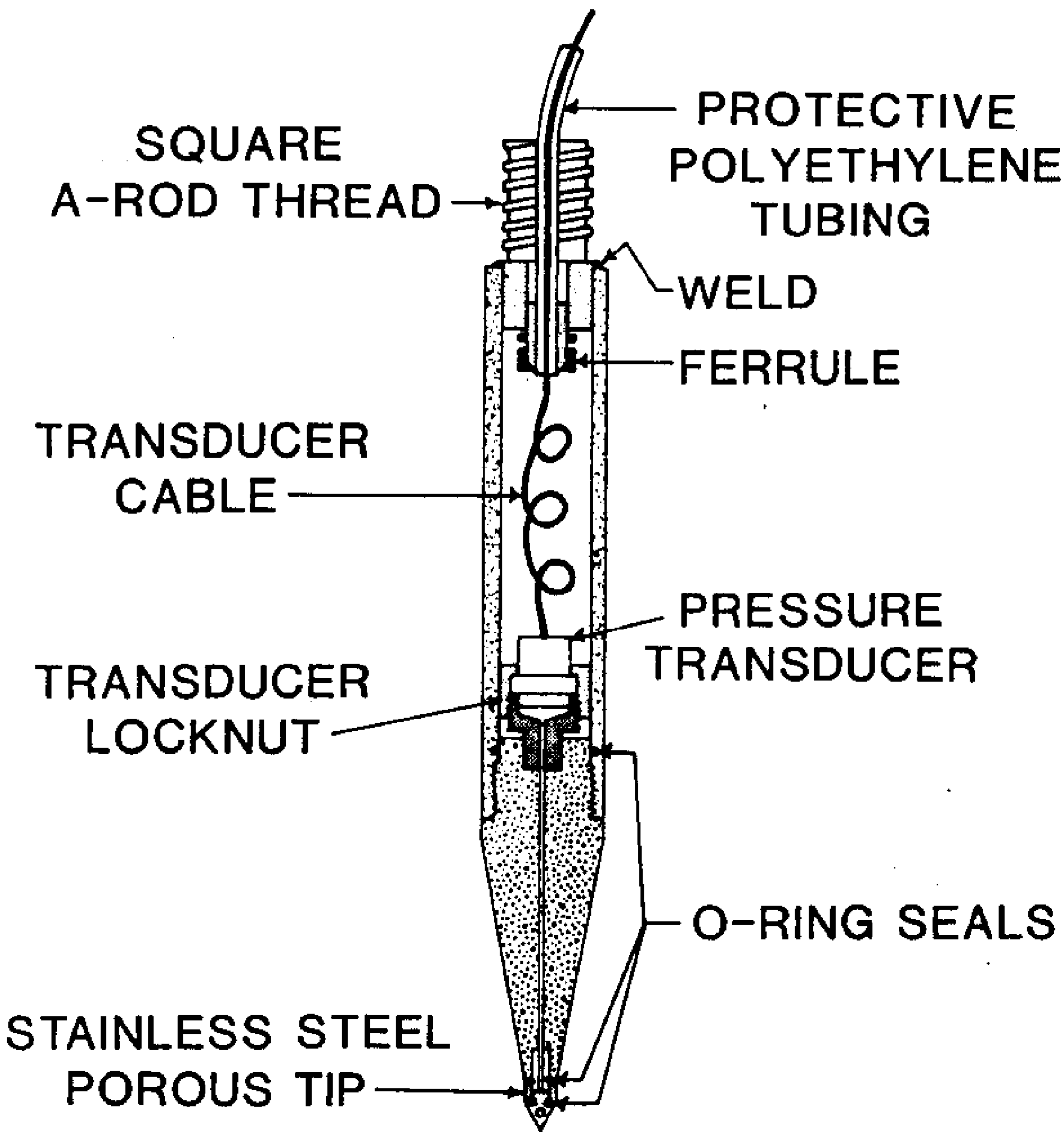
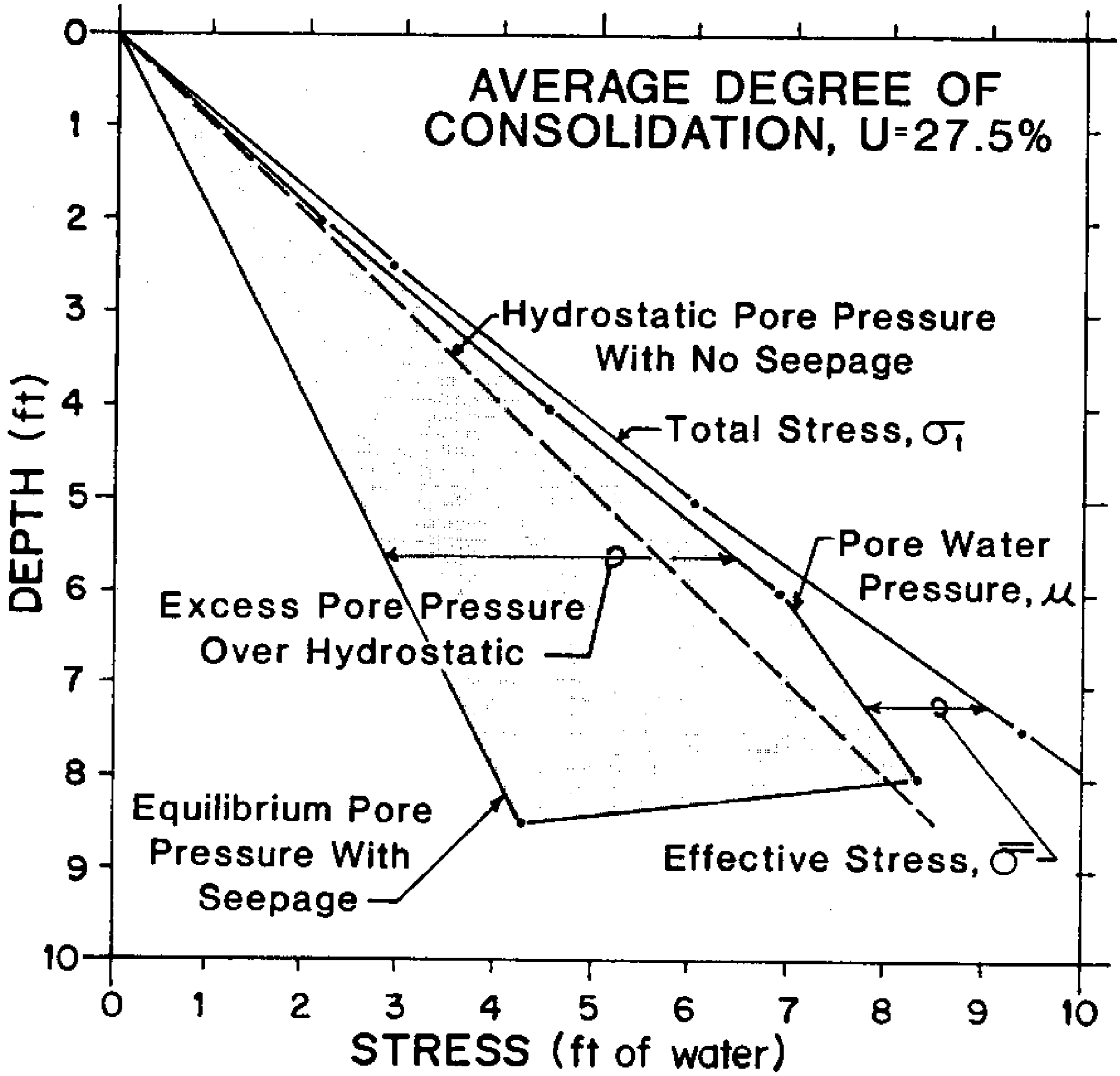

MAINSIDE: FINE FRACTION
Void Ratio
Volume (yd³)
Mass (tons)
MILLIONS (tons or yd³)
AVERAGE VOID RATIO, e
0
1
2
3
4
5
6
7
8
OCT
NOV
1978
DEC
JAN
FEB
MAR
APR
MAY
JUN
1979
CALENDAR TIME

Figure 5



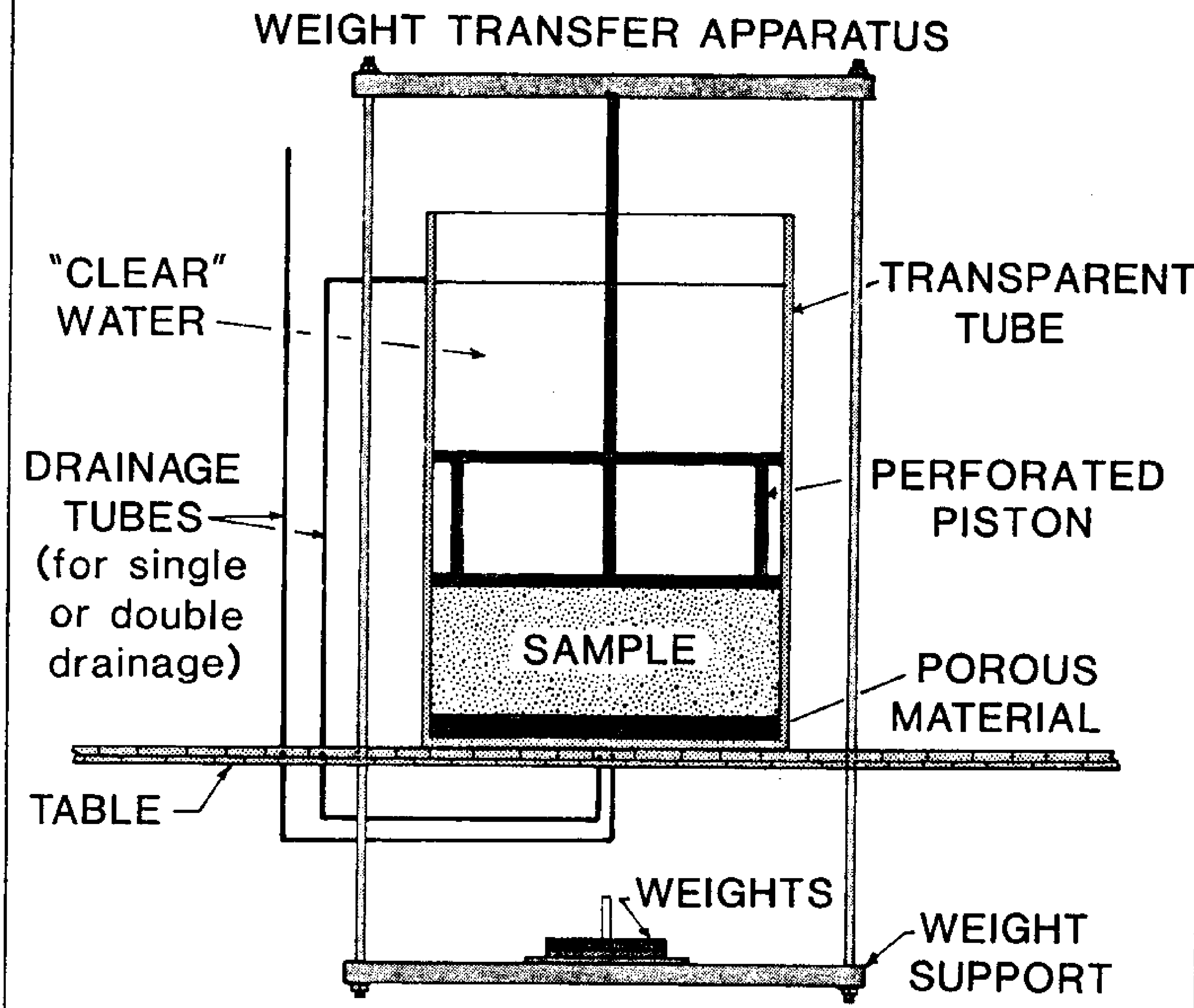
Schematic of Pore Pressure Probe

FIGURE 6



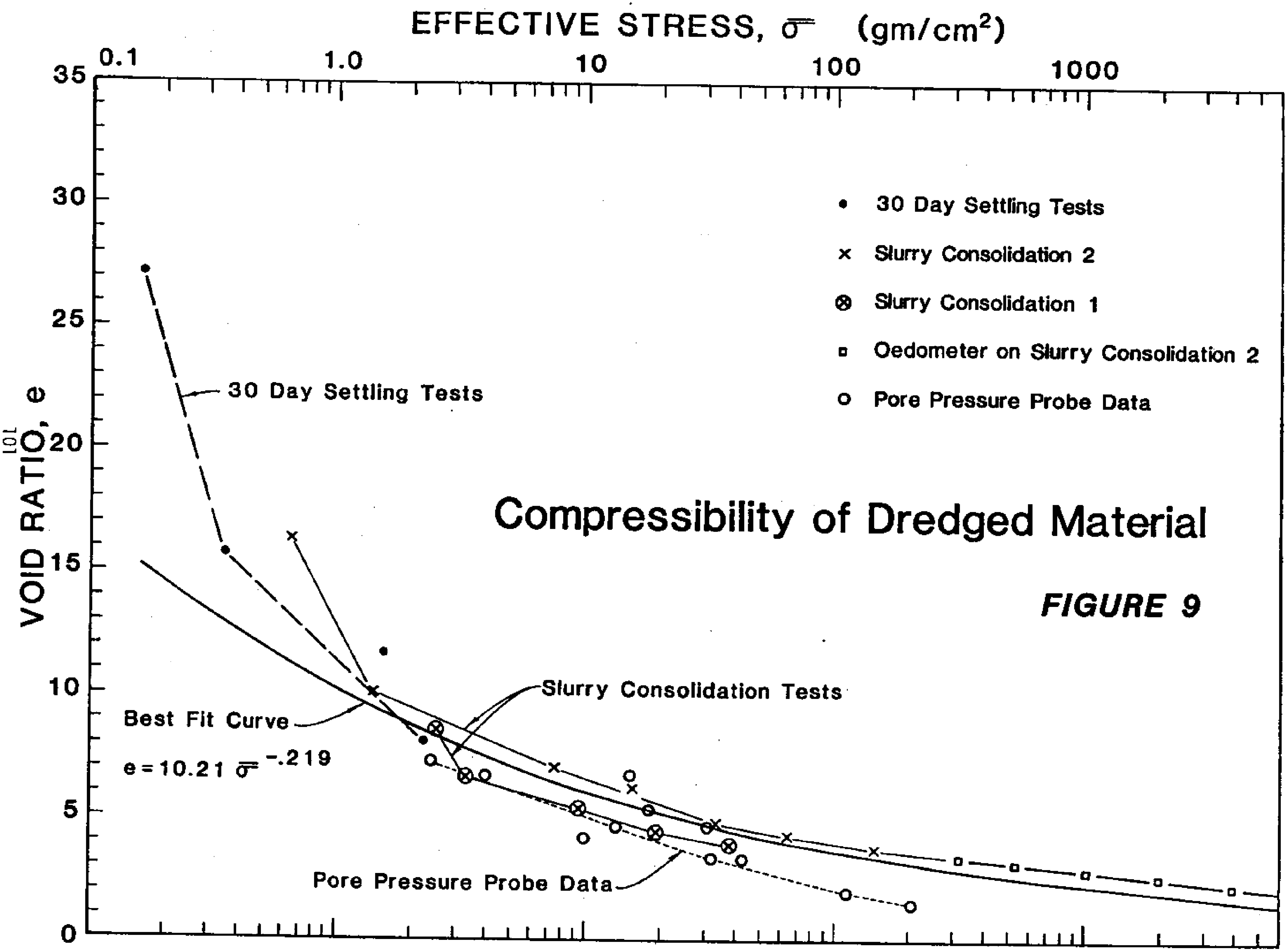
Profile of Pore Pressure, Total Stress, and Effective Stress

Figure 7



Schematic of Slurry Consolidometer

FIGURE 8



Compressibility of Dredged Material

FIGURE 9

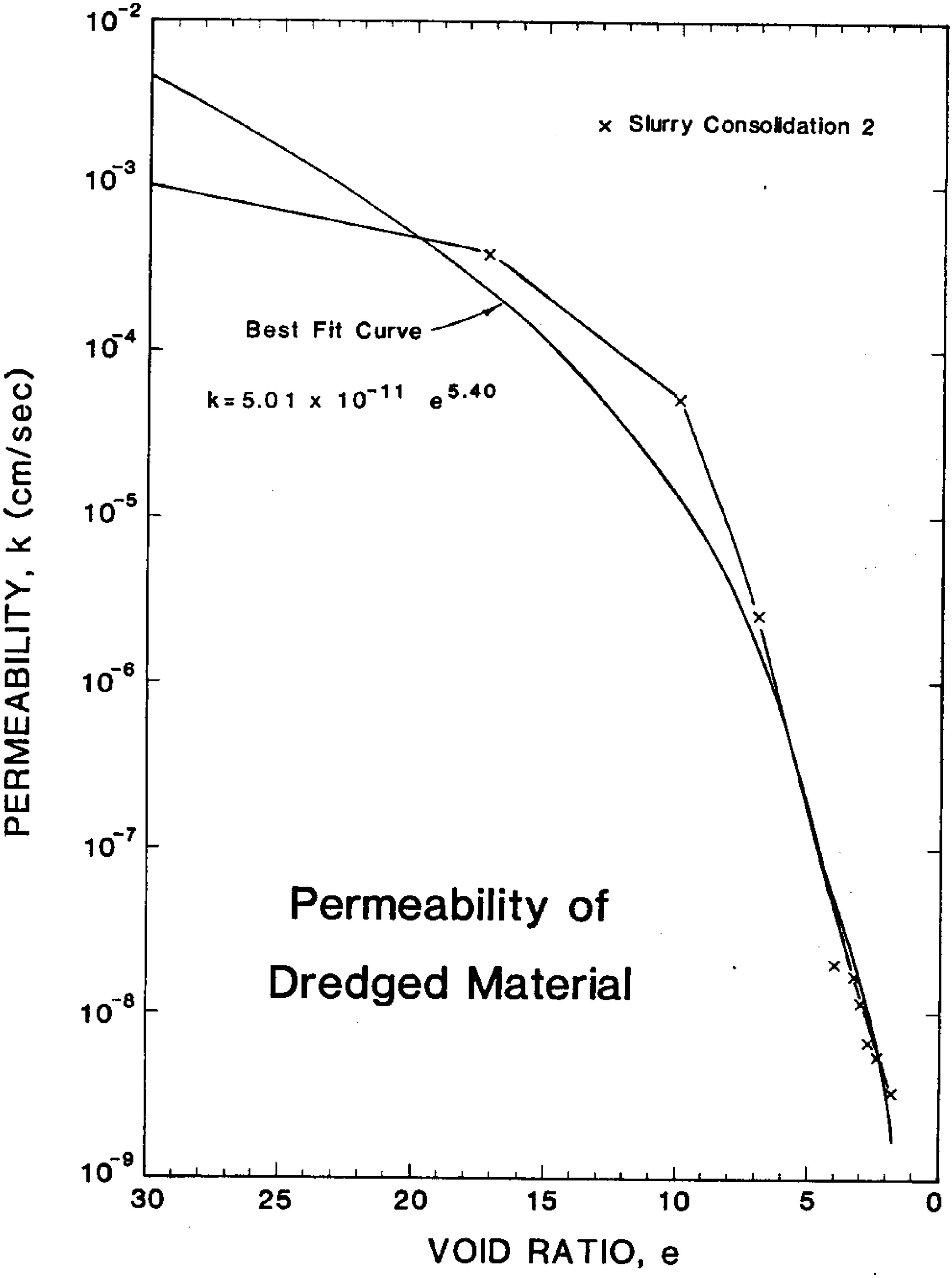
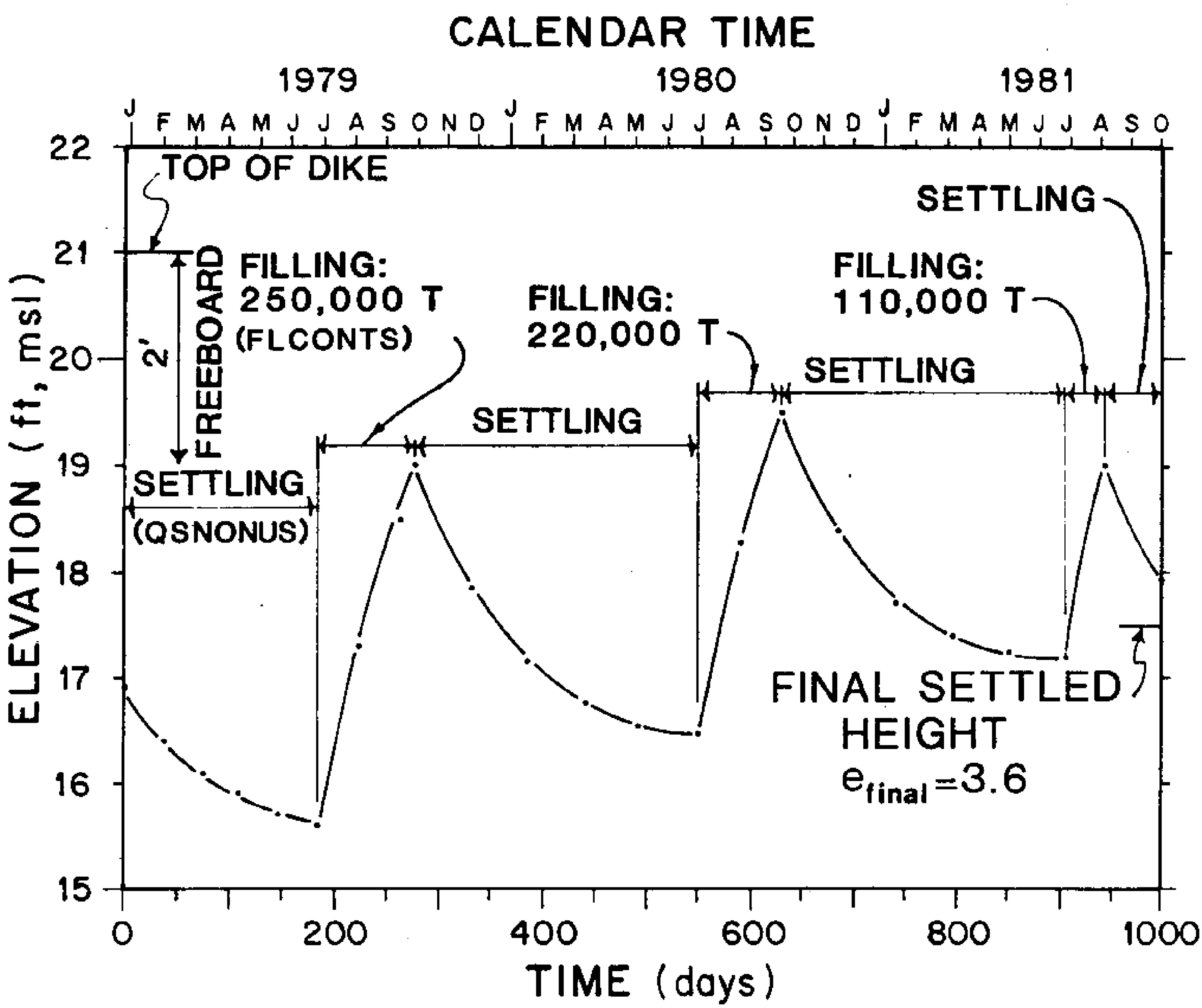

× Slurry Consolidation 2
Best Fit Curve
k = 5.01 x 10⁻¹¹ e^5.40
Permeability of
Dredged Material
PERMEABILITY, k (cm/sec)
VOID RATIO, e
10⁻²
10⁻³
10⁻⁴
10⁻⁵
10⁻⁶
10⁻⁷
10⁻⁸
10⁻⁹
30
25
20
15
10
5
0

Figure 10



Computer Analysis of Consolidation of Dredged Material

FIGURE 11

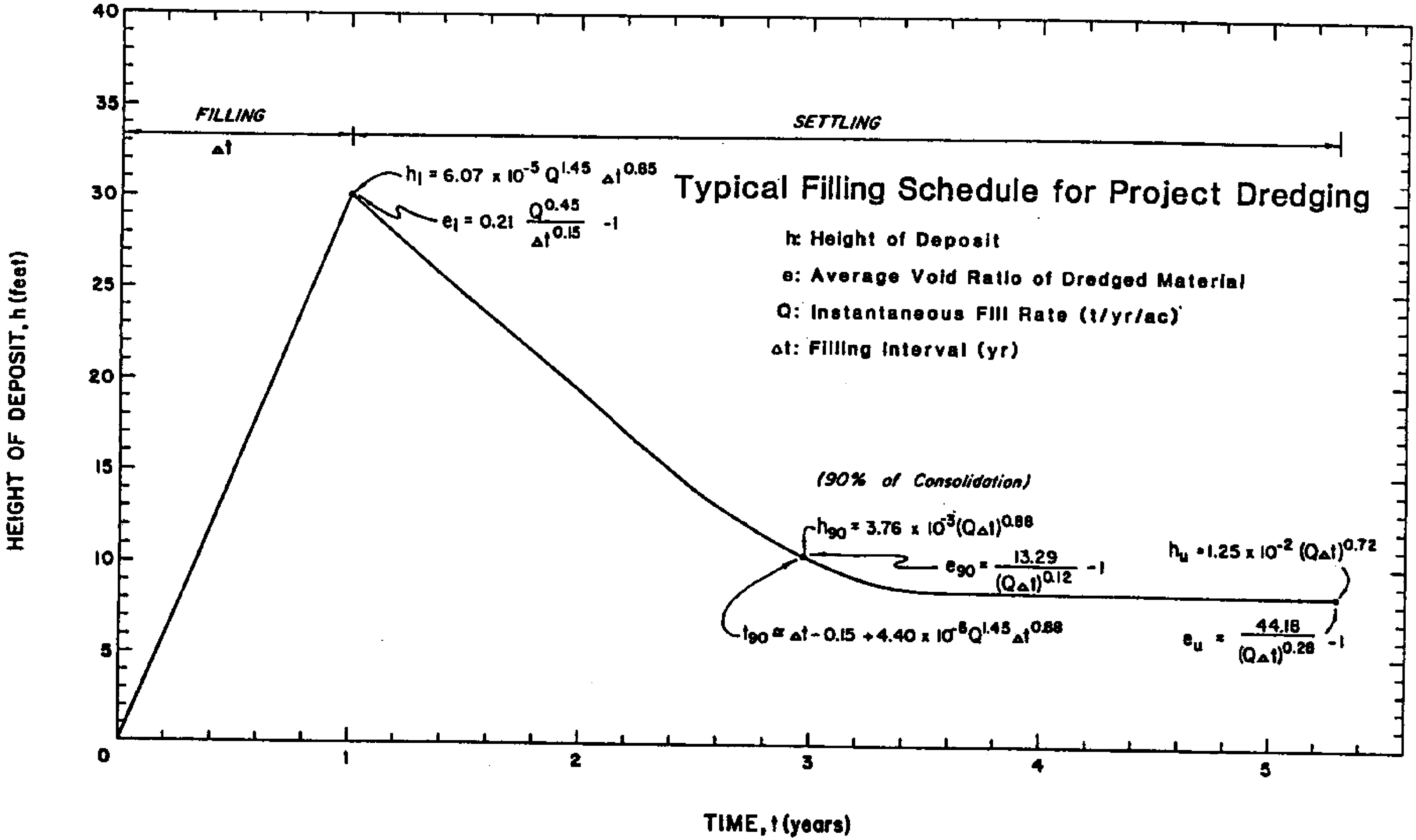

Typical Filling Schedule for Project Dredging
h: Height of Deposit
e: Average Void Ratio of Dredged Material
Q: Instantaneous Fill Rate (t/yr/ac)
Δt: Filling Interval (yr)
FILLING
Δt
SETTLING
$h_I = 6.07 \times 10^{-5} Q^{1.45} \Delta t^{0.85}$
$e_I = 0.21 \frac{Q^{0.45}}{\Delta t^{0.15}} - 1$
(90% of Consolidation)
$h_{90} = 3.76 \times 10^{-3} (Q \Delta t)^{0.88}$
$e_{90} = \frac{13.29}{(Q \Delta t)^{0.12}} - 1$
$t_{90} \cong \Delta t - 0.15 + 4.40 \times 10^{-6} Q^{1.45} \Delta t^{0.88}$
$h_u = 1.25 \times 10^{-2} (Q \Delta t)^{0.72}$
$e_u = \frac{44.18}{(Q \Delta t)^{0.28}} - 1$
HEIGHT OF DEPOSIT, h (feet)
TIME, t (years)
0
5
10
15
20
25
30
35
40
1
2
3
4
5

Figure 12

Typical Filling Schedule for Maintenance Dredging

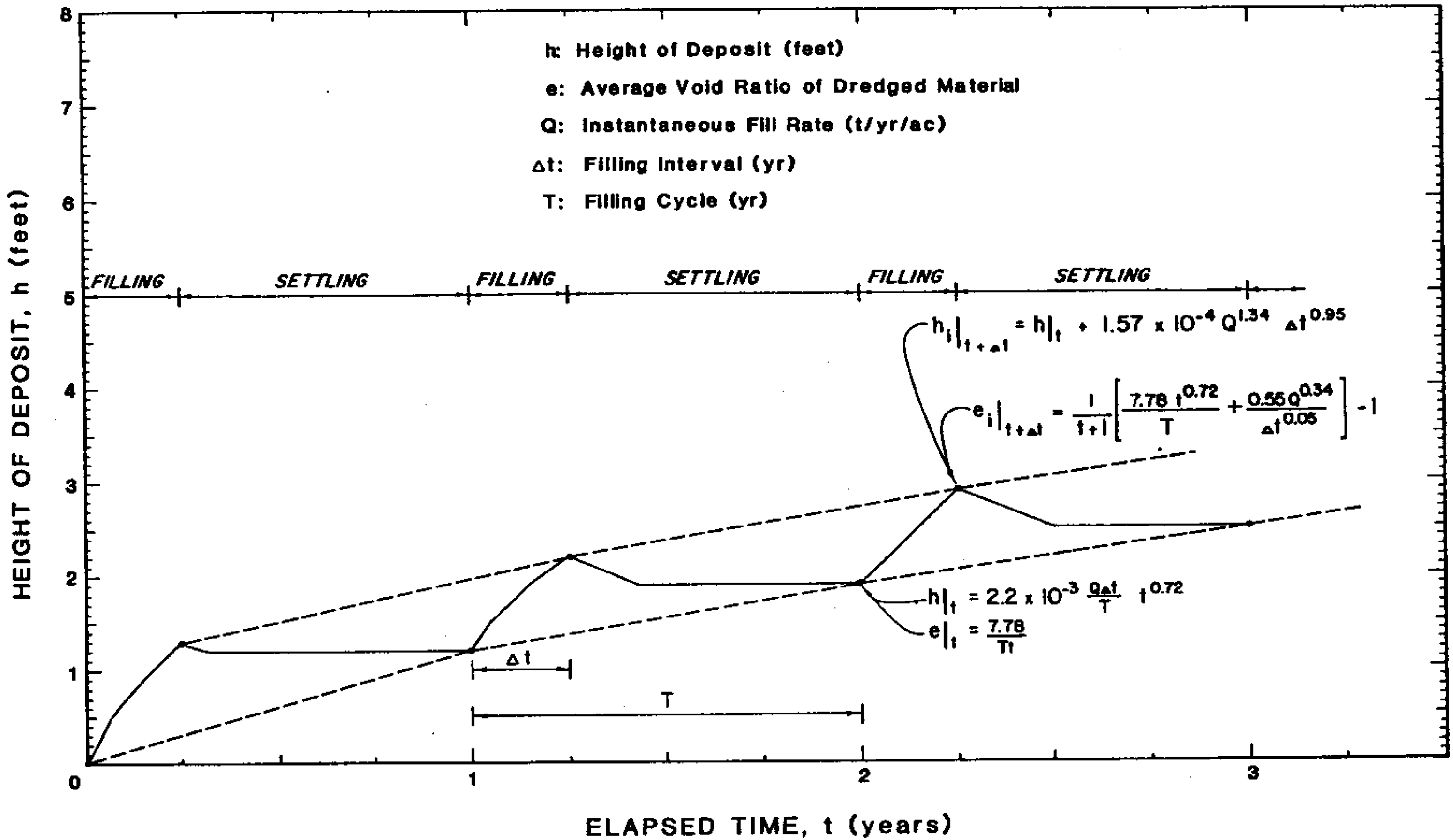


Figure 13

EARLY DREDGING ON THE TEXAS COAST

by

David Fenwick Bastian*

ABSTRACT

Having and maintaining a navigable channel to a port spelled the difference between prominence and oblivion. This paper will discuss the first dredging at Corpus Christi and the first dredges used at Corpus Christi and Galveston and will trace the fate of these dredges through the Civil War.

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As early as 1834 records show navigational problems along the Texas coast. Three years later, during the Texas war, Great Britain's consul in Mexico used the occasion to send Joseph T. Crawford to explore the Texas coast to look for promising harbors. From his survey he concluded that Corpus Christi did not hold much promise. Henry Laurence Kinney apparently thought otherwise because he established a trading post on Corpus Christi Bay by the end of the 1830's. Shallow water was, in fact, an impediment to traffic into Corpus Christi Bay exemplified when Texas was annexed by the United States, troops under General Zachary landed at St. Joseph's Island July 22, 1845 but because of the shoals they had great difficulty transferring to the mainland.[1]

Despite navigational problems Kinney's trading post became Corpus Christi on May 30, 1846 but because of these same problems the army moved south to Port Isabel in 1848. To protect his trade, Kinney's advertisements for new settlers resulted in semi-monthly packets plying in waters between Corpus Christi and New Orleans. This promoted merchants from both New Orleans and Corpus Christi to band together, believing that Corpus Christi could serve as the port city. A wagon route was established between Rio Grande City, Mier, Laredo, and Corpus Christi, and William Mann of William Mann and Company, General Commission and Forwarding Merchant, contracted for construction of a 170-foot wharf to ship and receive goods.[2]

Corpus Christi's position as a port was enhanced by the 1849 gold rush which promoted the building of another wharf and Indian hostilities in the interior resulted in the Texas Rangers establishing a post. The very next year, Fort Merrill was established for the army and Corpus Christi became a depot for military supplies.[3]

During the Mexican hostilities the United States government realized the deficiencies of navigable depths to the Texas mainland. In November of 1846, arrangements were made for the construction of a steam dredge for service at the mouth of the rivers of Texas. Expenses of the dredges were to "be set to the account of Mexican hostilities," according to the Quarter Master's report to the 30th Congress, I Session.

The dredge was built in Louisville, Kentucky, by J. Hulme under the direction of Lt. Col. S. H. Long of the U. S. Army Topographical Engineers. The LAVACA was fit for service by March of 1847. She was furnished with two dredge ladders, four mud scows and various articles of rigging (See Figure 1).

Cost of the completed dredge boat amounted to $20,000 according to the report:

"The cost of the dredge boat, including four mud scows, the materials for which are all prepared and ready for putting together; also three anchors, various articles for rigging and cordage, smith's and engineer tools, etc, has amounted to $18,372.04. The charge for safekeeping and protection, since the time of her completion amounts to about $60 per month. To which must be added the expenses of recaulking her boat tops, or sides above water, now in progress, together with the purchase of cooking apparatus, and other furniture for the accommodation of her captain and crew when employed; the whole of which will probably amount to about $1,500, making the whole cost of the dredge boat when furnished in all respects, about $20,000."

The following table shows the Quarter Master's statement of accounts from the dredge boat LAVACA.

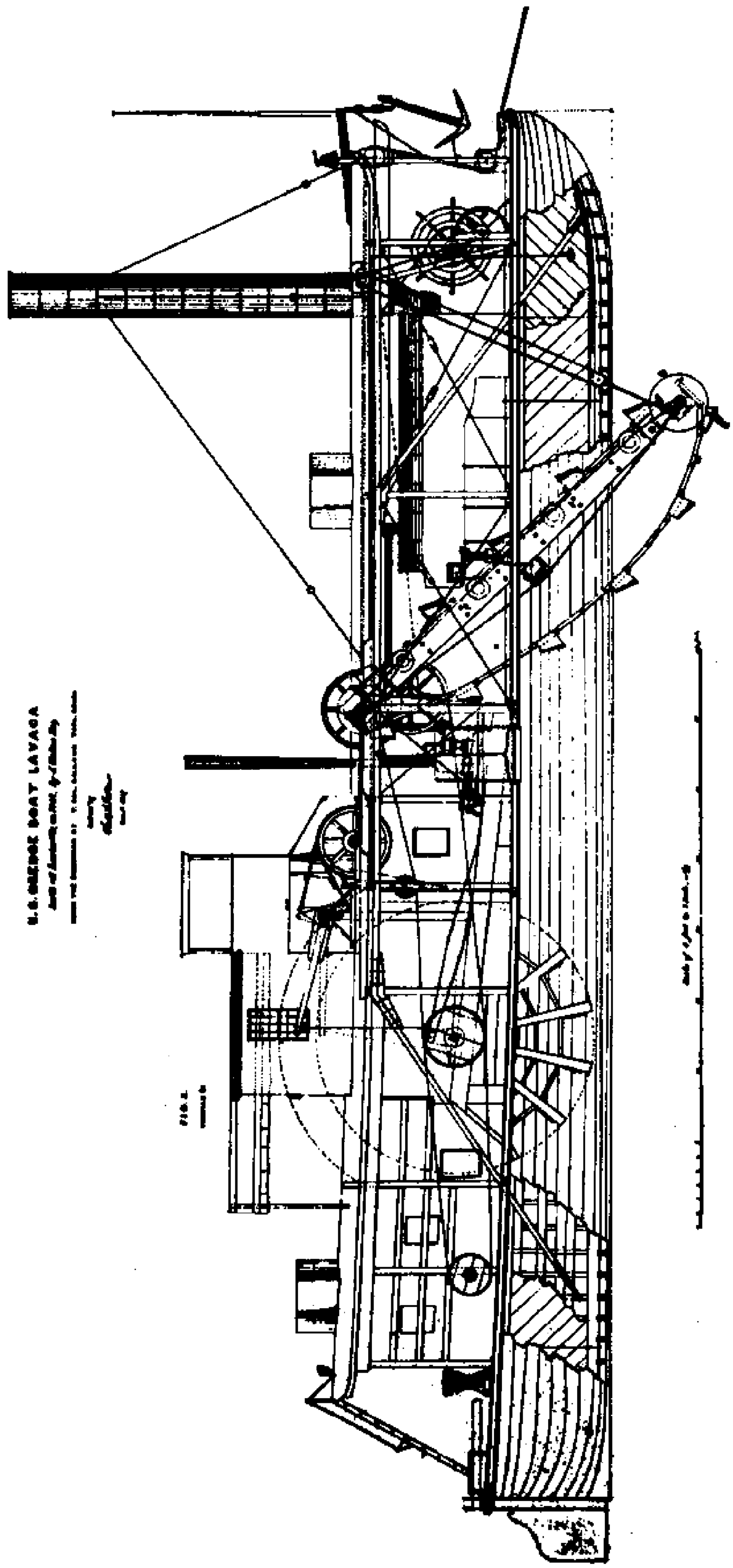


Figure 1. Dredgeboat LAVACA

TABLE 1

Receipts on account of steam dredge boat LAVACA 1846, November 16. Treasury draft No. 4425, on war warrant No. 6535, payable to order of Lieutenant Colonel Long, for	$ 5,000.00
1847, January 15. Treasury draft No. 5019, on war warrant No. 7205, payable as above, for	5,000.00
March 20. Treasury draft No. 5996, on war warrant No. 8058, payable as above for	8,000.00
Amounting to	18,000.00
To which may be added the amount of treasury draft No. 5363, on war warrant No. 7617, misdirected, returned as dead letter, reforwarded August 11th, and received August 16th of the current year, viz..	5,000.00
Total amount of receipts on account of dredge boat	23,000.00
Expenditures on account of steam dredge boat LAVACA 1847, January 1. Amount expended on account of dredge boat in 4th quarter of 1846	5,000.00
April 1. Amount expended on account of dredge boat in 1st quarter of 1847	5,020.00
July 1. Amount expended on account of dredge boat in 2d quarter of 1847	8,372.04
Amount of expenditures	18,392.04

In the Congressional report, it was revealed that during the LAVACA's trials "her efficiency in dredging to the depth of 9 or 10 feet below the surface of the water, has been proved adequate to the raising of at least 150 cubic yards of mud and sand per hour, and her speed through the water, when propelled by her own machinery, equivalent to at least 8 miles per hour." [4]

Despite her capabilities, for some reason, the LAVACA lay idle on the Ohio River at Louisville for the next eleven months.

The war between the United States and Mexico was settled by the Treaty of Guadalupe Hidalgo in February 2, 1848, but troops remained in Texas and the southwest along the border, and those troops needed supplies. Enterprising businessmen began to campaign for her use in deepening the channel into Aransas Bay and Corpus Christi, Texas.

J. H. Blood, owner of a mercantile firm in New Orleans, Louisiana, had by January of 1849 obtained the support for the project from the Texas and Louisiana delegations in Washington. Appeals were sent to William L. Marcy, U. S. Secretary of War, to permit Blood to use the dredge.

On January 2, 1849, Texas representative F. Pilsburg wrote:

"The review of this letter, Mr. Blood of New Orleans solicits your department to make some arrangement by which a "Dredging Machine" made and prepared for deepening the channel into Aransas Bay and Corpus Christi and which was ordered by the government to facilitate the transportation of supplies for the Army of Genl. Taylor."

"The Army, I believe, left the point before the "Machine" was completed, notwithstanding no army occupies the position which Genl. Taylor held. Yet large supplies are sent by the Government into that quarter for the troops on our Western Boundary of the United States. Could not the department under these circumstances cause the machine, now lying useless, to be put in operation. Mr. Blood is well acquainted with the soundings and is prepared to detail the amount of expense that would be required to deepen the channel sufficiently for the Steam Boats which ply between New Orleans and Matagorda Bay."[5]

In addition to Representative Pilsburg, Senators Sam Houston and Thomas Rusk of Texas joined Senator W. Downs and Representatives Isaac Morse, J. H. Harmanson, and Emile La Sere of Louisiana in the following request of January 2, 1849:

"Some gentlemen of Texas are anxious to make some arrangements to procure the use of a dredging machine constructed by the Quarter Masters Department to be used upon the coast of Texas. We understand that it is useless elsewhere and feel assured that it would be of great service in the Bays of Texas. It would not only advance the commercial interests but will be of important advantage to the Quarter Masters Department in reducing the price of any project which might be necessary to transport to the frontier."

"We hope therefore that they may be able to procure the use of the machine." [6]

The request was received on January 5. Robert E. Lee endorsed the request from a military point of view and routed it to his superior.

Meanwhile, Blood contacted Brigadier General Thomas S. Jesup directly on January 19, 1849:

"The Louisiana and Texas Delegation having addressed the Department with a view to consummate an arrangement, whereby the United States Dredging Machine now at Louisville may be assigned to the use of the undersigned, the object of this is to inform you that he is now prepared to make such a contract for the use of it on behalf of his mercantile firm J. H. Blood & Co. of New Orleans, on the following terms and conditions: to wit--:

"That he will receive, take charge of and place this machine in perfect order, and employ the same in opening the channel as far as practicable between Aransas and Corpus Christi Bays, after which he agrees to return it to the officer whom the Department may delegate to receive it, in good and perfect repair as when received, subject of course to the usual wear and tear incident on the use thereof." [7]

Blood was prepared to enter into bond to secure the use of the LAVACA and referenced Colonel H. L. Kinney and William Mann as security.

General Jesup recommended the project be approved with some conditions. In his letter to Secretary Marcy, he said:

"If individuals were not disposed to execute the work, I should consider it of sufficient public importance to recommend that it be done by the military. I therefore recommend that the Dredge Boat be placed in the hands of the gentlemen recommended by the Texas & Louisiana Delegations, on a proper guaranty that the boat be completed and put into a condition to execute the work at the expense of the persons desiring its service, and that the most energetic measures be adopted to execute the work and that the boat be returned within one year from the date of its arrival at Aransas Bay should the Quarter Masters Department require it."[8]

This massive campaign ended anticlimatically on January 22, 1849 when Marcy, the Secretary of War, wrote on the back of the Rusk, Houston, et. al. letter "approved, 22 January, W. L. Marcy."[9]

In 1851, money was raised by private subscription to open a channel through Aransas Pass to Corpus Christi thus creating the gateway for Mexican trade. Channel work was commenced assumedly by Col. Kinney who is recorded to have bought a dredging machine in 1848. More probably the dredge referenced was the LAVACA which was made available sometime after January 1849.[10]

It was on February 16, 1852, that a charter was taken out by James K. McCreary, Ebenezer Allen, Henry L. Kinney, Forbes Britton, Frederick Belden, William Mann, William H. Jones, and Walter Merriam. The Corpus Christi Navigation Company proposed to take over the channel work done in 1851, and issued stock certificates to the citizens who had advanced those funds. It further proposed to continue the work and make the channel available for steamships and other vessels from the town of Saluria, on Matagorda Bay, through to Corpus Christi. Tolls were to be charged on all vessels passing through this channel at the rate of $1.00 per foot over and above three feet, etc.[11]

On May 15, 1854, the committee on deepening or cutting a channel from Aransas Pass to Corpus Christi made a report.

Mayor C. R. Hopson cast the deciding vote to authorize the channel, after Aldermen I. R. Peterson, Benjamin F. Neal, McMartin, Blucher, Love and Somers Kinney had reached a deadlock.

The committee's proposal was to issue bonds in the name of the city for $50,000, drawing 8 per cent interest, for digging the channel "as provided for and contemplated in the charter granted by the legislature during the session of 1853 and 1854."

Purchaser of the bonds would make a survey and begin work on the channel, which, the contract provided, would be cut to not less than 5 feet deep at ordinary tides during the first year. Thereafter the channel would be deepened to 6 feet within three months, and the contractor could cut it on down to 10 feet.

The contractor would collect all tolls on the channel until the bonds and interest had been paid off. The city would get 10 per cent of the profit, the rest would amortize the bonds.[12]

It was a fairly simple proposition, and at the next meeting Alderman Somers Kinney accepted it, taking the bonds at 20 cents on the dollar. He didn't know it, but he was heading for trouble.

That was 1854, and he had to cut the channel to 5 feet by the spring of 1855. The old records don't give details of his difficulties, but

January 15, 1855, Alderman Barnard had to come to his rescue.

Barnard got the aldermen to extend the contract to January 1, 1856, because "said Somers Kinney had met with misfortunes, hinderances and delays unforeseen by the wrecking of the Dredging Machine."[13]

The extension belatedly set the width for the channel: 60 feet, to be widened as commerce might require.

By June 23, 1855, the board of aldermen was rangling over terms of the contract and requested Kinney to submit a written proposition. Two days later they extended his time to August 1, 1856, provided that by February 1 "he shall have a good and serviceable dredging machine actually at work ... and that he continue from day to day actually and energetically engaged in completion of his contract ... and cut channels through the places known as 'Mud Flats' and 'Mouth of the Shell Flats' to depths of 8 or 9 feet." [14]

The city government evidently wanted action, and weren't getting it.

Kinney seems to have found the job too much to handle by himself. Therefore, he arranged a business deal with Dean S. Howard of Lyonsdale, New York, who had just patented a dredge January 9, 1855, to provide a dredge and take over the dredging project. Howard was an experienced dredge builder and operator having had previous contracts with the Corps of Engineers on the Great Lakes. Howard described his business arrangement:

"I have got a charter or a job under a charter from the state of Texas to open a channel about a mile long leading into Corpus Christi harbor for which I am to receive tolls on all vessels that pass through it at the rate of 5 cents on every barrel bulk of freight."

"I expect to complete it so far as to begin to take toll by the 1st of May next but the out lay will be so large that I must get up some little capital to keep me along until the tolls will pay. If you can help me any let me know soon at Lyonsdale."[15]

On October 10, 1855, the alderman of Corpus Christi approved the Howard dredging contract.

More extensions of time followed. On January 14, 1856, the aldermen repealed the August 1 deadline if Kinney would have a dredge by March 1 and be at work by May 1.[16]

But by February 5, 1856, Howard reported the status of the dredge:

"Nothing of consequence has yet transpired to afford anything in the way of business gratifying to communicate. My goods and machinery or lumber for the Dredge has not yet arrived but I still hope they are all safe."[17]

Howard subsequently enlarged his company, taking in D. E. Watrous and J. C. Riddle, and by April 28, 1856, three extensions of time had been granted.

Realizing that further inducements would have to be held forth if the project was to be put over, the citizens of Corpus Christi petitioned the State Legislature, and accordingly in August of that year the Channel Land Grant found its way into the statutes, donating to Dean S. Howard, Somers Kinney, Daniel Watrous and John C. Riddle sixteen sections of land for each mile of ship channel, not exceeding seven miles, eight feet deep by one hundred feet wide, through the mud flats between Aransas Pass and Corpus Christi.[18]

By November the dredge was still uncompleted. Howard explained to his brother:

"My business is going on well, I have the Dredging machine nearly ready to launch and the machinery all on the way from New York. I have until the 1st of April next to begin the canal which is all the time I want. Shall probably finish it before next Fall. After I have kept it open six months I receive from the state in addition to the tolls of 5¢ a barrel 8 stations of land and when I have kept it open 18 months I get 8 stations more for each mile of canal not exceeding 7 miles which makes 112 miles of land in all in addition to the Tolls of 5 cents for every barrel of freight passing through it."[19]

Howard's dredge was completed in 1857 (See Figures 2 and 3). He described it:*

Figure 1 is a side view of a dredging machine calculated to deposit in lighters, over the stern or at the sides, as occasion may require. It is also calculated to work with any shape bucket best adapted to the material to be excavated.

Figure 2 is an end view of the same showing the facilities for lateral deposit. The same letters refer to like parts in each.

A is the driving chain-wheel, geared to the engine by the wheels and shafting, shown in Figures 1 and 2.

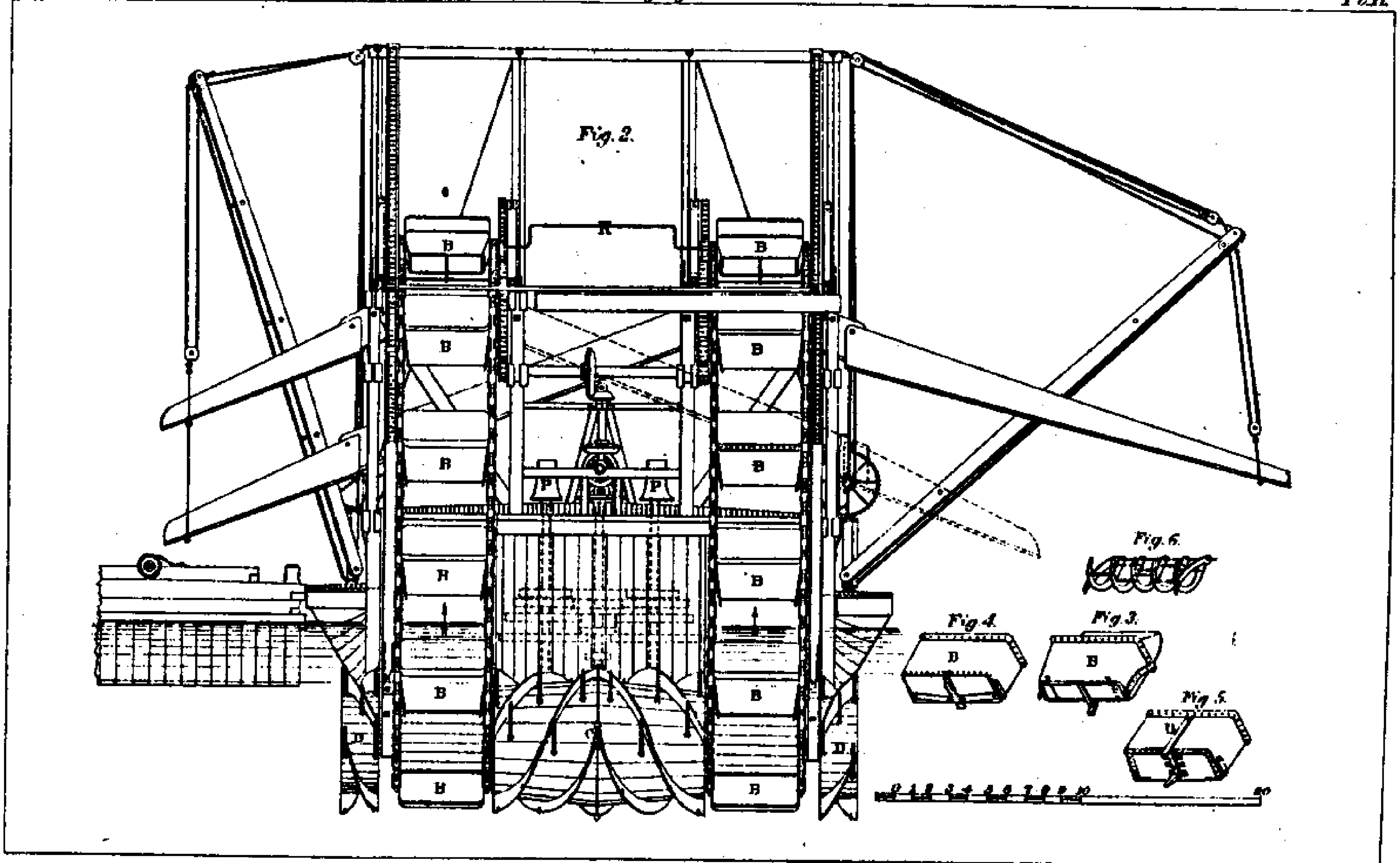
B designates the buckets attached to the chains shown in Figures 1 and 2.

C represents the center cylinders, with the spiral scrapers and hooks, for loosening and conveying the material to be excavated, from the center, each way, to the buckets.

* Howard's figure references are shown in Figures 2 and 3 of this paper.

FIGURE 2

Jr. of Fr. Inst. Vol. LVII. — Dredging Machines. — Pl. II.



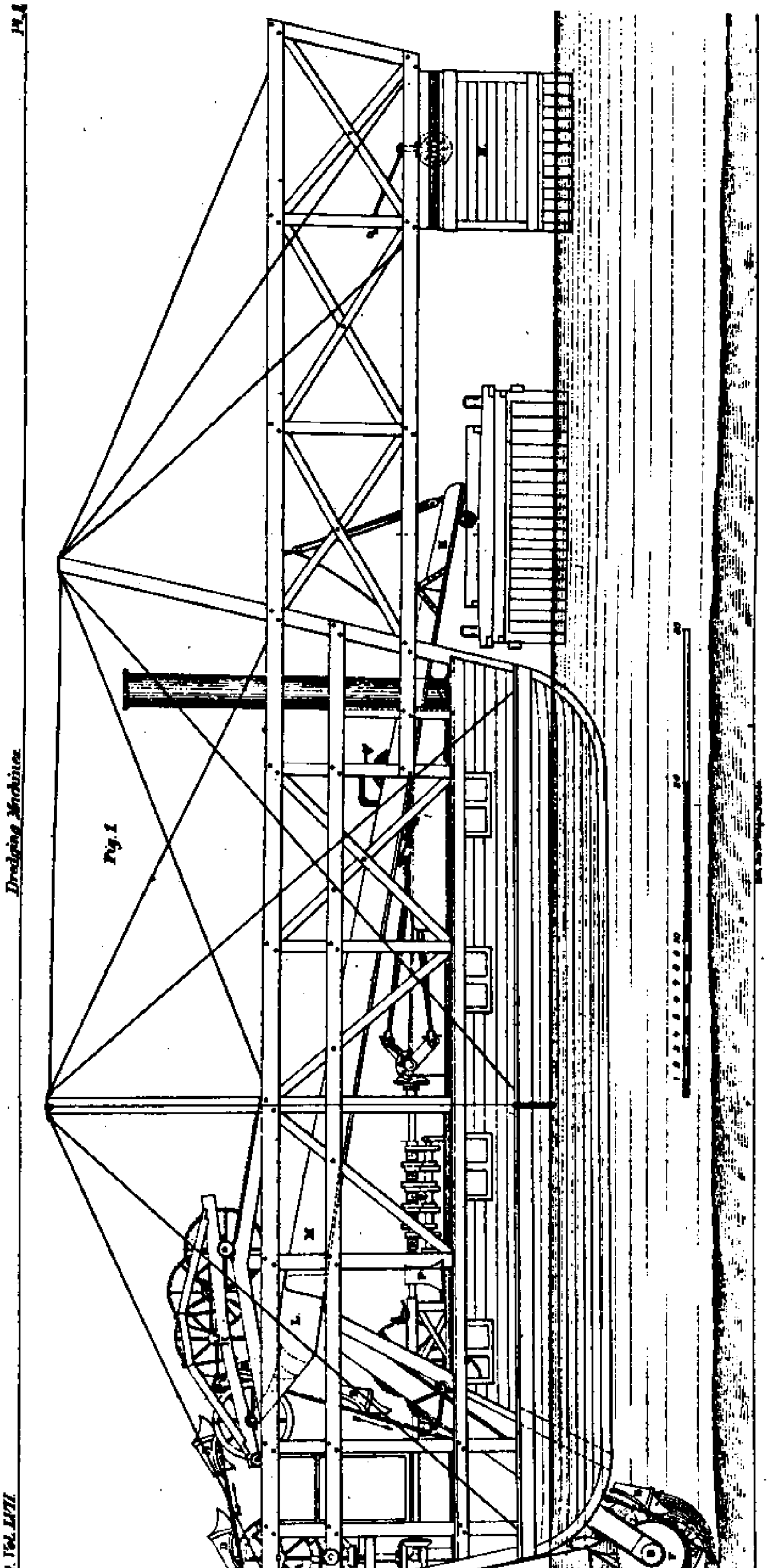


FIGURE 3

D, in Figure 2, represents short cylinders on the ends of the cylinder shaft, E, Figure 1, with spiral scrapers for conveying the material to be excavated, from the outside to the buckets.

The short cylinder on the port side of the dredge, Figure 1, is left off, to show the lower chain-wheels and the lower attachments to the ways.

F is one of the lower chain-wheels, attached to the cylinders and driven by the chains, in the same manner as the chains are driven by the driving chain-wheel, A, by cams fitting into alternate links of the chain.

G is one of the flange-wheels seen in Figure 1, over which the chain passes before descending to the lower chain-wheels, F.

H is one of the movable frames, which suspends the flange-wheels, G, on rollers running upon ascending ways, provided with powerful purchase-wheels, terminating in a pinion working in a descending rack, shown in Figure 1.

This purchase is worked by a cog-wheel on the flange-wheel shaft (not shown in the drawing), working in a wallower-wheel, J, when thrown into gear, for the purpose of raising the buckets from the bottom when required, by forcing the flange-wheels, G, up the inclined ways. When the machine is not in motion, the wallower-wheel may be thrown out of gear, and worked by hand, with a long double crank, K, Figure 2.

By the motion of the buckets in the direction of the arrows, the excavated material is brought up; and if it is to be deposited over the stern, it is dumped into the movable spout, L, Figure 1, which is attached to the axis of the flange-wheel, G, by the extension of its sides, and moves freely within the stationary spout, M, allowing the flange-wheel, G, to be drawn up, when necessary, with the movable frame, H, without changing its proper position, for receiving the contents of the buckets as they pass over the flange-wheel, G.

N, Figure 1, is the counter balance, situated about 25 feet aft of the hull, attached to it by a truss frame and hog chains, for the purpose of balancing the weight of the machinery necessarily placed ahead of the hull, to enable the dredge to clear its own way. It is also used for transferring the lighters, by attaching the empty one to the outside of the counter balance, and the loaded one to the inside; then by the level, O, which works the gear rollers between the counter balance and the truss frame, the two lighters are changed about, the empty one inside, under the spout, M, ready to be filled, and the loaded one outside, ready to be transferred to the dumping place. It is also used as a water tank for the engine, which may be filled in the morning, before the water has been disturbed by the dredging, sufficiently to afford clean water during the day.

B, Figure 3, is a perspective view of a bucket used in this arrangement for depositing over the stern. It is provided with a loose bottom, which drops with the load about two inches, rendering the discharge perfectly certain when at work in the worst kind of material.

When the situation of the work is such as to require the deposit to be made on the bank, or into the lighters alongside, a bucket like B, Figure 4, is put on the chain, in place of B, Figure 3, which dumps into the lateral spouts, situated between the driving chain-wheel, A, and the flange-wheel, G, by the tripping of the latch which lets fall the whole under side of the bucket, hinged to the bolt that fastens the bucket to the chain. This ensures the discharge of the most difficult material.

The short receiving spouts under the buckets, and the one between them, are hung on pivots in the center, so that either end may be elevated, and the contents of both sets of buckets discharged on either side, or both sides, as may be desired.

Whenever the extent of the work requiring a lateral deposit is sufficient, the hull of the dredge may be built long enough, and lean enough aft, to balance the machinery forward, without the counter balance.

When the work is situated in water deep enough in front of the dredge to float the loaded lighters, the buckets like B, Figure 5, which dump through the bottom, are brought into requisition, the latch to these may be tripped anywhere on their perpendicular way up, and discharged into a short vibrating spout, which conveys the material directly into the lighter, placed much nearer than it can be in any other arrangement. Thus, saving power in proportion to the height of discharge.

Figure 6 is a gang of hooks, sometimes put upon the chains between the buckets, when working in hard, coarse material, like cobblestone, shale, or hard-pan. The chain is so constructed that any shaped bucket, or any other device for loosening the material, that might be found in practice to be preferable, may be put upon it. All the articles here represented have been fully tested, and found very useful in their places. No sacrifice of power, or of economy in working, has been required to enable us to use all these appliances on the same machine; on the contrary, the perpendicular position of the working part of the chains, and their passing around three drums instead of two, are great improvements under all circumstances.

The perpendicular application of power secures a great economy of friction, in the wear of rollers, ways etc, especially where no lubricating material can be used, nor the wearing parts secured from the destructive action of sand and water, which must always be present in dredging.

The third drum, which constitutes the flange-wheels, G, enables us to raise the buckets from the bottom, in the manner above mentioned, without changing the perpendicular position of the working part of the chain. It also furnishes the best possible position for the discharge of buckets like B, Figure 3, at all times when that kind of bucket is in use. Also, that of B, Figure 4, which dumps between the two upper drums, requiring nearly a horizontal position to discharge.

The bucket B, Figure 5, is equally well accommodated. This one may be dumped anywhere on the perpendicular part of the chain, by raising or lowering the vibrating spout which trips the latch. This is the most economical of all the different buckets, where circumstances are favorable for its use, as it dumps with perfect freedom all kinds of material, and is discharged at a much less elevation on account of its better relative position with respect to the lighters.

In working these machines, a pully is anchored at a convenient distance ahead, with a feed line passing from one of the feed capstans, P, through the pully and back to the other capstan, either one of which, or both, may be worked by the adjustable machinery to any required motion, by a change in the series of clutches at Q, or a series of wheels below deck; while the spiral cylinders below water, with the buckets, clear the way to the full width of the dredge, and to the depth required, bringing up the material and depositing it in lighters, or on the banks, or at any distance in any direction horizontally, not exceeding one in twelve of the altitude overcome, by supporting a spout, lined with sheet iron, of the length required, on a movable support, that it may coincide with the feed motion of the dredge.[20]

Howard's dredge was an initial success. The aldermen held a special meeting October 29, 1857. Their jubilant record reads:

"Whereas D. S. Howard & Co. having so far progressed in the great work of cutting a ship channel between Aransas Pass and Corpus Christi Bay as to admist vessels to this place drawing 6 1/2 feet of water, therefore:

"Be it resolved that looking upon the enterprise as one of the greatest importance to Western Texas, we have every reason to hope that with proper encouragement from the state, the same will be brought to a speedy completion. We have confidence in the dredging machine, the permanency of the channel, and that it will be dredged to a depth of 10 feet."

"Resolved: That we recognize Captain Matson of the schooner Union as the pioneer in a direct trade between this city and New Orleans, that the reduction of the cost of freight by 50 per cent already is indicative of future commercial advantages, and that this enterprise meets with our approbation."[21]

For a time it looked as if this land grant would be the means of giving them a channel, but alas, matters seemed to have dragged along, some six extensions of time had been asked and granted, and finally in April 1858, the city, it seems, entered into a contract with Dean S. Howard & Company, who were to surrender the $50,000 bonds for cancellation, and in lieu thereof eight per cent bonds to the amount of $500,000 were to be issued.

Howard's letters to his brother continue to indicate vast indebtedness. In June 1859 he broke from that theme long enough to inject, "We ... have got the dredging machine on to the work again which is to be finished by the first of November."[22]

Apparently the project was not completed as specified. Howard's letters cease after mid-summer 1859, although ten years later when writing in the Journal of the Franklin Institute he claimed that "the channel was finished to sixty-four feet in width" but didn't say when or to what depth.[23]

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5. Letters Received, Engineers Department, National Archives.

6. Ibid.

7. Ibid.

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13. Ibid., January 15, 1855.

14. Corpus Christi Times, February 5, 1846.

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16. Minutes, City of Corpus Christi, January 14, 1856.

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18. Allhands, op. cit., p. 61.

19. Howard to his brother, November 21, 1856.

20. D. S. Howard, "The Dredging Machine," The Journal of the Franklin Institute, Vol. 87, 1869, pp. 239-243.

21. Minutes, City of Corpus Christi, October 29, 1857.

22. Howard to his brother, June 1859.

23. Howard, op. cit., p. 243.

SLURRY PUMP AND PIPELINE PERFORMANCE TESTING

AT THE GEORGIA IRON WORKS HYDRAULIC LABORATORY

by

Graeme R. Addie[1]

ABSTRACT

1. DESCRIPTION OF FACILITY

A general description of the complete facility covering the 4", 8", and 18" diameter pipe slurry loops, the system instrumentation and the computerized method of data collection that provides for up to 32 different readings and allows monitoring, display, and collection of both pump and pipeline performance at the same time.

2. SETTLING AND NON-SETTLING PIPELINE TESTING

A description of the method of carrying out pipeline tests, the calculation and presentation of results along with the details of some of the results collected in the lab.

3. CENTRIFUGAL SLURRY PUMP PERFORMANCE ON SLURRY

An explanation of the effect of slurries on the water performance of a centrifugal pump and presentation of some of the results collected showing the effect on the head and efficiency.

4. SLURRY PIPELINE DESIGN AND SCALE-UP METHODS

A brief outline of the methods of slurry pipeline design and scale-up being followed at the GIW Laboratory for applications and comparison of data.

5. SLURRY PUMP DESIGN DEVELOPMENTS

A description of the development work being carried out into pump design and in particular the work on different head quantity, characteristic curves, and high efficiency designs.

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1. DESCRIPTION OF FACILITY

In February, 1975, the President of Georgia Iron Works Company, Mr. Tom Hagler decided to build a new hydraulic laboratory to water performance test all of the companies' centrifugal slurry pumps.

Equipment procurement started in March, 1975, by ordering of a mini computer data acquisition system.

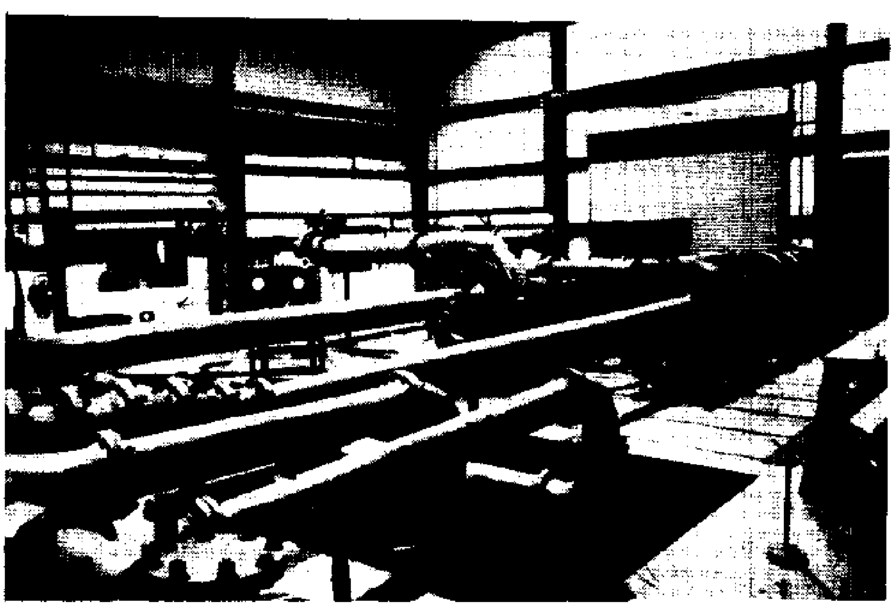
Work commenced on the lab in July,1976, by erection of a Butler Building and the laying down of a 20,000 gallon concrete sump.

By late October, a facility compromising two water test loops of 8" and 18" pipe diameter was in operation carrying out head quantity and suction performance tests on pumps up to 16" in size.

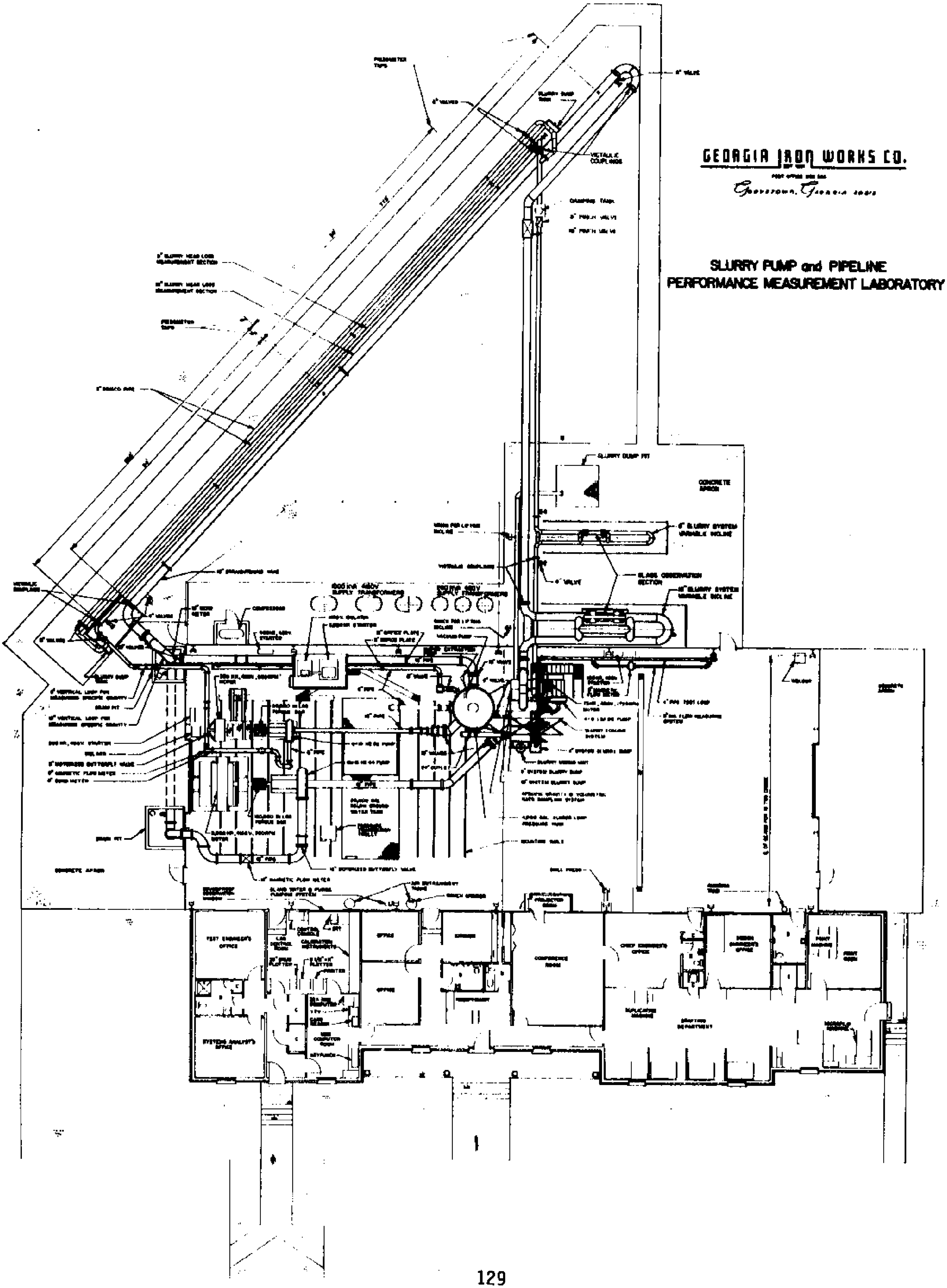
In August of the following year, an 8" diameter pipe slurry loop was installed to enable pipe friction and deposition velocity of a slurry to be measured in an 8" inside diameter pipe.

With the encoragement of Reserve Mining in Minnesota and IMCC in Florida, this was followed in March, 1978, by the addition of over 300 feet of 18" diameter pipe to make up an 18" diameter pipe slurry system complete with head loss and deposition velocity observation sections.

The office and shop areas of the lab were again expanded in April, 1978, to provide further expansion to the test sections and staff facilities.



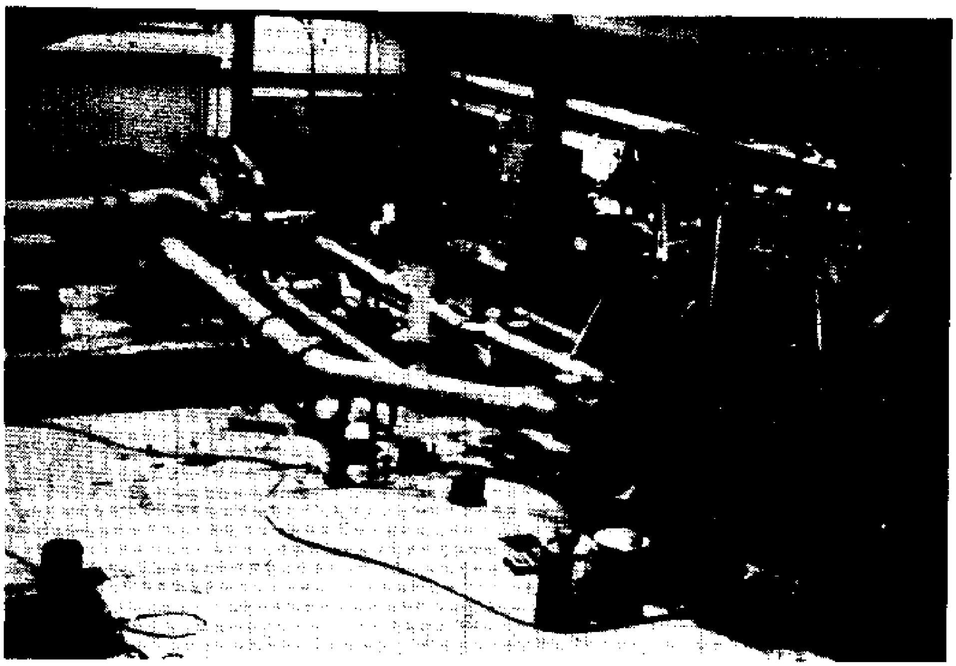
PUMPS SET UP INSIDE LAB FOR TESTING


GEORGIA IRON WORKS CO.
SLURRY PUMP and PIPELINE
PERFORMANCE MEASUREMENT LABORATORY

The GIW Hydraulic Laboratory is now one of the largest and most sophisticated labs of its type in the world. With multiple slurry test loop systems, a wide variety of instrumentation, and a unique multi channel, extremely flexible and high speed system of monitoring, displaying, and storing data.

The GIW Hydraulic Laboratory is set up to carry out both pump and pipeline performance testing on water and slurry.

Three test loop sizes of 4", an 8", and an 18" diameter pipeline are available for open sump pump head quantity or pipeline tests using water or slurry.



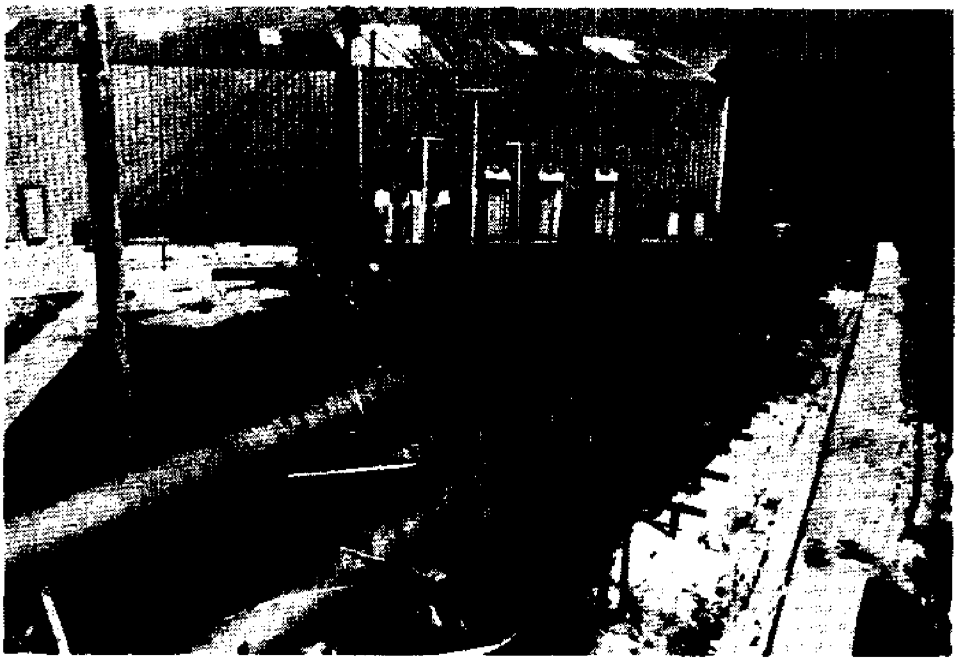
OVERALL VIEW OF INSIDE LAB

Each system has its own pump and electric motor drive. The 4" system has it's own 4x6LSA25 GIW pump with 75HP 480 volt motor. The 8" system has an 8x10HE32 GIW pump with a 200HP 480 volt motor. The 18" system has an 18x18HE44 GIW pump with a 2000HP 4160 volt motor. A separate 400HP 480V starter is available for additional testing.

Drive train arrangement is V-belt for the 4" system pump, V-belt or direct for the 8" system pump, and direct for the 18" system pump.

Each system is provided with offset taper pieces and adjustable sections so that other sizes of pumps can set up and be tested with a minimum of change.

Maximum flow in the 18" system is 30,000 GPM.



HEAD LOSS AND INCLINE SECTIONS FOR
4", 8" AND 18" SLURRY SYSTEMS

Each of the slurry systems is laid out as a large right angled triangle connecting into an open elevated slurry tank at the right angle corner. Each system has its own separate slurry tank, but these are connected together by the 4" system recirculation pump. The slurry systems each are made up of over 300 feet of piping.

Closed loop pump cavitation performance testing is possible in the 8" and 18" systems.

The closed loop test systems use the same suction and pump discharge pipe sections that are used in slurry test systems. Separate return sections incorporating the orifice plates and a special sealed tank complete the water test closed loops. The sealed tank is set up so it can be pressurized or evacuated of either air or water.

Closed loop to slurry system changeover in both 8" and 18" systems is arranged by transferring a section of suction piping and throwing valves on the discharge side.

18" GLASS FLOW OBSERVATION SECTION

Head loss in the slurry system is measured over 100 foot long straight sections of pipe set clear of any bends or obstructions.

The 4", 8" and 18" systems are equipped with glass pipe sections for observations and deposition velocities. These are located in special sections of pipe that may be inclinded in any position from horizontal up to vertical. The inclines may be varied during a test.

The 4" system is designed to allow wear testing. Equipment is available to allow strain gauge, vibration, noise, and viscosity measurement.

The Data Acquisition System used in the lab is comprised of a specially configured 13 bit accuracy (8191 Digit), 32 channel analogue to digital conversion unit and a 32K, 16 bit word mini computer, with control console, printer and plotter peripherals.

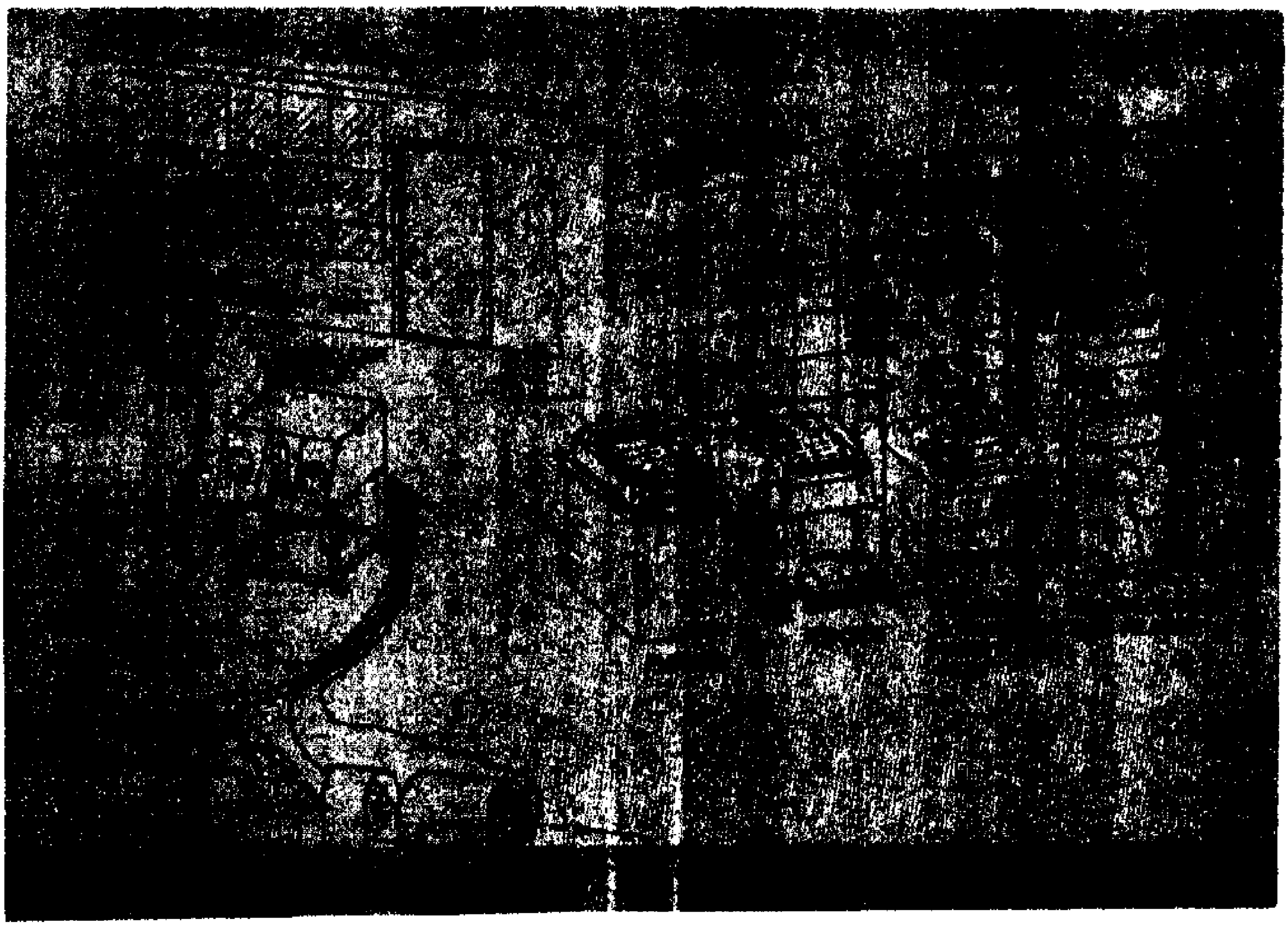
This unit is set up to monitor the output from up to 32 transducers and at regular intervals convert the transducer outputs into readings and results for display on the operator's console. The unit is also programmed so that any set of readings that are displayed may be recorded as a data point, and that up to 15 data points per test, may be recorded. At the end of the test, the readings are stored on casette, printed out, and plotted as required.

Conversion of the transducer output is carried out at a rate of 4400 readings per second. Digital values are collected, averaged, and converted over a monitoring time that can be varied at the operator's consoles from 0 to 200 seconds.

The averaged digital values for each transducer are converted into the different pressure, flow, and torque readings at the end of each monitoring period.

The resultant efficiencies and friction factors required are also calculated at the end of each monitoring period and held in temporary storage with them.

The averaged readings and calculated results are then displayed on the operator's console and a new monitoring period commenced.



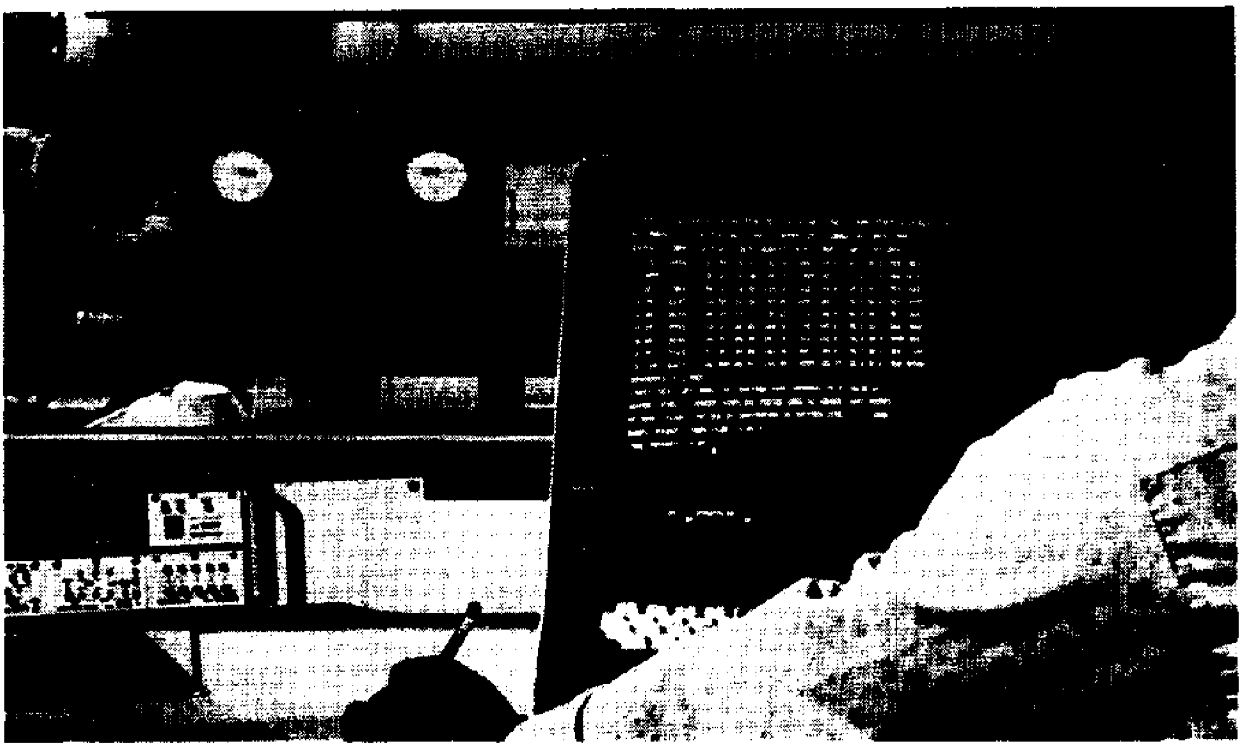
TESTING A PUMP USING THE MINI COMPUTER SYSTEM

At the end of each monitoring period, new readings and results are calculated and the screen refreshed.

A data point is normally taken to be a set of readings taken at steady flow conditions.

Steady flow conditions are assumed to exist when the monitored readings and results remain reasonably constant over several monitoring periods.

Recording of the steady flow condition readings and results for all 32 channels is carried out by depressing a single key of the operator's console. This stores the readings and results for all channels permanently and incriments the display line feed ready for the next line of monitored output.



MONITORING PUMP PERFORMANCE AT THE CONSOLE

A test is usually made up of 10 to 15 points taken at different system steady flows.

The console display and printed output are varied to suit the type of test being carried out. The console display normally includes only key readings such as pump head flow and efficiency along with pipeline velocity head loss and slurry specific gravity. Basic test types allowed are either pump head quantity on water only, sigma suction performance on water only, or slurry test.

Slurry tests are normally tests of both pipeline and pump performance on slurry since the mini system makes it easy for both pump head and power to be monitored simultaneously with pipeline velocity, head loss, and specific gravity.

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PUMP TEST DATA FOR    GIW SLURRY COURSE        CH  USE  NUO  SOURCE INSTRUMENT                                   GEORGIA IRON WORKS CO.
------------------                             --  ---  ---  ------ ----------                                  P.O. BOX 626
PROJECT               18" SAND SLURRY TEST      1        DISC DIFF HD ROSEMOUNT 0-240FT H2O 0226991E1  1.000     GROVETOWN,
PUMP TYPE             GIW 18"/18" ME 44         2        WATT METER   LANDIS AND GYR              1E3  1.000     GEORGIA           30813
CLIENTS ORDER NO   18" DIA. SLURRY LOOP         3 NULL FLOW 8" MAG  FISCHER & PORTER 8"0PM010094        .000     ------------------------
ASSEMBLY DRAWING NO                 18390       4        SUCT TRANSDR FOXBORO E13DM 0-34'H20 12080D1E2  1.000
SHELL    DRAWING NO                 18770       5        DISC TRANSDR.FOXBORO E11DM 1000'H20 06209A1E1  1.000
IMPELLER DRAWING NO                 18470       6 LOW  FLOW BEND   .FOXBORO E13DM 0-20 H20 12080D1E2    1.000
IMPELLER DIAM  44" 85=5.875" 34 DEG OUT         7 HIGH FLOW BEND   .FOXBORO E13DM 0-74'H20 12080D1E2    1.000     WORK ORDER NO  GIW COURSE
                                                8 NULL TORQ TRQ LIL LEBOW/DAYTRONIC LITTLE102680         .000     SERIAL NO         5012 3514
PUMP ROTATION                                   9        RPM  TRQ BAR LEBOW/DAYTRONIC RPM     102680    1.000     TEST CURVE NO     # 31 -79
HYDROSTATIC PSI =450              WITNESSED    10 NULL BHP  LIL BAR.LEBOW/DAYTRONIC HP 8" 040077         .000     TEST DATE         06/20/79
                                               11 HIGH LOSS 77.5 FT ROSEMOUNT 0-18 FT H20 12080D1E2    1.000
                                               12 AVE  S.G. U-SECDN ROSEMOUNT-1-11 FT H20-12080D1E2      .500
TEST CONSTANTS                                 13 LOW  LOSS 77.5 FT ROSEMOUNT 0- 9 FT H20 12080D1E3    1.000     DRIVER DETAIL
--------------                                 14      LOSS INCLINE-ROSEMOUNT 0- 2 FT H20 12080D1E3    1.000     -------------
  .00 INS DIA          IN   .00 INS PIPE       15 AVE  S.G. U-SECUP ROSEMOUNT 0- 5 FT H20 12080D1E3     .500     TYPE              LAB MOTOR
  1 FT H20 =      .0 US GPM USING              16      FLOW 18" MAG FISCHER & PORTER18"0PM0327991E-1 1.000     MAKE           GENERAL ELEC
BEND HT CORR =-1.4 FT  CONST = 4959.0          17      TEMP 18" DIS PADGETT 18"DISCH  (F) 0617BC1E1   1.000     SERIAL NO           5311957
DISCHARGE PIPE DIAMETER = 17.28 INS            18      TEMP  8" DIS.PADGETT 8" DISCH  (F) 0617BC1E1   1.000     FRAME SIZE          5368400
METER  .00' ABOVE PUMP DATUM, TAP 2.30'        19      SOLT SG.U 61-PADGETT U-LOOP61  (F) 0617BC1E1   1.000     RPM =  360     BHP = 2000.
SUCTION   PIPE DIAMETER = 17.28 INS            20      SOLT SG.D 64 PADGETT U-LOOP64  (F) 0617BC1E1   1.000     4160 VOLTS 3 PHASE 60 CPS
METER  .00' ABOVE PUMP DATUM, TAP  .00'        21 NULL S.G. SAMPLER.                                    .000
STATIC WATER LEVEL   .00 FT ABOVE P/D          22 NULL SLVL SAMPLER.                                    .000
WATTMETER MULT = 800. BAROMETER = 29.6'        23 NULL STIM SAMPLER.                                    .000     SCALED PERFORMANCE FACTRS
HEAD LOSS = 77.50 FT OF 17.28 INCH DIAM        24 NULL FLOW 8" DOP  LEEDS AND NORTHRUP 8" 07248         .000     --------------------------
S.G. TAPS 12.00' APART G = 32.14 FT/S/S        25      FLOW 18" DOP.LEEDS AND NORTHRUP 18"07248 1E-1 1.000     SPEED OR RATIO            .00
SOLIDS SG 2.65 OF 270. MICRONS S.D.=1.6        26 NULL FLOW BEND   .FOXBORO E13DM 0-20 H20 12080D1E2    .000     IMP TURN DOWN RATIO       .00
FOR VEL CALCS PIPE DIAMETER =    .00 INS       27      TORQ TRQ BIG LEBOW/DAYTRONIC BIG   10760E1E-1 1.000     MERIDNL WIDTH RATIO       .00
SAMPLER! HT CORR = .00' AREA =   .00 SQ'       28      BHP  BIG BAR.LEBOW/DAYTRONIC HP 18"040077      1.000     SCALE RATIO               .00
BALANCE VOLUME CONSTANT =    .00 CUBIC '       29      BHP  TRQ*RPM                                   1.000     BEP REF        .HD,     .RPM
                                               30      BHP  CALC'ED.FROM 200 HP MOTOR KILOWATTS       1.000     EFFICIENCY     .0%  BY   .00
                                               31      SUCT TRANSDR.ROSEMOUNT-30-30FT H20-06209A1E2   1.000
                                               32      RPM  LEUZE  .                                  1.000
```

TEST RESULTS

NO	VELOCITY Vs FT/S	FLOW Qs GAL/MIN	TEMP Ts F	S.G. Sw	S.G. Sm	VOLUME CONC. Cv %	WEIGHT CONC. Cw %	MASS Ms TON/HR	REYNOLDS NUMBER Re	PIPELINE LOSSES im FT/FT	iw FT/FT	FRICTION FACTRS Fm	Fw SAME Re	HAZEN WALLMS C	im-iw / Sm-Sw	TIME t HH.MM
1	19.64	14355.4	107.1	.992	1.15	9.8	22.4	731.5	.419E+07	.0635	.0532	.0132	.0128	134.	.0638	14.58
2	16.70	12208.4	109.1	.992	1.18	11.5	28.3	914.2	.354E+07	.0509	.0384	.0149	.0128	121.	.1140	17.05
3	14.46	10567.2	109.1	.993	1.17	10.5	33.0	734.2	.306E+07	.0599	.0290	.0213	.0127	100.	.1533	17.08
4	12.73	9301.2	108.0	.993	1.15	9.6	22.1	593.1	.271E+07	.0517	.0226	.0237	.0129	90.	.1831	17.14
5	11.59	8475.3	107.8	.993	1.14	8.7	20.4	492.2	.244E+07	.0500	.0188	.0303	.0130	91.1	.2155	17.17
6	10.21	7462.3	107.4	.993	1.12	7.4	17.5	365.5	.214E+07	.0465	.0146	.0370	.0130	82.	.2604	17.19
7	9.10	6655.3	108.3	.993	1.11	7.3	17.4	324.0	.193E+07	.0427	.0117	.0420	.0131	77.	.2547	17.26
8	6.95	5076.9	108.4	.993	1.08	3.2	8.1	107.3	.147E+07	.0324	.0069	.0594	.0133	66.	.4823	17.32
9	4.30	3139.8	108.0	.993	1.00	.3	.7	5.3	.708E+06	.0093	.0027	.0467	.0130	70.	1.5359	17.34

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GEORGIA IRON WORKS CO.                                   COMMENTS
TEST ENGINEER J.R. MAFFETT  DATE 06/20/79                FOUNDRY SAND PLUS 3 BARRELS OF PEBBLES FOR GIW SLURRY COURSE
                                                         STATIONARY BED OBSERVED TO FORM AT 9.2 FT/SEC
WITNESSED BY  GROUP 1       FOR  GIW SLURRY COURSE       NO HYSTERESIS, FINES APPEARED TO SETTLE SLIGHTLY AHEAD. GROUP 1
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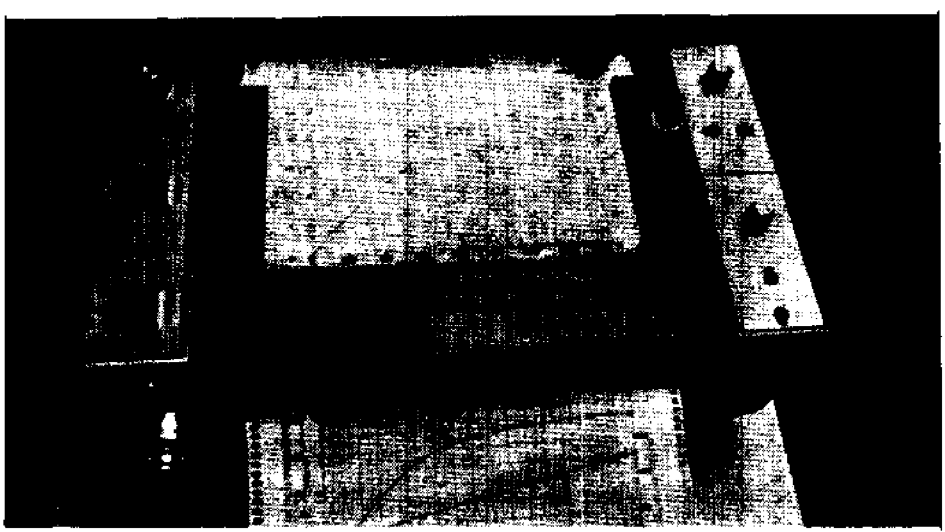
TYPICAL SLURRY TEST PRINTOUT

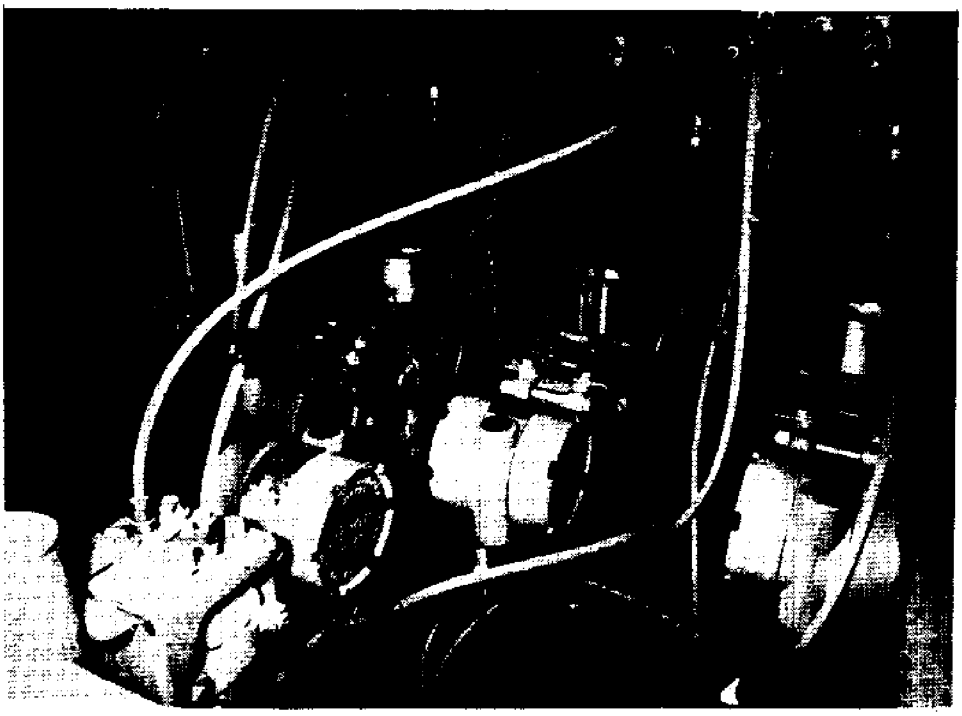
Printed output is available immediately after a test is completed. It covers all readings taken and includes such calculated values as Reynolds Number pipe friction, fluid temperature, and volume concentration of slurry.

Because of output sheet space limitations, printed output for slurry tests are usually separated into pump oriented and pipe oriented output.

All test data is stored on cassette for later use. Plotted output is produced automatically for pump head quantity and efficiency performance. Metric plots and printouts are available by calling on special options. The mini computer is also programmed to allow speed change or size scaling adjustments to tested performance.



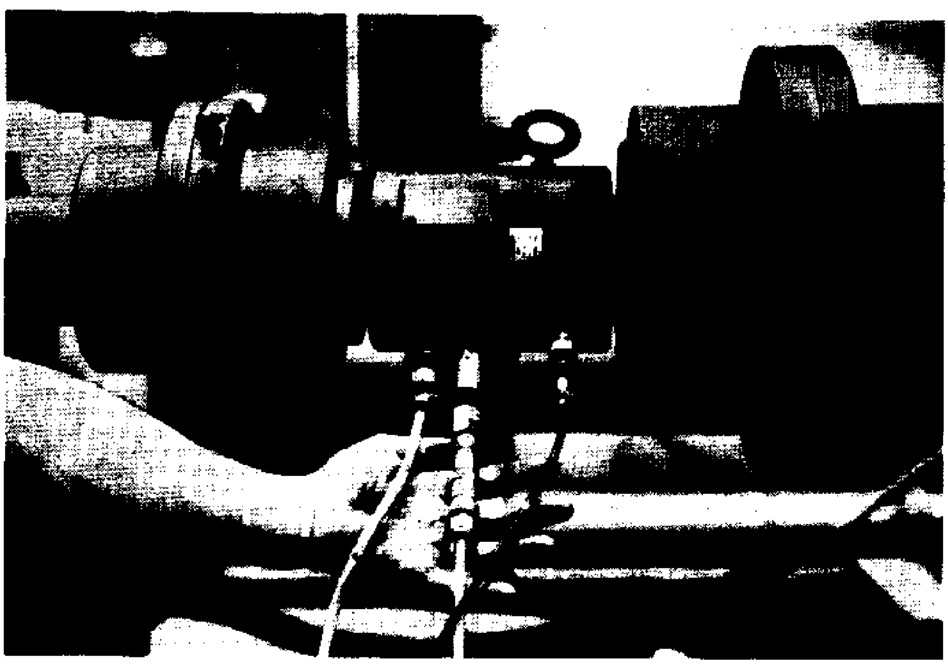
PUMP TEST PLOTTED OUTPUT



ROSEMOUNT PRESSURE TRANSDUCERS

Over 30 different precision transducer-type instruments are available for use with the mini computer system. Each of these has an analogue output that can be connected into the analogue to digital conversion unit. A special patching system is provided so that the instrumentation for a test may be varied easily.

Sufficient transducers are provided so that for a normal test, every reading is duplicated. Pizometer points are also duplicated so that every primary reading is backed up by some secondary reading. The mini computer has been programmed to allow readings and results from primary and secondary sources to be seen alternatively at a single key command. This enables the test engineer to see immediately if any readings are wrong because of blocked pizometers, etc.



LEBOW SERIES 1100 TORQUE BARS.

Each loop system has its own magnetic flowmeter, doppler flowmeter, bend meter, and orifice plate. Power may be measured from a two element type wattmeter or from one of the three strain gauge type torque bars. A minimum of four temperature transducers are allowed for in the system and eleven high accuracy capacitance type pressure transducers are provided for a variety of pump head and pipeline loss readings.

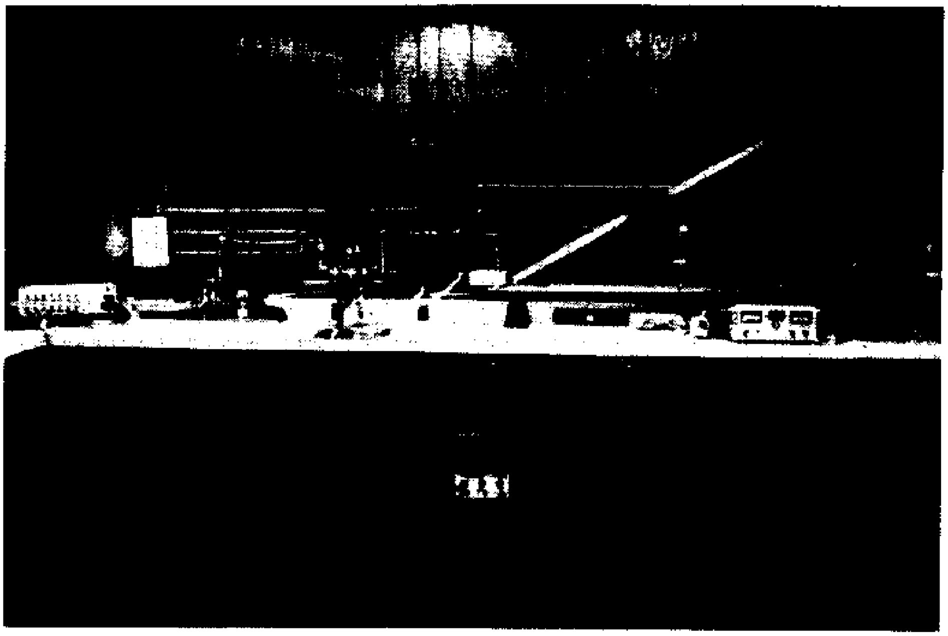
A large tank is available for flow calibration. Specific gravity is measured by the pressure difference across the two legs of a vertical U-loop section. A full pipe cross section sample cutting device is available, complete with sample weight, sample volume, and time over which it is cut for recording specific gravity and volume rate measurement also.

In addition to the transducer type instruments, the lab is equipped with a full set of manual gauges and meters sufficient to allow a pump or pipeline test to be carried out without the use of the A to D unit or computer.

Calibration of all the instruments used in the GIW lab is carried out routinely by the most accurate methods possible. Pressure transducer calibrations are carried out using a dead weight tester, an inclined mercury manometer, and a vertical water column - depending on the range of the transducer.

The magnetic flowmeter calibration is carried out using a secondary calibrator. The orifice co-efficiencies are determined by calculation. Both units are also checked against a change in volume in the 20,000 gallon sump over a timed period.

8" and 18" SAMPLER CUTTERS



CALIBRATION EQUIPMENT

The torque bar is calibrated using a precision resistor and an electrical resistance method specified by the manufacturer.

These methods have been checked by comparison with applied torques. All calibrations are filed and stored on cassette for later reference as required.

Control of all tests is carried out from within the test engineer's office using motorized valves and solenoid operated controls. Even the sample device is operated remotely using pneumatic cylinders and solenoid operated valves from the operator's console.

Once a system is loaded with material, it is only necessary for the test engineer to leave his control console to observe any stratification or deposition velocity at the glass.

The simultaneous nature of the readings and the back-up provided by the GIW data aquisition system ensures much greater accuracy than has hitherto been possible. The large number of measurements possible enables a test to include all the variables involved and in particular, it is possible to make a slurry test a test of both the pipeline and the pump on solids.

A test in the GIW system using the above equipment is able to be carried out much faster than has ever been possible in the past. A slurry test at several concentrations that would normally take a week, can be carried out in the GIW system in one day.

GIW has available a large Burroughs computer and a data line between the Lab and the Burroughs computer.

Results of tests carried out in the Hydraulic Lab can be transmitted to the GIW Burroughs Main Frame Computer for more detailed analysis and comparison.

2. SETTLING AND NON-SETTLING SLURRY PIPELINE TESTING

The power required to drive a mass rate of flow of solid liquid mixture through a pipeline is essential information in the design of any pipeline transportation system.

This can normally be separated into the power absorbed by the resistance to flow inside the pipe and the energy given to the fluid as a result of any level change.

The determination of the power required to change the level of a given mass rate of flow is usually easily obtained and is related directly to the final level change and the mass rate of flow involved.

The resistance to the flow inside the pipe is subject to more variables and requires more detailed consideration.

The first law of thermodynamics states that:

The change in heat + the change in work = change in internal energy

$$\delta Q + \delta W = \delta U$$

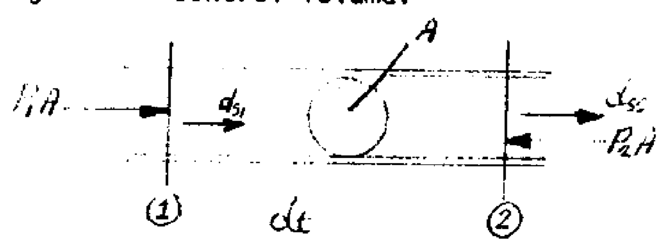
If we consider the case of a horizontal section of pipe (assuming we are clear of all bends and obstructions), the potential energy, the kinetic energy, and the internal energy will remain unchanged, thus equation (1) reduces to

$$\delta Q = -\delta W$$

meaning that all the work expended in pushing the fluid through the pipe is dissipated as heat.

It is not possible to measure the temperature change with sufficient accuracy to measure the work expended.

Consider again our control volume.



The work in time dt

$$\delta W = P_1 \times A \times ds_1 - P_2 \times A \times ds_2$$

where the fluid is incompressible and V is the mean velocity:

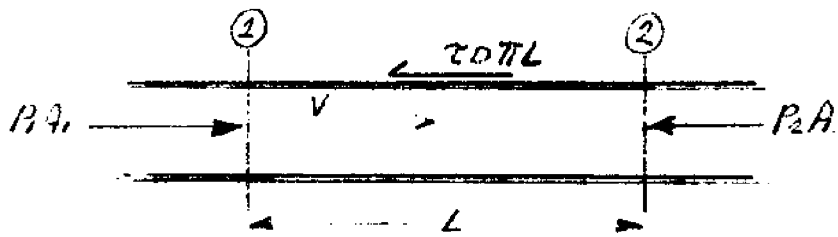
$$\delta W = P_1 \times A \times V \times dt - Pb \times A \times V \times dt$$

$$= Pa \times Q \times dt$$

$$\frac{\delta W}{dt} = (P_1 - P_2)Q$$

which gives us an expression for power in terms of the pressure difference across a control section of pipe.

The force involved in pushing the liquid through the pipe is resisted by friction at the pipe walls. Using Newton's Second Law of Motion, we may derive another expression for a control volume in terms of the pressure found and the pipe friction.



$$P_1 \times A - P_2 \times A - \tau_0 D\pi L = 0$$

$$\delta P_{1-2} = \frac{4\tau_0 L}{D}$$

where τ_0 is the pipe wall boundary shear stress.

The boundary shear stress can be considered as a skin drag force on which

$$FD = A \times \tau_0 \propto \rho \frac{V^2}{2} \times A \text{ where}$$

ρ = fluid density
V = reference velocity
F_D = drag force

introducing a drag co-efficient constant or friction factor f gives us

$$\tau_0 = f \frac{\rho V^2}{2}.$$

Now substituting this back into the expression for pressure difference in terms of the pipe wall boundary shear stress, we get:

$$\delta P_{1-2} = 4f \frac{\rho V^2}{2} \times \frac{L}{D}$$

Using the relation $w = \rho g$ and $P = \rho g H$ where w = specific weight (lb)(τ3)

ρ = (slugs/ft^3)

P = pressure (lb/ft^3)

H = liquid level (ft)

converting the pressure loss into head loss in level of liquid units we get

$$H_{f\,(1-2)} = \frac{\delta P(1-2)}{\rho g} = \frac{4f\,\rho V^2}{2xgx\rho} \times \frac{L}{D} = \frac{4fLV^2}{2gD}$$

The friction factor f in the above expression depends on the Reynolds Number and the relative roughness.

ie $f = \phi(N_R, e)$ where D = pipe dia. (ft)

V - velocity inside pipe (ft/sec)

μ = viscosity (lb sec/ft^2)

k = roughness height (ft)

N_R = Reynolds Number

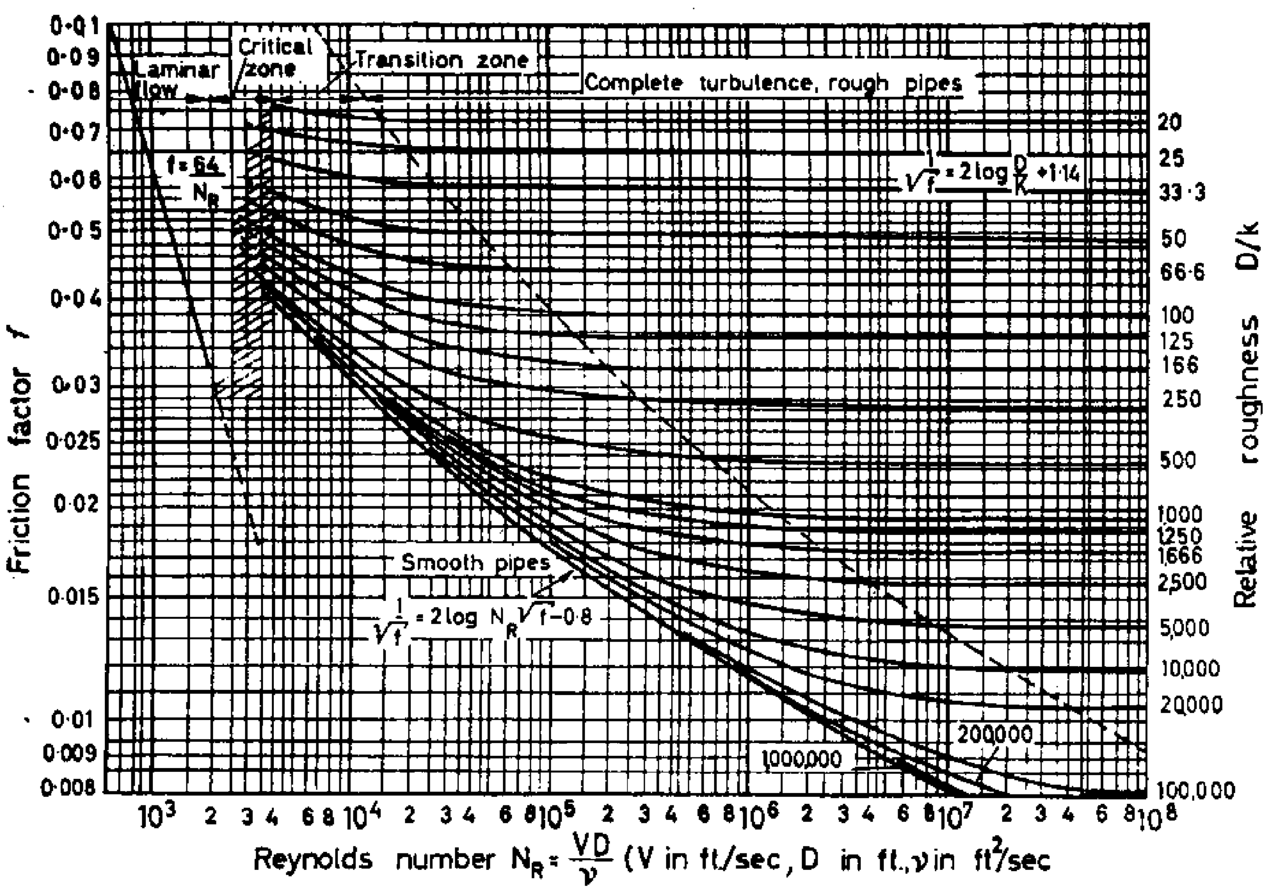
Noting that: $N_R = \frac{\rho VD}{\mu}$ and $e = \frac{k}{D}$

For the Reynolds Number values of greater than 3000 (for turbulent flow), the friction factor for water flowing in pipes can be found from the following relation developed by C. F. Colebrock.

$$\frac{1}{\sqrt{f}} = -2\log_{10}\left[\frac{\epsilon}{3.7D} + \frac{2.51}{R\sqrt{f}}\right]$$

and for values less than 2000 corresponding to laminar flow, it can be found from the Weisbach Equation: $f = \frac{64}{Re}$

Both of these are combined in the Moody Diagram shown below.



The Moody Diagram covers all Newtonian fluids regardless of viscosity and temperature. It and the previous theory enable some understanding of the mechanism of flow and the variables involved.

A solids liquid mixture is not a Newtonian fluid, but approaches the Newtonian fluid in characteristics for a range of solids and applications.

A comparison of head loss and friction factor values of the solids liquid mixture and the carrier fluid is the main means of collecting and analyzing slurry pipe friction data used in the GIW Hydraulic Lab and described later.

Another less reliable but easier method of estimating and evaluating pipe friction head loss values for Newtonian Fluids is the Hazen and Williams C value defined as follows:

$$hf = 0.2083\frac{(100)^{1.85}}{(C)^{1.85}} \times \frac{\text{FLOW GPM}^{1.85}}{\text{Pipe I.D.}^{4.8655}}$$

where hf is in ft of liquid, flow is in GPM (US)

The "C" value in this case is a number varying from about 100 (rough pipe) up to about 150 (smooth pipe) which can be used to categorize the absolute roughness. C values remain reasonably constant in the range of Reynolds values 1×10^5 to 2.5×10^6. Where a solids liquid mixture acts as a pseudo fluid, it is possible to use "c" values to categorize pipe friction and estimate head loss values.

The "C" value is used widely in the mining industry. Pipe friction data collected in the GIW Hydraulic Lab is compared with "C" value charts as another means of analyzing data.

The GIW Hydraulic Lab has three pipe test systems of 4", 8", and 17.25 inch inside diameter pipe. Long straight horizontal sections are set up in each of these to enable pressure from across a given test section to be measured.

The test sections in each of the 4", 8", and 17.25" diameter systems are 30 feet, 50 feet, and 77.5 feet long, respectively, with straightening sections preceeding these of 30 feet, 60 feet, and 55 feet each.

The pressure measurements are recorded with a number of different high accuracy Rosemount differential transducers that are located in a transducer trolley.

Piezometers are made up out of ½" NTP fittings with 1/8" diameter edge free holes into the pipe. These are connected to sediment traps which are fitted with drain holes and vent taps. The pressure lines from the transducers are of 5/16" I. D. nylon tube.

A clear water flushing manifold is set up in the transducer trolley so that each piezometer line may be purged clear of solids through individual isolation valves. Purge water is provided by a separate high pressure gland water pump. The purge manifold can be connected to the inclinded manometer and dead weight tester for calibration of all the transducers at the same time.

Solenoid operated air vent tappings are provided at the top of each pipe section. The piezometers are located at 45 degress to the vertical. Two piezometers complete with sediment trap lines and transducers are set up on each line so that a primary and secondary pressure drop reading may be obtained. The primary reading may have a low range and high range transducer connected in for increased accuracy if required.

A GIW pipeline test of a particular slurry commences with adding a given concentration of material in the system and then circulating, using one of the centrifugal pumps provided.

Flow is varied by throttling with a pinch or butterfly type valve. Care is taken to see that any pump gland or flushing system does not dilute the slurry.

Head loss measurements are taken at a number of different steady flows from 0 ft/sec velocity up to about 20 ft/sec in steps of about 2 ft/sec.

Steady flow is assumed to exist when variations in flow and head loss of successive readings vary by less than 1%.

Readings are recorded using the mini computer data acquisition system described previously. The averaging period used for the transducers is usually about 10 seconds.

Flow measurement is recorded at the same time. The primary flow reading is normally taken from the magnetic flow meter. Secondaries usually included are the doppler and the bend meter. A sampler device is available but is usually limited to low flows or where something is leading the test engineer to doubt one of his other flow meters.

When sufficient readings have been collected over the range of flows necessary monitoring is stopped and any particular test comments noted in the comment section for later printing out.

The computer then calculates, prints and plots the results as shown on attached sample sheets NO. M 50-79.

The value of i_m is the head loss per foot of pipe in units of ft of water pressure. The value of i_w is the head loss per ft of the same pipe for a previous water test (from stored data) corrected for temperature and at the same Reynolds number.

PUMP TEST DATA FOR GIW SLURRY COURSE

PROJECT	18" SAND SLURRY TEST
PUMP TYPE	GIW 18"/18" HE 44
CLIENTS ORDER NO	18" DIA. SLURRY LOOP
ASSEMBLY DRAWNG NO	1839D
SHELL DRAWNG NO	1877D
IMPELLER DRAWNG NO	1847D
IMPELLER DIAM	44" B5=5.875" 34 DEG OUT
PUMP ROTATION	
HYDROSTATIC PSI =450	WITNESSED

TEST CONSTANTS

.00 INS DIA IN .00 INS PIPE
1 FT H2O = .0 US GPM USING
BEND HT CORR =-1.0 FT CONST = 4959.0
DISCHARGE PIPE DIAMETER = 17.28 INS
METER .00' ABOVE PUMP DATUM, TAP 2.30'
SUCTION PIPE DIAMETER = 17.28 INS
METER .00' ABOVE PUMP DATUM, TAP .00'
STATIC WATER LEVEL .00 FT ABOVE F/D
WATTMETER MULT = 800. BAROMETER = 29.6"
HEAD LOSS = 77.50 FT OF 17.28 INCH DIAM
S.G. TAPS 12.00' APART G = 32.14 FT/S/S
SOLIDS SG 2.65 OF 270. MICRONS S.D.=1.6
FOR VEL CALCS PIPE DIAMETER = .00 INS
SAMPLER: HT CORR = .00' AREA = .00 SQ'
BALANCE VOLUME CONSTANT = .00 CUBIC '

CH	USE	RDG	SOURCE	INSTRUMENT	
1		DISC	DIFF HD	ROSEMOUNT 0-240FT H20 02269B1E1	1.000
2		WATT	METER	LANDIS AND GYR 1E3	1.000
3	NULL	FLOW	8" MAG	FISCHER & PORTER 8'GPM01089A	.000
4		SUCT	TRANSDR	FOXBRO E13DM 0-34'H20 12088B1E2	1.000
5		DISC	TRANSDR.	FOXBRO E11GM 1000'H20 06209A1E1	1.000
6	LOW	FLOW	BEND	.FOXBRO E13DM 0-20 H20 12088B1E2	1.000
7	HIGH	FLOW	BEND	.FOXBRO E13DM 0-74'H20 12088B1E2	1.000
8	NULL	TORQ	TRQ LIL	LEBOW/DAYTRONIC LITTLE10268H	.000
9		RPM	TRQ BAR	LEBOW/DAYTRONIC RPM 10268B	1.000
10	NULL	BHP	LIL BAR.	LEBOW/DAYTRONIC HP 8' 040877	.000
11	HIGH	LOSS	77.5 FT	ROSEMOUNT 0-18 FT H20 12088B1E2	1.000
12	AVE	S.G.	U-SECDN	ROSEMOUNT-1-11 FT H20-12088B1E2	.500
13	LOW	LOSS	77.5 FT	ROSEMOUNT 0- 9 FT H20 12088B1E3	1.000
14		LOSS	INCLINE.	ROSEMOUNT 0- 2 FT H20 12088B1E3	1.000
15	AVE	S.G.	U-SECUP	ROSEMOUNT 0- 5 FT H20 12088B1E3	.500
16		FLOW	18" MAG	FISCHER & PORTER18'GPM03279B1E-1	1.000
17		TEMP	18" DIS	PADGETT 18'DISCH (F) 06178C1E1	1.000
18		TEMP	8" DIS.	PADGETT 8" DISCH (F) 06178C1E1	1.000
19		SGLT	SG.U #1.	PADGETT U-LOOP#1 (F) 06178C1E1	1.000
20		SGLT	SG.D #4	PADGETT U-LOOP#4 (F) 06178C1E1	1.000
21	NULL	S.G.	SAMPLER.		.000
22	NULL	SLVL	SAMPLER.		.000
23	NULL	STIM	SAMPLER.		.000
24	NULL	FLOW	8" DOP	LEEDS AND NORTHRUP 8" 07248	.000
25		FLOW	18" DOP.	LEEDS AND NORTHRUP 18'07248 1E-1	1.000
26	NULL	FLOW	BEND	.FOXBRO E13DM 0-20 H20 12089B1E2	.000
27		TORQ	TRQ BIG	LEBOW/DAYTRONIC BIG 10268E1E-1	1.000
28		BHP	BIG BAR.	LEBOW/DAYTRONIC HP 18'040877	1.000
29		BHP	TRQ*RPM		1.000
30		BHP	CALC'ED.	FROM 200 HP MOTOR KILOWATTS	1.000
31		SUCT	TRANSDR.	ROSEMOUNT-30-30FT H20-06209A1E2	1.000
32		RPM	LEUZE	.	1.000

GEORGIA IRON WORKS CO.
P.O. BOX 626
GROVETOWN,
GEORGIA 30813

WORK ORDER NO	GIW COURSE
SERIAL NO	5012 3514
TEST CURVE NO	T 50 -79
TEST DATE	06/20/79

DRIVER DETAIL

TYPE	LAB MOTOR
MAKE	GENERAL ELEC
SERIAL NO	5511957
FRAME SIZE	5368480
RPM = 360	BHP = 2000.
4160 VOLTS 3 PHASE 60 CPS	

SCALED PERFORMANCE FACTRS

SPEED OR RATIO	.00
IMP TURN DOWN RATIO	.00
MERIDNL WIDTH RATIO	.00
SCALE RATIO	.00
BEP REF .HG,	.RPM
EFFICIENCY .0% BY	.00

TEST RESULTS

NO	FLOW MEASUREMENT: DIFF. ' H2O	FLOW US GPM	HEAD MEASUREMENT: DISCH ' H2O	SUCTN ' H2O	TOT HD ' FLUID	DRIVE INPUT, SPEED, OUTPUT: WATT METER	KILO-WATTS	RPM	BHP	PUMP: WATER HP OUT	EFFCY %	VEL FT/S	S.G.	HEAD LOSS /100'	HAZN WLLM	FRCTN: FACTR:	TEMP (F)	TIME HH,MM
1:	.00:	14738.2:	79.35:	-24.84:	70.84:	.413:	330.7:	360:	386.6:	296.4:	76.7:	20.2 :	1.12 :	5.40:	150.:	.0109:	101.4:	15.49
2:	.00:	11910.0:	88.87:	-12.36:	77.07:	.354:	283.3:	360:	322.2:	268.4:	83.4:	16.3 :	1.16 :	4.66:	134.:	.0140:	101.8:	15.55
3:	.00:	10636.0:	90.16:	-7.79:	78.41:	.329:	263.0:	360:	294.4:	243.1:	82.6:	14.6 :	1.15 :	4.44:	122.:	.0168:	102.1:	16.00
4:	.00:	9622.4:	90.83:	-4.99:	79.36:	.313:	250.1:	360:	276.3:	221.5:	80.2:	13.2 :	1.15 :	4.63:	108.:	.0215:	101.9:	16.03
5:	.00:	8746.7:	91.12:	-3.34:	80.06:	.299:	238.8:	360:	262.0:	202.0:	77.1:	12.0 :	1.14 :	4.51:	99.:	.0255:	102.2:	16.05
6:	.00:	8184.9:	91.48:	-2.08:	81.19:	.288:	230.6:	360:	250.7:	189.7:	75.7:	11.2 :	1.13 :	4.42:	93.:	.0289:	102.4:	16.07
7:	.00:	7722.2:	91.55:	-1.17:	81.57:	.281:	224.6:	360:	241.2:	179.1:	74.3:	10.6 :	1.13 :	4.32:	89.:	.0318:	102.7:	16.11
8:	.00:	6705.0:	92.30:	.36:	84.94:	.261:	208.8:	360:	220.6:	156.6:	71.1:	9.2 :	1.09 :	3.87:	81.:	.0386:	102.9:	16.15
9:	.00:	5906.2:	91.80:	1.28:	86.41:	.245:	196.1:	360:	203.7:	137.1:	67.4:	8.1 :	1.06 :	3.57:	73.:	.0475:	103.0:	16.17
10:	.00:	4870.5:	90.51:	2.95:	88.13:	.223:	178.7:	360:	179.9:	111.4:	62.0:	6.7 :	1.03 :	2.97:	65.:	.0603:	103.3:	16.19
11:	.00:	3739.7:	90.05:	4.34:	89.95:	.205:	163.7:	360:	159.3:	85.0:	53.4:	5.1 :	1.00 :	1.40:	74.:	.0494:	103.5:	16.21

GEORGIA IRON WORKS CO.
TEST ENGINEER J.R. MAFFETT DATE 06/20/79

WITNESSED BY GROUP 1 FOR GIW SLURRY COURSE

COMMENTS

FOUNDRY SAND IN 18" SYSTEM FOR GIW SLURRY COURSE
STATIONARY BED OBSERVED TO FORM AT 8.9 FT/SEC
NO DIFFICULTIES EXPERIENCED

GROUP 1

PUMP TEST DATA FOR GIW SLURRY COURSE

PROJECT 18" SAND SLURRY TEST
PUMP TYPE GIW 18"/18" HE 44
CLIENTS ORDER NO 18" DIA. SLURRY LOOP
ASSEMBLY DRAWNG NO 1839D
SHELL DRAWNG NO 1877D
IMPELLER DRAWNG NO 1847D
IMPELLER DIAM 44" B5=5.875" 34 DEG OUT

PUMP ROTATION
HYDROSTATIC PSI =450 WITNESSED

TEST CONSTANTS

.00 INS DIA IN .00 INS PIPE
1 FT H20 = .0 US GPM USING
BEND HT CORR =-1.0 FT CONST = 4959.0
DISCHARGE PIPE DIAMETER = 17.28 INS
METER .00' ABOVE PUMP DATUM, TAP 2.30'
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SAMPLER: HT CORR = .00' AREA = .00 SQ'
BALANCE VOLUME CONSTANT = .00 CUBIC '

CH	USE	RDG	SOURCE INSTRUMENT	
1		DISC	DIFF HD ROSEMOUNT 0-240FT H20 0226981E1	1.000
2		WATT	METER LANDIS AND GYR 1E3	1.000
3	NULL	FLOW	8" MAG FISCHER & PORTER 8"GPM01089A	.000
4		SUCT	TRANSDR FOXBRO E13DM 0-34'H20 12088B1E2	1.000
5		DISC	TRANSDR.FOXBRO E11GM 1000'H20 06209A1E1	1.000
6	LOW	FLOW	BEND .FOXBRO E13DM 0-20 H20 12088B1E2	1.000
7	HIGH	FLOW	BEND .FOXBRO E13DM 0-74'H20 12088B1E2	1.000
8	NULL	TORQ	TRQ LIL LEBOW/DAYTRONIC LITTLE10268H	.000
9		RPM	TRQ BAR LEBOW/DAYTRONIC RPM 10268R	1.000
10	NULL	BHP	LIL BAR.LEBOW/DAYTRONIC HP 8" 040877	.000
11	HIGH	LOSS	77.5 FT ROSEMOUNT 0-18 FT H20 12088B1E2	1.000
12	AVE	S.G.	U-SECDN ROSEMOUNT-1-11 FT H20-12088B1E2	.500
13	LOW	LOSS	77.5 FT ROSEMOUNT 0- 9 FT H20 12088B1E3	1.000
14		LOSS	INCLINE.ROSEMOUNT 0- 2 FT H20 12088B1E3	1.000
15	AVE	S.G.	U-SECUP ROSEMOUNT 0- 5 FT H20 12088B1E3	.500
16		FLOW	18" MAG FISCHER & PORTER18"GPM03279B1E-1	1.000
17		TEMP	18" DIS PADGETT 18"DISCH (F) 06178C1E1	1.000
18		TEMP	8" DIS.PADGETT 8" DISCH (F) 06178C1E1	1.000
19		SGLT	SG.U #1.PADGETT U-LOOP#1 (F) 06178C1E1	1.000
20		SGLT	SG.D #4 PADGETT U-LOOP#4 (F) 06178C1E1	1.000
21	NULL	S.G.	SAMPLER.	.000
22	NULL	SLVL	SAMPLER.	.000
23	NULL	STIM	SAMPLER.	.000
24	NULL	FLOW	8" DOP LEEDS AND NORTHRUP 8" 07248	.000
25		FLOW	18" DOP.LEEDS AND NORTHRUP 18"07248 1E-1	1.000
26	NULL	FLOW	BEND .FOXBRO E13DM 0-20 H20 12088B1E2	.000
27		TORQ	TRQ BIG LEBOW/DAYTRONIC BIG 10268E1E-1	1.000
28		BHP	BIG BAR.LEBOW/DAYTRONIC HP 18"040877	1.000
29		BHP	TRQ*RPM	1.000
30		BHP	CALC'ED.FROM 200 HP MOTOR KILOWATTS	1.000
31		SUCT	TRANSDR.ROSEMOUNT-30-30FT H20-06209A1E2	1.000
32		RPM	LEUZE .	1.000

GEORGIA IRON WORKS CO.
P.O. BOX 626
GROVETOWN,
GEORGIA 30813

WORK ORDER NO GIW COURSE
SERIAL NO 5012 3514
TEST CURVE NO M 50 -79
TEST DATE 06/20/79

DRIVER DETAIL

TYPE LAB MOTOR
MAKE GENERAL ELEC
SERIAL NO 5511957
FRAME SIZE 5368480
RPM = 360 BHP = 2000.
4160 VOLTS 3 PHASE 60 CPS

SCALED PERFORMANCE FACTRS

SPEED OR RATIO .00
IMP TURN DOWN RATIO .00
MERIDNL WIDTH RATIO .00
SCALE RATIO .00
BEP REF .HG, .RPM
EFFICIENCY .0% BY .00

TEST RESULTS

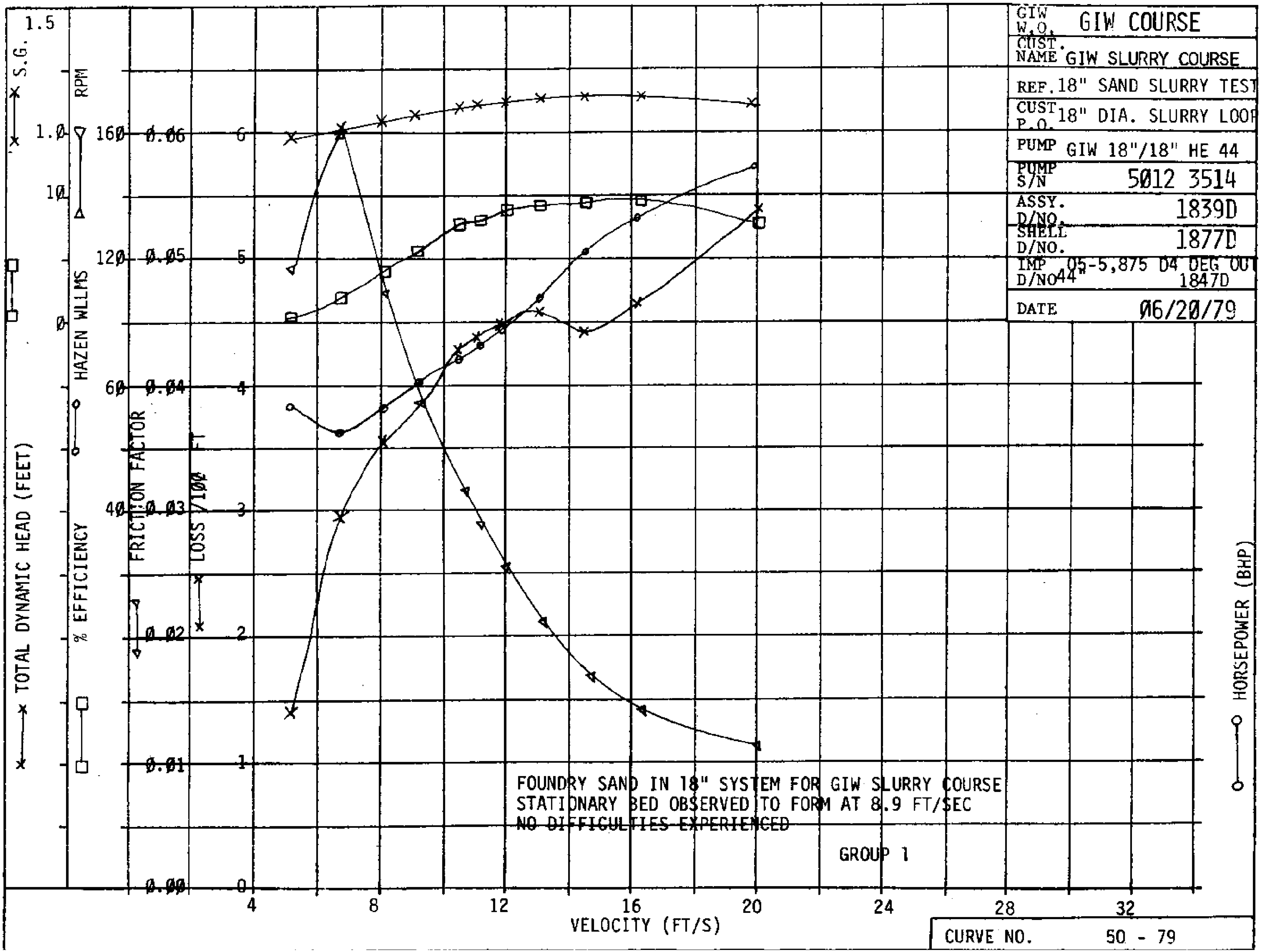
NO	VELOCITY Vm FT/S	FLOW Qm GAL/MIN	TEMP Tm F	S.G. Sw	S.G. Sm	VOLUME CONC. Cv %	WEIGHT CONC. Cw %	MASS Ms TON/HR	REYNOLDS NUMBER Re	PIPELINE LOSSES Im FT/FT	PIPELINE LOSSES Iw FT/FT	FRICTION FACTRS Fm	FRICTION FACTRS Fw SAMF Re	HAZEN WLLMS C	Im-Iw ----- Sm-Sw	TIME t HH.MM
1	20.16	14738.2	101.4	.994	1.12	7.9	18.5	760.8	.400E+07	.0540	.0561	.0109	.0128	150.	-.0166	15.49
2	16.29	11910.0	101.8	.994	1.16	9.9	22.6	782.4	.325E+07	.0466	.0368	.0140	.0128	134.	.0598	15.55
3	14.55	10636.0	102.1	.994	1.15	9.7	22.2	684.1	.291E+07	.0444	.0294	.0160	.0129	122.	.0934	16.00
4	13.16	9622.4	101.9	.994	1.15	9.4	21.6	597.8	.263E+07	.0463	.0242	.0215	.0129	108.	.1426	16.02
5	11.97	8746.7	102.2	.994	1.14	9.0	20.8	520.0	.239E+07	.0451	.0200	.0255	.0130	99.	.1692	16.05
6	11.20	8184.9	102.4	.994	1.13	8.2	19.3	447.7	.224E+07	.0442	.0176	.0289	.0130	93.	.1952	16.07
7	10.56	7722.2	102.7	.994	1.13	8.0	18.8	408.0	.212E+07	.0432	.0157	.0318	.0130	89.	.2095	16.11
8	9.17	6705.0	102.9	.994	1.09	5.8	14.0	256.2	.185E+07	.0382	.0119	.0386	.0131	81.	.2755	16.15
9	8.08	5906.2	103.0	.994	1.06	4.2	10.6	166.6	.163E+07	.0357	.0095	.0475	.0132	73.	.3747	16.17
10	6.66	4870.5	103.3	.994	1.03	2.1	5.3	66.7	.134E+07	.0297	.0064	.0603	.0134	65.	.6034	16.19
11	5.12	3739.7	103.5	.994	1.00	.5	1.2	11.4	.103E+07	.0140	.0038	.0494	.0136	74.	1.3357	16.21

GEORGIA IRON WORKS CO.
TEST ENGINEER J.R. MAFFETT DATE 06/20/79
WITNESSED BY GROUP 1 FOR GIW SLURRY COURSE

COMMENTS

FOUNDRY SAND IN 18" SYSTEM FOR GIW SLURRY COURSE
STATIONARY BED OBSERVED TO FORM AT 8.9 FT/SEC
NO DIFFICULTIES EXPERIENCED

GROUP 1


GIW W.O. GIW COURSE
CUST. NAME GIW SLURRY COURSE
REF. 18" SAND SLURRY TEST
CUST P.O. 18" DIA. SLURRY LOOP
PUMP GIW 18"/18" HE 44
PUMP S/N 5Ø12 3514
ASSY. D/NO. 1839D
SHELL D/NO. 1877D
IMP D/NO 05-5,875 D4 DEG OUT 44" 1847D
DATE Ø6/2Ø/79
FOUNDRY SAND IN 18" SYSTEM FOR GIW SLURRY COURSE
STATIONARY BED OBSERVED TO FORM AT 8.9 FT/SEC
NO DIFFICULTIES EXPERIENCED
GROUP 1
VELOCITY (FT/S)
CURVE NO. 50 - 79
S.G.
RPM
HAZEN WLLMS
FRICTION FACTOR
LOSS /1ØØ FT
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
HORSEPOWER (BHP)

The f_m and f_w values are similar to the i_m and i_w values except that they are pipe friction factor values calculated using head loss in ft of slurry for f_m and the previous water test value for the f_w. The $\frac{i_m - i_w}{S_m - S_w}$ is the solids effect of the slurry in the pipeline where the slurry specific gravity S_m and the water specific gravity S_w are used to correct the result into a dimensionaless term.

The plots produced are self explanatory and are as shown on attached sample. It should be noted that each data point is included and the line between each point is a best fit according to a moving spline relation.

To date, over 44 slurry tests have been carried out in the hydraulic lab. Eighteen of these have been in the 18" pipe diameter system and the remainder in the 8" diameter pipe system. Nineteen of these tests have been for outside customers and the remainder for GIW research. The materials tested in the slurry loops have varied from heavy phosphate slimes through taconite tailings, taconite ore, coarse pebble, foundry sand to phosphate rejects and phosphate ore.

Slurries and tests associated with them may be catagorized into two main types namely the settling or a non-settling type.

The settling type exhibits some deposit velocity at which the particles will fall to the bottom of the pipe. This will vary with the particle sizes. The material sizing will result in this occurring over a range of velocities. Above this, the material will move as a sliding bed then varying degrees of stratification as the velocity in the pipeline increases.

A good example of a settling slurry is a foundry sand. Test results on a foundry sand slurry are shown in attached graph of head loss versus velocity test M 71-78 in an 18" diameter pipe at a slurry s.g. of approximately 1.2.

It was noted in this case that the slurry was stratified at velocities below 15 f/sec and that a fixed bed formed at velocities below 9 ft/sec. As the test loop is a fixed inventory system, the deposition can also be seen as a drop of s.g. in the system.

Most settling slurries exhibit similar characteristics with the deposition velocity, the head loss and the degree of stratification varying with the sizing of the material, the concentration, the sizing distribution, the carrier fluid rheology and the size of pipeline.

A few of the settling slurry tests carried out in the hydraulic lab and illustrating the above are listed on the next page.

PUMP TEST DATA FOR TEST IN 18"SLURRY SY

PROJECT	'C'VALUE FOR WATER
	G.I.W HYD LAB DATA
PUMP TYPE	20/24"LSA 48 (NEW)
CLIENTS ORDER NO	K739270 GIW BIL 3363
ASSEMBLY DRAWING NO	1731D
SHELL DRAWING NO	1783D
IMPELLER DRAWNG NO	1775D 4 VANE 30 OUT
IMPELLER DIAM	47.75" B2=9.125 30 DEG
PUMP ROTATION	CLOCKWISE
HYDROSTATIC PSI =STID	YES WITNESSED

INSTRUMENTS

```
     DISC DIFF HD ROSEMOUNT 0-100    PSI 04198A2.309
     WATT METER    LANDIS AND GYR              1.000
     FLOW MAGFLOW FISCHER & PORTER 18"   10MAR 1.000
     SUCT TRANSDR FOXBRO E13DM 0-30 "HG 24FEB 1.048
     FLOW DOPPLER.LEEDS AND NORTHRUP    JUNE 11.000
LOW  FLOW ORIFICE.FOXBRO E13DM 0-20 H2O 05248A1.000
     TEMP TEM:SGU.PADGETT             (F) 06178C1.000
     TORQ TRQ BAR LEBOW/DAYTRONIC              1.000
     RPM  TRQ BAR LEBOW/DAYTRONIC              1.000
     TEMP TEM:SGD.PADGETT             (F) 06178C1.000
     SG.U U-SECTN ROSEMOUNT 0-18 FT H2O 06078C1.000
     SG.D U-SECTN ROSEMOUNT 0- 5 FT H2O 06178B1.000
     LOSS 77 FT   ROSEMOUNT 0- 9 FT H2O 06078C1.000
     LOSS 50 FT  .ROSEMOUNT 0- 2 FT H2O 06178B1.000
     LOSS 50 FT  .ROSEMOUNT 0- 5 FT H2O 06178B1.000
     TEMP TEM:DIS PADGETT             (F) 06178C1.000
```

GEORGIA IRON WORKS CO.
P.O. BOX 626
GROVETOWN,
GEORGIA 30813

WORK ORDER NO	4658
SERIAL NO	5012-3363
TEST CURVE NO	M 71 -78
TEST DATE	06/23/78

DRIVER DETAIL

TYPE	LAB MOTOR
MAKE	GENERAL ELEC
SERIAL NO	
FRAME SIZE	5368480
RPM = 360	BHP = 2000.

4160 VOLTS 3 PHASE 60 CPS

SCALED PERFORMANCE FACTRS

SPEED OR RATIO	.00
IMP TURN DOWN RATIO	.00
MERIDNL WIDTH RATIO	.00
SCALE RATIO	.00

OPERATING CONDITIONS

	FIELD	LAB
R.P.M.	470.	360.
FLOW	27500.	20000.
T.D. HEAD	140.	90.
EFFICIENCY	83.	.
KWHRS/1000	.000	.000
NPSH	.0	.0
SP. GRAVITY	1.00	.00

TEST CONSTANTS

18.00 INS DIAM VLOCITY IN 18.00 INS DIAM PIPE
1 FT OF WATER = 4959. US GPM USING CODE C89
DISCHARGE PIPE DIAMETER = 19.25 INS
METER IS .00' ABOVE PUMP DATUM, TAP IS 2.75'
SUCTION PIPE DIAMETER = 19.25 INS
METER IS .00' ABOVE PUMP DATUM, TAP IS .00'
STATIC WATER LEVEL IS .00' ABOVE PUMP DATUM
HEAD LOSS LENGTH = 78. FEET OF 17.28 INCH DIA
S.G. TAP SPACING = 12.00 FT G = 32.14 FT/S/S
SOLIDS SG 2.65 SIZE 290. MICRONS STD DVN 1.5

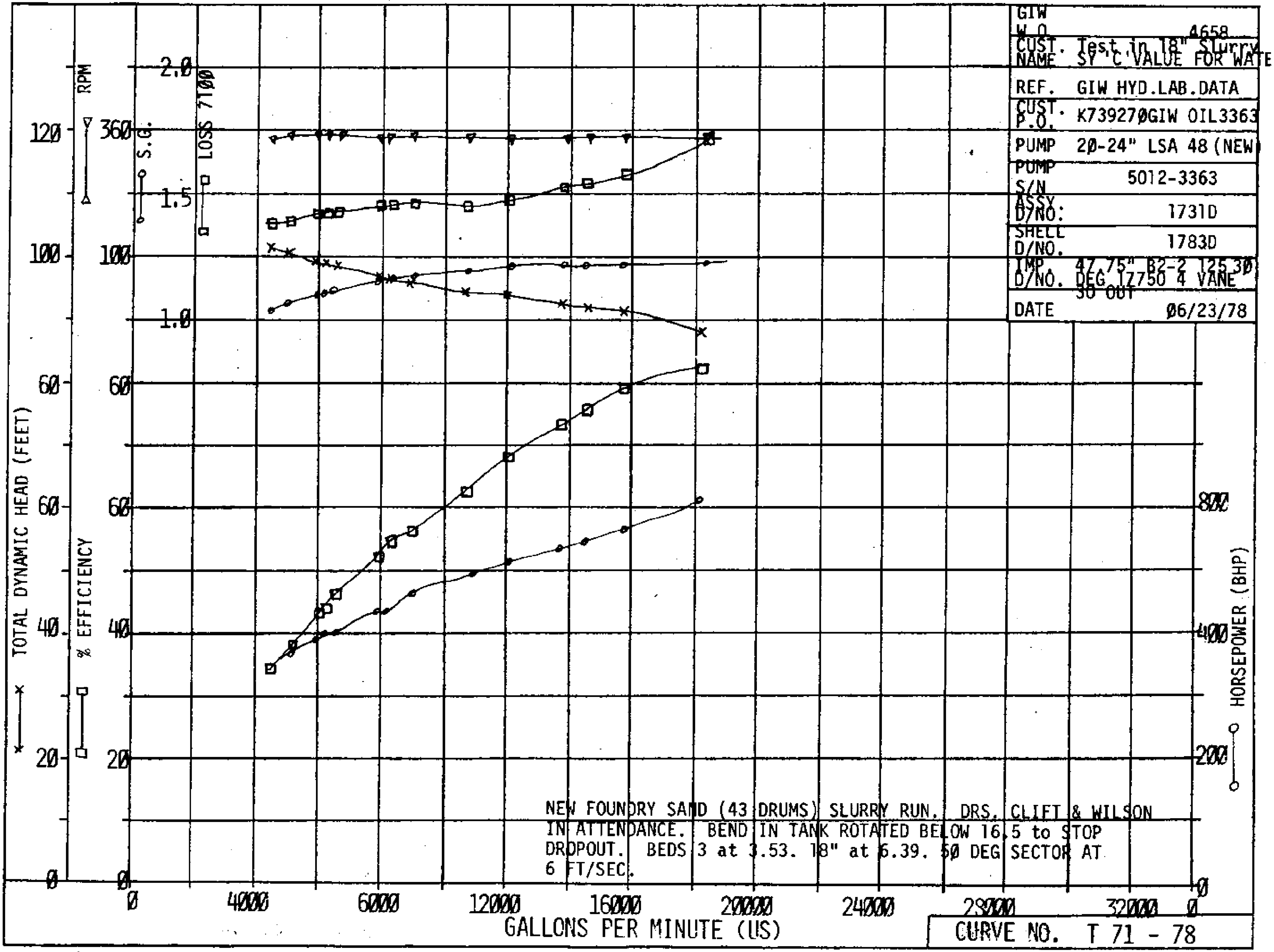
TEST RESULTS

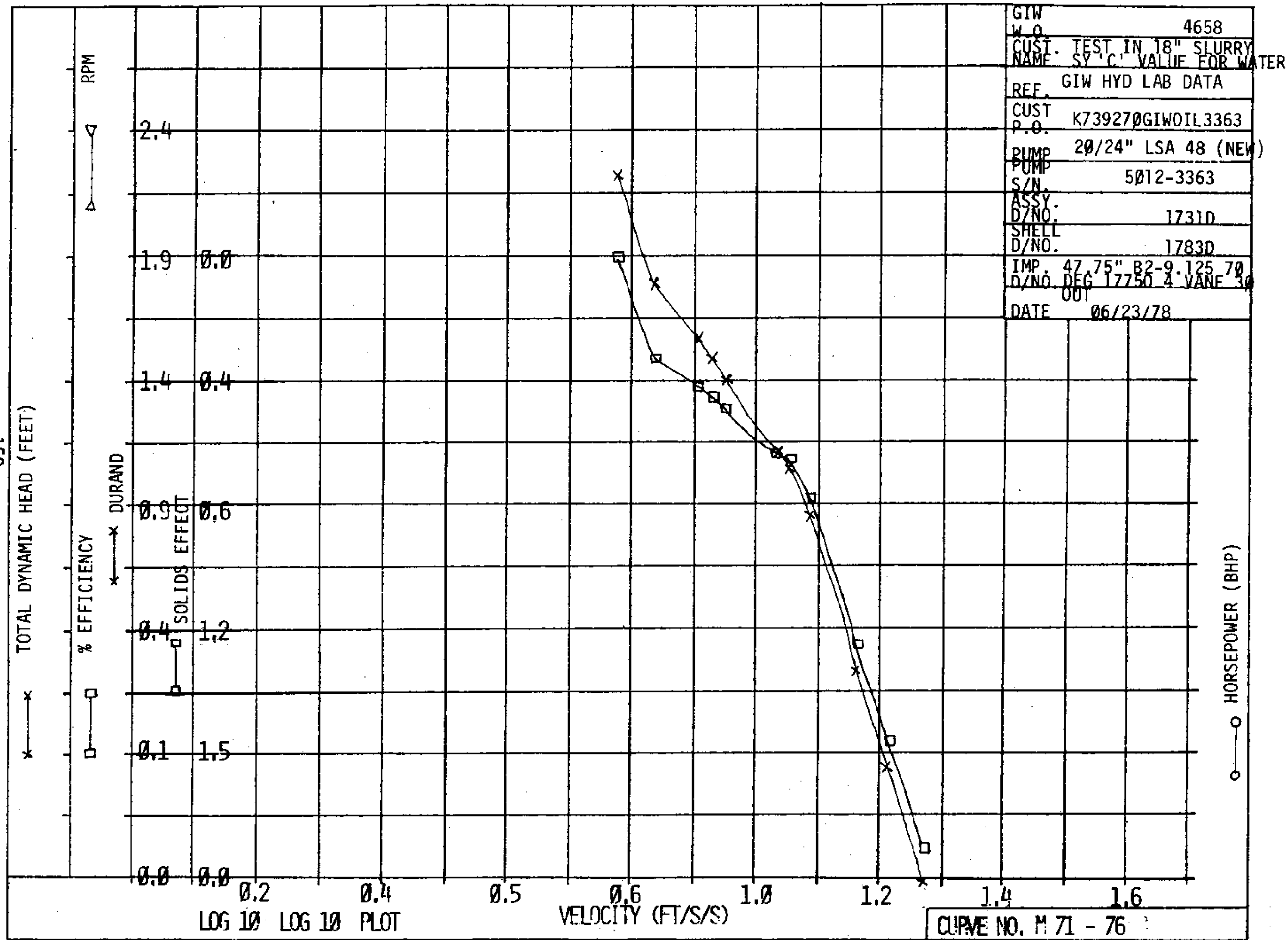
NO	VEL MAGFLOW	SG LINE TEMP	LINE DENS	S.G. (U-LEG)	VOLUME CONC.	SLURRY TEMP	REYNOLDS NUMBER	LOSS-H2O /100 FT	BNDRY DRAG	H2O DRAG (SAME R)	DURAND ET AL	SOLIDS EFFECT	HAZEN WLLMS	VEL RENO	VEL DOPPLR	TIME HH.MM
1	25.21	125.	.989	1.23	.146	95.	.469E+07	9.47	.0139	.0135	.2	.02	146.	24.0	.0	13.49
2	21.74	123.	.989	1.22	.138	97.	.415E+07	6.61	.0130	.0135	-.3	-.02	152.	20.2	.0	13.57
3	20.01	117.	.991	1.22	.136	100.	.394E+07	5.89	.0137	.0136	.1	.00	149.	18.9	.0	14.04
4	18.88	120.	.990	1.21	.135	102.	.376E+07	5.37	.0140	.0136	.2	.01	147.	17.9	.0	14.08
5	16.50	121.	.990	1.21	.130	103.	.332E+07	4.36	.0149	.0137	.7	.03	143.	15.4	.0	14.13
6	14.74	125.	.989	1.19	.119	105.	.301E+07	3.88	.0167	.0138	1.8	.06	135.	13.9	.0	14.26
7	12.39	125.	.989	1.17	.110	105.	.255E+07	4.13	.0251	.0139	7.3	.17	109.	11.8	.0	14.33
8	10.83	123.	.989	1.15	.095	106.	.225E+07	3.95	.0314	.0140	13.1	.23	97.	10.2	.0	14.39
9	8.53	123.	.989	1.10	.066	104.	.177E+07	3.45	.0441	.0143	31.6	.36	80.	8.3	.0	14.43
10	8.15	123.	.989	1.09	.060	107.	.170E+07	3.32	.0467	.0143	37.6	.39	78.	8.2	.0	14.49
11	6.89	123.	.989	1.05	.039	106.	.142E+07	2.59	.0507	.0145	63.2	.47	74.	6.9	.0	14.52
12	6.00	123.	.987	1.02	.019	105.	.124E+07	2.46	.0638	.0147	178.2	1.02	65.	6.1	.0	14.57
13	9.00	125.	.987	1.10	.070	108.	.189E+07	3.44	.0396	.0142	25.7	.32	85.	8.6	.0	15.08
14	11.40	124.	.987	1.15	.095	108.	.241E+07	4.05	.0291	.0140	11.3	.22	100.	10.6	.0	15.17

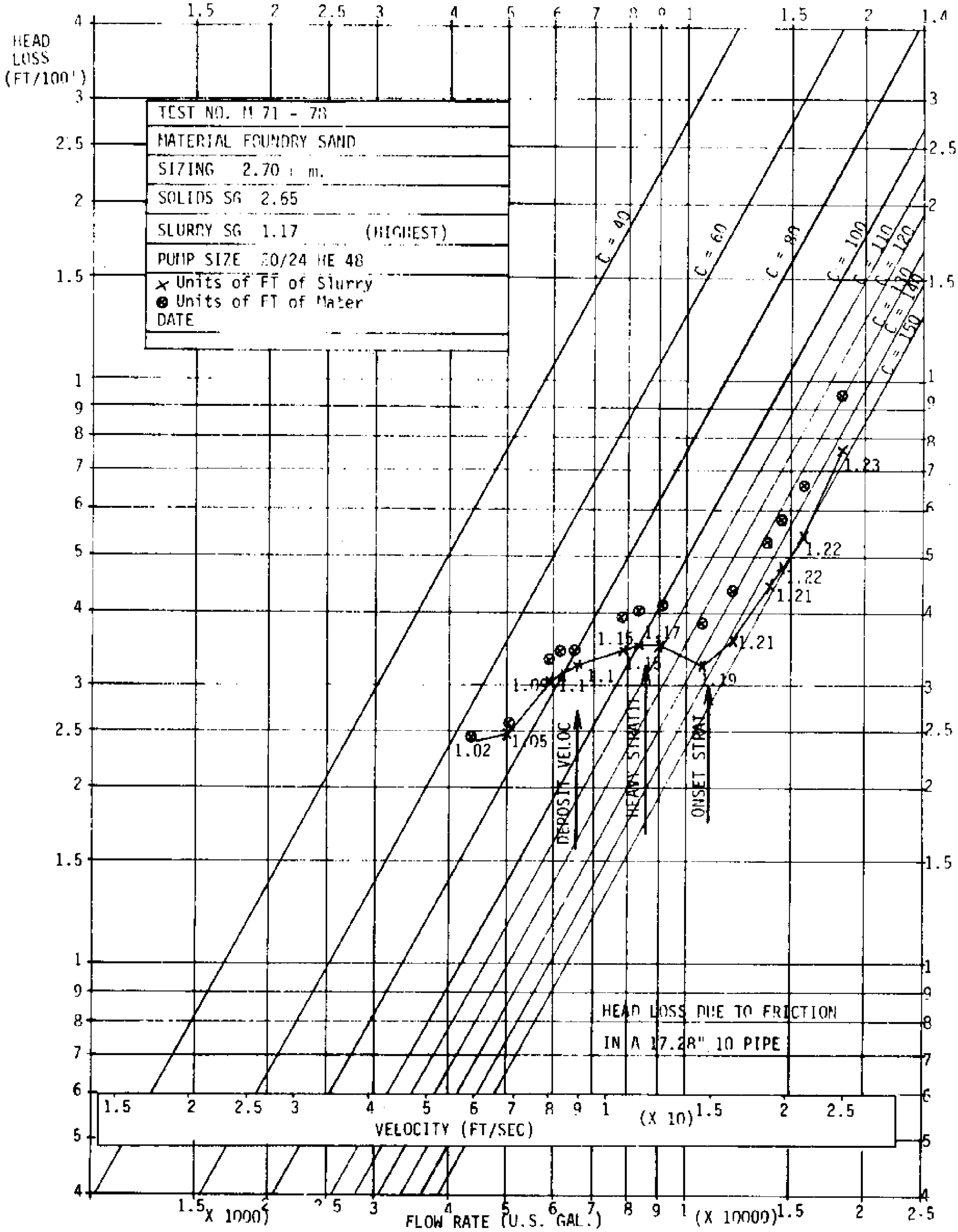
GEORGIA IRON WORKS CO.
TEST ENGINEER G.R.ADDIE DATE 06/23/78
WITNESSED BY FOR GIW

COMMENTS

NEW FOUNDRY SAND (43 DRUMS) SLURRY RUN. DRS CLIFT & WILSON IN ATTENDANCE. BEND IN TANK ROTATED BELOW 16.5 TO STOP DROPOUT. RODS: 3" AT 8.53, 10" AT 6.89, 50 DEG. SECTOR AT 6FT/SEC.


GIW W.O. 4658
CUST. NAME Test in 18" Slurry SY 'C' VALUE FOR WATE
REF. GIW HYD.LAB.DATA
CUST. P.O. K739270GIW OIL3363
PUMP 20-24" LSA 48 (NEW)
PUMP S/N 5012-3363
ASSY. D/NO: 1731D
SHELL D/NO. 1783D
IMP. D/NO. 47.75" B2-2 125.30 DEG 17750 4 VANE 30 OUT
DATE 06/23/78
NEW FOUNDRY SAND (43 DRUMS) SLURRY RUN. DRS. CLIFT & WILSON IN ATTENDANCE. BEND IN TANK ROTATED BELOW 16.5 to STOP DROPOUT. BEDS 3 at 3.53. 18" at 6.39. 50 DEG SECTOR AT 6 FT/SEC.
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
RPM
S.G.
LOSS 7100
HORSEPOWER (BHP)
GALLONS PER MINUTE (US)
CURVE NO. T 71 - 78


GIW W.O. 4658
CUST. NAME TEST IN 18" SLURRY SY 'C' VALUE FOR WATER
REF. GIW HYD LAB DATA
CUST P.O. K73927ØGIWOIL3363
PUMP 2Ø/24" LSA 48 (NEW)
PUMP S/N. 5Ø12-3363
ASSY. D/NO. 1731D
SHELL D/NO. 1783D
IMP. D/NO. 47.75" B2-9.125 7Ø DEG 17750 4 VANE 3Ø OUT
DATE Ø6/23/78
RPM
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
DURAND
SOLIDS EFFECT
HORSEPOWER (BHP)
2.4
1.9
1.4
Ø.9
Ø.4
Ø.1
Ø.Ø
Ø.Ø
Ø.4
Ø.6
1.2
1.5
Ø.Ø
Ø.2
Ø.4
Ø.5
Ø.6
1.Ø
1.2
1.4
1.6
LOG 1Ø LOG 1Ø PLOT
VELOCITY (FT/S/S)
CURVE NO. M 71 - 76


HEAD LOSS (FT/100')
TEST NO. M 71 - 78
MATERIAL FOUNDRY SAND
SIZING 2.70 mm.
SOLIDS SG 2.65
SLURRY SG 1.17 (HIGHEST)
PUMP SIZE 20/24 HE 48
Units of FT of Slurry
Units of FT of Water
DATE
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
DEPOSIT VELOC
HEAVY STRAT
ONSET STRAT
HEAD LOSS DUE TO FRICTION
IN A 17.28" ID PIPE
VELOCITY (FT/SEC)
(X 10)
FLOW RATE (U.S. GAL.)
X 1000
(X 10000)

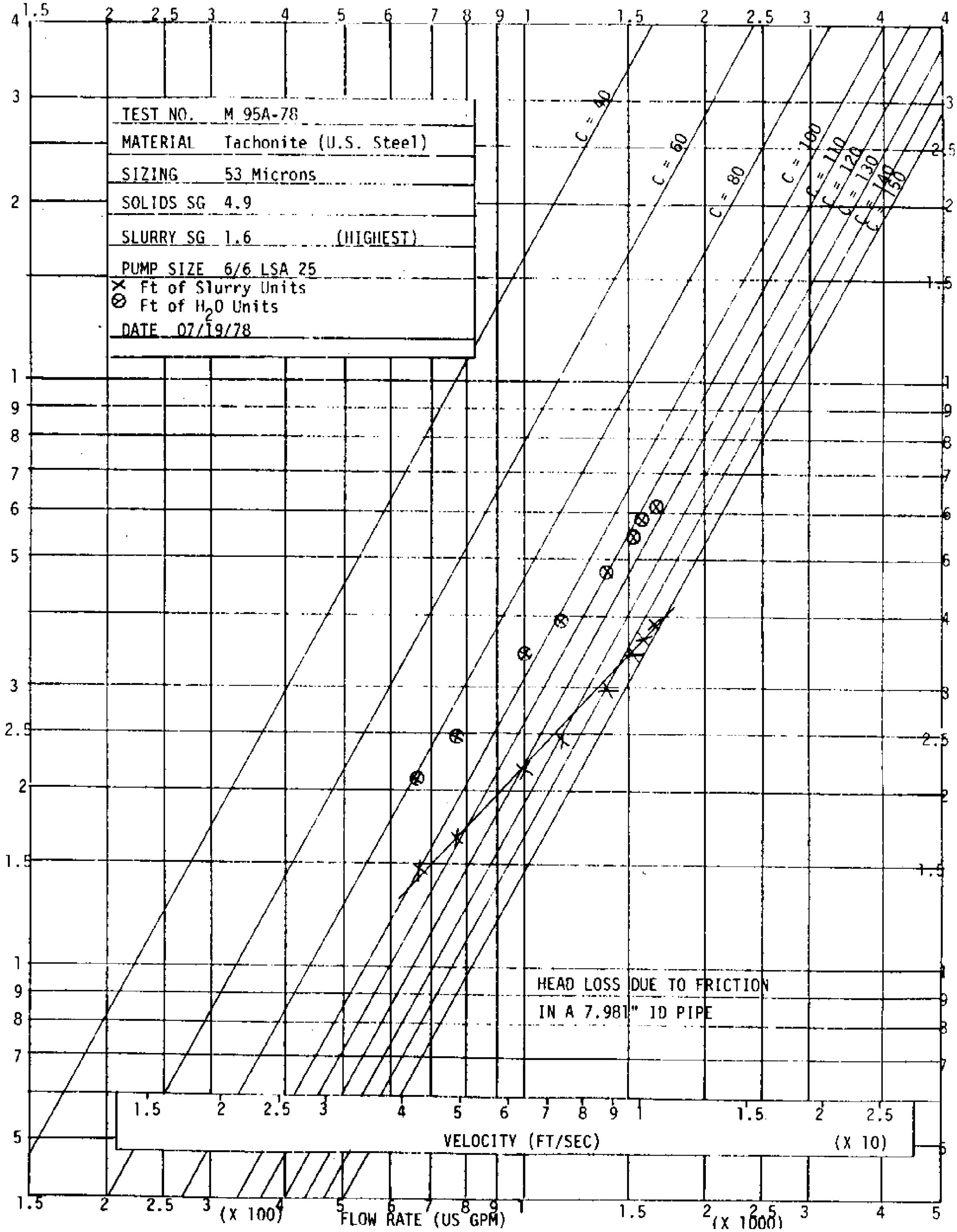
Test No.	Material	Sizing	Slurry Sg	Deposit Velocity	Pipe Size
M95A-78	Taconite Ore	55 Microns	1.6		8"
M51 -78	Fdy Sand Coarse Pebbles	3/8" Dia & 70 Microns	1.18	9.1 ft/sec	18"
M91 -79	Phosphate Ore	250 Microns	1.28	4.3 ft/sec	18"
M102-78	Foundry Sand	270 Microns	1.34	5 ft/sec	8"
M118-78	Phosphate Reject	560 Microns	1.18	8 ft/sec	8"
M44 -79	3/8" Dia. Pebble	6000 Microns	1.16	8 ft/sec (started duning)	8"

The non-settling slurries need to be categorized further in order to analyze these fully. In general, they are comprised of fine particle materials, and exhibit little or no deposition velocity. Their characteristics are usually related to their rheology. The viscosity of the slurry may vary during a test if the temperature varies. It is often necessary to control this. The transition from laminar to turbulent flow will normally be seen during a test.

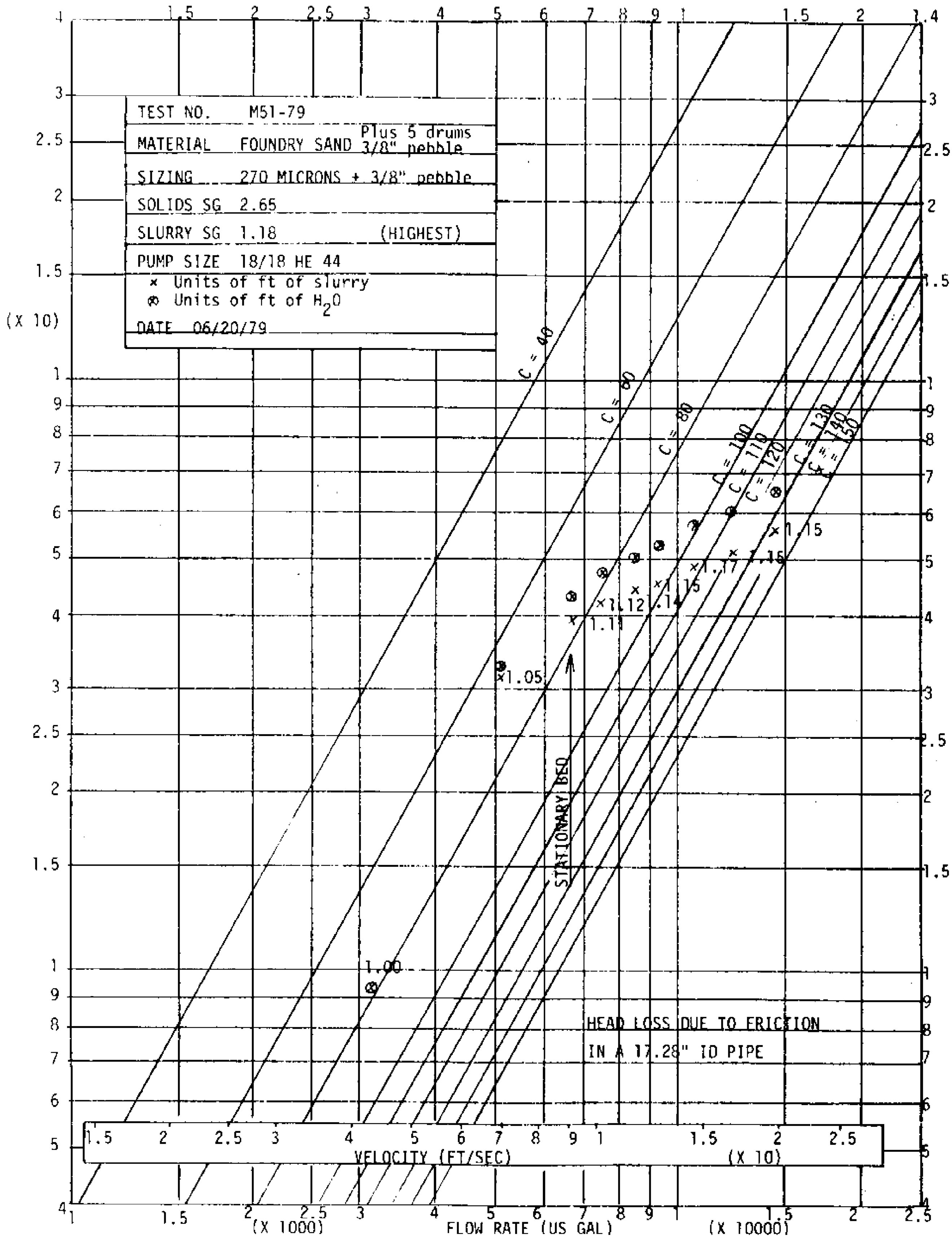
A few of the non-settling type slurry tests carried out in the hydraulic lab are as follows:

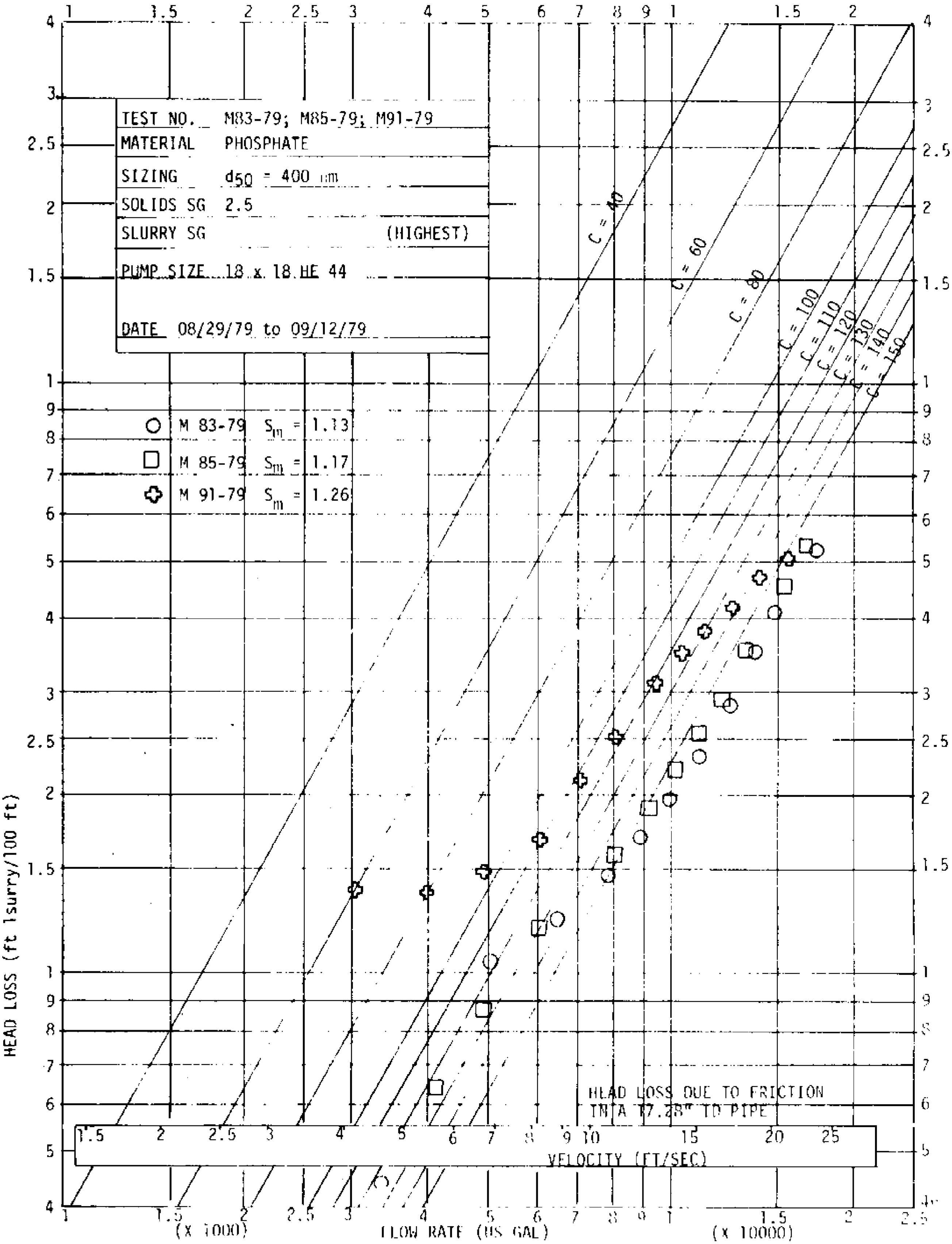
Test No.	Material	Sizing	Slurry Sg	Deposit Velocity	Pipe Size
M79 -78	Phosphate Slimes	0.5 Microns	1.01		18"
M108-78	Heavy Phosphate Slimes	0.5 Microns	1.49		8"
M52 -79	Kaolin	50 Microns	1.64		8"

The pipe friction characteristics of these vary but are typically as shown in the plots on the Kaolin Slurry Test M52-79 attached.



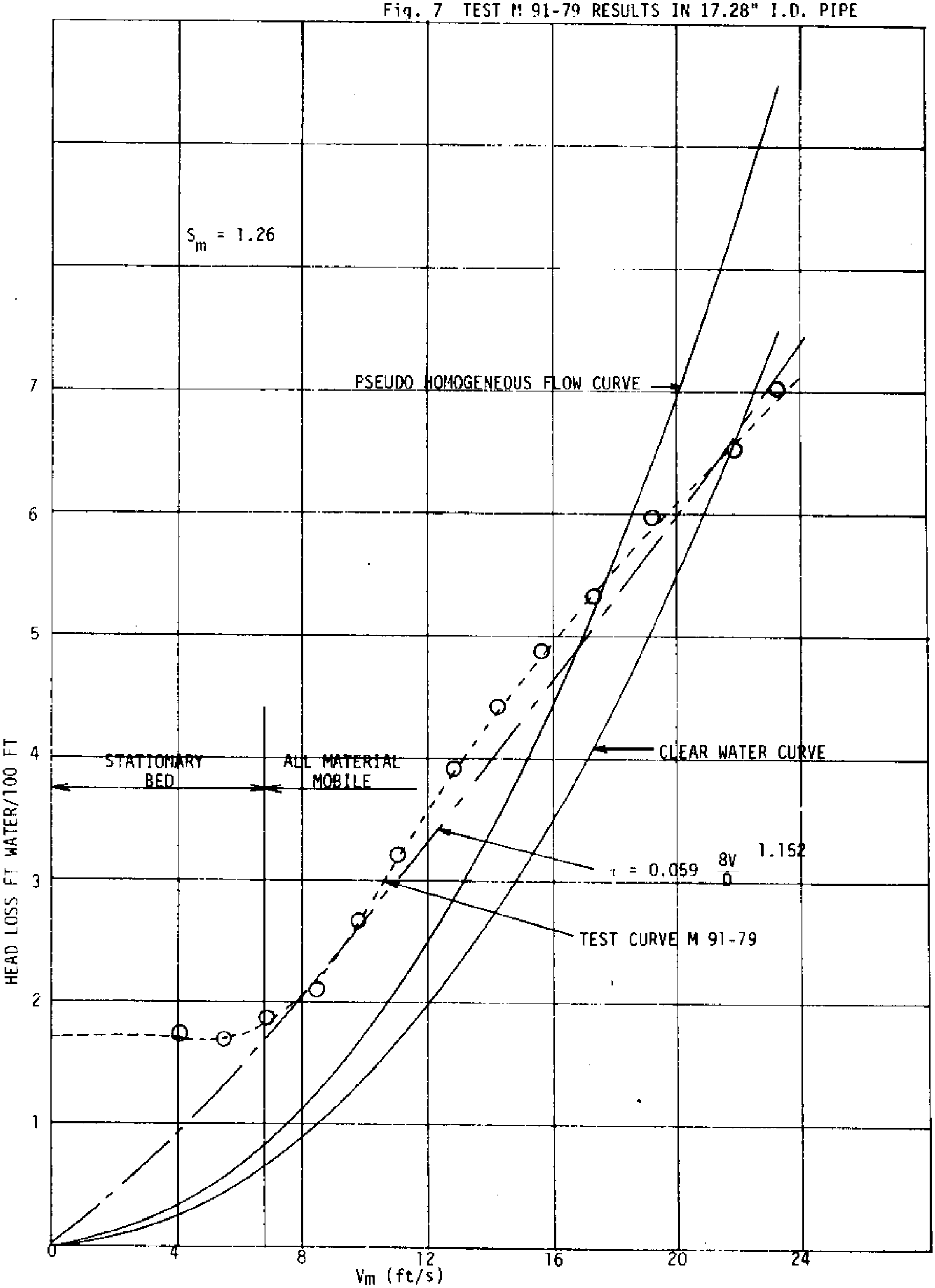
TEST NO. M 95A-78
MATERIAL Tachonite (U.S. Steel)
SIZING 53 Microns
SOLIDS SG 4.9
SLURRY SG 1.6 (HIGHEST)
PUMP SIZE 6/6 LSA 25
X Ft of Slurry Units
⊗ Ft of H_2O Units
DATE 07/19/78
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
HEAD LOSS DUE TO FRICTION
IN A 7.981" ID PIPE
VELOCITY (FT/SEC)
(X 10)
(X 100)
FLOW RATE (US GPM)
(X 1000)

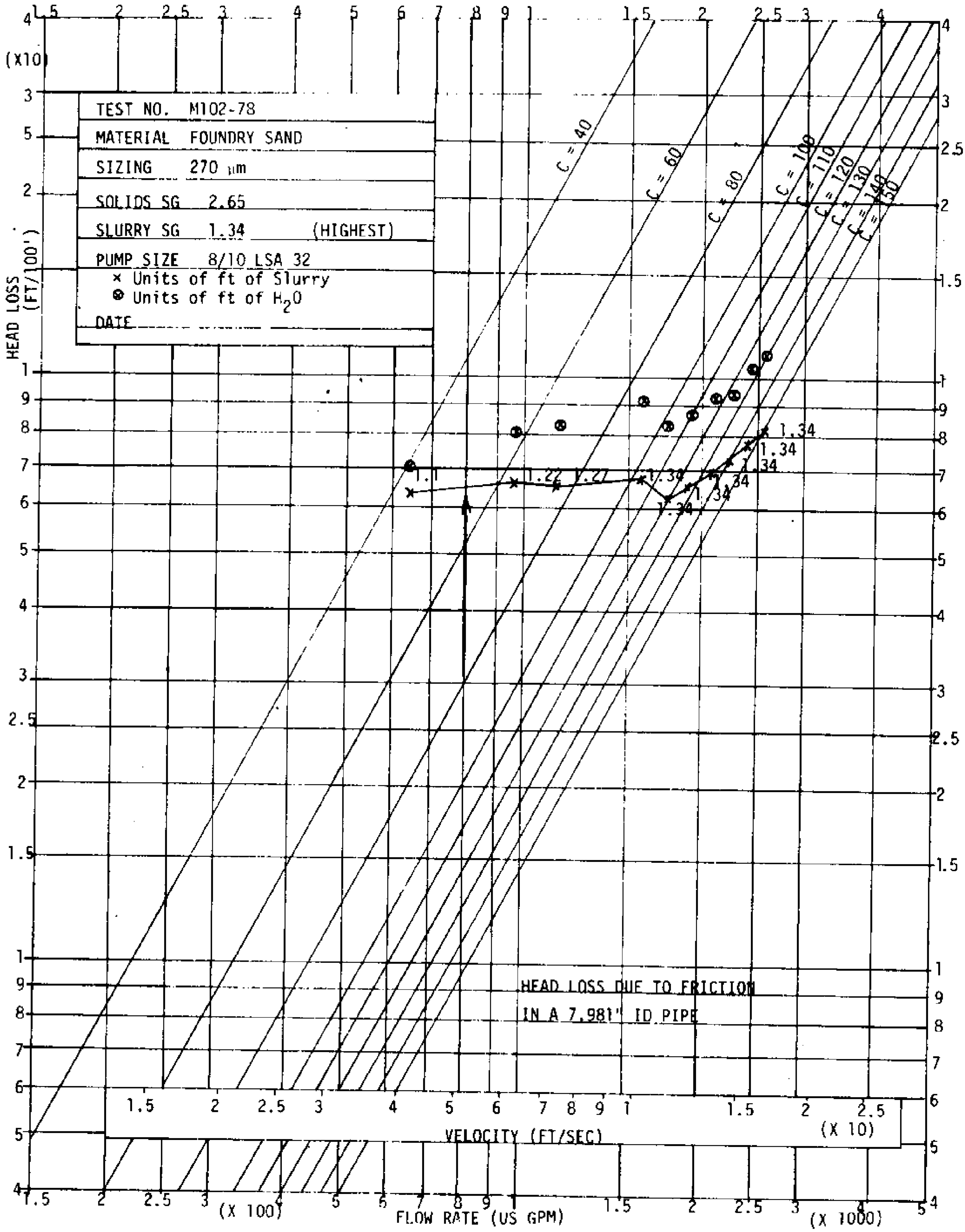

TEST NO. M51-79
MATERIAL FOUNDRY SAND Plus 5 drums 3/8" pebble
SIZING 270 MICRONS + 3/8" pebble
SOLIDS SG 2.65
SLURRY SG 1.18 (HIGHEST)
PUMP SIZE 18/18 HE 44
× Units of ft of slurry
⊗ Units of ft of H2O
DATE 06/20/79
(X 10)
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
1.00
1.05
1.11
1.12
1.14
1.15
1.17
1.16
STATIONARY BED
HEAD LOSS DUE TO FRICTION
IN A 17.28" ID PIPE
VELOCITY (FT/SEC)
(X 10)
(X 1000)
FLOW RATE (US GAL)
(X 10000)

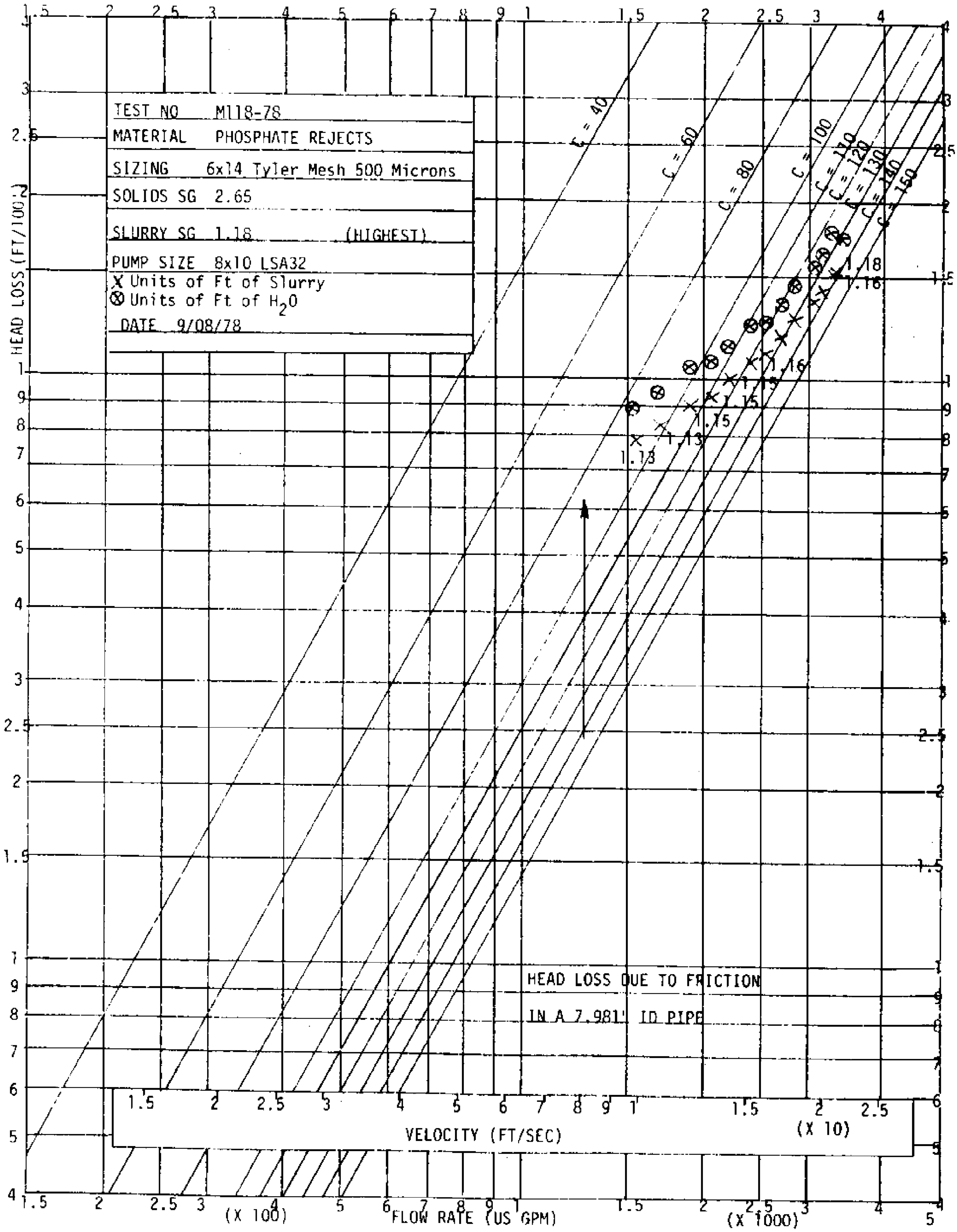


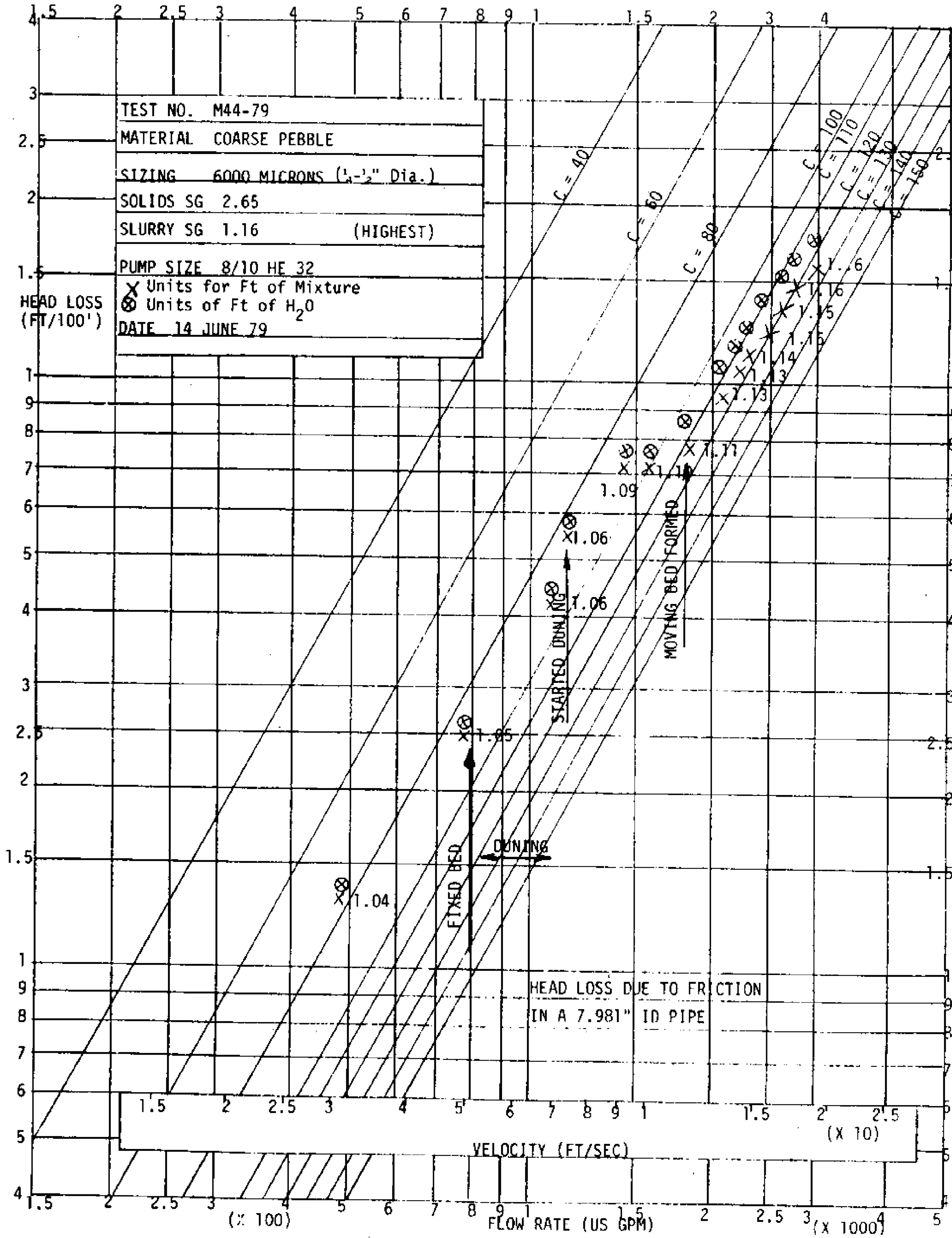
TEST NO. M83-79; M85-79; M91-79
MATERIAL PHOSPHATE
SIZING d_{50} = 400 μm
SOLIDS SG 2.5
SLURRY SG (HIGHEST)
PUMP SIZE 18 x 18 HE 44
DATE 08/29/79 to 09/12/79
M 83-79 S_m = 1.13
M 85-79 S_m = 1.17
M 91-79 S_m = 1.26
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
HEAD LOSS DUE TO FRICTION
IN A 17.28" ID PIPE
VELOCITY (FT/SEC)
HEAD LOSS (ft slurry/100 ft)
FLOW RATE (US GAL)
(x 1000)
(x 10000)

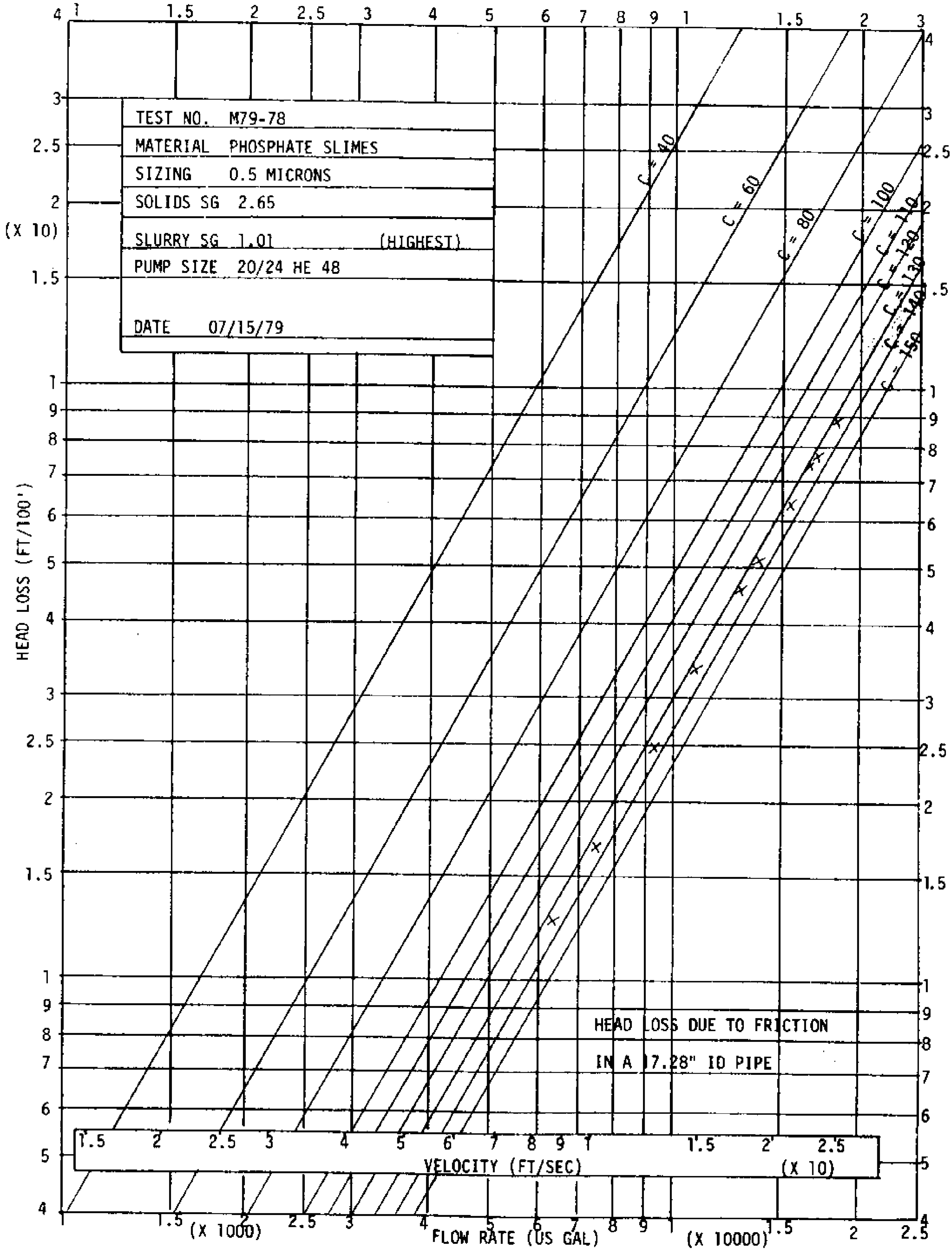
Fig. 7 TEST M 91-79 RESULTS IN 17.28" I.D. PIPE

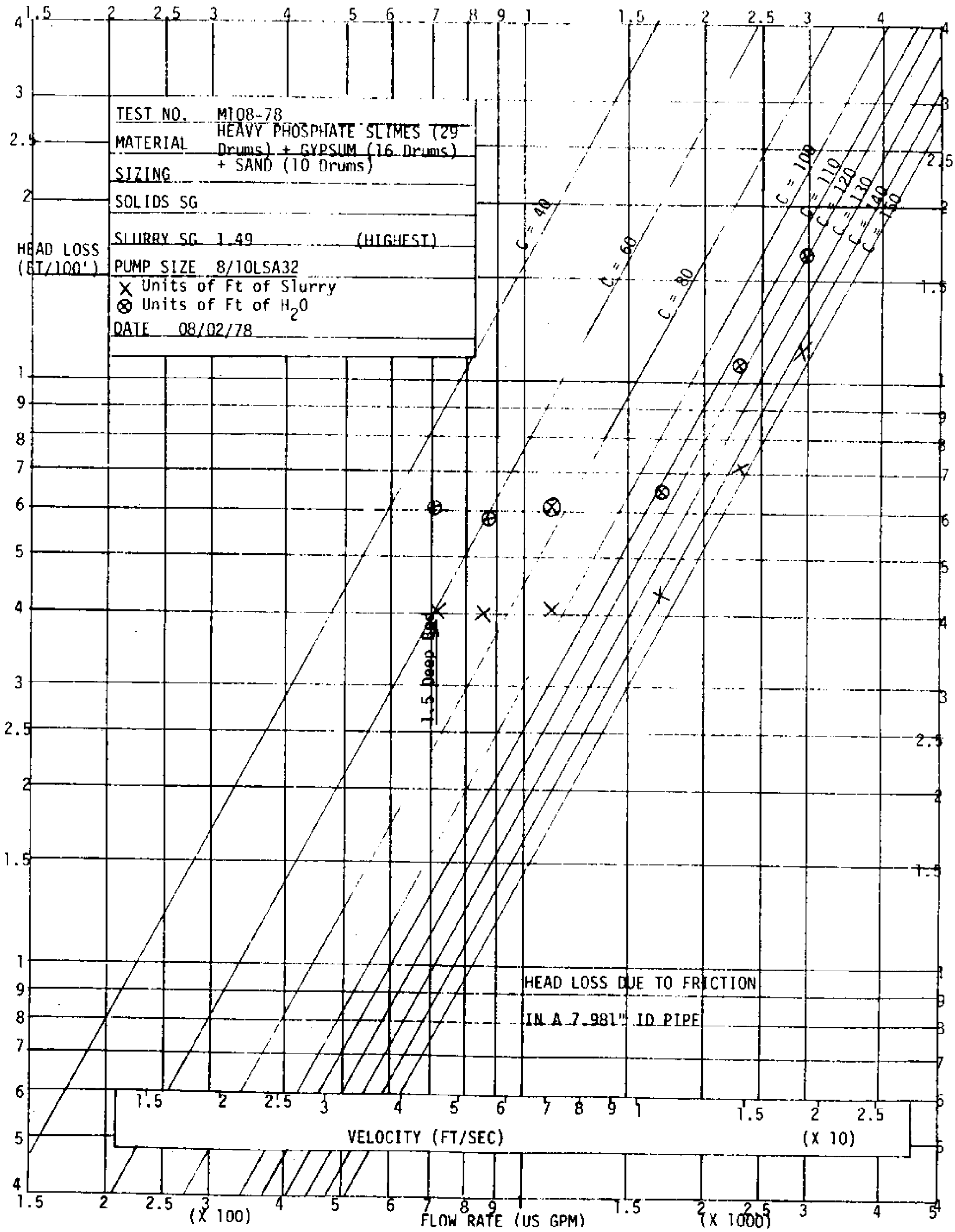



TEST NO. M102-78
MATERIAL FOUNDRY SAND
SIZING 270 μm
SOLIDS SG 2.65
SLURRY SG 1.34 (HIGHEST)
PUMP SIZE 8/10 LSA 32
× Units of ft of Slurry
⊗ Units of ft of H2O
DATE
HEAD LOSS (FT/100')
(X10)
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
HEAD LOSS DUE TO FRICTION
IN A 7.981" ID PIPE
VELOCITY (FT/SEC)
(X 10)
FLOW RATE (US GPM)
(X 100)
(X 1000)


TEST NO M118-78
MATERIAL PHOSPHATE REJECTS
SIZING 6x14 Tyler Mesh 500 Microns
SOLIDS SG 2.65
SLURRY SG 1.18 (HIGHEST)
PUMP SIZE 8x10 LSA32
X Units of Ft of Slurry
⊗ Units of Ft of H2O
DATE 9/08/78
HEAD LOSS (FT/100')
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
1.13
1.15
1.16
1.18
HEAD LOSS DUE TO FRICTION
IN A 7.981" ID PIPE
VELOCITY (FT/SEC)
(X 10)
FLOW RATE (US GPM)
(X 100)
(X 1000)


TEST NO. M44-79
MATERIAL COARSE PEBBLE
SIZING 6000 MICRONS (¼-½" Dia.)
SOLIDS SG 2.65
SLURRY SG 1.16 (HIGHEST)
PUMP SIZE 8/10 HE 32
X Units for Ft of Mixture
⊗ Units of Ft of H2O
DATE 14 JUNE 79
HEAD LOSS (FT/100')
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
FIXED BED
DUNING
STARTED DUNING
MOVING BED FORMED
HEAD LOSS DUE TO FRICTION
IN A 7.981" ID PIPE
VELOCITY (FT/SEC)
(X 10)
FLOW RATE (US GPM)
(X 100)
(X 1000)


TEST NO. M79-78
MATERIAL PHOSPHATE SLIMES
SIZING 0.5 MICRONS
SOLIDS SG 2.65
SLURRY SG 1.01 (HIGHEST)
PUMP SIZE 20/24 HE 48
DATE 07/15/79
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
HEAD LOSS (FT/100')
(X 10)
HEAD LOSS DUE TO FRICTION
IN A 17.28" ID PIPE
VELOCITY (FT/SEC)
(X 10)
FLOW RATE (US GAL)
(X 1000)
(X 10000)



TEST NO. M108-78
MATERIAL HEAVY PHOSPHATE SLIMES (29 Drums) + GYPSUM (16 Drums) + SAND (10 Drums)
SIZING
SOLIDS SG
SLURRY SG 1.49 (HIGHEST)
PUMP SIZE 8/10LSA32
X Units of Ft of Slurry
⊗ Units of Ft of H_2O
DATE 08/02/78
HEAD LOSS (FT/100')
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
1.5 Deep Bed
HEAD LOSS DUE TO FRICTION
IN A 7.981" ID PIPE
VELOCITY (FT/SEC)
(X 10)
FLOW RATE (US GPM)
(X 100)
(X 1000)

```
PUMP TEST DATA FOR     GIW SLURRY COURSE
------------------
PROJECT                NON-SETTLING KAOLIN
PUMP TYPE                 GIW 8'/10' HE 32
CLIENTS ORDER NO     8' DIA. SLURRY LOOP
ASSEMBLY DRAWNG NO                   1851D
SHELL    DRAWNG NO                   1849D
IMPELLER DRAWNG NO 1396C  30 DEG OUTLET
IMPELLER DIAM     32"  B2=2"  34 DEG OUT

PUMP ROTATION
HYDROSTATIC PSI =               WITNESSED

TEST CONSTANTS
--------------
  .00 INS DIA          IN    .00 INS PIPE
  1 FT H2O =      .0 US GPM USING
BEND HT CORR =   .5 FT  CONST = 1045.0
DISCHARGE PIPE DIAMETER =  7.98 INS
METER  .00' ABOVE PUMP DATUM, TAP 1.50'
SUCTION   PIPE DIAMETER = 10.02 INS
METER  .00' ABOVE PUMP DATUM, TAP  .00'
STATIC WATER LEVEL    .00 FT ABOVE P/D
WATTMETER MULT =  80. BAROMETER = 29.6"
HEAD LOSS = 50.00 FT OF  7.98 INCH DIAM
S.G. TAPS  8.00' AFART G = 32.14 FT/S/S
SOLIDS SG 2.65 OF  50. MICRONS S.D.=1.0
FOR VEL CALCS PIPE DIAMETER =    .00 INS
SAMPLER: HT CORR = .00' AREA = 6.15 SQ'
BALANCE VOLUME CONSTANT =    .00 CUBIC '
```

```
CH  USE   RDG  SOURCE INSTRUMENT
--  ---   ---  ------ ----------
 1        DISC DIFF HD ROSEMOUNT 0-240FT H2O 02269B1E1  1.000
 2        WATT METER   LANDIS AND GYR              1E3  1.000
 3        FLOW 8' MAG  FISCHER & PORTER 8'GPM01089A     1.000
 4        SUCT TRANSDR.FOXBRO E13DM 0-34'H2O 12088B1E2  1.000
 5        DISC TRANSDR.FOXBRO E11GM 1000'H2O 06209A1E1  1.000
 6  LOW   FLOW BEND   .FOXBRO E13DM 0-20 H2O 12088B1E2  1.000
 7  HIGH  FLOW BEND   .FOXBRO E13DM 0-74'H2O 12088B1E2  1.000
 8        TORQ TRQ LIL LEBOW/DAYTRONIC LITTLE10268H     1.000
 9        RPM  TRQ BAR LEBOW/DAYTRONIC RPM    102688    1.000
10        BHP  LIL BAR.LEBOW/DAYTRONIC HP 8' 040877     1.000
11  HIGH  LOSS 50 FT   ROSEMOUNT 0-18 FT H2O 12088B1E2  1.000
12  AVE   S.G. U-SECDN ROSEMOUNT                         .500
13  LOW   LOSS 50 FT   ROSEMOUNT 0- 9 FT H2O 12088B1E3  1.000
14        LOSS INCLINE.ROSEMOUNT 0- 2 FT H2O 12088B1E3  1.000
15  AVE   S.G. U-SECUP ROSEMOUNT 0- 5 FT H2O 12088B1E3   .500
16  NULL  FLOW 18' MAG.FISCHER & PORTER18'GPM03279B1E-1  .000
17        TEMP 18' DIS.PADGETT 18'DISCH  (F) 06178C1E1  1.000
18        TEMP 8' DIS  PADGETT 8' DISCH  (F) 06178C1E1  1.000
19        SGLT SG.U #1.PADGETT U-LOOP#1  (F) 06178C1E1  1.000
20        SGLT SG.D #4 PADGETT U-LOOP#4  (F) 06178C1E1  1.000
21  SUM   S.G. SAMPLEW.WEIGHT                  LBS       .000
22  NULL  SLVL SAMPLEL                                   .000
23        STIM SAMPLET TIME               SEC           1.000
24        FLOW  8' DOP.LEEDS AND NORTHRUP 8' 07248      1.000
25        FLOW  8' DOP.L & N HYDRA        8' 07248      1.000
26  NULL  FLOW BEND   .FOXBRO E13DM 0-20 H2O 12088B1E2   .000
27  NULL  TORQ TRQ BIG.LEBOW/DAYTRONIC BIG   10268E1E-1  .000
28  NULL  BHP  BIG BAR.LEBOW/DAYTRONIC HP 18'040877      .000
29        BHP  TRQ*RPM                                  1.000
30        BHP  CALC'ED.FROM 200 HP MOTOR KILOWATTS      1.000
31        SUCT TRANSDR.ROSEMOUNT-30-30FT H20-06209A1E2  1.000
32  NULL  RPM  LEUZE  .                                  .000
```

```
GEORGIA IRON WORKS CO.
P.O. BOX 626
GROVETOWN,
GEORGIA                 30813
-----------------------

WORK ORDER NO   GIW COURSE
SERIAL NO        5012 3514
TEST CURVE NO     M 52 -79
TEST DATE         06/21/79

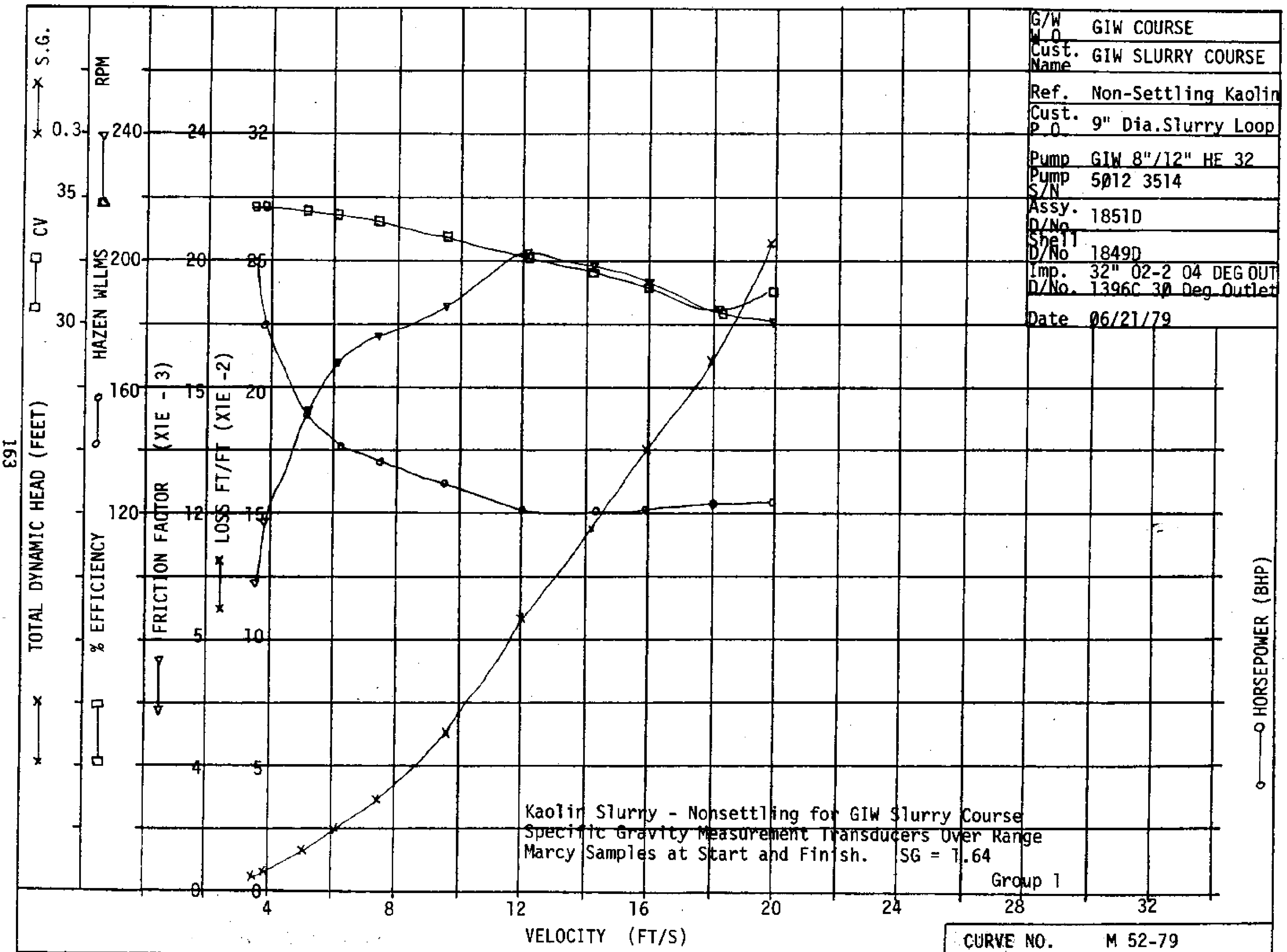
DRIVER DETAIL
-------------
TYPE                LAB MOTOR
MAKE             GENERAL ELEC
SERIAL NO              250662
FRAME SIZE      16  AS-13617
RPM =  600       BHP =  200.
 480 VOLTS 3 PHASE 60 CPS

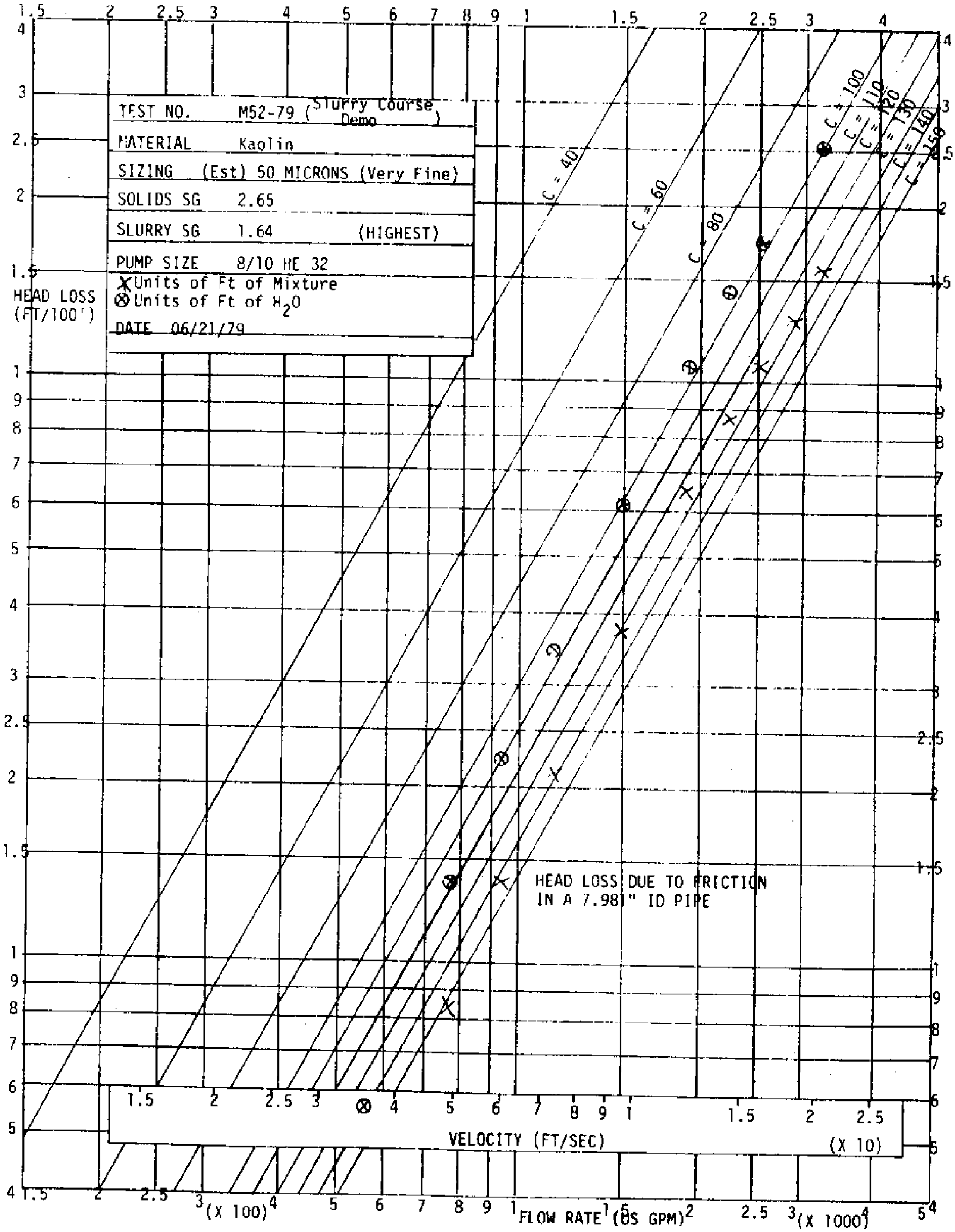
SCALED PERFORMANCE FACTRS
-------------------------
SPEED OR RATIO             .00
IMP TURN DOWN RATIO        .00
MERIDNL WIDTH RATIO        .00
SCALE RATIO                .00
BEP REF          .MG,     .RPM
EFFICIENCY       .0%  BY   .00
```

TEST RESULTS

NO	VELOCITY Vm FT/S	FLOW Qm GAL/MIN	TEMP Tm F	S.G. Sw	S.G. Sm	VOLUME CONC. Cv %	WEIGHT CONC. Cw %	MASS Ms TON/HR	REYNOLDS NUMBER Re	PIPELINE LOSSES Im FT/FT	PIPELINE LOSSES Iw FT/FT	FRICTION FACTRS Fm	FRICTION FACTRS Fw SAME Re	HAZEN WLLMS C	Im-Iw ----- Sm-Sw	TIME t HH.MM
1	19.95	3110.2	95.6	.995	1.53	32.2	55.8	664.6	.172E+07	.2573	.1224	.0181	.0132	123.	.2532	13.53
2	17.99	2804.0	96.6	.995	1.50	30.8	54.2	573.3	.157E+07	.2102	.1000	.0185	.0132	123.	.2161	13.57
3	15.89	2476.8	96.8	.995	1.53	32.3	55.9	530.6	.139E+07	.1749	.0787	.0194	.0133	121.	.1802	13.58
4	14.06	2192.5	97.2	.995	1.55	33.3	57.1	484.4	.123E+07	.1419	.0621	.0198	.0134	120.	.1448	14.00
5	12.01	1871.5	97.5	.995	1.56	34.3	58.2	426.4	.106E+07	.1069	.0458	.0203	.0136	120.	.1075	14.01
6	9.44	1472.1	97.6	.995	1.58	35.4	59.4	346.2	.835E+06	.0612	.0290	.0186	.0139	129.	.0550	14.03
7	7.24	1129.4	98.1	.995	1.60	36.5	60.5	273.4	.643E+06	.0345	.0175	.0176	.0143	135.	.0282	14.06
8	5.96	928.9	98.4	.995	1.60	36.9	60.9	227.2	.531E+06	.0224	.0121	.0168	.0146	141.	.0170	14.07
9	4.91	765.8	98.6	.995	1.61	37.1	61.2	188.8	.438E+06	.0138	.0084	.0152	.0149	151.	.0087	14.08
10	3.62	563.9	98.6	.995	1.61	37.4	61.4	140.0	.323E+06	.0057	.0048	.0115	.0156	180.	.0015	14.08
11	3.32	518.1	98.8	.995	1.62	37.5	61.5	128.9	.297E+06	.0040	.0041	.0096	.0157	201.	-.0001	14.09

```
GEORGIA IRON WORKS CO.                            COMMENTS
TEST ENGINEER J.R. MAFFETT  DATE 06/21/79          KAOLIN SLURRY - NONSETTLING FOR GIW SLURRY COURSE
                                                   SPECIFIC GRAVITY MEASUREMENT TRANSDUCERS OVER-RANGE
WITNESSED BY  GROUP 1     FOR  GIW SLURRY COURSE   MARCY SAMPLES AT START AND FINISH, SG=1.64       GROUP 1
```


G/W W.O. GIW COURSE
Cust. Name GIW SLURRY COURSE
Ref. Non-Settling Kaolin
Cust. P.O. 9" Dia.Slurry Loop
Pump GIW 8"/12" HE 32
Pump S/N 5Ø12 3514
Assy. D/No. 1851D
Shell D/No 1849D
Imp. D/No. 32" 02-2 04 DEG OUT 1396C 3Ø Deg Outlet
Date Ø6/21/79
TOTAL DYNAMIC HEAD (FEET)
CV
S.G.
% EFFICIENCY
HAZEN WLLMS
RPM
FRICTION FACTOR (X1E - 3)
LOSS FT/FT (X1E -2)
HORSEPOWER (BHP)
Kaolin Slurry - Nonsettling for GIW Slurry Course
Specific Gravity Measurement Transducers Over Range
Marcy Samples at Start and Finish. SG = 1.64
Group 1
VELOCITY (FT/S)
CURVE NO. M 52-79


TEST NO. M52-79 (Slurry Course Demo)
MATERIAL Kaolin
SIZING (Est) 50 MICRONS (Very Fine)
SOLIDS SG 2.65
SLURRY SG 1.64 (HIGHEST)
PUMP SIZE 8/10 HE 32
X Units of Ft of Mixture
⊗ Units of Ft of H_2O
DATE 06/21/79
HEAD LOSS (FT/100')
C = 40
C = 60
C = 80
C = 100
C = 110
C = 120
C = 130
C = 140
C = 150
HEAD LOSS DUE TO FRICTION IN A 7.981" ID PIPE
VELOCITY (FT/SEC)
(X 10)
FLOW RATE (US GPM)
(X 100)
(X 1000)

The above are just a few samples of the types of tests carried out in the GIW Hydraulic Lab.

Analysis of this data and its use in pipeline design is another subject about which we are just beginning to achieve a greater understanding.

3. <u>Centrifugal Slurry Pump Testing and Performance on Solids.</u>

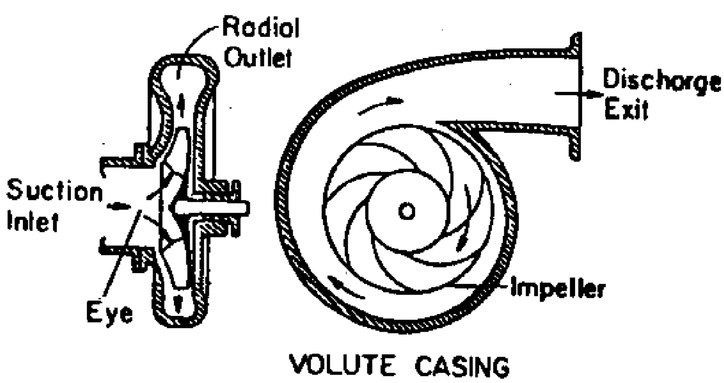
Pumping may be defined as the addition of energy to a fluid to move it from one point to another. It is not, as frequently thought, the addition of pressure. Because energy is capacity to do work, adding it to a fluid causes the fluid to do work, normally flowing through a pipe or rising to a higher level.

A centrifugal pump is a machine consisting of a set of rotating vanes enclosed within a housing or casing. The vanes impart energy to a fluid through centrifugal force. Thus, stripped of all refinements, a centrifugal pump has two main parts: (1) a rotating element, including an impeller and a shaft, and (2) a stationary element made up of a casing, stuffing box, and bearings.

To effect a flow the liquid is forced either by atmospheric or other pressure into a set of rotating vanes. These vanes constitute an impeller that discharges the liquid at a higher velocity at its periphery. The velocity is then converted into pressure energy by means of a volute or by a set of stationary diffusion vanes surrounding the impeller periphery. Pumps with volute casings are generally called volute pumps; whereas, those with diffusion vanes are called diffuser pumps.

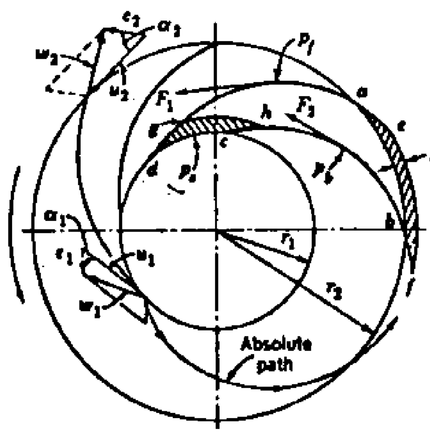
Centrifugal pumps may comprise of a number of stages of impeller and collector and may exist in a wide variety of arrangements to suit different applications.

Slurry pumps are normally single stage end suction type. They are usually radial or mixed flow type. They commonly have volute type collectors, but these are often modified to concentric or semi-concentric to reduce the effect of wear on the shell.

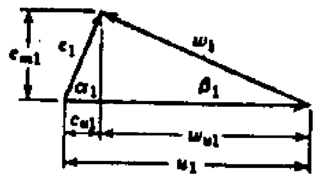


VOLUTE CASING

An expression for the theoretical head of a centrifugal pump is obtained by applying the principle of angular momentum to the mass of liquid going through the impeller passages. This principle states that the time rate of change of angular momentum of a body with respect to the speed of rotation is equal to the torque of the resultant force on the body with respect to the same axis.

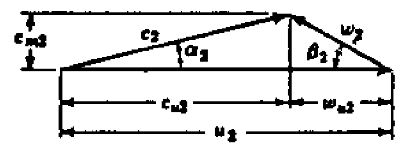


Forces and velocities in an impeller.



(a)

(a) Entrance velocity triangle;



(b)

(b) discharge velocity triangle.

Assuming there is no loss of head between the impeller and the point where the total dynamic head is measured, this power is available as the pump output of an idealized pump.

$$T\omega = \frac{Q\gamma}{g}\,\omega(r_2c_2\cos\alpha_2 - r_1c_1\cos\alpha_1)$$

U_2 = tangent velocity

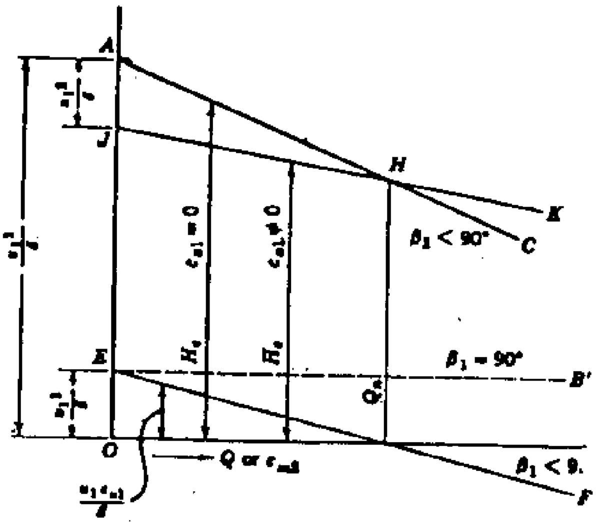
C_{u2} = Absolute velocity

Eliminating Q, we get an expression for head.

$$H_i = \frac{u_2\,c_{u2} - u_1c_{u1}}{g}$$

Since all hydraulic losses between the points where the actual total dynamic head of a pump is measured have been disregarded, the head H_i is a theoretical head; the equation is known as Euler's equation.

The head-capacity curve of an ideal pump is a straight line. For a given discharge vane angle, a single line will represent the characteristics of pump of all specific speeds when plotted to dimensionless scales.

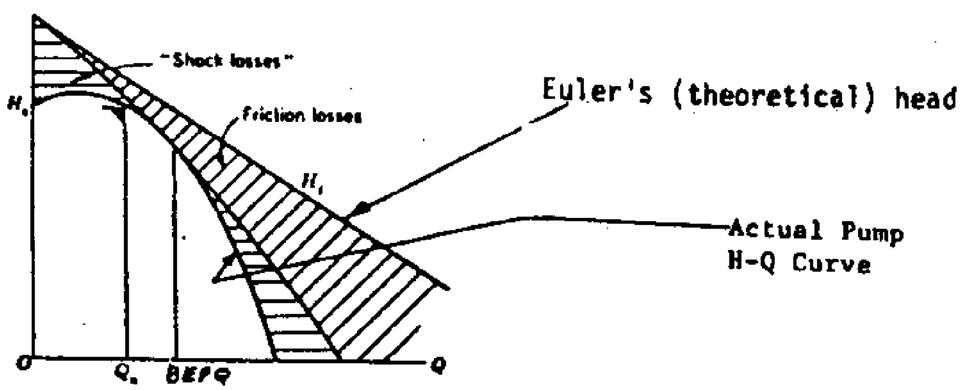

Euler's head-capacity characteristics.

Friction and other losses occur that reduce the theoretical head produced by the impeller.

The collector or shell of a pump has the task of converting the velocity energy of the water at the periphery of the collector back into pressure energy.

In an idealized pump we assume there are no losses in the casing. In practice, losses due to friction and turbulence occur here also.

Hydraulic losses for the selected proportions of the essential passages designed will determine the head-capacity curve of the actual pump. In general, for a constant speed, the head-capacity curve can be obtained by subtracting losses from the input head of an idealized pump noting that the inlet angle shock losses will be zero only for one flow.



· Q-H curve is obtained by subtraction of hydraulic losses from input head.

In order to evaluate pump performance on slurries, it is necessary to carry out water performance tests on the pumps and then repeat these shortly thereafter on the slurry being used in the comparison.

Pump performance can vary with inferior piping and instrumentation arrangements so performance testing needs to be carried out to standards if it is to be an accurate representation of that pump's performance.

All the GIW test results that are used for comparison purposes are carried out in accordance with the Hydraulic Institute Standard under laboratory conditions.

Tests are carried out in a pipework loop system that includes a large tank that will get rid of air and help keep the liquid cool. Long straight lengths are provided before and after the pump and any instruments to minimize any turbulence effects.

A full characteristic is necessary so a test is usually carried out at constant speed recording performance of head and power at a dozen or so different flows out to about 120% of the pumps best efficiency flow.

Variation in system resistance is achieved by throttling with a butterfly valve near the discharge of the pump and/or a pinch valve downstream of the head loss measurement section.

Pump tests carried out in the GIW Lab normally use one of the three magnetic flow meters as the prime flow measuring device and a bend meter and a doppler meter for backup.

A full pipe cross section sampler device is available to check the delivered flow rate. Flow rate is taken from a level change (volume) in a sample tank and a time. A load cell is set up so the specific gravity can be derived also from the volume and mass of the sample in the tank.

Discharge and suction pressures are taken from a differential pressure transducer across the pump and separate suction and discharge pressure transducers as backup instruments.

All pressure transducers are set up with their own sediment traps and clear water purge systems using clear nylon pressure tubing.

Power is taken from one of the three precision type LeBow torque bars and the backup is provided by measuring the power into the motor and using the motor efficiency.

Specific gravity is measured using an inverted "U" tube and the differential pressure across an upstream and a downstream leg. The reading obtained here is backed up by the sampler device noted earlier.

PUMP TEST DATA FOR GIW SLURRY COURSE

PROJECT 18' SAND SLURRY TEST
PUMP TYPE GIW 18"/18" HE 44
CLIENTS ORDER NO 18' DIA. SLURRY LOOP
ASSEMBLY DRAWNG NO 1839D
SHELL DRAWNG NO 1877D
IMPELLER DRAWNG NO 1847D
IMPELLER DIAM. 44' 85=5.875' 34 DEG OUT

PUMP ROTATION
HYDROSTATIC PSI =450 WITNESSED

TEST CONSTANTS

.00 INS DIA IN .00 INS PIPE
1 FT H2O = .0 US GPM USING
BEND HT CORR =-1.0 FT CONST = 4959.0
DISCHARGE PIPE DIAMETER = 17.28 INS
METER .00' ABOVE PUMP DATUM, TAP 2.30'.
SUCTION PIPE DIAMETER = 17.28 INS
METER .00' ABOVE PUMP DATUM, TAP .00'
STATIC WATER LEVEL .00 FT ABOVE P/D
WATTMETER MULT = 800. BAROMETER = 29.6"
HEAD LOSS = 77.50 FT OF 17.28 INCH DIAM
S.G. TAPS 12.00' APART G = 32.14 FT/S/S
SOLIDS SG 2.65 OF 270. MICRONS S.D.=1.6
FOR VEL CALCS PIPE DIAMETER = .00 INS
SAMPLER: HT CORR = .00' AREA = .00 SQ'
BALANCE VOLUME CONSTANT = .00 CUBIC '

CH	USE	RDG	SOURCE	INSTRUMENT	
1		DISC	DIFF HD	ROSEMOUNT 0-240FT H2O 02269B1E1	1.000
2		WATT	METER	LANDIS AND GYR 1E3	1.000
3	NULL	FLOW	8' MAG	FISCHER & PORTER 8'GPM01089A	.000
4		SUCT	TRANSDR	FOXBRO E13DM 0-34'H2O 12088B1E2	1.000
5		DISC	TRANSDR	FOXBRO E11GM 1000'H2O 06209A1E1	1.000
6	LOW	FLOW	BEND	FOXBRO E13DM 0-20 H2O 12088B1E2	1.000
7	HIGH	FLOW	BEND	FOXBRO E13DM 0-74'H2O 12088B1E2	1.000
8	NULL	TORQ	TRQ LIL	LEBOW/DAYTRONIC LITTLE10268M	.000
9		RPM	TRQ BAR	LEBOW/DAYTRONIC RPM 10268B	1.000
10	NULL	BHP	LIL BAR	LEBOW/DAYTRONIC HP 8" 040877	.000
11	HIGH	LOSS	77.5 FT	ROSEMOUNT 0-18 FT H2O 12088B1E2	1.000
12	AVE	S.G.	U-SECDN	ROSEMOUNT-1-11 FT H2O-12088B1E2	.500
13	LOW	LOSS	77.5 FT	ROSEMOUNT 0- 9 FT H2O 12088B1E3	1.000
14		LOSS	INCLINE	ROSEMOUNT 0- 2 FT H2O 12088B1E3	1.000
15	AVE	S.G.	U-SECUP	ROSEMOUNT 0- 5 FT H2O 12088B1E3	.500
16		FLOW	18' MAG	FISCHER & PORTER18'GPM0327981E-1	1.000
17		TEMP	18' DIS	PADGETT 18'DISCH (F) 06178C1E1	1.000
18		TEMP	8' DIS.	PADGETT 8' DISCH (F) 06178C1E1	1.000
19		SGLT	SG.U #1	PADGETT U-LOOP#1 (F) 06178C1E1	1.000
20		SGLT	SG.D #4	PADGETT U-LOOP#4 (F) 06178C1E1	1.000
21	NULL	S.G.	SAMPLER.		.000
22	NULL	SLVL	SAMPLER.		.000
23	NULL	STIM	SAMPLER.		.000
24	NULL	FLOW	8' DOP	LEEDS AND NORTHRUP 8" 07248	.000
25		FLOW	18' DOP.	LEEDS AND NORTHRUP 18"07248 1E-1	1.000
26	NULL	FLOW	BEND	FOXBRO E13DM 0-20 H2O 12088B1E2	.000
27		TORQ	TRQ BIG	LEBOW/DAYTRONIC BIG 10268E1E-1	1.000
28		BHP	BIG BAR.	LEBOW/DAYTRONIC HP 18"040877	1.000
29		BHP	TRQ*RPM		1.000
30		BHP	CALC'ED.	FROM 200 HP MOTOR KILOWATTS	1.000
31		SUCT	TRANSDR.	ROSEMOUNT-30-30FT H2O-06209A1E2	1.000
32		RPM	LEUZE		1.000

GEORGIA IRON WORKS CO.
P.O. BOX 626
GROVETOWN,
GEORGIA 30813

WORK ORDER NO GIW COURSE
SERIAL NO 5012 3514
TEST CURVE NO T 51 -79
TEST DATE 06/20/79

DRIVER DETAIL

TYPE LAB MOTOR
MAKE GENERAL ELEC
SERIAL NO 5511957
FRAME SIZE 5368480
RPM = 360 BHP = 2000.
4160 VOLTS 3 PHASE 60 CPS

SCALED PERFORMANCE FACTRS

SPEED OR RATIO .00
IMP TURN DOWN RATIO .00
MERIDNL WIDTH RATIO .00
SCALE RATIO .00
BEP REF .HG. .RPM
EFFICIENCY .0% BY .00

TEST RESULTS

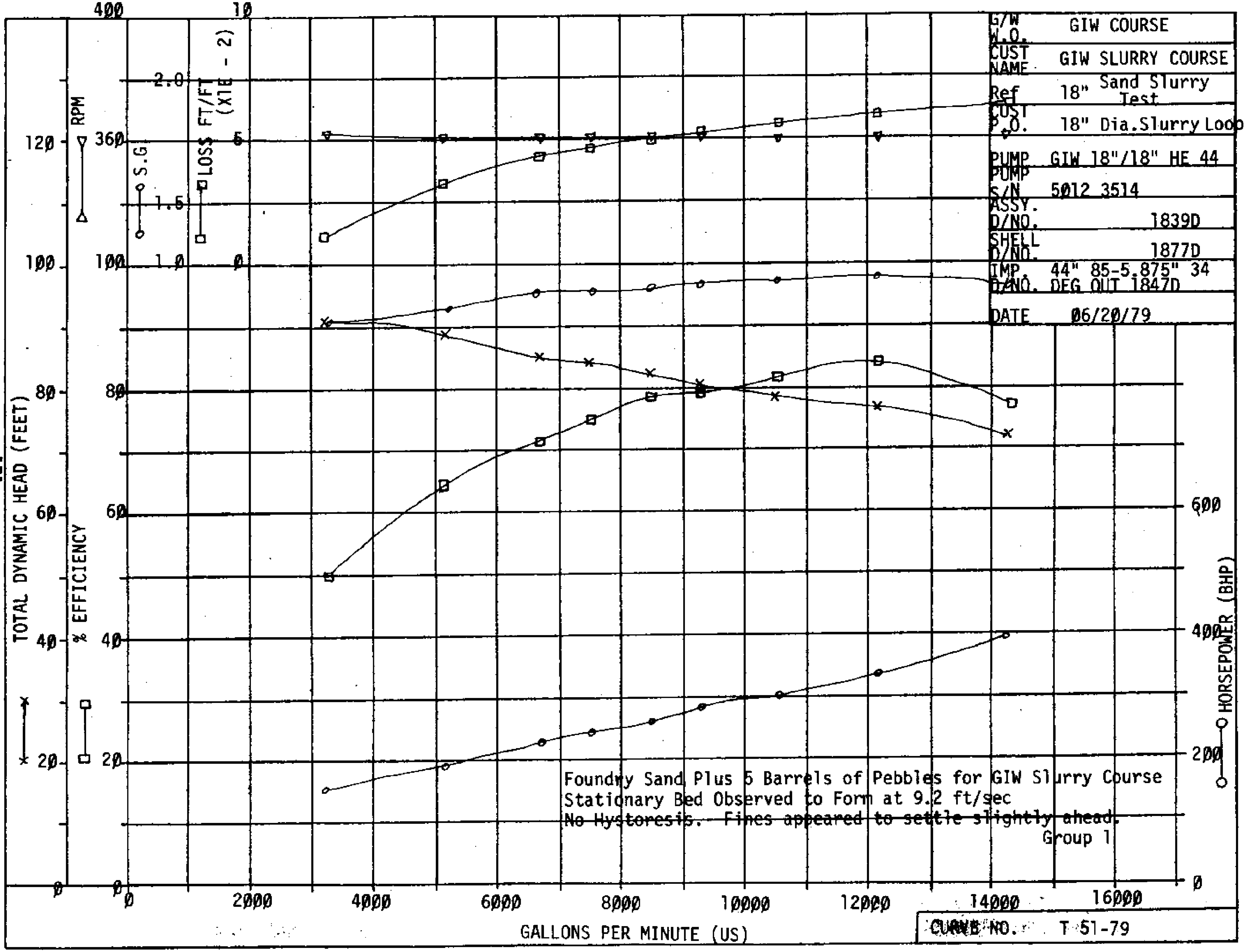
NO	FLOW MEASUREMENT: DIFF. ' H2O	FLOW US GPM	HEAD MEASUREMENT: DISCH ' H2O	SUCTN ' H2O	TOT HD ' FLUID	DRIVE INPUT, SPEED, OUTPUT: WATT METER	KILO-WATTS	RPM	BHP	PUMP: WATER HP OUT	EFFCY %	VEL FT/S	S.G.	HEAD LOSS /100'	HAZN WLLM	FRCTN FACTR:	TEMP (F)	TIME HH.MM
1	.00	14355.4	81.74	-24.73	71.11	.415	332.3	360	391.2	297.6	76.1	19.6	1.15	6.35	136.	.0159	109.1	16.58
2	.00	12208.4	89.02	-12.39	75.82	.360	288.2	360	330.7	275.7	83.4	16.7	1.18	5.99	121.	.0202	109.1	17.05
3	.00	10567.2	89.98	-7.69	77.50	.330	264.4	360	297.6	241.1	81.1	14.5	1.17	5.59	108.	.0255	109.1	17.08
4	.00	9301.2	91.37	-5.17	79.64	.311	249.0	360	276.1	215.4	78.1	12.7	1.15	5.17	98.	.0308	108.8	17.14
5	.00	8475.2	92.19	-3.15	81.31	.291	233.2	360	254.7	198.0	77.8	11.6	1.14	5.00	91.	.0363	107.8	17.17
6	.00	7462.3	92.83	-1.36	83.49	.276	220.7	360	237.4	175.4	74.0	10.2	1.12	4.65	82.	.0444	107.4	17.19
7	.00	6655.3	93.37	.01	84.04	.263	210.5	360	223.6	157.4	70.4	9.1	1.11	4.27	77.	.0513	108.3	17.28
8	.00	5076.9	91.69	2.79	87.81	.228	182.4	360	184.9	117.7	63.7	6.9	1.05	3.24	66.	.0713	108.4	17.37
9	.00	3139.8	89.76	5.59	90.03	.195	156.1	361	148.2	71.2	48.1	4.3	1.00	.93	78.	.0560	108.0	17.36

GEORGIA IRON WORKS CO.
TEST ENGINEER J.R. MAFFETT DATE 06/20/79

WITNESSED BY GROUP 1 FOR GIW SLURRY COURSE

COMMENTS

FOUNDRY SAND PLUS 5 BARRELS OF PEBBLES FOR GIW SLURRY COURSE
STATIONARY BED OBSERVED TO FORM AT 9.2 FT/SEC
NO HYSTERESIS, FINES APPEARED TO SETTLE SLIGHTLY AHEAD. GROUP 1


G/W W.O. GIW COURSE
CUST NAME GIW SLURRY COURSE
Ref 18" Sand Slurry Test
CUST P.O. 18" Dia.Slurry Loop
PUMP GIW 18"/18" HE 44
PUMP S/N 5012 3514
ASSY. D/NO. 1839D
SHELL D/NO. 1877D
IMP. D/NO. 44" 85-5.875" 34 DEG OUT 1847D
DATE 06/20/79
Foundry Sand Plus 5 Barrels of Pebbles for GIW Slurry Course
Stationary Bed Observed to Form at 9.2 ft/sec
No Hystoresis. Fines appeared to settle slightly ahead.
Group 1
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
RPM
S.G
LOSS FT/FT (X1E - 2)
HORSEPOWER (BHP)
GALLONS PER MINUTE (US)
CURVE NO. T 51-79

All the different instrument readings are monitored, displayed and stored by the GIW mini computer data acquisition system described previously.

The total head developed by a pump is equal to the pump discharge pressure less the suction pressure plus the velocity head given to the water in common units of feed of liquid.

$$TDH = H_{\text{Disch press}} - H_{\text{Suct press}} + \frac{Vel^2 disc}{2g} - \frac{Vel^2\ suct}{2g}$$

If the inlet and outlet branches are the same diameter, the velocity head will be zero. If the suction and discharge gauges are at pipe centerline and these are at different levels, then a correction need be applied to bring all relative to the pump centerline.

For a perfect fluid, the TDH produced by a pump (refer Eulers' Equation) will remain the same with varying SG provided the units are in feet of liquid level.

The power given to the liquid in the American system of units is usually calculated in units of horsepower. Water horsepower (WHP) is found from using total developed head in ft of liquid and flow in USGPM.

$$\text{Water horsepower (WHP)} = \frac{\text{Total Developed Head (ft of liquid)} \times \text{Qty(gpm)} \times \text{SG of lqd.}}{3960}$$

it follows also, that Pump Efficiency = $\frac{\text{WHP}}{\text{BHP input to pump}} \times 100$

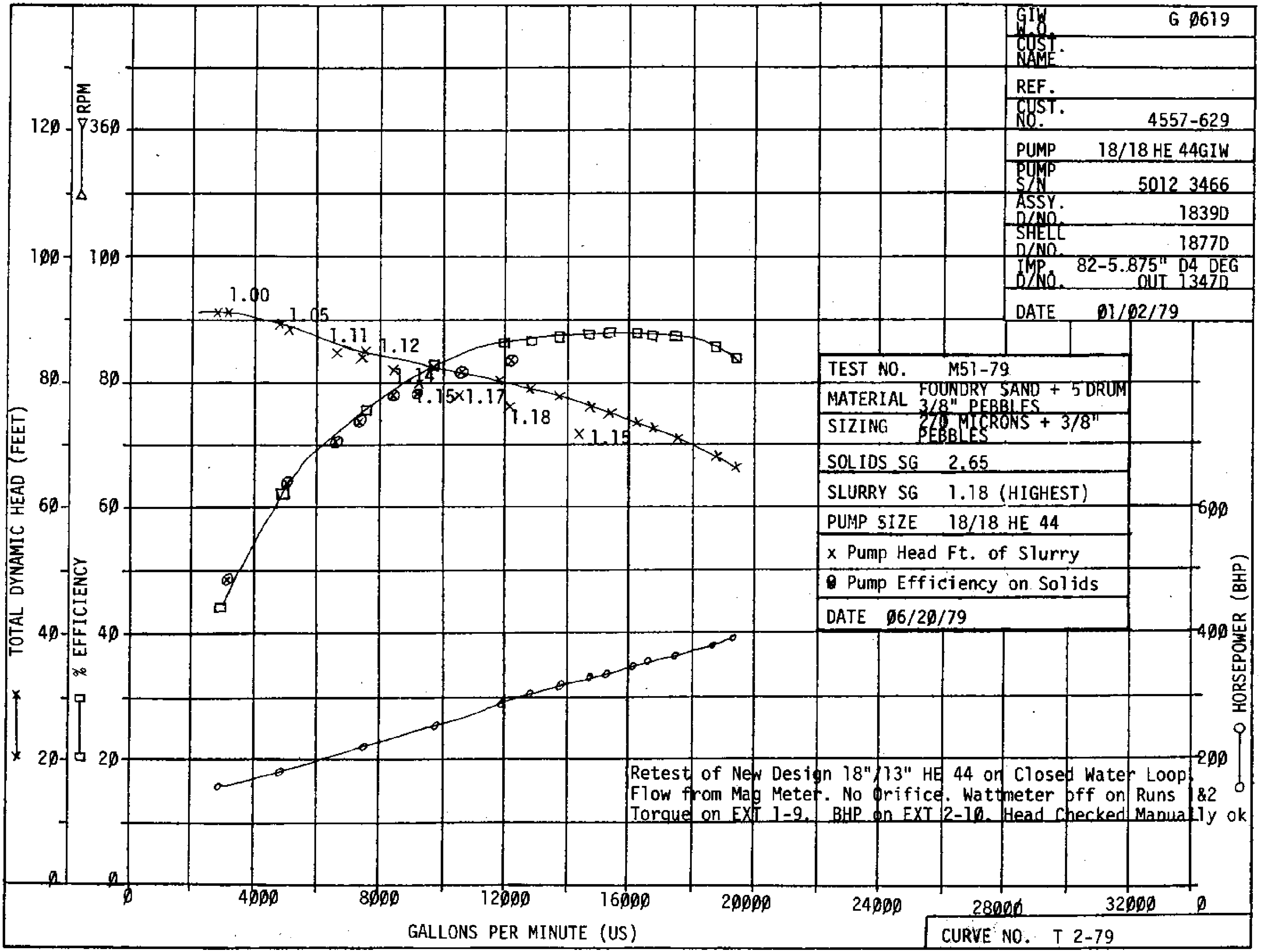
The pump performance recorded during a test on a typical settling slurry is as shown on the data sheet for test M 51-79 attached. This sheet shows the normal method of presenting slurry pump test results carried out in the GIW Lab.

As noted, the pump TDH is shown in units of ft of liquid or slurry.

For the case above, the pump head and efficiency data points have been plotted over the original water performance curve T 2-79 attached so a comparison can be made.

A slurry is not a perfect fluid, so the head produced is not the same as that of a fluid of a higher specific gravity.

It does, however, approach this for a wide variety of cases so it is used in practice as a first estimate of pump performance on slurry.



GIW W.O. G Ø619
CUST. NAME
REF.
CUST. NO. 4557-629
PUMP 18/18 HE 44GIW
PUMP S/N 5012 3466
ASSY. D/NO. 1839D
SHELL D/NO. 1877D
IMP. D/NO. 82-5.875" D4 DEG OUT 1347D
DATE Ø1/Ø2/79
TEST NO. M51-79
MATERIAL FOUNDRY SAND + 5 DRUM 3/8" PEBBLES
SIZING 27Ø MICRONS + 3/8" PEBBLES
SOLIDS SG 2.65
SLURRY SG 1.18 (HIGHEST)
PUMP SIZE 18/18 HE 44
x Pump Head Ft. of Slurry
Pump Efficiency on Solids
DATE Ø6/2Ø/79
Retest of New Design 18"/13" HE 44 on Closed Water Loop.
Flow from Mag Meter. No Orifice. Wattmeter off on Runs 1&2
Torque on EXT 1-9. BHP on EXT 2-1Ø. Head Checked Manually ok
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
RPM
HORSEPOWER (BHP)
GALLONS PER MINUTE (US)
CURVE NO. T 2-79
1.00
1.05
1.11
1.12
1.14
1.15
1.17
1.18
1.15

The pseudo fluid approach as this is generally called, is one where it is assumed that the head produced in ft of slurry is the same as the head of ft of water produced except for a loss called the solids effect.

In the case shown the head of pseudo fluid and efficiency data performance points are below the original water performance curve showing a "Solids Effect". It is noted, however, that the pressure produced by the pump is still considerably higher than the original water performance pressures.

This loss or solids effect varies with the pump size and design, the type of slurry, the size of particles, the concentration and the viscosity of the slurry.

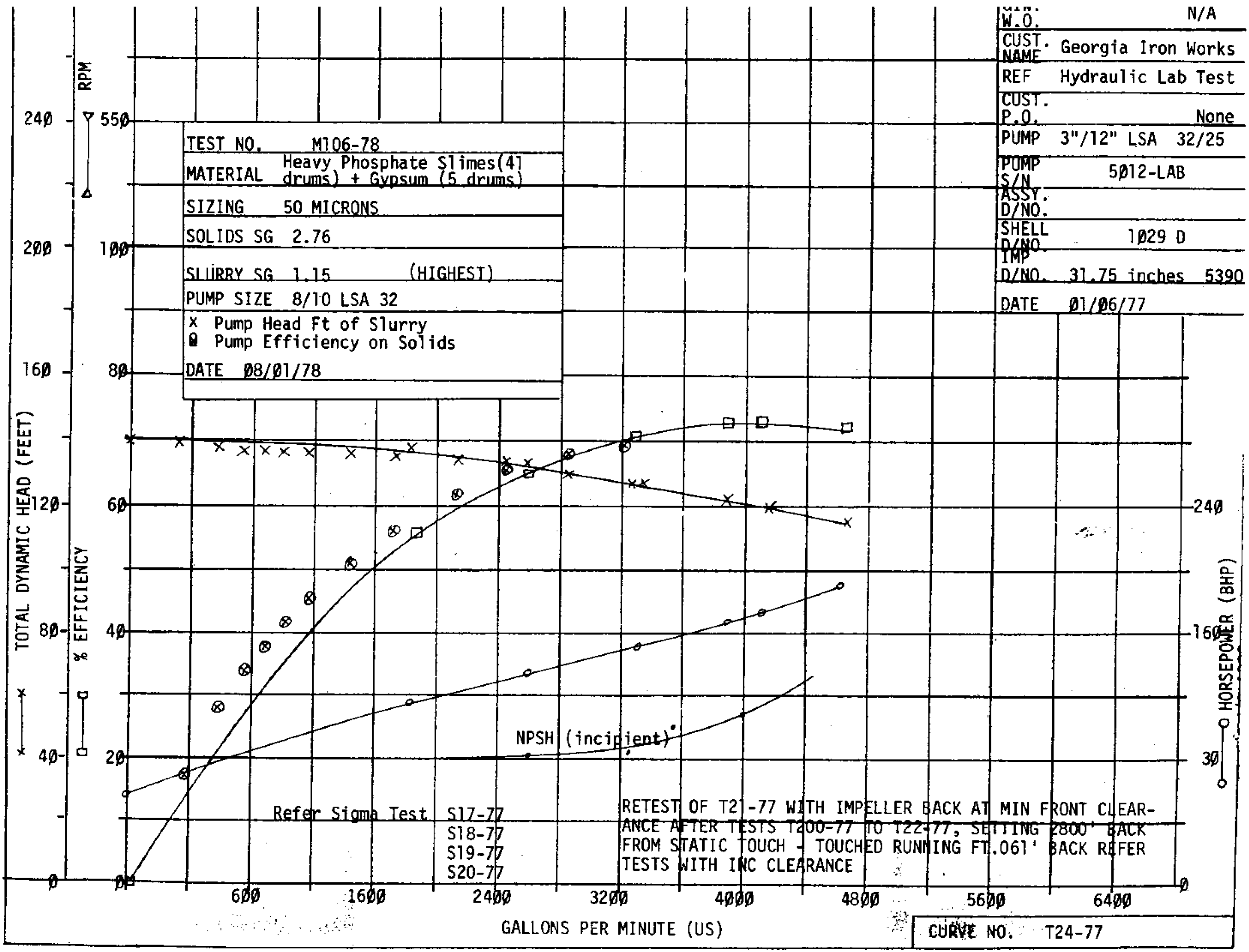
In practice, it is found that for non-settling slurries where the particle size is small, the solids SG is not high and the viscosity is not greatly different from water, that the effect on the head and efficiency is negligible or less than 1%.

A number of the tests carried out in the GIW Lab on slurries composed of small sized particle solids are shown below compared with the water performance of the pump. In general, these are non-settling slurries. Units used in all cases are ft of pseudo fluid.

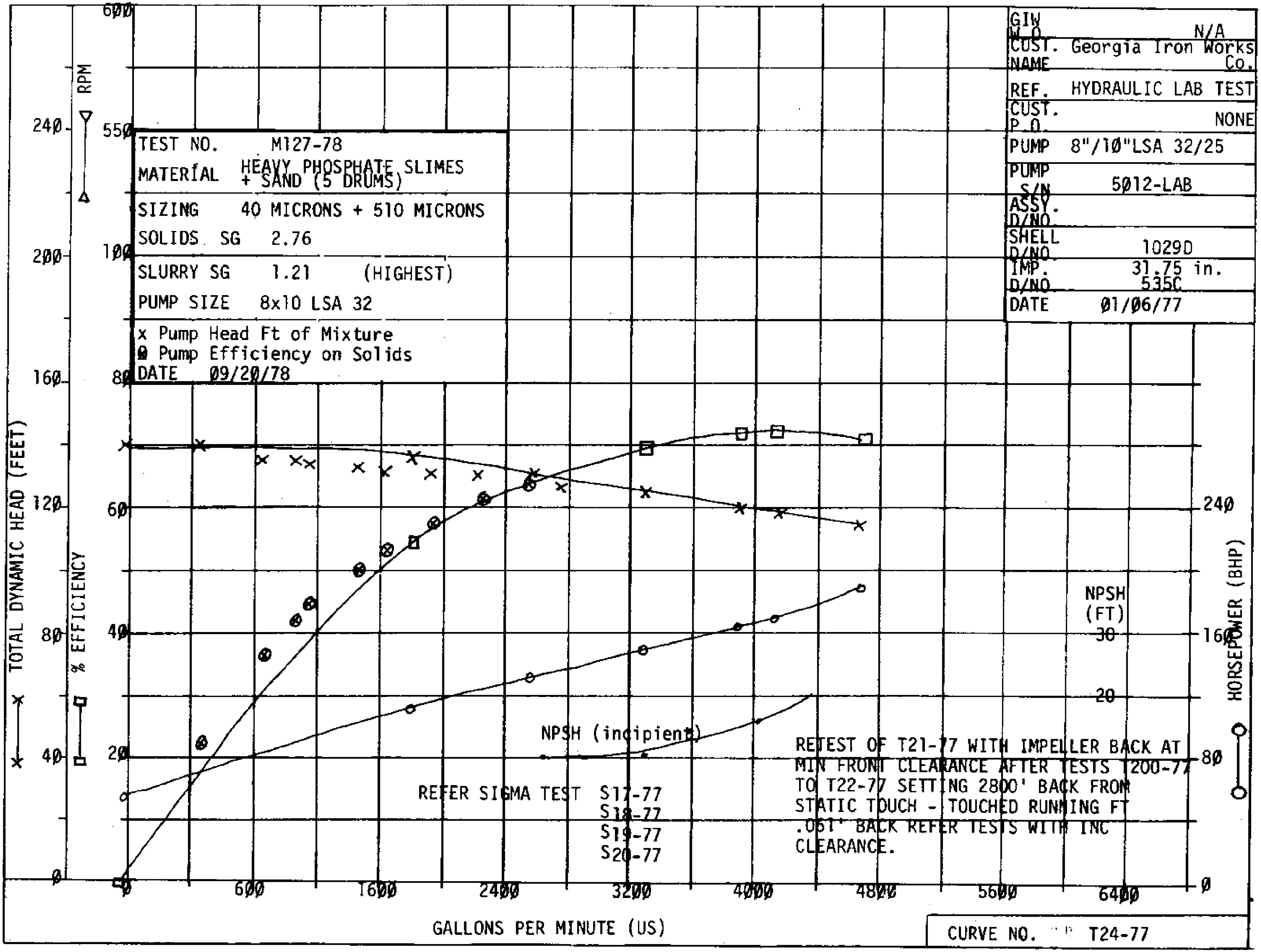
Test No.	Material	Sizing	Slurry SG	Pump Size	Max Head Loss %	Max Eff Loss %
M106-78	Heavy Phosphate Slimes	50 Microns	1.15	GIW 8x10LSA32	Less than 1%	
M127-78	" "	40 Microns	1.21	GIW 8x10LSA32	" " "	
M 36-79	" "	50 Microns	1.21	GIW 8x10HE 32	" " "	
M52A-79	Kaolin Slurry	50 Microns	1.64	GIW 8x10HE 32	1½%	4%

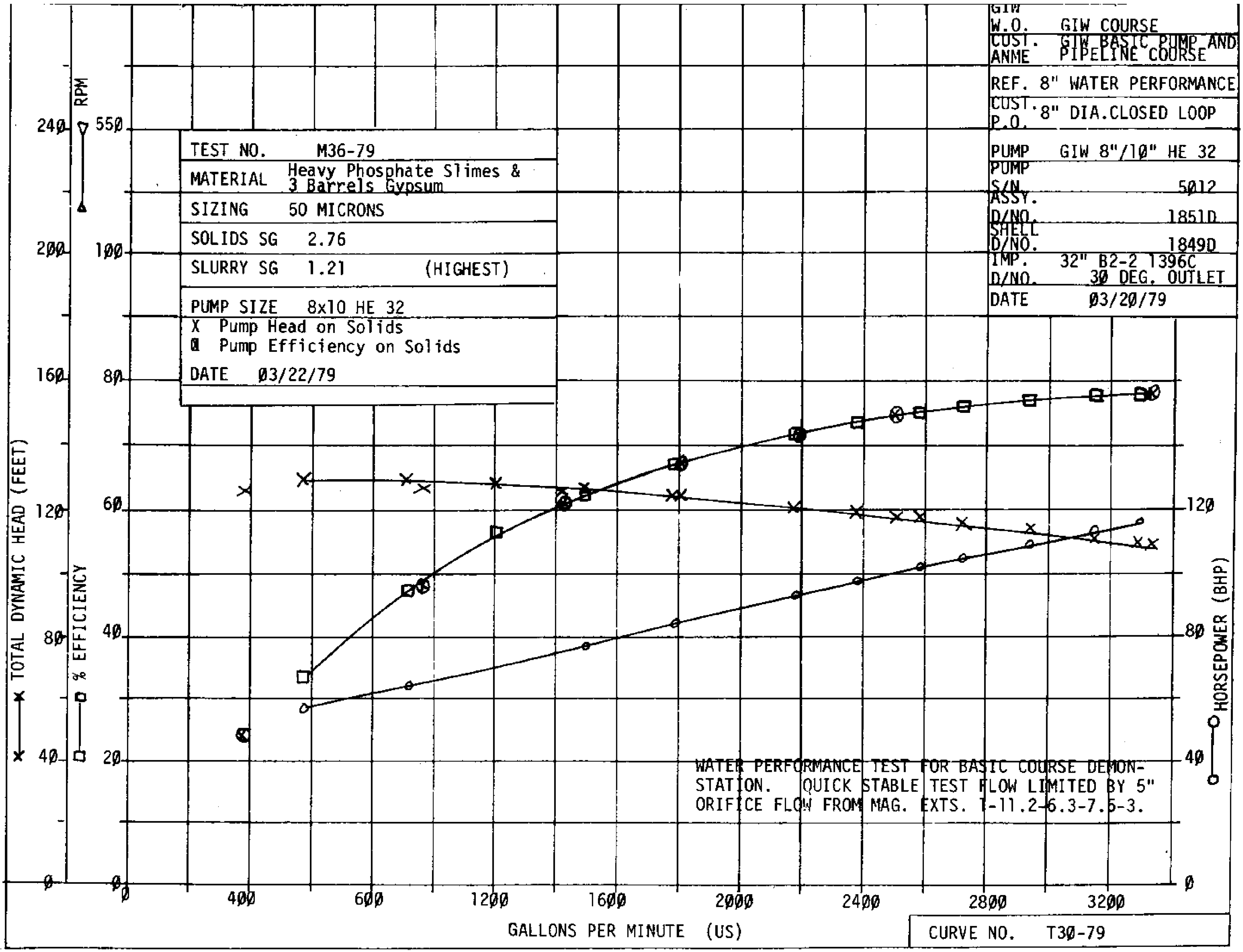
Where the particle size is increased the effect on both head and efficiency also increases. The effect is greatest where the particle is largest, the solids SG is highest and/or the concentration highest.

The following are a number of tests carried out in the lab which illustrate the effect of particle size and concentration. Again the loss values noted are in comparison with a water performance test. The flows at which these occurred were the maximum for the test and were in the range 80-110% of the pump best efficiency flow.

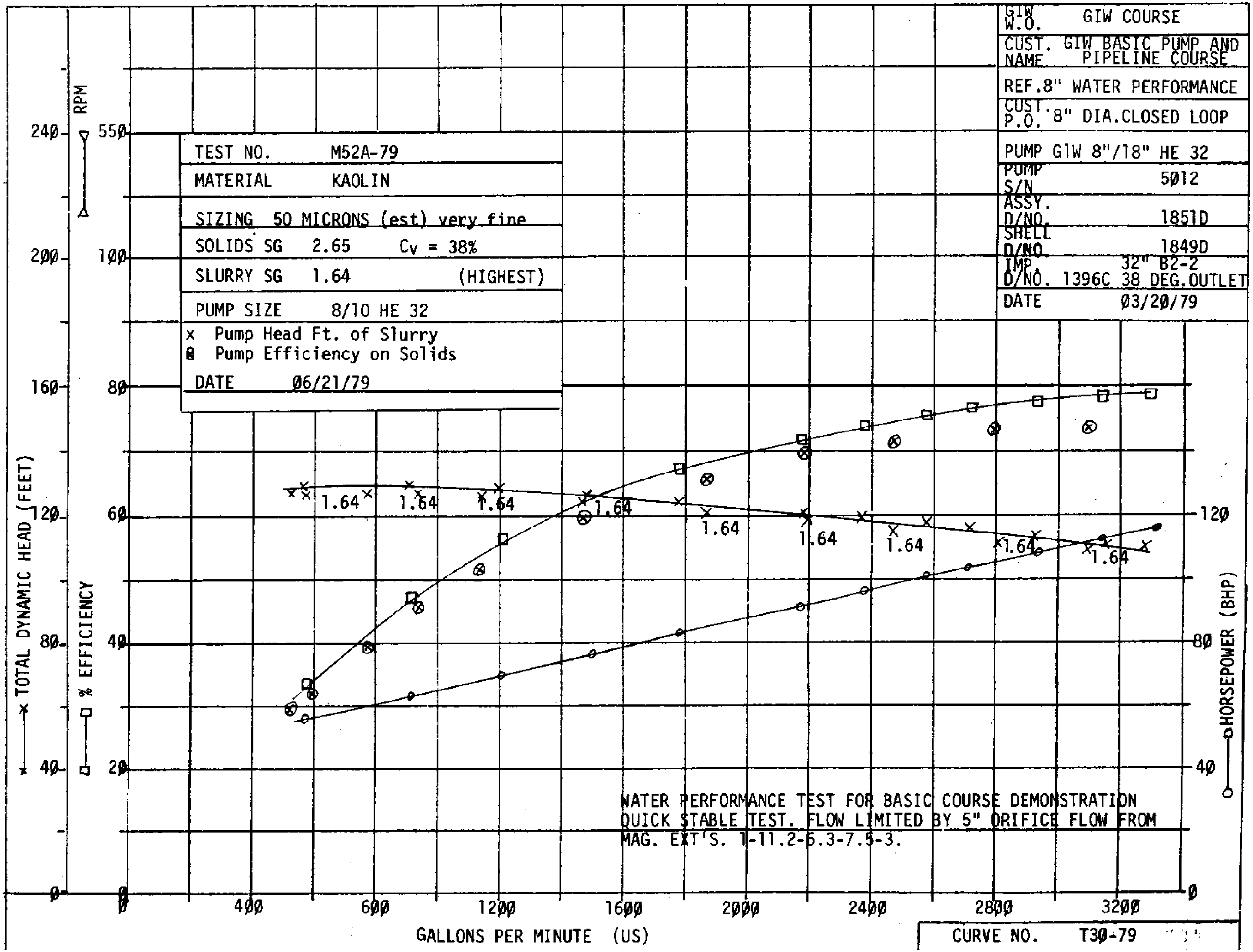


G.I.W. W.O. N/A
CUST. NAME Georgia Iron Works
REF Hydraulic Lab Test
CUST. P.O. None
PUMP 3"/12" LSA 32/25
PUMP S/N 5012-LAB
ASSY. D/NO.
SHELL D/NO. 1029 D
IMP D/NO. 31.75 inches 5390
DATE 01/06/77
TEST NO. M106-78
MATERIAL Heavy Phosphate Slimes(4 drums) + Gypsum (5 drums)
SIZING 50 MICRONS
SOLIDS SG 2.76
SLURRY SG 1.15 (HIGHEST)
PUMP SIZE 8/10 LSA 32
x Pump Head Ft of Slurry
⊗ Pump Efficiency on Solids
DATE 08/01/78
RPM
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
HORSEPOWER (BHP)
NPSH (incipient)
Refer Sigma Test S17-77 S18-77 S19-77 S20-77
RETEST OF T21-77 WITH IMPELLER BACK AT MIN FRONT CLEARANCE AFTER TESTS T200-77 TO T22-77, SETTING 2800' BACK FROM STATIC TOUCH - TOUCHED RUNNING FT.061' BACK REFER TESTS WITH INC CLEARANCE
GALLONS PER MINUTE (US)
CURVE NO. T24-77


GIW W.O. N/A
CUST. NAME Georgia Iron Works Co.
REF. HYDRAULIC LAB TEST
CUST. P.O. NONE
PUMP 8"/10"LSA 32/25
PUMP S/N 5012-LAB
ASSY. D/NO.
SHELL D/NO. 1029D
IMP. D/NO. 31.75 in. 535C
DATE 01/06/77
TEST NO. M127-78
MATERIAL HEAVY PHOSPHATE SLIMES + SAND (5 DRUMS)
SIZING 40 MICRONS + 510 MICRONS
SOLIDS SG 2.76
SLURRY SG 1.21 (HIGHEST)
PUMP SIZE 8x10 LSA 32
x Pump Head Ft of Mixture
⊗ Pump Efficiency on Solids
DATE 09/20/78
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
RPM
HORSEPOWER (BHP)
NPSH (FT)
NPSH (incipient)
REFER SIGMA TEST S17-77 S18-77 S19-77 S20-77
RETEST OF T21-77 WITH IMPELLER BACK AT MIN FRONT CLEARANCE AFTER TESTS T200-77 TO T22-77 SETTING 2800' BACK FROM STATIC TOUCH - TOUCHED RUNNING FT .061" BACK REFER TESTS WITH INC CLEARANCE.
GALLONS PER MINUTE (US)
CURVE NO. T24-77
0
600
1600
2400
3200
4000
4800
5600
6400
40
80
120
160
200
240
20
60
100
550
600
30
20
80
160
240



GIW
W.O. GIW COURSE
CUST. NAME GIW BASIC PUMP AND PIPELINE COURSE
REF. 8" WATER PERFORMANCE
CUST. P.O. 8" DIA.CLOSED LOOP
PUMP GIW 8"/10" HE 32
PUMP S/N. 5012
ASSY. D/NO. 1851D
SHELL D/NO. 1849D
IMP. D/NO. 32" B2-2 1396C 30 DEG. OUTLET
DATE 03/20/79
TEST NO. M36-79
MATERIAL Heavy Phosphate Slimes & 3 Barrels Gypsum
SIZING 50 MICRONS
SOLIDS SG 2.76
SLURRY SG 1.21 (HIGHEST)
PUMP SIZE 8x10 HE 32
X Pump Head on Solids
Ø Pump Efficiency on Solids
DATE 03/22/79
RPM
550
240
200
160
120
80
40
0
100
80
60
40
20
0
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
HORSEPOWER (BHP)
120
80
40
0
0
400
600
1200
1600
2000
2400
2800
3200
GALLONS PER MINUTE (US)
WATER PERFORMANCE TEST FOR BASIC COURSE DEMON-STATION. QUICK STABLE TEST FLOW LIMITED BY 5" ORIFICE FLOW FROM MAG. EXTS. 1-11.2-6.3-7.5-3.
CURVE NO. T30-79

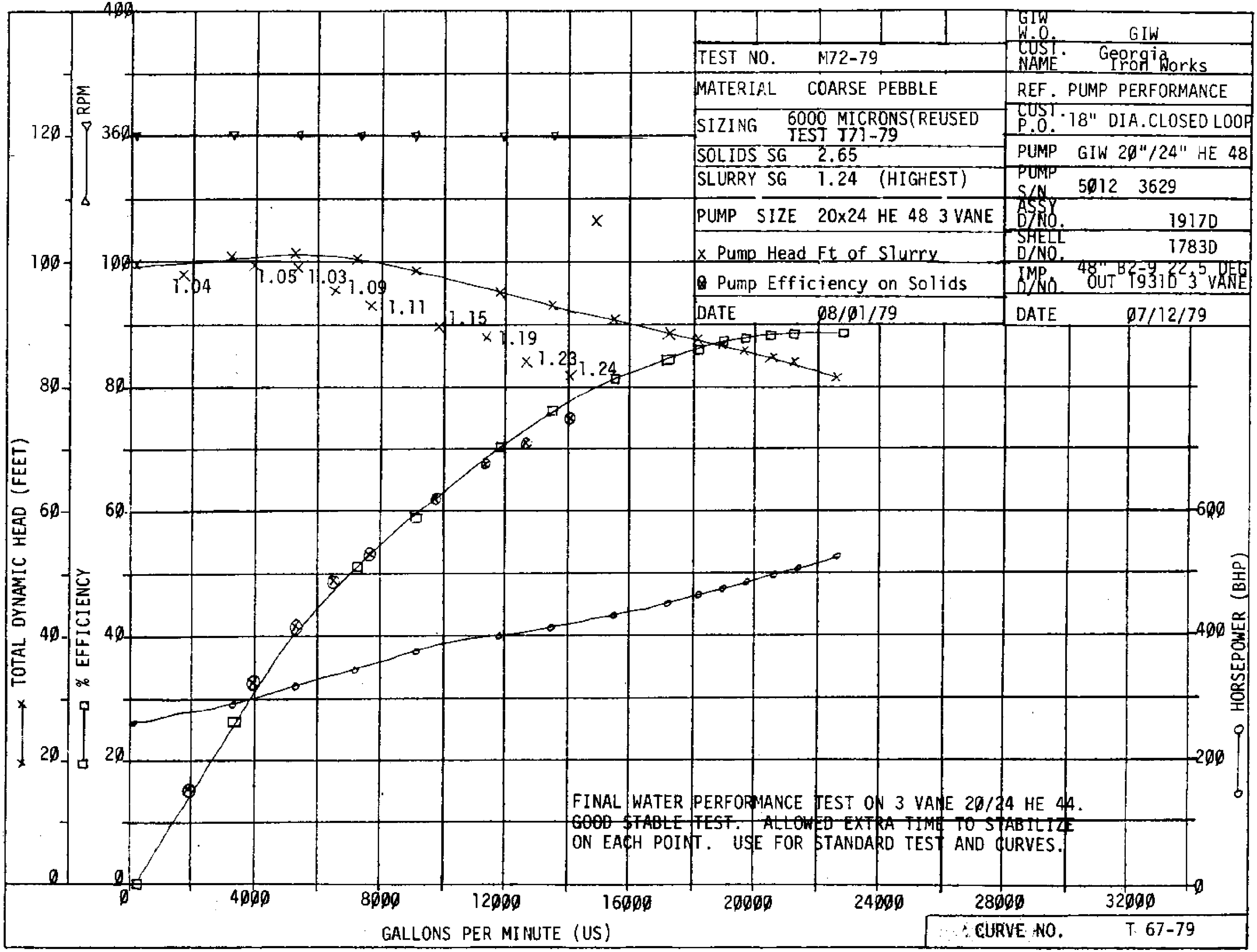

GIW W.O. GIW COURSE
CUST. NAME GIW BASIC PUMP AND PIPELINE COURSE
REF. 8" WATER PERFORMANCE
CUST. P.O. 8" DIA. CLOSED LOOP
PUMP GIW 8"/18" HE 32
PUMP S/N 5012
ASSY. D/NO. 1851D
SHELL D/NO. 1849D
IMP. D/NO. 1396C 32" B2-2 38 DEG. OUTLET
DATE 03/20/79
TEST NO. M52A-79
MATERIAL KAOLIN
SIZING 50 MICRONS (est) very fine
SOLIDS SG 2.65 Cv = 38%
SLURRY SG 1.64 (HIGHEST)
PUMP SIZE 8/10 HE 32
x Pump Head Ft. of Slurry
Pump Efficiency on Solids
DATE 06/21/79
RPM
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
HORSEPOWER (BHP)
GALLONS PER MINUTE (US)
WATER PERFORMANCE TEST FOR BASIC COURSE DEMONSTRATION
QUICK STABLE TEST. FLOW LIMITED BY 5" ORIFICE FLOW FROM
MAG. EXT'S. 1-11.2-6.3-7.5-3.
CURVE NO. T30-79

Test No.	Material	Sizing	Slurry SG	Pump Size	Max Head Loss %	Max Eff Loss %
M 72 -79	3/8" Pebble	6000 Microns	1.24	20x24HE48(GIW)	11%	4%
M118 -78	Phosphate Reject	500 Microns	1.18	8x10LSA32(GIW)	9%	7%
M 44 -79	3/8" Pebble	6000 Microns	1.16	8x10HE32(GIW)	8%	5%
M 50 -79	Foundry Sand	270 Microns	1.15	18x18HE44(GIW)	4%	3%
M102 -78	Foundry Sand	270 Microns	1.34	8x10LSA32(GIW)	10%	4%

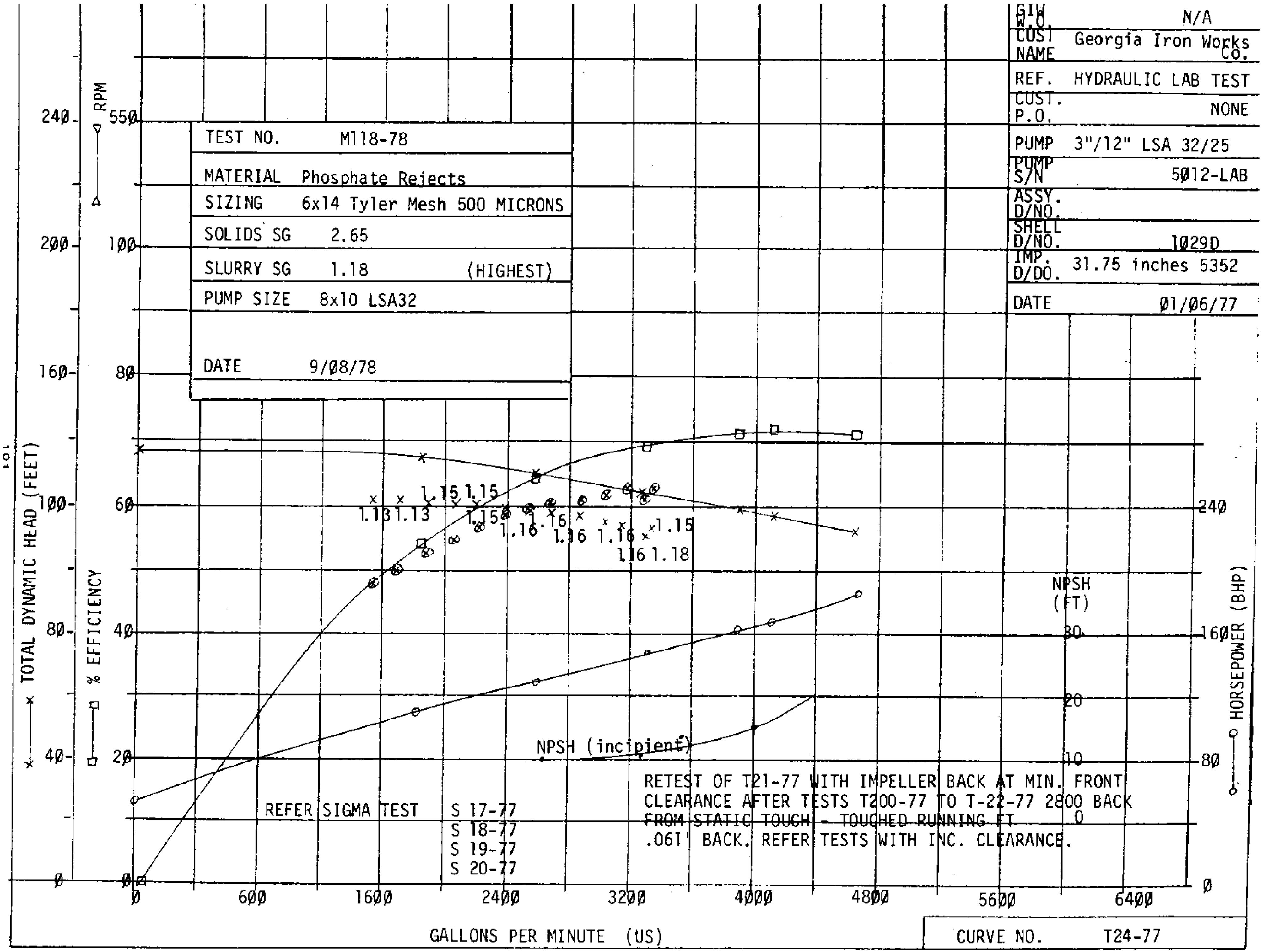
Solids SG can have a significant effect on pump performance as shown below in the results of a test carried out on a taconite slurry.

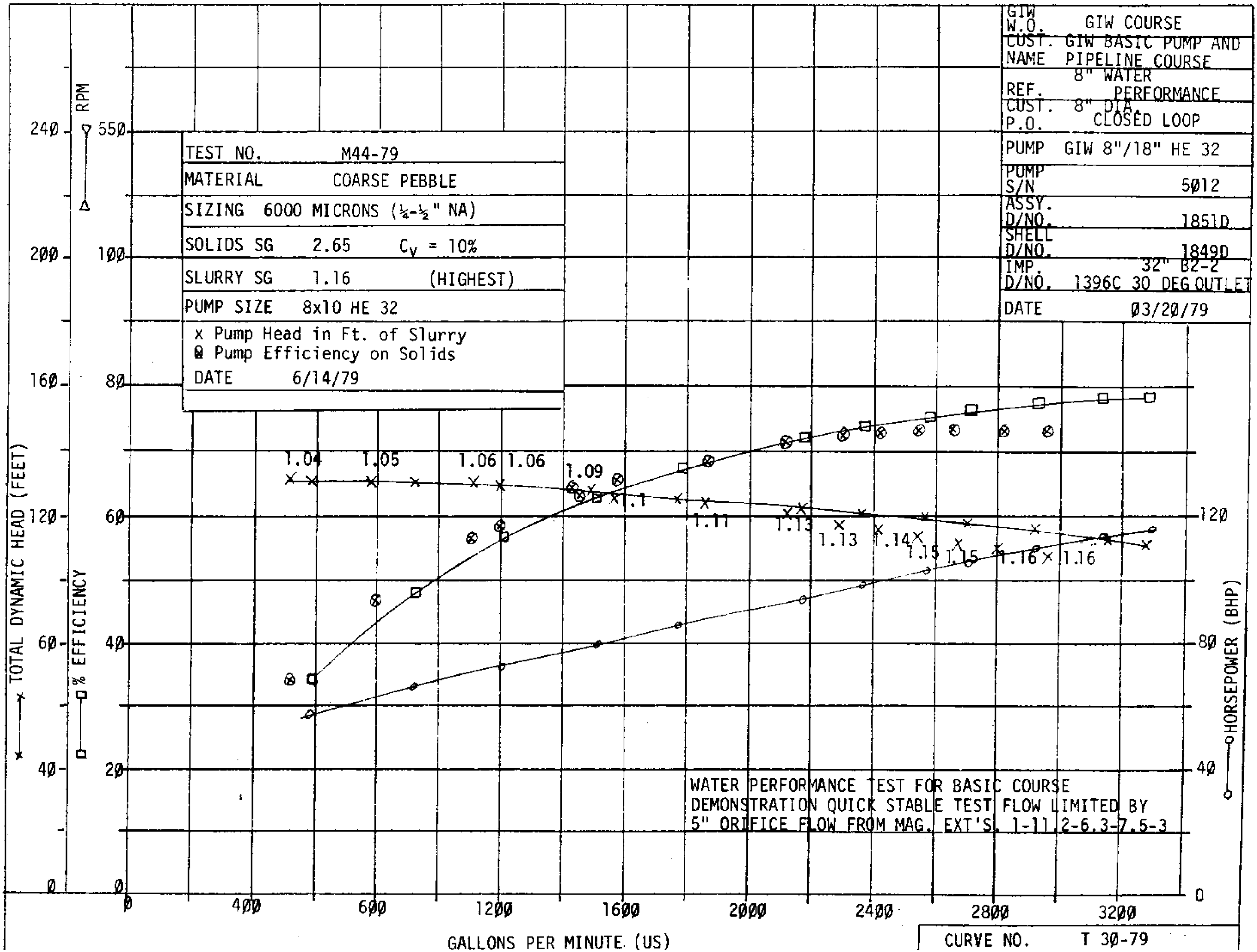
Test No.	Material	Sizing	Slurry SG	Pump Size	Max Head Loss %	Max Eff Loss %
M 95A-78	Taconite Ore	53 Microns	1.6	6x6LSA25(GIW)	14%	5%

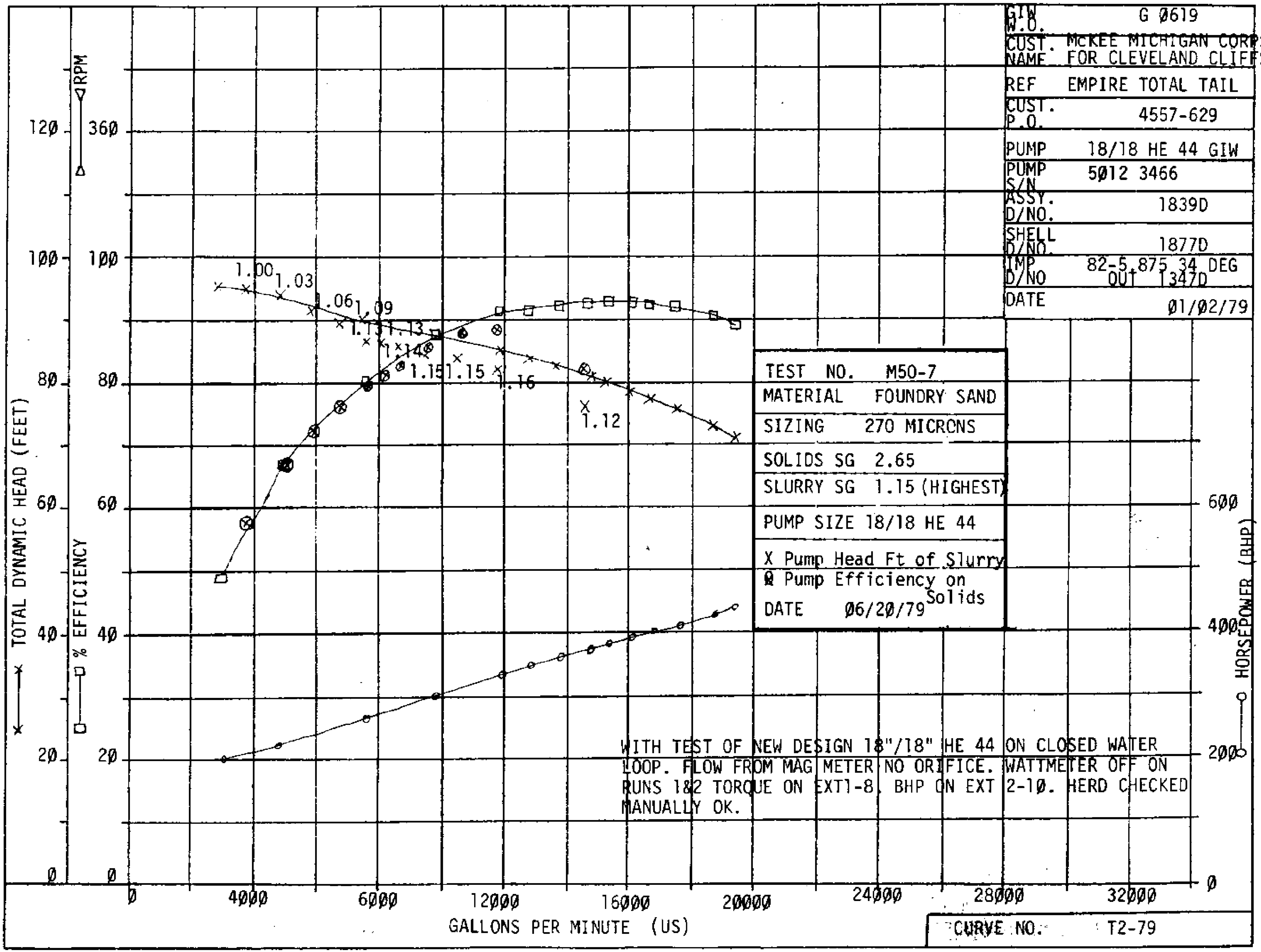
Much more data is needed to be collected before we can understand and predict the performance of a centrifugal slurry pump whilst pumping solids. GIW is continuing with a program of pump and pipeline solids testing with the aim of achieving a greater understanding of this subject.

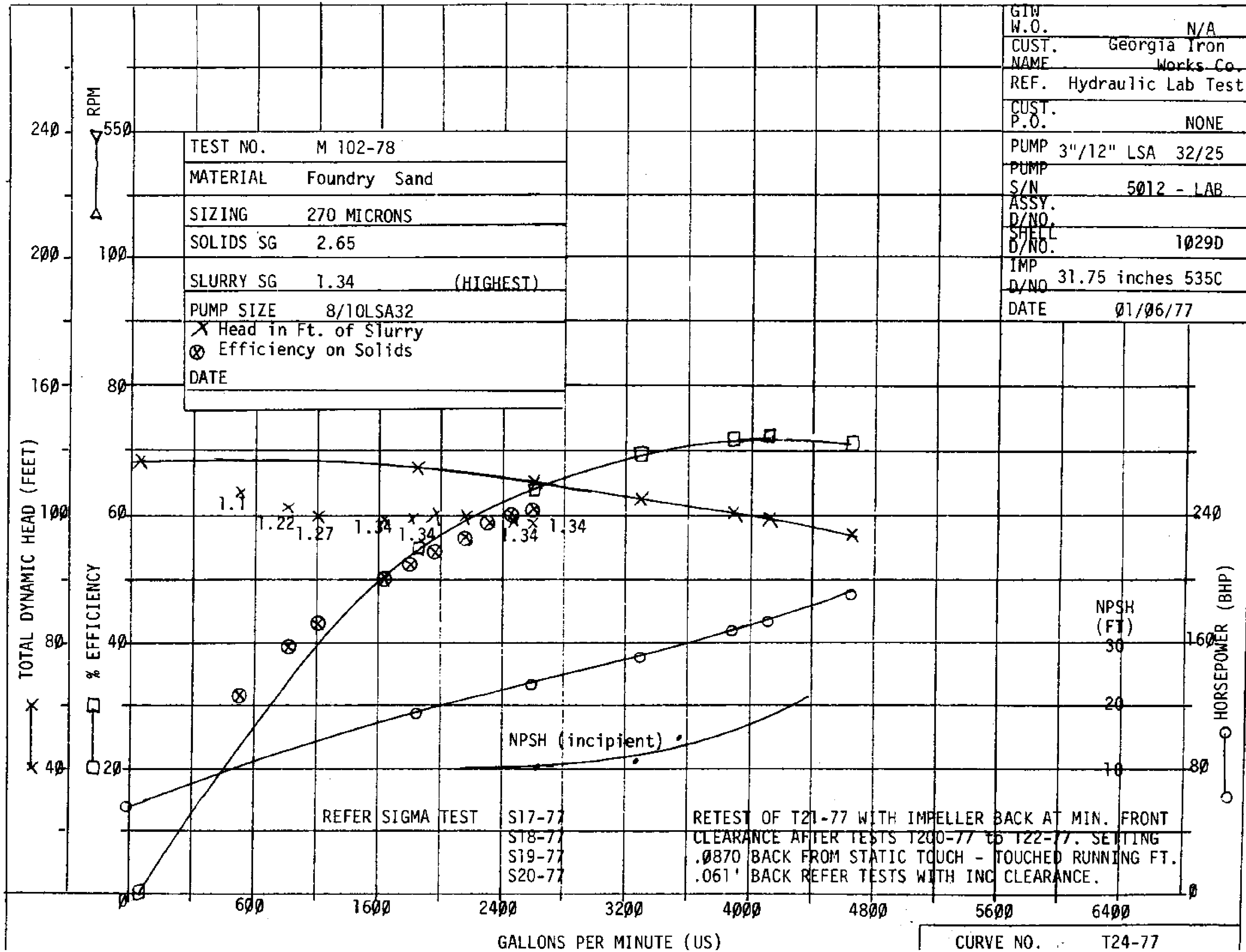


TEST NO. M72-79
MATERIAL COARSE PEBBLE
SIZING 6000 MICRONS(REUSED TEST T71-79
SOLIDS SG 2.65
SLURRY SG 1.24 (HIGHEST)
PUMP SIZE 20x24 HE 48 3 VANE
x Pump Head Ft of Slurry
⊗ Pump Efficiency on Solids
DATE 08/01/79
GIW W.O. GIW
CUST. NAME Georgia Iron Works
REF. PUMP PERFORMANCE
CUST. P.O. 18" DIA.CLOSED LOOP
PUMP GIW 20"/24" HE 48
PUMP S/N 5012 3629
ASSY D/NO. 1917D
SHELL D/NO. 1783D
IMP. D/NO. 48" B2-9 22.5 DEG OUT 1931D 3 VANE
DATE 07/12/79
1.04
1.05
1.03
1.09
1.11
1.15
1.19
1.23
1.24
FINAL WATER PERFORMANCE TEST ON 3 VANE 20/24 HE 44.
GOOD STABLE TEST. ALLOWED EXTRA TIME TO STABILIZE
ON EACH POINT. USE FOR STANDARD TEST AND CURVES.
RPM
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
HORSEPOWER (BHP)
GALLONS PER MINUTE (US)
0
4000
8000
12000
16000
20000
24000
28000
32000
0
20
40
60
80
100
120
360
400
200
600
CURVE NO. T 67-79


GIW W.O. N/A
CUST NAME Georgia Iron Works Co.
REF. HYDRAULIC LAB TEST
CUST. P.O. NONE
PUMP 3"/12" LSA 32/25
PUMP S/N 5Ø12-LAB
ASSY. D/NO.
SHELL D/NO. 1Ø29D
IMP. D/DO. 31.75 inches 5352
DATE Ø1/Ø6/77
TEST NO. M118-78
MATERIAL Phosphate Rejects
SIZING 6x14 Tyler Mesh 500 MICRONS
SOLIDS SG 2.65
SLURRY SG 1.18 (HIGHEST)
PUMP SIZE 8x10 LSA32
DATE 9/Ø8/78
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
RPM
HORSEPOWER (BHP)
NPSH (FT)
NPSH (incipient)
GALLONS PER MINUTE (US)
REFER SIGMA TEST S 17-77 S 18-77 S 19-77 S 20-77
RETEST OF T21-77 WITH IMPELLER BACK AT MIN. FRONT CLEARANCE AFTER TESTS T200-77 TO T-22-77 2800 BACK FROM STATIC TOUCH - TOUCHED RUNNING FT 0 .061" BACK. REFER TESTS WITH INC. CLEARANCE.
CURVE NO. T24-77


GIW W.O. GIW COURSE
CUST. NAME GIW BASIC PUMP AND PIPELINE COURSE
REF. 8" WATER PERFORMANCE
CUST. P.O. 8" DIA. CLOSED LOOP
PUMP GIW 8"/18" HE 32
PUMP S/N 5Ø12
ASSY. D/NO. 1851D
SHELL D/NO. 1849D
IMP. D/NO. 32" B2-2 1396C 30 DEG OUTLET
DATE Ø3/2Ø/79
TEST NO. M44-79
MATERIAL COARSE PEBBLE
SIZING 6000 MICRONS (¼-½" NA)
SOLIDS SG 2.65 C_V = 10%
SLURRY SG 1.16 (HIGHEST)
PUMP SIZE 8x10 HE 32
x Pump Head in Ft. of Slurry
⊗ Pump Efficiency on Solids
DATE 6/14/79
RPM
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
HORSEPOWER (BHP)
GALLONS PER MINUTE (US)
1.04
1.05
1.06
1.06
1.09
1.1
1.11
1.13
1.13
1.14
1.15
1.15
1.16
1.16
WATER PERFORMANCE TEST FOR BASIC COURSE
DEMONSTRATION QUICK STABLE TEST FLOW LIMITED BY
5" ORIFICE FLOW FROM MAG. EXT'S. 1-11,2-6,3-7,5-3
CURVE NO. T 3Ø-79

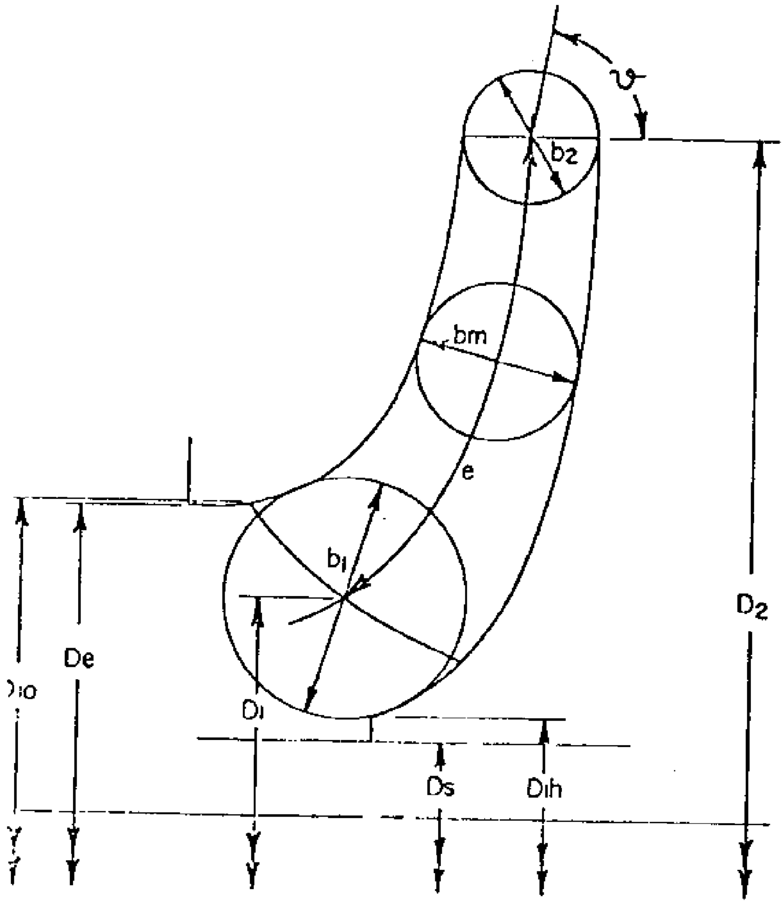

GIW W.O. G Ø619
CUST. NAME McKEE MICHIGAN CORP FOR CLEVELAND CLIFF
REF EMPIRE TOTAL TAIL
CUST. P.O. 4557-629
PUMP 18/18 HE 44 GIW
PUMP S/N 5Ø12 3466
ASSY. D/NO. 1839D
SHELL D/NO. 1877D
IMP D/NO 82-5.875 34 DEG OUT 1347D
DATE Ø1/Ø2/79
TEST NO. M50-7
MATERIAL FOUNDRY SAND
SIZING 270 MICRONS
SOLIDS SG 2.65
SLURRY SG 1.15 (HIGHEST)
PUMP SIZE 18/18 HE 44
X Pump Head Ft of Slurry
⊗ Pump Efficiency on Solids
DATE Ø6/2Ø/79
WITH TEST OF NEW DESIGN 18"/18" HE 44 ON CLOSED WATER LOOP. FLOW FROM MAG METER NO ORIFICE. WATTMETER OFF ON RUNS 1&2 TORQUE ON EXT1-8. BHP ON EXT 2-1Ø. HERD CHECKED MANUALLY OK.
1.00
1.03
1.06
1.09
1.13
1.13
1.14
1.15
1.15
1.16
1.12
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
RPM
36Ø
HORSEPOWER (BHP)
GALLONS PER MINUTE (US)
CURVE NO. T2-79


GIW W.O. N/A
CUST. NAME Georgia Iron Works Co.
REF. Hydraulic Lab Test
CUST. P.O. NONE
PUMP 3"/12" LSA 32/25
PUMP S/N 5Ø12 - LAB
ASSY. D/NO.
SHELL D/NO. 1Ø29D
IMP D/NO 31.75 inches 535C
DATE Ø1/Ø6/77
TEST NO. M 102-78
MATERIAL Foundry Sand
SIZING 270 MICRONS
SOLIDS SG 2.65
SLURRY SG 1.34 (HIGHEST)
PUMP SIZE 8/10LSA32
Head in Ft. of Slurry
Efficiency on Solids
DATE
RPM
TOTAL DYNAMIC HEAD (FEET)
% EFFICIENCY
HORSEPOWER (BHP)
NPSH (FT)
NPSH (incipient)
GALLONS PER MINUTE (US)
REFER SIGMA TEST S17-77 S18-77 S19-77 S20-77
RETEST OF T21-77 WITH IMPELLER BACK AT MIN. FRONT CLEARANCE AFTER TESTS T200-77 to T22-77. SETTING .Ø870 BACK FROM STATIC TOUCH - TOUCHED RUNNING FT. .061' BACK REFER TESTS WITH INC CLEARANCE.
CURVE NO. T24-77

PUMP TERMS (General)

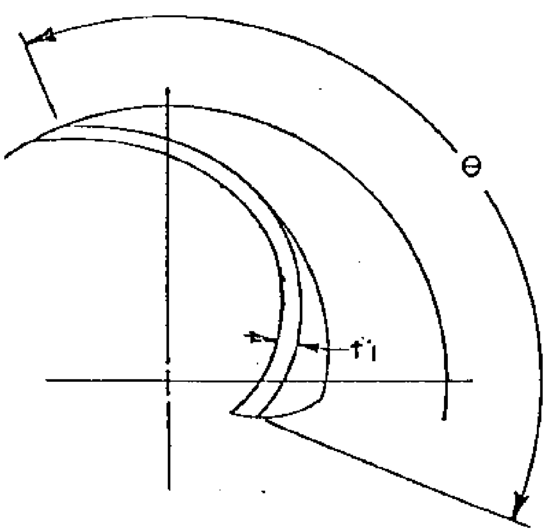
Q	=	flow rate through pump (gpm)
H	=	total head produced across pump (ft)
N or n	=	pump rotational speed (rpm)
BEPQ	=	flow rate at best efficiency (gpm)
BEPH	=	total head at BEPQ flow (ft)
η	=	pump efficiency
N_s(US)	=	American specific speed
T	=	Pump shaft torque (ft/lb)
ω	=	shaft angular velocity (rad/sec)
g	=	acceleration due to gravity (32.2 ft/sec^2)
γ	=	fluid density (lb/ft^3)
ρ	=	fluid density (slugs/ft^3 where $\gamma = \rho g$)
Pf	=	pressure on front of vane
Pb	=	pressure on back of vane
Pd	=	pressure at outlet of impeller
Ps	=	pressure at inlet of impeller
Hi	=	theoretical head produced by a pump
He	=	Eulers' head
BHP	=	pump input power

PUMP IMPELLER TERMS



Prefixes

2 at outside diameter
1 at inside (mean) diameter
m at mean diameter or meridinal
3 at shell tongue.



D_{1av} = average inlet diameter (ins)

D_2 = outlet diameter (ins)

r_2 = mean radius at outlet (ins)

Ψ = (psi) A dimensionless head constant

U_2 = tangential velocity at OD (ft/sec)

Cm_2 = meridinal radial velocity at outlet (ft/sec)

ηh = hydraulic efficiency

κ = (kappa) slip constant

b_1 = meridinal diameter at inlet (ins)

b_m = mean meridinal diameter(ins)

r_1 = mean radius at inlet (ins)

e = mean meridinal distance from inlet to outlet (ins)

C_2 = absolute outlet velocity (ft/sec)

W_2 = relative outlet velocity (ft/sec)

β_2 = vane outlet angle (deg.)

α_2 = absolute outlet angle (deg)

Cu_2 = tangential component velocity of absolute outlet velocity (ft/sec)

ζ = (zeta) absolute whirl

Z = number of vanes

b_2 = meridinal diameter at outlet (ins)

t_2 = vane tip length at outlet (ins)

De = eye diameter (ins)

θ = vane sweep angle (deg)

β_1 = vane inlet angle (deg)

C_1 = absolute inlet velocity (ft/sec)

W_1 = relative inlet velocity (ft/sec)

U_1 = tangential velocity at inlet (ft/sec)

d_1 = absolute inlet angle (deg)

Cm_1 = meridinal radial velocity at inlet

t_1 = vane thickness at inlet (ins)

t = vane thickness (in)

V = stream tube radius (in)

d = stream tube width (in)

η_v = volumetric efficiency

s = no of stream tubes

H_s = head at shut of flow (ft)

S = scale ratio

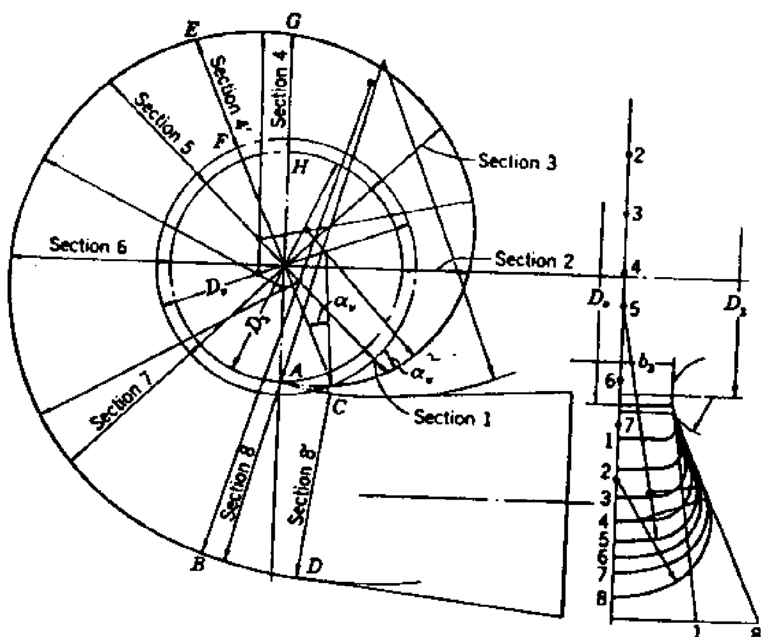
PUMP CASING TERMS

D_3 = volute base circle (in)

b_3 = volute width (in)

D_v = volute tongue dia (in)

α_v = volute angle (deg)



Pump volute casing

BUILDING AND MANAGEMENT OF DREDGED MATERIAL ISLANDS FOR USE BY WILDLIFE IN NORTH AMERICA

by

Mary C. Landin[1]

ABSTRACT

The more than 2000 dredged material islands in North America have become of immense value to North American sea and wading bird species. These islands are located primarily in the United States Intracoastal Waterway System, the larger rivers, and in the Great Lakes where colony-nesting birds congregate by the hundreds of thousands each year to nest. Since their traditional, natural colony sites have largely been disturbed by human activities, the birds have turned to use of manmade islands constructed of dredged material. Research in the U. S. Army Corps of Engineers Dredged Material Research Program has shown that islands can be built which will attract these bird species, including several that are endangered in North America. It is a feasible practice and one of great benefit to the wildlife using an island, to manage existing islands according to techniques set forth in Soots and Landin (1978). Simple modification of dredging schedules and dredged material placement accomplishes changes in habitat, plant successional stages, elevations, and stabilization, and can offer new land areas for wildlife use. Dike construction and modification, substrates, location, and configuration of the islands are important. Advantages and disadvantages to island construction and management are discussed, as well as several management techniques which may be carried out in a dredging operation.

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Introduction

In the past 100 years, more than 2000 islands have been constructed and/or altered by placement of dredged material deposits in United States waterways. Many more have been built that are no longer in existence due to erosion by wave, wind, and tidal actions, especially along the Florida to Texas stretch of the Gulf coast. Most of these islands have been built by the U. S. Army Corps of Engineers as a result of in-house, contracted, or permitted dredging. Most were built during the late 1930s and early 1940s when the U. S. Intracoastal Waterway System was constructed, and they often have been added to during maintenance dredging since that time (Landin 1978a).

These islands were not built with wildlife use as a consideration and they are not frequently inhabited by larger wildlife species. Rather, smaller mammals and rodents and numerous bird species including songbirds, raptors, waterfowl, shorebirds, and others use the islands. Many of these birds are migratory and use the islands primarily for feeding and resting; however, over a period of the past fifty years the islands have become of immense value to a unique wildlife group, the North American species of sea and wading birds. This phenomenon has largely resulted from human impacts on these birds through destruction, alteration, and displacement of habitats needed for their life requirements. Taken as a group because of their colony nesting habits, biological studies of these species indicate that these species need privacy, nesting sites with specific vegetation, substrate, and elevation characteristics, freedom from nest predators, and adequate feeding areas as the most basic of their requirements (Soots and Landin 1978). There are many other more subtle factors, such as interspecies relationships and courtship behaviors involved, that do not need to be discussed within the scope of this paper but are discussed in Soots and Landin (1978). Presently, there are hundreds of thousands of sea and wading birds nesting on dredged material islands, representing at least 37 species and ranging in percentage of their total population from minimal use to 99 percent use depending upon location, availability of habitat, and human disturbance factors (Table 1). Several of these species, including reddish egrets (Dichromanassa rufescens), roseate spoonbills (Ajaia ajaja), gull-billed terns (Gelochelidon nitotica), brown pelicans (Pelecanus occidentalis), least terns (Sterna albifrons), Caspian terns (Sterna caspia), and others, are considered rare, threatened, or endangered nationally or are on state or region-wide endangered species lists (Soots and Landin 1978).

Research and Results

As part of the work of the Dredged Material Research Program (DMRP), sponsored by the Office, Chief of Engineers and conducted by the U. S. Army Engineer Waterways Experiment Station (WES), Task 4F of the DMRP Habitat Development Project was initiated under the title "Island Habitat Development." This task was charged with the responsibility of locating and documenting the existence of and use by wildlife on Corps dredged material islands throughout U. S. continental waters (Smith 1978). In addition, comparisons were made between dredged material and natural islands, between diked and undiked islands, and other features inherent to dredged material substrates such as vegetation succession and soil formation rates and characteristics.

Table 1

Colonial Waterbird Species Found Nesting on Dredged Material Islands in Seven Regions of the Corps-Maintained Waterways[1]

Species	Regions[2]					
	TX	FL	NC	NJ	GL	PNW
White pelican (Pelecanus erythrorhynchos)	X					
Brown pelican (Pelecanus occidentalis)	X	X	X			
Double-crested cormorant (Phalocrocorax auritus)		X	X			
Olivaceous cormorant (Phalacrocorax olivaceus)	X					
Anhinga (Anhinga anhinga)		X				
Great blue heron (Ardea herodias)	X	X	X	X		X
Green heron (Butorides striatus)	X	X	X	X		
Little blue heron (Florida caerulea)	X	X	X	X		
Cattle egret (Bulbulcus ibis)	X	X	X	X	X	
Reddish egret (Dichromanassa rufescens)	X	X				
Great egret (Casmeroduis albus)	X	X	X	X		
Snowy egret (Egretta thula)	X	X	X	X		
Louisiana heron (Hydranassa violacea)	X	X	X	X		
Black-crowned night heron (Nycticorax nycticorax)	X	X	X	X	X	
Yellow-crowned night heron (Nyctanassa violacea)	X	X	X	X		
White-faced ibis (Plegadis chihi)	X					
Glossy ibis (Plegadis falcinellus)	X	X	X	X		
White ibis (Eudocimus albus)	X	X	X			
Roeseate spoonbill (Ajaia ajaja)	X	X				
Glaucous-winged gull (Larus glaucescens)						X
Great black-backed gull (Larus marinus)			X	X		
Herring gull (Larus argentatus)			X	X	X	
Western gull (Larus occidentalis)					X	
Ring-billed gull (Larus delewarensis)					X	X
Laughing gull (Larus atricilla)	X	X	X	X		
Gull-billed tern (Gelochelidon nitotica)	X	X	X	X		
Forster's tern (Sterna forsteri)	X		X	X	X	
Common tern (Sterna hirundo)		X	X	X	X	
Roseate tern (Sterna dougallii)		X	X	X		
Least tern (Sterna albifrons)	X	X	X	X		
Royal tern (Sterna maxima)	X	X	X			
Sandwich tern (Sterna sandivicensis)	X	X	X			
Caspian tern (Sterna caspia)	X	X	X		X	X
Black tern (Chilidonias niger)					X	X
Artic tern (Sterna paradisaea)						X
Black skimmer (Rynchops niger)	X	X	X	X		
Wood ibis (Mycteria americana)		X				

1 The upper Mississippi River study is not listed since none of the nesting colonies found were located on dredged material.

2 TX = Texas; FL = Florida; NC = North Carolina; NJ = New Jersey; GL = Great Lakes; PNW = Pacific Northwest.

(From Soots and Landin 1978)

Studies were carried out in seven regions (New Jersey coast, North Carolina estuaries, entire Florida coastline, entire Texas coastline, entire coasts of Oregon and Washington, entire U. S. Great Lakes shoreline, and the Upper Mississippi River between Lock and Dams 1-26 from Alton, Illinois to Minneapolis, Minnesota). The studies resulted in twelve technical reports including a synthesis report on the development and management of avian habitat on dredged material (Buckley and McCaffrey 1978; Chaney et al. 1978; Landin 1978b, 1978c; Lewis and Lewis 1978; Parnell et al. 1978; Peters et al. 1978; Scharf 1977; Scharf et al. 1978; Schrieber and Schrieber 1978; Soots and Landin 1978; Thompson and Landin 1978). Management techniques developed by Soots and Landin (1978) are used as the basis for this paper.

The Task 4F studies brought out facts that, although many colonies of birds presently are nesting on dredged material islands, there are numerous characteristics of these islands that could be improved by management to enhance the available habitat, and there are several ways dredging operations can be altered to benefit the numerous sea and wading birds and other wildlife on dredged material islands.

Dredged Material Island Development and Management

Development and management of dredged material islands for avian wildlife will usually provide essential habitat for the smaller mammals and rodents that use the islands as well, and covers a broad spectrum of techniques. Basically, development and/or management of an island for colonial sea and wading birds is concerned with habitat manipulation, habitat establishment, and habitat protection. Manipulation of habitats, by far the most likely technique to be used, for the engineer, would include proper placement of dredged material to maintain or reestablish habitats, increase the size of existing islands or stabilize islands, and/or change configuration, elevation, vegetation, and other island features for more desirable habitats. Manipulation of habitats would also include, for the biologist, establishment of new vegetation and management of existing vegetation on islands through various agronomic and horticultural techniques.

Establishment of new habitats is needed when nesting habitat is lacking and new islands must be created, with the resulting need for vegetation establishment; when nesting habitat is expanded by an addition to an existing island which must be established with vegetation; or when undesirable nesting habitats (vegetation) occurring on islands must be cleared out and desirable vegetation established in their place.

Habitat protection may be accomplished by island posting or fencing for isolation. Most bird species are already protected by law, but their habitats are not protected except during the time they are occupied by the nesting birds. Year-round protection to prevent destruction of habitat from year to year and seasonal protection to prevent nesting colony disruption by humans and predators are necessary.

Management of existing islands has been demonstrated to be an effective disposal technique and wildlife management practice. Considerable potential exists for the disposal of dredged material and the improvement of critical avian habitat. Management of existing dredged material islands is encouraged because the potential environmental impacts of disposing on an existing site are less than those of developing new islands.

The Use of Dredging Operations on Existing Islands

The Corps of Engineers has provided habitat for at least 37 species of sea and wading birds in North America since the agency first created dredged material islands. Since that time, islands have been kept in various stages of plant succession through dredged material deposition from channel maintenance operations. These operations have had a significant positive impact on bird breeding populations.

Through proper planning the positive impact of regular maintenance dredging could be increased. Since past dredging operations have been carried out with little or no regard for nesting birds, many areas do not have adequate diversity of nesting habitats. Some areas lack ground nesting habitats while others lack woody habitats. The DMRP Task 4F reports listed the following habitat needs that could be provided by dredging operations in regions studied:

1. The Texas coast needs additional habitat for ground nesting birds, especially in the Galveston Bay area. Nesting habitats for tree nesting birds is needed in the Laguna Madre area.

2. The Florida coast needs more bare ground areas and sparse to medium herbaceous plant cover for ground nesting species including terns, skimmers, gulls, and shorebirds.

3. North Carolina needs woody habitats for tree nesting species, especially near river mouths and inlets. The state also needs bare ground habitat for terns in coastal areas.

4. The New Jersey coast needs bare ground areas and sparse plant cover for ground nesting birds, and all kinds of woody habitat for tree nesting species.

5. The U. S. Great Lakes shoreline and islands need sparse plant cover habitat for common terns and herring gulls, and woody habitats for all the tree nesting colonial birds in the region.

6. The Upper Mississippi River basin has a great need for bare sandy beaches protected from human use so that the interior least tern (Sterna albifrons interior) could once again find suitable nesting habitat in that river system.

Similar habitat needs exist in other coastal and riverine areas where Corps islands are located that should be investigated to determine the types of habitats that are needed. The rate at which various habitats appear on an island after receiving dredged material and an estimate of their longevity have been determined in the DMRP Task 4F reports and at least two other studies (Soots and Parnell 1975 and Coastal Zone Resources Corporation 1977). Once site-specific needs are known, nesting habitat management can easily become a part of the regular maintenance dredging process. To maintain target habitat diversity for certain bird species, islands in any given area would have to be selected to receive periodic deposition of dredged material. Restrictions against dredged material deposition on all or parts of some islands may be necessary in order to allow habitats for tree nesting birds to develop or to preserve existing habitats. The feasibility of these management recommendations has already been demonstrated by the Corps' Wilmington District. They have been practicing such management on a local, annual basis for several years and have developed a long-range colonial sea and wading bird management plan for the lower Cape Fear River estuary which includes maintenance dredging and placement and timing of dredged material depositions on existing islands.

Building New Islands

Construction of new islands would be desirable under some condition. If it has been demonstrated that there is a need for nesting habitat in an area lacking suitable islands, and if the benefits for the birds will exceed any negative effects of construction of an island to benthic organisms and current flow, then an island could be built. However, islands should not be placed in areas where they would be used for recreational purposes during the breeding season, thus eliminating or severely reducing their wildlife value.

In most areas there is no need for more islands for colonial nesting birds or other forms of wildlife. Management of existing islands should be given first priority. There are areas, however, where additional nesting habitats would be beneficial to sea and wading birds and existing dredged material and natural islands are not available to fulfill that need. Establishment of need should be determined by consultation with knowledgeable wildlife biologists or by field studies. Generally, construction of new islands for wildlife will not be feasible unless it can be demonstrated that the anticipated positive impacts on the target species will outweigh any negative impacts on the environment. However, it would be desirable to construct a limited number of new islands in various regions of the United States for study purposes and to obtain baseline data. As more natural away without the protective sandbag dike.

Lewis and Dunstan (1975) proposed a design for artificial islands for Florida wading birds, and this design was incorporated into the dredged material disposal plan for the Tampa Harbor Deepening Project. Two islands were built through use of this design (Lewis 1977); this design is similar to the same one used by Wilmington District and recommended by Soots and Landin (1978). In addition to these two new islands, an extension was added to Sunken Island, an existing dredged material island and a National Audubon Society sanctuary, in Tampa Bay in December 1977. The site was used during the first breeding season by royal terns (Sterna maxima), least terns, and black skimmers since bare sites are extremely scarce in the area. The habitat has remained the same through Audubon management efforts and in 1978 the terns once again nested, but in 1979 only laughing gulls (Larus atricilla) nested there and the terns were not present (R. R. Lewis, III, personal communication, 1979, Mangrove Systems, Inc., Tampa, Florida). This was probably due to the aggressiveness of the gulls and the fact that they are predators of tern nests. Any new islands built in the Tampa Bay area or on the west coast of Florida will probably result in heavy use by ground nesting birds if it can be managed as bare ground habitat and protected from predators.

Once the need and feasibility of development of a new island has been established, the next step to be considered is the actual design of the island. Three basic aspects must be considered: site location, timing of development, and physical design.

Site location of an island should be worked out with knowledgeable wildlife biologists and concerned agencies to establish the best location. Building an island in an area that does not conform to the biological and engineering specifications outlined in this paper would fail to produce the desired wildlife habitat. The islands must be placed where the birds will be isolated from predators and sites are taken over by man, strategic placement of new sites may become more valuable as a management tool. The present knowledge of bird utilization is based solely on empirical observations of existing dredged material islands, and more baseline data are needed.

In addition to establishment of need, the feasibility of new island construction will be dependent on the concerns of Federal and State agencies and the private sector. These concerns vary considerably among the regions of the country. However, it has been proven that construction of new islands for birds and other forms of wildlife is feasible. The WES built an island in the tidal freshwater reaches of the James River in Virginia for purposes of marsh restoration and wildlife use (Cole 1978). The Wilmington Corps District constructed two islands in Core Sound, North Carolina (Anonymous 1977). These two islands were unique in that they were the first to be constructed and placed in a manner to deliberately create habitat for colonial sea birds and aquatic life, and they were retained by the use of large nylon sand-filled bags. The sites were designed so that during future maintenance dredging of the nearby navigation channel, materials may be added to them within the existing sandbag retainers, and more sandbags may be added to create higher retainment dikes. The kidney shape of the islands form a small cove where it is expected that a marsh will develop and benthic organisms will thrive. The marsh was given a boost by the planting of smooth cordgrass (Spartina alterniflora) and saltmeadow cordgrass (Spartina patens) around the perimeter. The islands were placed in an area with adequate shallow water and food resources but with a scarcity of bare ground nesting habitat. Gull-billed terns, common terns (Sterna hirundo), least terns, and black skimmers (Rynchops niger) nested on the islands during the first breeding season after construction (Parnell et al. 1978). Unfortunately, in 1979 vandals cut the sandbags of one of the islands in Core Sound with knives. They succeeded in destroying the island because it had not yet stabilized and it eroded human disturbances unless the islands are going to be actively protected by wardens. With active protection, colonies of sea and wading birds have been successful close to human activities and have provided tourist attractions that could be observed from outside the colony (Collins et al. 1979 and Landin 1978b).

Timing of island development is important. Ideally an island should be built during the fall or winter preceding the initiation of the next breeding season. The birds generally do not use a site until after the initial sorting of fine materials by wind and water. If it is built in the spring, this sorting will not have had time to take place, and any colony of birds trying to nest there will not be successful. Frequently, their eggs will be covered by drifting fine material. In addition, they cannot use a site of silty dredged material until it has had adequate time to dewater.

The physical design of an island is important. In general, islands must be permanently emergent at high water levels; birds have been found nesting on all sizes and shapes of islands as long as they met this crucial breeding requirement. However, observations of hundreds of bird colonies on dredged material islands and the kinds of islands they select has led to the following four categories of recommendations concerning size, configuration, substrate, and elevation.

Ideally, new islands should be no smaller than two hectares and no larger than twenty hectares; however, birds have been found nesting on both smaller and larger islands. Islands larger than twenty hectares would be difficult to manage and would be more likely to support predator populations such as coyotes, snakes, foxes, and feral cats and dogs. Islands between the two extremes could be easily managed and considerable habitat diversity could be achieved on them. Generally, the greater the amount of habitat diversity to be maintained for wildlife populations, the larger the island should be.

The configuration of an island will depend on the target wildlife species. Steep slopes such as those found on dikes should be avoided for all species. A slope no greater than a one-meter rise per thirty linear meters has been recommended by Chaney et al. (1978). Substrate configurations for the ground nesting species are shown in Table 2. Two species which are shown on the table may be used to illustrate the importance of slope: royal terns must have access to the water for their young, either by removal of the dike or by creation of a sloped traffic lane down to the water's edge; least terns will not nest on sites that are steeply sloped and will not nest on flat areas. Both species must have bare ground habitat with little or no vegetation that is gently sloped to prevent eggs rolling from nest scrapes. Further data on the effect of diking on bird use is presented by Parnell et al. (1978) in the North Carolina DMRP Task 4F study. There is also some evidence that the formation of a bay or pond with the island makes it especially attractive to nesting birds.

The general nesting substrate of colonial bird species is shown in Tables 3 and 4. Some substrates are preferred over others by certain species. Generally, coarser material such as sand or cobble make better nesting substrates due to their greater stability. Fine material such as silts and clays are subject to wind and rain erosion, and usually have the added problems associated with desiccation cracks, settling, and ponding. A mixture of sand and shell material makes good nesting substrate for most of the ground nesting birds which prefer bare ground areas where these species of birds historically nested before being forced off by human use. Fine, unstable dredged material may be stabilized to form suitable nesting substrate by adding coarse materials such as shells over its surface or by planting a ground cover on the material to provide vegetation for those species which prefer that kind of habitat, such as the Forster's tern (*Sterna forsteri*). Tree nesting species obviously prefer woody substrates, and these trees and shrubs

Table 2

Preferred Configuration of Nesting Substrates for Ground Nesting Species on Dredged Material Islands

Species	Substrate Configuration					
	Macrotopography			Microtopography		
	Flats	Slopes	Domes	Ridges	Lumps	Other
White pelican	✓		✓			
Brown pelican	✓		✓			
Glaucous-winged gull	✓	✓	✓			✓[a]
Great black-backed gull		✓	✓	✓		
Herring gull		✓	✓	✓		
Western gull	✓	✓	✓			✓[a]
Ring-billed gull	✓	✓	✓			
Laughing gull	✓	✓				✓[b]
Gull-billed tern	✓	✓	✓	✓	✓	✓[a]
Forster's tern	✓					✓[b]
Common tern	✓	✓	✓	✓	✓	✓[b]
Roseate tern	✓	✓	✓	✓	✓	
Least tern	✓	✓		✓	✓	
Royal tern		✓	✓			
Sandwich tern		✓	✓			
Caspian tern	✓		✓	✓		
Black tern	✓					✓[a]
Black skimmer	✓	✓		✓		

[a] Shows a preference for nesting on or adjacent to debris in the colony site.

[b] Frequently nests on drift material left by lunar or wind tides.

✓ Denotes occurrence of preferred configuration for each species.

(From Soots and Landin 1978)

Table 3

Common Interspecific Associations of Ground Nesting Colonial Waterbirds Found Breeding on Dredged Material Islands in Six of the Seven Study Regions

White pelican																
Glaucous-winged gull	0															
Great black-backed gull	0	0														
Herring gull[a]	0	0	+													
Western gull	0	+	0	0												
Ring-billed gull	0	0	0	+	0											
Laughing gull	-	0	-	+	0	0										
Gull-billed tern	-	0	-	-	0	0	-									
Forster's tern	-	0	-	-	0	+	-									
Common tern	-	0	-	-	0	-	-	+	-							
Roseate tern	0	0	-	-	0	0	-	-	-	+						
Least tern[b]	-	0	-	-	0	0	-	-	-	-	-					
Royal tern	-	0	-	-	0	0	-	-	-	-	-	-				
Sandwich tern	-	0	-	-	0	0	-	-	-	-	-	-	+			
Caspian tern	-	-	-	-	-	-	-	+	-	+	-	-	+	+		
Black tern	0	0	0	0	0		0	0	0	0	0	0	0	0	-	
Black skimmer[b]	-	0	-	-	0	0	-	+	-	+		-	-	-	+	0

\+ Found nesting together in mixed species colonies.

\- Breeding ranges overlap but were not found in intermixed colonies.

0 Breeding ranges do not overlap in the study regions.

[a] Has been observed nesting in trees and on man-made structures.

[b] Has been observed nesting on flat, gravel-covered rooftops and paved parking lots.

(From Soots and Landin 1978)

Table 4

Common Interspecific Associations of Arboreal Nesting Colonial Waterbirds Found Nesting on Dredged Material Islands

Brown pelican[a]																	
+	Double-crested cormorant[a,b]																
0	0	Olivaceous cormorant[b]															
+	+	0	Anhinga														
-	+	+	+	Great blue heron[b,c]													
-	-	+	-	-	Green heron[c]												
-	-	?	+	-	-	Little blue heron											
-	-	+	+	-	-	+	Cattle egret										
-	-	-	+	-	-	+	+	Reddish egret[c]									
-	-	+	+	+	-	-	-	+	Great egret[b,c]								
-	-	+	+	-	-	+	+	+	+	Snowy egret							
-	-	-	+	-	-	+	+	+	+	+	Louisiana heron[c]						
-	-	+	+	-	-	+	+	+	-	+	+	Black-crowned night heron[c]					
-	-	0	+	-	-	+	+	+	-	+	+	+	Yellow-crowned night heron				
-	0	0	0	+	-	+	+	+	-	+	+	-	0	White-faced ibis			
-	-	0	+	-	-	+	+	+	-	+	+	+	-	0	Glossy ibis[c]		
-	-	?	+	-	-	+		+	-	-	-	-	-	+	+	White ibis[c]	
-	-	+	+	-	-	+	+	+	-	+	+	+	+	0	+	+	Roseate spoonbill

\+ Close association.

\- Nest on same island but not usually intermingled.

0 Were not found nesting on the same island together.

? Insufficient data to make an evaluation.

[a] Frequently nests on the ground.

[b] Nests on man-made structures.

[c] Occasionally nests on the ground.

(From Soots and Landin 1978)

often colonize best on silty, more fertile substrates. Selected plant species of shrubs and trees which are discussed later in this paper may be planted on the sites since there are several plant species which seem to be preferred over others by tree nesting birds. If plant propagation is to be a part of a management scheme, these species should be given first consideration.

Elevations of constructed islands should be high enough to prevent flooding of the areas that could be used by wildlife, especially for nesting. However, elevations do not need to be so high that the substrate will not become stabilized due to wind erosion. Examples of this are evident in the Cape Fear River in North Carolina and the Upper Mississippi River where sandy dredged material was piled so high above the water table that no vegetation will grow, and the islands have the appearance of giant sand dunes which cause sand drift problems on adjoining areas or blow back into the shipping channel where they have to be redredged. Generally, the optimal elevation for an island is between one and three meters. The desirable elevation to be achieved will depend on texture of the exposed dredged material, wind exposure, and the habitat objectives or target species. Coarser materials may stabilize at higher elevations than finer materials. An example of this is Monks Island in the North Carolina Intracoastal Waterway which has an elevation of 3.8 meters. It supports the largest tern colony in the state, and they were successful there because the sand and shell was coarse enough to provide stability even though the island would have been considered too high in other circumstances. If islands could be constructed of coarser material for ground nesting birds, then it would be acceptable in some cases to exceed the recommended elevation. In general, the higher the elevation, the slower the island will be colonized by plants. Therefore, lower elevations to achieve plant cover for some ground nesting species and all tree nesting species should be considered where those are the target wildlife species and where substrates are of fine textured material. It should be remembered that given the proper substrates and vegetation for nesting, that none of the species using dredged material islands for nesting choose one elevation over another as long as they are above the tide or flood lines.

Dredged Material Island Additions

Additions to islands may be a useful management tool if valuable nesting sites are altered by erosion until they have to be abandoned eventually. Additions to such an island will prolong its usefulness as nesting habitat. Additions to islands which are covered with vegetation will increase habitat diversity by providing some bare ground habitat, at least temporarily, for those forms of wildlife requiring bare ground. In south Florida, additions may be done in such a manner that encourages the growth of mangroves, which provide excellent nesting sites for wading birds (Lewis and Lewis 1978).

The author has observed colonization of additions to existing islands on many occasions, and several of the DMRP Task 4F reports recommended additions to specific islands in those regions addressed. Texas, Florida, and North Carolina, as well as other states along the Atlantic and Gulf coasts, would benefit from additions made to islands in those areas. In general, bare ground habitat that is undisturbed by humans and thus available for nesting birds and other wildlife is scarce along the entire Gulf and Atlantic coasts.

Protection of Bird Colonies

Since the primary users of dredged material islands are the sea and wading birds which nest in colonies, and the lack of isolation and protection is one of the primary problems these birds face, this group of species would be greatly benefitted by the provision of protection of colonies and resting areas. They are already protected under the U. S. Migratory Bird Treaty Act and its amendments. In addition, some states have laws and regulations designed to give protection. It has been shown over and over throughout the United States in numerous reports that, in general, protected colonies are successful and unprotected colonies are not. Every federal and state agency and individual has the responsibility to see that its actions are not in violation of laws which protect nesting birds. To insure compliance with the law, maintenance operations involving placement of dredged material should be conducted in a manner which will not disturb the bird colonies. Management should include proper care during placement of dredged material, surveying, and dike construction.

Public education concerning the vulnerability of colonial nesting birds has the potential of being a valuable management tool. Through various public affairs channels, the general public could be made aware of the value of dredged material islands to colonial birds. At the same time they could be informed that the continued disposal of dredged material may be a viable management option.

Other protective measures of colonies which are valuable management tools include posting of colonies with signs, fencing, designation of certain colonies as sanctuaries, limiting of scientific study (and thus disturbance of the birds by constant surveillance), and control of wildlife predators such as raccoons, foxes, and feral animals.

Vegetation on Dredged Material Islands

There are a number of plant species that could be planted on new or existing dredged material islands that would increase the island's attractiveness to wildlife and especially to colonies of nesting sea and wading birds. These plant species are named and specific growth habits are outlined in Soots and Landin (1978) and shown in Table 5. To date, little attention has been paid to planting vegetation on dredged material islands to be used for wildlife habitat; rather, the purpose of planting has been to stabilize the substrate and prevent wind and wave erosion (Woodhouse et al. 1972; Lewis and Dunstan 1974a, 1974b). The DMRP's

Table 5

Plant Species that can be Planted on Dredged Material Islands for Colonial Waterbird Nesting Habitat

Species	Occurrence Range*	BS	SH	MH	DH	HS	ST	SF	F**
Saltmeadow cordgrass (*Spartina patens*)	6		x						
Seaside pasapalum (*Paspalum virginatum*)	2		x						
Saltgrass (*Distichlis spicata*)	6		x						
Evening primrose (*Oenothera humifusa*)	5,6		x						
Camphorweed (*Heterotheca subaxillaris*)	2,5		x						
Horseweed (*Erigeron canadensis*)	2,5		x						
Beach pea (*Strophostyles helvola*)	2		x	x					
Sedge(s) (*Carex* spp.)	5,6		x	x					
Rush(es) (*Scirpus* spp.)	5,6		x	x					
Smartweed(s) (*Polygonum* spp.)	5,6		x	x					
Fescue(s) (*Festuca* spp.)	5,6		x	x	x	x			
Knotweed(s) (*Polygonum* spp.)	5,6		x						
Spurge(s) (*Euphorbia* spp.)	5,6		x	x					
Sea ox-eye (*Borrichia frutescens*)	2		x						
Sea blite (*Suaeda maritima*)	2		x						
Dog fennel (*Eupatorium capillifolium*)	5,6			x	x	x			
Scotch broom (*Cytisus scoparius*)	4,5			x	x	x			
Broomsedge (*Andropogon virginicus*)	5,6			x	x	x			
American beachgrass (*Ammophila breviligulata*)	6			x	x	x			
Wild rye (*Elymus virginicus*)	5,6			x	x	x			
Sea oats (*Uniola paniculata*)	2			x	x	x			
Peppergrass (*Lipidium virginicum*)	5,6		x	x	x	x			
Croton (*Croton punctatus*)	5,6		x	x	x	x			
Purple top (*Tripalsis purpurea*)	5,6			x	x	x			
Beach panic grass (*Panicum ararum*)	6			x	x	x			
Reed canary grass (*Phalaris arundinacea*)	5,6			x	x	x			
Goldenrod(s) (*Solidago* spp.)	5,6				x	x			
Ragweed(s) (*Ambrosia artemisiifolia*)	5,6				x	x			
Switchgrass (*Panicum virginatum*)	2,5				x	x			
Marsh elder (*Iva frutescens*)	2					x	x	x	
Groundsel tree (*Baccharis hamilifolia*)	2					x	x	x	
Wax myrtle (*Myrica cerifera*)	2					x	x	x	x
Bayberry (*Myrica pennsylvania*)	3					x	x	x	
Shrub verbena (*Lantana camara*)	2,5					x	x	x	
Wild indigo (*Baptisia leucophaea*)	2,5					x	x	x	
Yaupon (*Ilex vomitoria*)	2					x	x	x	x

(Continued)

* 1 = extreme southern United States (freeze intolerant), 2 = mid-south (south of Virginia), 3 = northern United States only, 4 = western United States only, 5 = freshwater conditions only, 6 = entire Utites States.

** BS = bare substrate, SH = sparse herb, MH = medium herb, DH = dense herb, HS = herb-shrub, ST = shrub thicket, SF = shrub-forest, F = forest.

(From Soots and Landin 1978)

Table 5 (Concluded)

Species	Occurrence Range*	BS	SH	MH	DH	HS	ST	SF	F**
Huisache tree (*Acacia smallii*)	4,5					x	x	x	x
Brazilian pepper (*Schinus terebinthifolius*)	1					x	x	x	x
White mangrove (*Laguncularia racemosa*)	1					x	x	x	x
Red mangrove (*Rhizophora mangle*)	1					x	x	x	x
Black mangrove (*Avicennia germinans*)	1					x	x	x	x
Oleander (*Nerium oleander*)	5,6					x	x	x	
Eastern red cedar (*Juniperus virginiana*)	2,5					x	x	x	x
Live oak (*Quercus virginiana*)	2							x	x
Saltcedar (*Tamarix gallica*)	2,4						x	x	x
Sand pine (*Pinus clausa*)	2							x	x
Loblolly pine (*Pinus taeda*)	2,5							x	x
Hackberry (*Celtis occidentalis*)	5,6							x	x
Australian pine (*Casuarina equisitifolia*)	1							x	x
Eastern cottonwood (*Populus deltoides*)	5,6							x	x
Peachleaf willow (*Salix amygdaloides*)	3,5					x	x	x	
Nutsedge(s) (*Cyperus* spp.)	5,6		x	x					

Habitat Development Project established plantings of herbaceous vegetation for wildfowl grazing (Crawford and Edwards 1978), and completed a study of available and feasible plant material for use on upland dredged material disposal sites for stabilization and for general wildlife habitat (Coastal Zone Resources Division 1978; Hunt et al. 1978; Landin 1978b, 1978c). Depending upon the wildlife species' specific requirements, a variety of suitable plants could be incorporated into a management plan for dredged material islands. For ground nesting species using bare substrate, no plantings would be necessary; however, some ground nesting species use sparse, medium, or dense herbaceous cover, and plantings of the selected species listed in the above reports would be recommended. Tree nesting species would need to have habitats provided for them if the regions where the islands are being built have limited amounts of the desired habitat. It is frequently desirable to plant tree and shrub species on islands so that the desired habitat can be achieved much faster, and so that the species preferred by the target wildlife species will be the ones that are most abundant on the islands. Woody habitat will require several years (5-30) to develop on islands, and in some regions may never develop due to the climatic conditions of the regions (Chaney et al. 1978).

Another aspect of vegetation on dredged material islands is that sometimes vegetation must be controlled in order to provide the proper or desired habitat for target wildlife species. This would be necessary if habitat for ground nesting species was scarce and there was an abundance of other available habitats. Some of these areas could be cleared of vegetation to provide bare ground or sparse herbaceous cover to allow nesting by the target wildlife species. This would also be necessary when undesirable tree and shrub species have colonized islands, and are not of use to wildlife. An example of this is where the exotic tree Australian pine (Casuarina equisitifolia) has become heavily colonized on dredged material islands in Florida. It is of little use to any wildlife on these islands but nesting double-crested cormorants (*Phalacrocorax auritus*) and even these species prefer other trees for nesting before the exotic pine. Efforts to control the tree by National Audubon Society on its sanctuary islands have already begun using chain saws for removal of trees (R. R. Lewis, III, personal communication, 1979, Mangrove Systems, Inc., Tampa, Florida).

Some of the methods of vegetation control that have been successfully tried on dredged material islands include mechanical removal (tractors, tillers, chain saws, axes), hand removal (pulling up plants by their roots), controlled burning, and applications of herbicides. Controlled burning is not very successful in that new growth of herbaceous plants usually begins immediately after burning is completed. Herbicides should be carefully applied according to directions, and they have been found to be extremely effective on islands in North Carolina (Worsham et al. 1974).

Development and Management Problems

There are numerous potential problems that may be encountered in the building and/or management of dredged material islands. A key to success in the early planning stages is cooperation from federal, state, and local agencies with regulatory authorities. Federal and state agencies whose primary responsibility is the protection of fisheries resources frequently express concern that island development could cover aquatic feeding and spawning areas. Such problems could be eliminated or lessened through careful planning to locate the island away from productive aquatic areas or by developing an existing island without expanding its size.

Agencies concerned with preservation of natural areas may prevent dredging in parks and wilderness areas. When dredging is necessary in such areas to maintain channels, the placement of the dredged material is often a major item of contention. Management of existing islands in such areas is possible and may be a satisfactory alternative disposal technique.

State game and fish agencies are concerned primarily with game animals because their source of revenue is usually derived from the sale of hunting and fishing licenses. The management of nongame animals has received low priority in many states, a situation that may hinder cooperation in island development and management. In addition, local concern because of desires for fisheries and recreational uses of waterways and islands may cause problems. Many of these problems can be anticipated and be resolved through educational programs and public meetings.

Planning and Engineering

The development of specifications for dredged material disposal to develop islands for sea and wading bird habitat and simultaneously satisfy the need to dispose of a given amount of dredged material requires considerable care for an agency such as the Corps. Dredging contractors should be given specific instructions as to exact locations, time of disposal, size of deposit, elevation of deposit, and movement of disposal pipes in order to assure that plans are carried out. Onsite monitoring is highly desirable and necessary when the disposal is onto an island that has a bird colony or other highly vulnerable wildlife species.

Specifications such as silt curtains or temporary dikes may be required to limit the extent of the spread of dredged material. If a dike is built on an existing island and filled, then the dike should be at least partially removed or breached to allow ground access to the water by young birds. This will generally require the return of earth-moving equipment to the site. Diking disrupts bird use, and it is recommended that dikes be constructed just prior to disposal and be removed or partially removed as soon as possible after filling.

Periodic post-disposal monitoring to determine the after effects of disposal and succession of plant and bird life on the site and surrounding areas will provide useful information for future disposal operations within a given area.

Planners should realize that long-range plans which cover regions with populations of sea and wading bird colonies and numbers of dredged material islands, while highly desirable, are not often accurate. The reason for this is that it is difficult to predict the impacts of such factors as industrial development, erosion, recreational use, changes in wildlife use patterns and populations, and climatic events such as hurricanes. Such plans may include consideration of long-term disposal needs, management of vegetation succession, monitoring of colony and wildlife success, and target wildlife species management. All island management plans should be periodically reevaluated and revisions made to account for changes in habitat and wildlife use, populations, and needs.

Public Awareness and Cooperation

The public is seldom aware of wildlife needs. The public does not know, for example, that fishing or swimming off an island with a wading or sea bird colony will severely damage that colony in a matter of minutes through agitation of the adults and young which results in the young birds fleeing the nests, the adults abandoning the nests, and eggs being exposed to summer heat which can cook them in less than five minutes (Soots and Landin 1978). Education of the public can be achieved in several ways, and would be extremely beneficial. This can be accomplished by posting of large signs at colonies that warn away recreationalists and intruders and at the same time explain in simple words why this is necessary. Signs worded such as "BEWARE. STAY AWAY. WADING BIRD NESTING COLONY. HUMANS WILL DAMAGE THIS COLONY AND ARE NOT WELCOME. BEWARE" would help prevent human intrusion into colony sites except by the worst of vandals and the most persistent of wildlife researchers and bird watchers.

Another means of informing the public is via an information campaign in local newspapers, radio, and television. Public service announcements and programs reach a wide number of people, and can be of real benefit. This means can also be used to inform people about laws concerning protection on nongame migratory birds, since many people are not aware of these laws' existence and consider songbirds and birds other than game species fair targets year-round.

One of the best ways to reach the public is through an information program in local primary and secondary schools. Children have strong feelings about nature and can be quite effective in reaching their parents. Furthermore, what they learn as children will usually stick with them as adults.

Last but certainly not least, the public should be made to feel a part of management efforts and to realize that not only is dredging a necessity to maintain navigation channels, but that properly used dredged material can be very beneficial to wildlife. Positive public opinion regarding disposal operations of dredged material in the United States may improve public acceptance and understanding of dredged material disposal operations, and allow more of this resource to be developed for the benefit of American wildlife.

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CENTER FOR DREDGING STUDIES

by John Huston, P.E.*

ABSTRACT

A need for a center where new ideas and new hardware in dredging technology could be tested, where basic research could be conducted, where special problems could be solved, and where men from the industry could obtain academic training was recognized for many years. To meet this need, a Center for Dredging Studies was established at Texas A&M University.

The Center for Dredging activities is described in detail.

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CENTER FOR DREDGING STUDIES

by

John Huston, P.E.

Whenever I am going to prepare a paper or an article, I always remember an adage someone told me years ago . . . both should be like a woman's skirt - - short enough to be interesting, but long enough to cover the subject. I hope what follows will qualify.

INTRODUCTION

Now the title of this paper indicates I'm going to discuss the Center for Dredging Studies. However, in doing so I am first going to review for you the events leading up to the forming of the Center and why it was created. This will be, you might say, an academic subject. Unfortunately, academics have in the past not been one of dredging's main interests - - to say the least.

As a matter of fact, my own experience in being introduced to dredging bears this out. On signing up I was told to report to a certain dredge at a certain location on a certain day. Upon doing so, I was introduced around and then put to work. From that time on for many years my memory fails me if I was ever given or advised of any available information on dredging.

DREDGING'S ACADEMIC HISTORY

This laissez-faire attitude toward academics was not uncommon in industry in the early days. The concensus regarding education then might best be brought out by Somerset Maugham's story, "The Verger," It tells about a janitor who used to clean St. Peter's church until a young Vicar discovered he was illiterate and fired him. Jobless he invested his meager savings in a tiny tobacco shop, where he prospered, bought another, expanded and ended up with a chain worth $150,000. One day his banker said to him, "You've done well for an illiterate, but think where you would be if you could read and write." Scratching his head the little man replied, "Well, I'd be janitor of St. Peter's Church."

The janitor's reflection was similar to that of the dredging pioneers. They saw the development of the dredge happen through chance, and random trial and error, rather than by plan. Experience was their teacher in the practical aspects of dredging. So they left the academics to others.

This academic syndrome existed in dredging for a long time. Some of it still remains. For example. Compared to other industries of comparable size, the dredging industry still remains near the bottom of the list of time, effort and money invested in study, research and development. Men are still placed upon the dredges and left in an educational vacuum. Application of the fundamentals they may have acquired in school is left to time and chance.

While the other professions, and the trades, have always relied for their propagation upon an apprentice system based upon study along with practice, the dredging industry has almost completely ignored the former.

DREDGING TODAY

Years ago we could drift along in our limicolous life and take things as they came. Today, however, in this age of the computer and the energy crunch, we are having to squeeze every bit we can out of what we have - - not only from our machines, but from our people as well. No longer can men be expected to learn about dredging just by experience.

Today we must look to tomorrow, not just today. We have to study, investigate and develop. To do this we need knowledge.

For this knowledge we must look elsewhere than just to the experience we may get from the deck of a dredge or the end of a pipeline.

Fortunately, in the past few years, we seem to have realized this, and have moved in that direction.

DEVELOPMENTS

The first effort was the conception of the World Dredging magazine. In the summer of 1965, for the first time in history a publication became available where dredgemen could learn of what was happening past their horizons.

With this magazine as a catalyst, the renaissance began. In the spring of 1967 a World Dredging Conference was held, and thirty-one papers were presented. Following close upon the Conference's heels, in the winter of 1967, the World Dredging Organization was formed. Here finally was the fulfillment of the need of dredgemen for a professional organization.

But things didn't stop there. With the founding of the professional organization came the realization that there was also a need for a place where industry could look for dredge-related information. Until then locating dredging information, what little there was, was nearly impossible.

CENTER FOR DREDGING STUDIES

To solve this need it was proposed that a Center for Dredging Studies be established - - a center where new ideas and hardware could be tested, where basic research could be conducted, where special problems could be solved, and where men from the industry could obtain academic training.

Consequently, as a result of conferences between educators, representatives of the dredging industry and officials from Texas A&M University, a proposal was prepared and presented to the Board of Directors of Texas A&M University for their consideration. In June of 1968 the proposal was approved, and the Center for Dredging Studies was officially created. Established in the College of Engineering, under the Department of Civil Engineering, administration and fiscal responsibility were placed under the guidance of the Texas Engineering Experiment Station.

THE UNIVERSITY

The decision for the Center to be located at Texas A&M was based upon several criteria, not all of them academic. Travel-wise, the University is about the easiest University in the U.S. to get to - - and around in. Just 30 minutes north of Houston, its expanding perimeter encumbered only by mesquite and chaparral, it lies sprawling on the Texas plain. Ten minutes after you step down from the flight from Houston you can be anywhere in the University you choose.

Ranking with the best of the country's engineering institutions, Texas A&M probably encompasses as many fields of engineering study as any in the world. Nowhere else in the country is there an assemblage of curriculum, men, knowledge and equipment so closely associated with dredging.

The Center for Dredging Studies is one of the activities of the University's Department of Civil Engineering, and is part of the Coastal, Hydraulic and Ocean Engineering group. Bachelor's, Master's and Doctor's degrees are obtainable in all.

With this environment the Center has been able to contribute measureably to the advancement of the dredging industry. In addition to conducting research, tests, and development for dredging companies, the government, and equipment suppliers, it has provided the dredging industry with an extensive educational program and information service.

RESEARCH AND DEVELOPMENT

In addition to its own hydraulic test equipment, the Center has access to the extensive hydromechanics laboratories of the University. These laboratories cover 15,000 square feet of floor space.

They include such major equipment as two-dimensional wave tanks and channel flumes and long-wave basins of various sizes, equipped with pendulum and push-plate generators.

The Center's own research and development facilities are also located in the hydromechanics laboratories in a recently completed 6600 square-foot addition. Among the equipment available is a large dredge-pump test loop, incorporating an 11-foot by 15-foot reservoir of approximately 800-cubic feet capacity. The pump pipe line test loop consists of three, 180-foot long pipes of 4, 6, and 8-inches diameter. Pressure taps and manometers monitor losses along the pipes' entire length.

The associated instrument system for the loop includes suction and discharge manometers, pressure gages and transducers, volumetric flowmeters, speed indicators, torquemeters and nuclear density meters. The loop system is designed to withstand a vacuum of 29 inches of mercury, making it adequate for cavitation studies.

With these extenstive facilities, the Center conducts basic and applied research to improve dredge-pump efficiency, cavitation performance, drag-arm and cutter efficiency and pump-casing wear characteristics. Studies are made in connection with the geometries, design criteria and standards for the jet and air-lift pumps, and for gas-removal systems. Head-loss information on pipes, elbows, wyes and ball joints is also obtained to improve the design of dredging systems.

EDUCATIONAL PROGRAM

The Center's educational program is comprehensive. Among the services provided are short courses, special courses, seminars, an abstracting service and library facilities.

Short Courses. Each year a regularly-scheduled dredging short course is conducted at the University. Lasting for one week the course covers basic information on dredging. Participants come from all over the world. Attendance at the courses has varied between 45 and 88, with approximately one fifth being from foreign countries.

Special Courses. In addition to the short courses, the Center sponsors special one-semester courses for the dredging industry. Courses on any specific subject at any level are arranged. The only criteria for the formation of these courses is that there be at least ten enrollees.

Each special course is individually designed to fit industry's needs. If a company wants a group of its employees to have information on some particular phase of dredging, a course can be scheduled to suit almost any time-phase on any dredging subject. For instance. A course can be designed to last one week, and be held at any time. If more extensive instruction is desired, this can be accommodated by extending the course over two or more consecutive weeks, or, a three-week course, say, spread across three semesters, a one-week portion being given each semester. As you can see, arrangements are completely flexible.

Lately there has been some thought given to providing "on-location" courses. In such a program the Center would provide instructors who would travel to and conduct a course at a site prescribed by the dredging company -- probably the company's headquarters. Although this program is still tentative, its implementation would present opportunities not previously available.

Seminars. Supplementing the short and special courses are dredging seminars. They are held yearly in rotating order at Houston, New Orleans and the University. These seminars provide the dredging industry with an opportunity to stay abreast of the latest developments in dredging. No limit is placed upon the papers presented, and the subjects have varied from fundamentals to the unique and the abstract.

Abstracting Service. As part of its continuing goal of information dissemination, the Center has established an abstracting service. Abstracts of all past and recent literature having to do with dredging are available to the industry. Catalogued in more than 50 basic areas of dredging, they cover nearly 1000 articles, reports, papers and books germane to dredging. The abstracts are updated and issued monthly.

The Center can provide the full text of any abstract of literature cited. Supplementing the abstracting service is a continually updated dredging bibliography.

Library. Housed in the University's main library, which has a seating capacity of 2000, are the principal research collections. They number in excess of 1,500,000 volumes. This collection, along with the University's regular engineering collection, makes up what is one of the finest technical libraries in the country.

There is also a technical report center. Access to this collection is through a cataloging system delineating author, title, issuing agency, participating agencies, report and contract number. In January, 1975, this collection was officially designated as the Technical Reports Center and was provided with a full-time staff. Areas of emphasis are transportation, oceanography and water resources and development. There are now over 10,000 catalogued reports.

The hydromechanics laboratories also have a special library for the Center, as well as the Coastal, Hydraulic and Ocean Engineering group. The list of international publications include the major water-oriented publications of the Netherlands, United Kingdom, Japan, France and the U.S.A.

PROFESSIONAL DEVELOPMENT

The Center contributes to professional development. Currently under consideration by engineering registration boards throughout the country are new educational requirements for renewal of professional licenses. Briefly, this is a new concept whereby the renewal of licenses will in the future be based upon a requirement of continuing professional development. Two states, Iowa and Minnesota, have already adopted this requirement, and it is expected that the others will soon follow.

To qualify for registration renewal under this new mandate, professional engineers will be required to obtain a specific number of "credit hours" of professional development each year. Three ways are available by which these hours can be accumulated: professional practice itself, formal academic-level study, and participation in the programs and activities sponsored by the technical and professional societies and institutions.

The CEU (continuing educational unit) is the nationally-used measuring unit for professional development credit. Generally speaking, ten hours of professional activity will be worth one CEU. Hence, attending a 1-day, 8-hour course would be worth 4/5th of a CEU, or, for week-long, 5-day short course, 4 CEU's. Attendance and participation in other courses and technical meetings would qualify proportionally.

For those of the dredging profession who are registered engineers, and for those who intend to be, the Center will provide a means to accumulate credits. The educational program of the

Center, in conjunction with our professional association, will provide credits for attendance at short and special courses, seminars, technical meetings and other technical activities.

SUMMARY

Now a long time ago an old country-preacher friend of mine gave me some advice. He told me that whenever I wrote an article or prepared a speech, it should have only three parts . . . (1) tell 'em what you're going to tell 'em, (2) tell 'em, and (3) tell 'em, what you've told 'em. I've completed the first two parts, so the third now follows.

Some forty-plus years ago when I first was exposed to dredging, I could find little or nothing about it - - there was an informational vacuum. This situation continued unabated until about 1965, when the first issue of World Dredging magazine was published. Things improved further when the World Dredging Conference was organized Following this was the establishment of the World Dredging Organization, and then the Center for Dredging Studies. There was now little else one could ask for. It was dredging's renaissance.

THE FUTURE

There is one thing that yet concerns me. We must be careful that none of these creations are allowed to die, or wither away to uneffectiveness. This could happen, you know. The world is replete with the failures of good intentions.

So, let me say this to you in closing. Support these organizations. Keep up your membership. When possible, present your thoughts, experiences and ideas through the forums available. If you are not heard, the organizations will perish.

Those of you who represent management of dredging companies, equipment suppliers, and manufacturers, and those in government as well, can do more. See that your company or organization is among those who contribute materially - - advertise in the magazine, provide ways for your employees to attend the short courses, special courses, seminars and association meetings - - and, last, but certainly not least, assist financially in the perpetuation of the Center for Dredging Studies. For every dollar given by industry to the Center, the Government will not only match it with two dollars, but will allow the donated dollar to be deducated from the donator's income tax.

If we all contribute in our own possible way, I am sure that the dredging profession's lantern will continue to remain out from under the bushel.

VARIATION OF POROSITY OF OCEAN BOTTOM SEDIMENT AS A FUNCTION OF DEPTH AND TIME

by

Louis J. Thompson[1]

ABSTRACT

As sediment is deposited on the ocean floor its weight loads the sediment buried below it. The buried sediment can only be compressed if the water can be expelled. As the sediment compresses the permeability decreases. If the permeability decreases faster than the sediment compresses the water is entrapped, high pore pressures must result, and consolidation almost stops.

The compressibility and permeability of seven different submarine sediments were measured directly. The maximum vertical stress was 10,000 psi. It was found that a power law function of porosity could adequately describe the intergranular stress and another power law function of the porosity could adequately describe the permeability.

The governing differential equation that gives the relationship between the sediment porosity and the independent variables depth and time has been derived. It is a quasilinear second order parabolic partial differential equation that has a unique solution. Its basis is equilibrium, conservation of mass, Darcy's law and the empirical equations for permeability and compressiblity.

The equation shows that for the very special case where the deposition rate is constant and if the depth is always measured from the moving mudline, the porosity of the sediment is independent of time and a function only of depth.

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FIELD BIOASSAYS FOR DETECTION OF CHRONIC IMPACTS OF DREDGED MATERIAL ON AQUATIC ORGANISMS

W. E. Pequegnat[1]

ABSTRACT

A major problem facing those who must assess the environmental effects of the disposal in the ocean of industrial and municipal wastes, including dredged materials, is determining whether given wastes elicit chronic deteriorative responses in important species of organisms. The full importance of such low-level, nonlethal effects is not known, but it is suspected that repeated elicitations may result in ecosystem changes as important as those caused by more easily determinable acute effects. Such considerations are important to the marine environment, where dumped pollutants may be quickly diluted to legal nonlethal concentrations, but may still bring forth cumulative chronic response patterns.

The principal objective of the present study has been to develop a method of assessing in the field the impacts of the disposal of various industrial and municipal wastes. The gauge of the significance of the impact is not mortality measures against time, but the increase or decrease in concentration of certain metabolic enzymes that signal whether an organism is under stress from a class of wastes. Also, by analyzing tissues of test and indigenous species for the accumulation of metals, PCBs, and high molecular weight hydrocarbons as well as for the enzymes, one gains an insight into the actual effect, if any, of the accumulation upon the whole organism. The test organisms are exposed for selected

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periods of time in the field in devices called Biotal Ocean Monitors (BOMs) of which there are two types, the free-floating Pelagic BOMs (P-BOMs) and stationary Benthic BOMs (B-BOMs). After exposure in test and control BOMs, the housed organisms, which range from bivalves and polychaetes to pelagic and demersal fishes, are assayed for enzyme induction.

At present we use the enzyme ATPase, which is found in cell mitochondria and responds particularly to excess biphenyls in the environment; catalase that is dissolved in the cytosol and responds to excesses of toxic metals; and cytochrome P-420 and P-450, which respond to cyclic and long-chain hydrocarbons. In addition we are studying the applicability of the adenylate energy charge system to this problem. The Biotal Ocean Monitoring System has been applied with promising results to analysis of impacts of municipal and industrial wastes, dredged material, and high-salinity brine discharges in the western Atlantic Ocean and Gulf of Mexico.

INTRODUCTION

Regulatory Call for Bioassays

In the United States during the past two years, laboratory bioassays have assumed a major role in regulating the disposal of varied wastes into marine waterways. The regulating agent is the Environmental Protection Agency (EPA), which is directed to protect the integrity of U. S. ocean waters in Section 102 of Public Law 92-532, the Marine Protection, Research, and Sanctuaries Act of 1972 (the Act). Accordingly, EPA issued the final version of regulations and criteria for ocean dumping in the Federal Register, Vol. 42, No. 7, 11 January 1977. Section 227.27(b) of the regulations calls for bioassays of most waste materials prior to issuance of a dumping permit. In May 1976, EPA published a manual which was revised in 1978 (U. S. EPA, 1978) detailing methods for bioassaying in the laboratory wastes other than dredged materials prior to issuance of a dumping permit. In July 1977 a manual describing techniques for bioassaying dredged materials was published jointly by EPA and the Corps of Engineers (U. S. EPA and U. S. Army Corps of Engineers, 1977).

Validity of Bioassay Results

Some difficulties have been encountered by various laboratories in attempting to carry out these bioassays. This has been particularly true of so-called solid phase bioassays of dredged materials. Partly this has been related to the particular species of organisms used in the tests. And certainly there is as yet no unanimity of opinion as to the most appropriate species for these tests. Other difficulties with survival have been related to the fact that most laboratories were equipped to work only with static rather than flow-through systems. Granted that flow-through systems may be superior to static systems and that other technical problems can be overcome, there are still more fundamental doubts that the ecological bioassays carried out in the laboratory are measuring what we need to know about dumping practices. Except in a few ideally situated laboratories, it is unlikely that laboratory tests can simulate field conditions. Yet major decisions on dumping, involving in some cases hundreds of thousands of dollars, have to be made from less than ideal information.

Potential of Field Bioassays

At present, mortality is the end-point of most laboratory bioassays. Those who question whether this is the proper gauge of the acceptability of materials for dumping or of the way in which the dumping is carried out cite the real possibility that some wastes approved for dumping may elicit chronic deteriorative responses in important species of organisms. We are uncertain as to the ultimate importance of such low-level, nonlethal effects on the biota of the receiving ecosystem, including man. Still such possibilities are important to those managers of the marine environment who feel that even though dumped pollutants may be diluted to legal, nonlethal concentrations in prescribed time periods they are still capable of evoking cumulative, chronic response patterns in important organisms. Three technical changes in the bioassay routine show promise of overcoming most of the present problems with laboratory bioassays. These are, first, that the bioassays be conducted in the field at the dumpsite during actual dumping operations; second, that indigenous species be exposed alongside of usual bioassay species; and, third, that metabolic enzymes be utilized to detect the potential of a waste to elicit chronic responses in test organisms.

Study Objectives

Stimulated by the above considerations, the principal objective of the present study has been to develop an early warning system to reveal when organisms are being seriously stressed by particular wastes and disposal practices well before fatality occurs or is inevitable. Chronic effects cannot be gauged by mortality unless tests are carried out for such long periods that costs become a controlling factor. Thus, if chronic effects impinge seriously upon human welfare, it is to our advantage to develop *in situ* tests that can be carried out in a short time, and in which the severity of the impact is measured by changes in the induction of selected metabolic enzymes that will signal that test organisms are under stress as compared with controls. Since most enzyme levels cannot be measured in the field, the method that TerEco Corporation has developed over the past five years is actually a field/laboratory technique.

MATERIALS AND METHODS

Development of Biotal Ocean Monitors

During the course of monitoring the physical impacts of the ocean incineration of organochlorine wastes by the incinerator ship M/T VULCANUS in

1974 and 1975 (Wastler et al., 1975), TerEco Corporation saw the need for a free-floating device in which organisms could be held in contact with the ambient environment containing waste products long enough for the possible development of subtle metabolic responses that would foreshadow chronic impacts. Since incineration was carried out in deep water, it was concluded that only organisms living in the water column would be impacted by unburned organochlorines carried to the sea surface in the plume of slack gases. Hence the floating monitoring laboratories that TerEco designed were called Pelagic Biotal Ocean Monitors or P-BOMs (Figure 1). When it was evident that the pelagic units formed the basis for a viable monitoring program, work was started in 1976 on a very different type of BOM that could be deployed to monitor high-density wastes such as dredged material that would tend to impact benthic organisms. These units were named Benthic Biotal Ocean Monitors or B-BOMs (Figure 2). Both units are not only equipped to hold a wide range of species, but are also designed to capture and hold indigenous species (Pequegnat et al., 1978 and 1979).

Field Activities

The field tests during which organisms were exposed to polluted dredged material and sewage sludge were conducted in the northwestern Atlantic Ocean off the coasts of New York and New Jersey in the New York Bight (Figure 3). The test organisms in the pelagic units ranged from fertilized sea-urchin eggs and phytoplankton to species of both shellfish and finfish. The master cylinder of the P-BOM is made of monofilament nylon woven into a 2 mm mesh. Smaller organisms, such as phytoplankters and shrimp, are housed in small-mesh nets hung in the master cylinder. Larger fish are free to swim in the large-mesh cylinder. All tests require the launching of a control BOM at an appropriate station outside the polluted zone.

After launching the BOM alongside the ship, each compartment is loaded with sufficient organisms to provide a statistically adequate sample for analysis. Also, as each species is introduced into the BOM, a sample is immediately sacrificed for the laboratory analysis of the pretest condition of the organisms. When loaded the test P-BOM is placed where it will be exposed to the pollutant involved and then, depending on the objective of the experiment, either set adrift or anchored in a place where it will receive further exposure. The control is placed in a position that will remain free of the pollutants for the 5-10 days of the monitoring. When set adrift, the P-BOMs are equipped with radio beacons for easy retrieval.

Loading of the benthic units is done either alongside the ship or, in shallow water, by divers after it has been placed on the bottom. The bottom of the B-BOMs is made of stainless steel mesh (2 mm) allowing the floor to be covered with a veneer of sediment. The drawers in the benthic units are loaded with polychaetes, bivalves, and grass shrimp, whereas the nylon upper compartment contains finfish. The traps in the base generally capture diverse species of fish, crabs, lobsters, and starfish, all of which can be analyzed for metals and hydrocarbons to ascertain what materials are being accumulated in their tissues.

Upon retrieval of both the P-BOMs and the B-BOMs, the organisms are prepared immediately for laboratory analysis. In some cases the whole organism is quick frozen in liquid nitrogen, whereas in others the liver is removed and frozen separately for enzyme analysis and other tissues prepared for trace metal or hydrocarbon analyses. By comparing the concentration of, say, toxic metals such as cadmium in the tissues of individuals of a species living (a) at the dumpsite, (b) held at the dumpsite in BOMs, and (3) individuals of the same species at the control site, one can determine whether or not bioaccumulation of critical compounds is taking place in key species. We know that bioaccumulation is a normal function, but what is not so easy to determine is whether or not the accumulation is stressing the organism. In other words, we are lacking a calibrator; the metabolic enzyme approach promises to fill this need.

Laboratory Analyses

The initial laboratory analyses of organisms exposed to organochlorine wastes in P-BOMs attempted to discern the presence of organochlorine wastes in exposed organisms. Later histological studies of various tissues of exposed and control organisms were added to the routine, but both endeavors required longer periods of exposure than were deemed practicable for monitoring work. Therefore, more subtle means of detecting stress early on in the exposure process were sought. As a result, initial tests were run on a suite of metabolic enzymes that it was anticipated would respond to particular classes of chemicals. The rationale for this search was simply that any living cell maintains itself by extracting energy from its environment and transforming it into chemical energy to drive biosynthetic reactions and many other energy-requiring ractions of which the various specialized cells of the body are capable. All of these activities are mediated through enzymes that are quite functionally specific. It is well known that changes occur at various points in the enzyme systems when an organism is under environmental stress. The initial response may be either an increase or decrease of enzyme levels in cells, say, of the liver. Eventually there may be a failure of the cell function catalyzed by the particular enzyme under study; prior to this the organism is likely to exhibit a chronic nonlethal response. Thus, the disposal of toxic chemical wastes in the aquatic environment, which is often periodic, will serve as the stimulus for chronic stress responses. Between disposals the organism may be able to rid itself of sufficient toxic material to exhibit only nonlethal responses. Even so, its subnormal response is signalling that if it is subjected to an intensification of the disposal activities, by increasing amounts per dump or by shortening the time between dumps, acute and lethal responses may be evoked (Pequegnat, 1978).

Enzymes Selected for Monitoring

After testing a dozen or so enzymes, TerEco selected three for field work. At present we are using ATPase, which is found in cell mitochondria and responds particularly well to excess biphenyls in the environment; catalase, which is dissolved in the cytosol and responds to excesses of toxic metals; and cytochrome P-420/450, which respond to metals but particularly to cyclic and long-chain hydrocarbons. In addition, TerEco has employed the adenylate energy charge system in a few field tests. The advantage of this latter technique, which involves analysis of ATP, ADP, and AMP, is that a complicated set of enzyme reactions are reduced to a single parameter that relates all control methanisms to the energy level of the cell, which is then expressed as the following ratio.

$$E.C. = \frac{\text{ATP} + \frac{1}{2}\text{ ADP (adenosindiphosphate)}}{\text{ATP + ADP + adenosinmonophosphate (AMP)}}$$

The energy charge of a healthy cell centers around 0.85. Only at and above this level can growth and reproduction occur. Viability is maintained between 0.8 and 0.5, but death occurs at levels below 0.5. Chlorinated hydrocarbons and some metals act as inhibitors of the electron transport system enzymes with a consequent lowering of the energy charge ratio.

RESULTS

ATPase Analyses

During the summer of 1978 tests of the effectiveness of ATPase as an indicator of stress were run on fish (*Fundulus grandis*) kept in B-BOMs at the Mud Dumpsite in the New York Bight (Figure 3). During this period dredged material taken from polluted New York Harbor was being dumped three times daily throughout the seven days of the test. B-BOMs were placed upstream of the dumpsite (control), in the site, and one mile downstream of it in the extended impact zone (Figure 4). The results are shown in Table 1.

Table 1

ATPase Levels in *Fundulus grandis* Liver

After 7 Days Exposure in the New York Mud Dumpsite

Location	No. of Fish Analyzed	ATPase (Units*/Mg. Protein Ave.
Upstream B-BOM	33	0.124
In Dumpsite	35	0.040
1 Mile Downstream	32	0.091

*One unit equals the amount of ATPase which will decompose 1 mg. of NADH (Nicotinamide adenine dinucleotide reduced form) in 1 minute.

Tests of sediments showed the presence of PCBs in the dumpsite and well downstream of it. This is reflected in the higher ATPase values in fish held in the dumpsite and downstream BOMs as compared with the upstream site.

Cytochrome P-420 and P-450

Values for this enzyme are expressed in terms of ηMol P-450 per mg of liver protein and ΔOD/min/mg liver protein. Again tests were run at te New York Mud Dumpsite, the Sewage Sludge Dumpsite, and at the control. At the latter dumpsite sewage sludge is dumped by barge several times a day. B-BOMs and P-BOMs were used in the sites and at the control site well east of dumpsites. The results of this summer 1978 test are shown in Table 2.

The cytochrome P-450 levels of the *Fundulus* exposed in P-BOMs are appreciably higher than levels of benthic fish in B-BOMs at both the Control and Mud Dumpsite (MD), whereas at the Sewage Sludge Site (SSS) the two values are relatively close. When compared to the control site values, the SSS fish show similar P-450 values at the pelagic level, whereas benthic levels are significantly higher. Pelagic fish at MD show the highest P-450 levels, whereas benthic fish show levels close to those of the control site benthic fish. Cytochrome P-420 levels are significantly higher in pelagic fish than benthics at both the SSS and MD, whereas at the control site the two values are relatively close. Also, we note that P-420 levels in pelagic fish at both dumpsites are greater than levels at the control. Since cytochrome P-420 is a solubilized

Table 2

Cytochrome P-450 Values from 90 *Fundulus* Livers

Exposed at New York Bight Dumpsite

July-August 1978

Site & Bom Type	nM P-450 per mg Liver Protein ($x 10^2$)	S.D.	P-420 ΔOD/min/mg liver protein ($x 10^2$)	S.D.
Control Site				
P-BOM	0.45	0.10	0.02	0.008
B-BOM	0.17	0.08	0.02	0.015
Sewage Sludge Site				
P-BOM	0.46	0.16	0.07	0.029
B-BOM	0.41	0.09	0.03	0.010
Mud Dumpsite				
P-BOM	1.16	0.70	0.13	0.04
B-BOM	0.25	0.16	0.01	0.003

form of P-450, the high P-420 levels in the pelagic fish of the dumpsites is perhaps indicative of even stronger induction of the P-450 system at these two sites where experimental fish were undoubtedly exposed to relatively high levels of hydrocarbon due to heavy ship traffic.

Catalase

Tests for catalase change in the killifish (*Fundulus grandis*) when exposed to results of the dumping of sewage sludge and dredged material were also carried out in the New York Bight in July-August 1978. The results are shown in Table 3.

The lower values of catalase in fish at the sewage and mud dumpsites compared to the upstream control site reflect stresses on the organism in response to excess toxic metals in these disposal environments.

Adenylate Energy Charge

The following data are the first field data that TerEco has derived from application of an energy charge analysis. They were obtained from the 1978 New York Bight study using the grass shrimp, *Palaemonetes pugio* as the test organism. Data are insufficient to permit drawing firm conclusions, but the following observations can be made from examination of Table 4.

First, all energy charge values are 0.67 or above. Life can be sustained between 0.5 and 0.8, but only at 0.85 can growth and reproduction occur.

Table 3

Catalase Values from 90 *Fundulus* Livers

Exposed at New York Bight Dumpsites

July-August 1978

Site & BOM Type	BU/mg Liver Protein ($x 10^2$)	s.d.
Control Site		
P-BOM	1.20	0.4
B-BOM	1.80	0.3
Sewage Sludge Site		
P-BOM	0.9	0.09
B-BOM	1.4	0.2
Mud Dumpsite		
P-BOM	0.5	0.02
B-BOM	0.9	0.01

Table 4

Adenylate Energy Charge P-BOM and B-BOM Samples

New York Bight Control, Sewage Sludge, and Mud Site

Test Organism: *Palaemonetes pugio*

July-August 1978

Site & Bom Type	Population Segment Wet Weight	Mean Energy Charge
Control Site		
P-BOM	<200 mg	.85 ± .06
	>200 mg	.70 ± .11
B-BOM	<200 mg	.77 ± .12
	>200 mg	.69 ± .11
Sewage Sludge		
P-BOM	<200 mg	.85 ± .05
	>200 mg	.67 ± .04
B-BOM	<200 mg	.80 ± .04
	>200 mg	.73 ± .05
Mud Dumpsite		
P-BOM	<200 mg	.87 ± .04
	>200 mg	.72 ± .05
B-BOM	<200 mg	.77 ± .01
	>200 mg	.68 ± .08

All of the smaller shrimp (<200 mg) averaged an E.C. of 0.85 or above, but the larger shrimp did not. In fact, in all cases the older shrimp had low E.C. values, but not to the point of lethality. Also, the B-BOM values are uniformly lower than those from P-BOMs. In addition to pollutant effects, the lower B-BOM values may also be related to a factor of depth at and below the thermocline where temperature is depressed with increasing depth.

DISCUSSION

Catalase and ATPase

The uses of Catalase and ATPase as measures of response to excess toxic metals and biphenyls in the environment are sufficiently straightforward as to need no further discussion at this time. However, we do plan to maintain them in our Biotal Ocean Monitoring System along with E.C. and cytochrome P-450, which are just now coming into more general use.

P-450 MFO System

Evidence is accumulating that the P-450 mixed function oxidase (MFO) system can be a useful indicator of the sublethal effects of petroleum hydrocarbon pollution on a variety of marine organisms. If so, it will become an important monitor of environmental impacts. Buhler (1966) provided the basis for this by demonstrating that detoxification of organic xenobiotics in fish can be related to hepatic metabolic activity. Since then evidence has accumulated that the cytochrome P-450 dependent

MFO enzyme system is the mechanism by which a variety of environmental pollutants (especially petroleum hydrocarbons and PCBs) are detoxified in the liver's of fish. Components of the cytochrome P-450 linked MFO system are bound to the membranes of endoplasmic reticulum with highest levels in the liver. When analyzed spectrophotometrically, the cytochrome P-450 hemoproteins in a CO-bound reduced state display a Soret absorption maximum peak near 450 nm, while the solubilized P-420 exhibits a Soret peak at 420 nm (Omura and Sato, 1964).

The biotransformation of sublethal concentrations of environmental pollutants by the MFP system is interesting due to the apparent inducibility of the system. Lidman et al. (1976) and Statham et al. (1978) demonstrated induction of increased P-450 and MFO enzyme levels in rainbow trout upon exposure to PCBs and polycyclic aromatic hydrocarbons, respectively. In a study of the estuarine fish Fundulus heteroclitus, Burns (1976) found induction of P-450 in laboratory fish exposed to phenylbutazone and some evidence of elevated P-450 levels in fish from polluted natural environments relative to populations from nonpolluted areas. Of particular interest to us is that Yarbrough and Chambers (1977) showed induction of the MFO system in mullet exposed to crude oil extract for only four days, and Stegeman (1978), monitoring P-450 in natural fish from an oil-contaminated marsh relative to fish from unpolluted environments.

Adenylate Energy Charge

Few reports exist as to the application of the adenylate energy charge system (E.C.) to pollution studies. But the generality of its potential use should make it a very sensitive and useful assay for low-level or chronic responses to toxicants. Brezonik et al. (1975) discuss the rapid response of ATP to additions of toxic substances and suggest the use of ATP in toxicity studies; hence the potential for pollution studies of certain types is well founded. The rapid drop in E.C. ratios noted by Chapman et al. (1971) as a response to oxidative phosphorylation inhibitors would suggest the use of this assay for chlorinated hydrocarbons or PCBs, since Pardini (1971) notes that their toxic effect is observed most readily as inhibition of electron transport system enzymes. Since a ratio is dimensionless, a direct comparison can be made from baseline to test to control conditions and from one collecting station to another without the ancillary sample series required were one to use ATP values alone. Accordingly, the advantages of its use would be its sensitivity for assaying detrimental effects of low levels of toxicants and the simplification of field sampling logistics.

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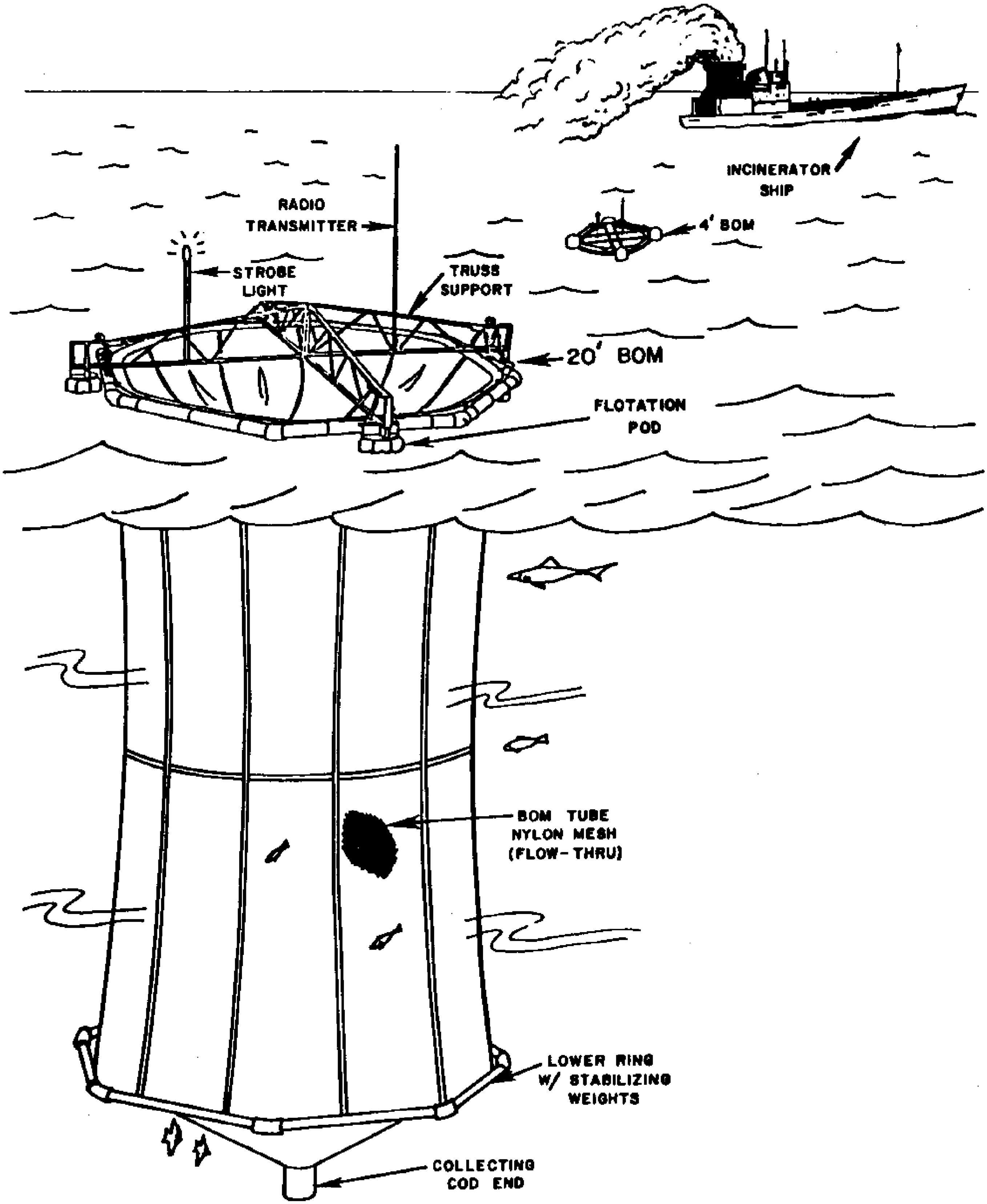

Figure 1. Pelagic Biotal Ocean Monitors (P-BOMs) of two sites (4 and 20 feet in diameter) deployed to monitor the incineration of toxic organochlorine residues of the manufacture of polyvinylchloride and other compounds. The BOMs, both controlled and exposed, contain a variety of organisms that are analyzed after the 5-7 day exposure.

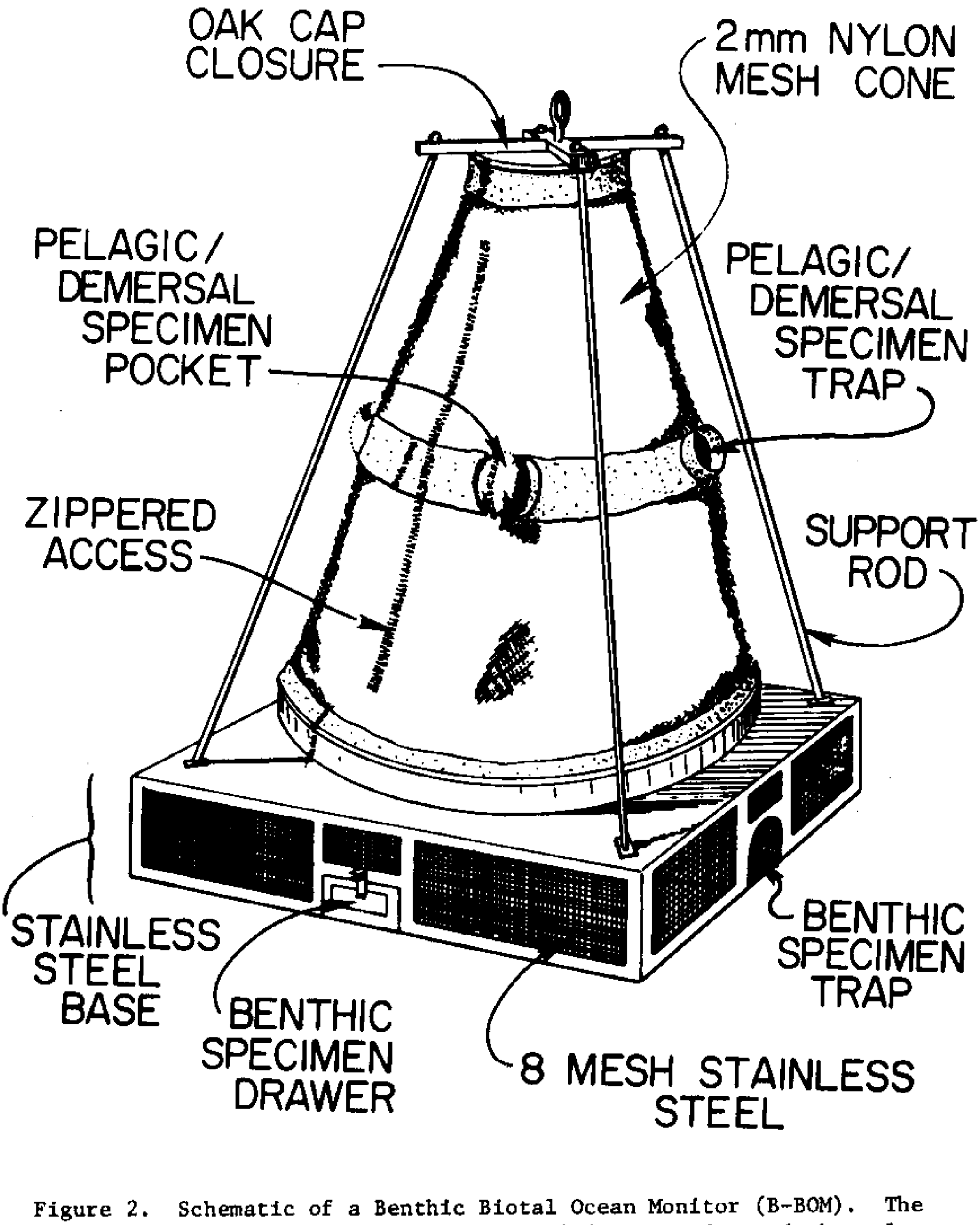

Figure 2. Schematic of a Benthic Biotal Ocean Monitor (B-BOM). The stainless steel mesh base is 4 feet on edge and the nylon mesh cone is 6 feet tall. The device is designed to hold a variety of organisms in steel drawers, nylon pockets, and free spaces. It is effective in trapping and holding apart various indigenous species.

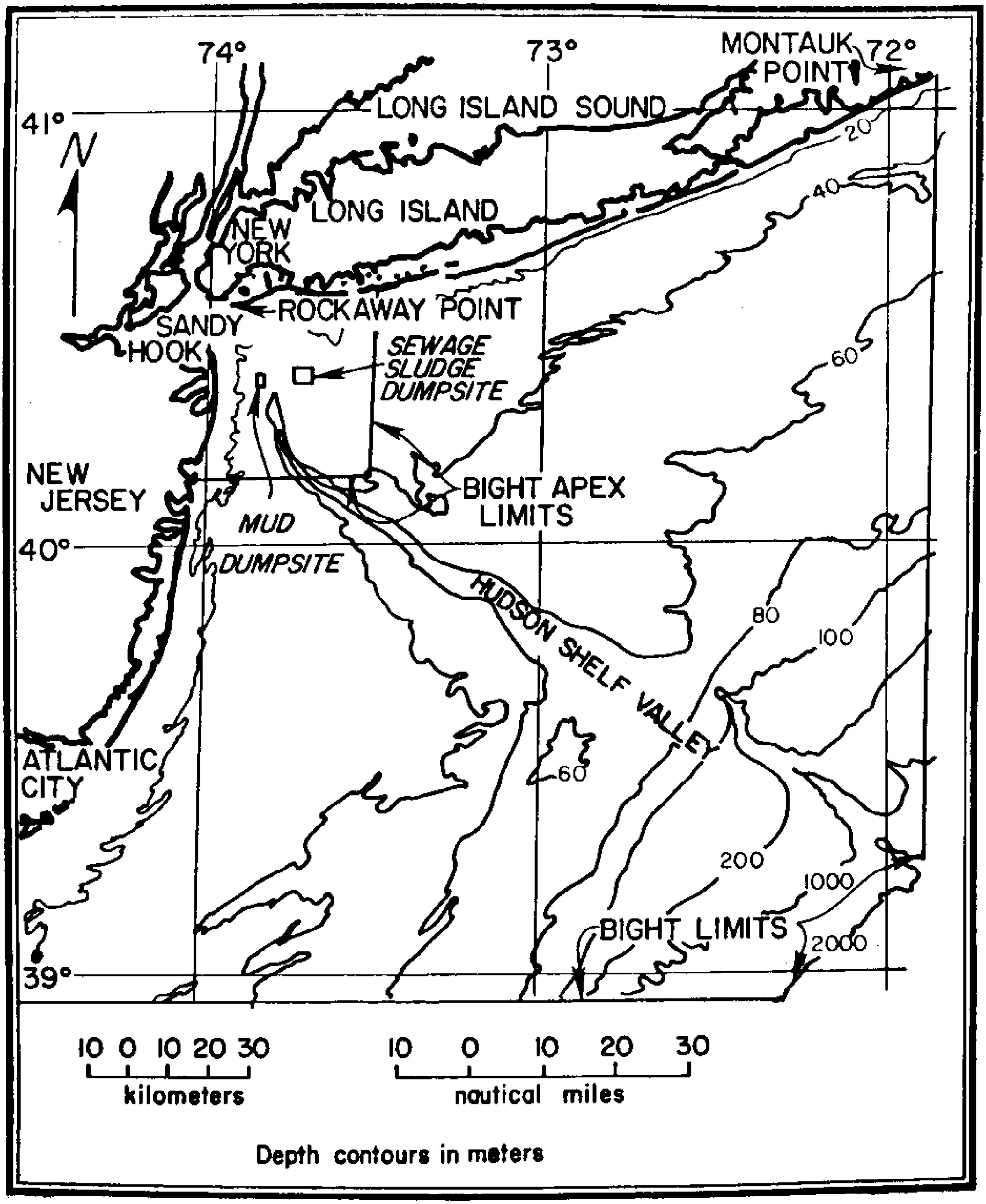

Figure 3. Bathymetric map of the New York Bight showing the location of the Mud Dumpsite and Sewage Sludge Dumpsite where both P-BOMs and B-BOMs were used in monitoring impacts.

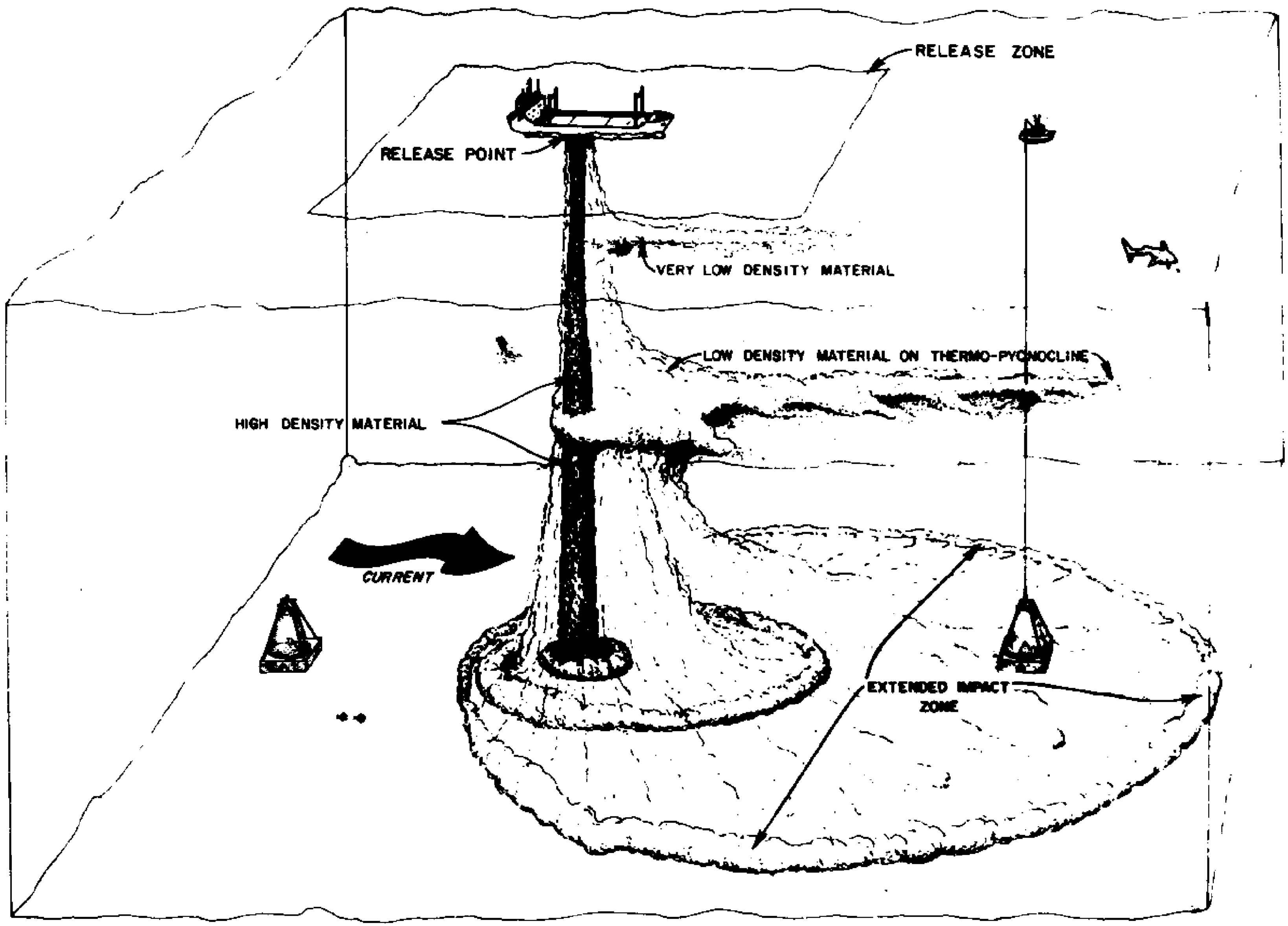

Figure 4. Conceptualization of the fate of dredged material released from a hopper dredge and the placement of control and test B-BOMs to monitor potential impacts.

CONTAMINATION OF JAMES RIVER

BED SEDIMENTS WITH KEPONE

by Charles A. Lunsford[1]

ABSTRACT

The Virginia State Water Control Board, along with various state and federal agencies, has been involved in addressing the Kepone contamination in the James River estuary. This paper discusses the SWCB's involvement in an intensive monitoring program to determine the extent of Kepone contamination of the bed sediments, and a study conducted to investigate the release, translocation and biological uptake of Kepone associated with dredging.

Kepone was detected in bed sediment samples collected over a 72 river mile area. The bulk of the contamination was confined to the top 16.51 cm. The greatest concentrations were detected in the Hopewell area which was the original source of contamination. Deposition sites, downstream of Hopewell, contained fine-grained sediments, rich in organic matter. The distribution pattern indicated that the turbidity maximum zone is a major deposition area for sediments moving seaward.

The dredging study was unable to detect an increase in the Kepone water column residues above ambient levels or demonstrate a biomagnification of Kepone by the test organism, estuarine bivalve, *Rangia cuneata*.

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INTRODUCTION

Historical Perspective

During the period 1966-1975, Kepone, or chlordecone, a persistent organochlorine insecticide similar to mirex was discharged into the James River estuary at Hopewell, Virginia. Kepone manufacture began at Allied Chemical Corporation's Semi-Works Plant in 1966 and continued there until 1974. During this period approximately 1.5 million pounds of Kepone were produced. Production then shifted to Life Science Products, Inc. which produced approximately 1.7 million pounds of the insecticide during 16 months of operation in 1974 and 1975.

The majority of the Kepone produced was shipped abroad to combat the Colorado potato beetle in Europe and the banana root borer in Central America. Its use in the United States was limited to a small volume placed in ant and roach traps at a maximum allowable level of 0.125% Kepone (Battelle, 1978).

In July, 1975, Life Sciences was closed under pressure by the State Health Department because of inadequate employee protection in the production of the toxic compound.

It was estimated by Battelle (1978) that 200,000 lbs. of Kepone was released to the environment through atmospheric emissions, wastewater discharges and bulk-disposal of off-specification batches. Present estimates indicate that there are 20,000 to 40,000 lbs. of Kepone deposited in the top 30 cm of the James River bed sediments (U.S. EPA, 1978).

EPA's Health Effects Research Laboratory in North Carolina conducted an environmental sampling in the James River estuary in late 1975. This was the initial indication of the extent of Kepone contamination in the James River estuary (U.S. EPA, 1978). Widespread contamination of the water column, bed sediments, finfish, shellfish and blue crabs was noted. Finfish, shellfish and blue crabs were found to exceed the United States, Food and Drug Administration's action levels for Kepone.

Due to these findings, in late 1975, the James River and its tidal tributaries were closed to commercial fishing except for the taking of shad/herring (*Alosa spp.*), channel catfish (*Ictalurus punctatus*) and the harvesting of female blue crabs (*Callinectes sapidus*). This was followed by a ban on sport fishing by order of the Governor. Some 157.5 km (85 mi) of the James River were affected by these closures.

The Virginia Kepone Task Force was established in December, 1975 to develop a comprehensive monitoring program with responsibilities divided between the State Water Control Board, State Health Department and the Virginia Institute of Marine Science (Bellanca and Gilley, 1977). This monitoring program focused on drinking water, surface water, groundwater, sediments, non-point source studies, finfish, shellfish and blue crabs. Presently this monitoring program is still being administered.

The State Water Control Board's monitoring efforts have centered on the Kepone contamination of the water column, bed sediments and finfish.

Also in late 1975, because of concern over the potential adverse impacts of dredging and disposal of Kepone contaminated sediments on aquatic resources, the Governor's Office declared a moratorium on dredging in the James River estuary and an Interagency Task Force on Dredging was formed. This Task Force was disbanded by the Governor in January, 1978. Presently, prior to the issuance of 401 dredging certificates, Kepone concerns are reviewed by a Federal/State permit coordination group.

James River Estuary

The James River is the southernmost and third largest tidal tributary of the Chesapeake Bay, in reference to the volume of river flow; the first two are the Susquehanna and Potomac river systems. It is a partially stratified estuary with the salt wedge extending approximately 64 km (40 mi) upstream from the river mouth, this distance varies according to the volume of river flow. Water movement in the estuary is dominated by freshwater flow and tidal action. The tidal reach extends to Richmond, 157.5 km (85 mi) upstream from its mouth at the Hampton Roads bridge tunnel.

Fishery resources in the James River are diverse, productive and include freshwater and marine fish. These resources were described by Battelle (1978). Commercial fishing grounds extend 65 nautical miles upstream to the Hopewell area. The freshwater and upper portion of the brackish water zones are extensively utilized by anadromous fish as spawning and nursery areas. The brackish water zones provide a nursery area for important commercial species that are taken within the Chesapeake Bay.

The lower estuary is a very productive shellfish area. The important commercial species include the hard clam, *Mercenaria mercenaria* and the eastern oyster, *Crassostrea virginica*. The James River is the most productive and the only major source of seed oysters in Virginia.

Another important fishery in the James River is the harvesting of the blue crab, *Callinectes sapidus*. Major nursery grounds are located between river mile 10 and river mile 34. (Va. Institute of Marine Science, 1973 cited by Battelle, 1978).

Sediment Monitoring

The monitoring of bed sediments to determine the distribution of Kepone in the James River estuary was initiated by the State Water Control Board in 1976. The resulting data have been a valuable input into several aspects of the Kepone contamination problem, these being: a) the evaluation by state and federal agencies of the feasibility of removing or stabilizing Kepone contaminated sediments, b) providing information to the Federal/State 401 dredging certificate group to aid in their evaluation of dredging and disposal activities in the James River estuary and c) provide annual comparison of Kepone bed sediment concentrations to detect any unexpected movement of the Kepone reservoir downstream of the turbidity maximum zone towards the lower James River and Chesapeake Bay.

Dredging Studies

Recent dredging projects in the James River estuary, using conventional dredging techniques, have presented an opportunity to monitor the dredging and disposal of Kepone contaminated sediments to determine the effects of the resuspension of such sediments. In July, 1976 a channel maintenance test dredging was conducted as the result of an agreement between the Commonwealth of Virginia and the United States Army Corps of Engineers, Norfolk District Office. Approximately 215,000 cubic yards of shoal material was hydraulically dredged from a freshwater area near Windmill Pt. in the James River and disposed of in an open water disposal site. Windmill Pt. is approximately 7 river miles downstream of Hopewell. This project was monitored by the State Water Control Board (R.A. Gregory, unpublished data). Three species of organisms were used to monitor Kepone uptake. There were: bluegill sunfish, *Lepomis macrochirus*; channel catfish, *Ictalurus punctatus;* and the brackish water bivalve, *Rangia cuneata.* This study was unable to demonstrate that the dredging significantly elevated the Kepone residues of the test organisms.

The Virginia Institute of Marine Science monitored the dredging of Kepone contaminated sediments for two projects in the lower James River. These included: dredging at the Surry Power Station in late 1977, early 1978, and the Skiffes Creek area, access channel to the Fort Eustis complex, conducted in June, 1979.

The most recent field investigation by the State Water Control Board concerning the effects of resuspending Kepone contaminated sediments during dredging was conducted in January, 1979. The dredging project involved the removal of 35,000 cubic yards of Kepone contaminated bottom sediments in Bailey Bay, Hopewell, Va. The purpose of dredging was to remove sediments that had silted in a raw water channel, as well as provide a depth of 26 feet below mean low water at a shipping pier to allow ships access. Dredging was accomplished by a hydraulic dredge, and the dredged materials were transported by pipeline to an open water disposal site in Bailey Bay.

The monitoring of Kepone contaminated bed sediments in the James River estuary from 1976-1978 is discussed in this paper. Also presented is a discussion of the field investigation conducted by the State Water Control Board concerning the dredging of Kepone contaminated sediments in Bailey Bay. The major objectives of this study were to: a) detect if Kepone residues in the water column were elevated above ambient levels during dredging and disposal and b) determine if conventional dredging and disposal methods elevated the Kepone levels in James River biota using the brackish water bivalve, *Rangia cuneata,* as the test organism.

MATERIALS AND METHODS

Sediment Monitoring

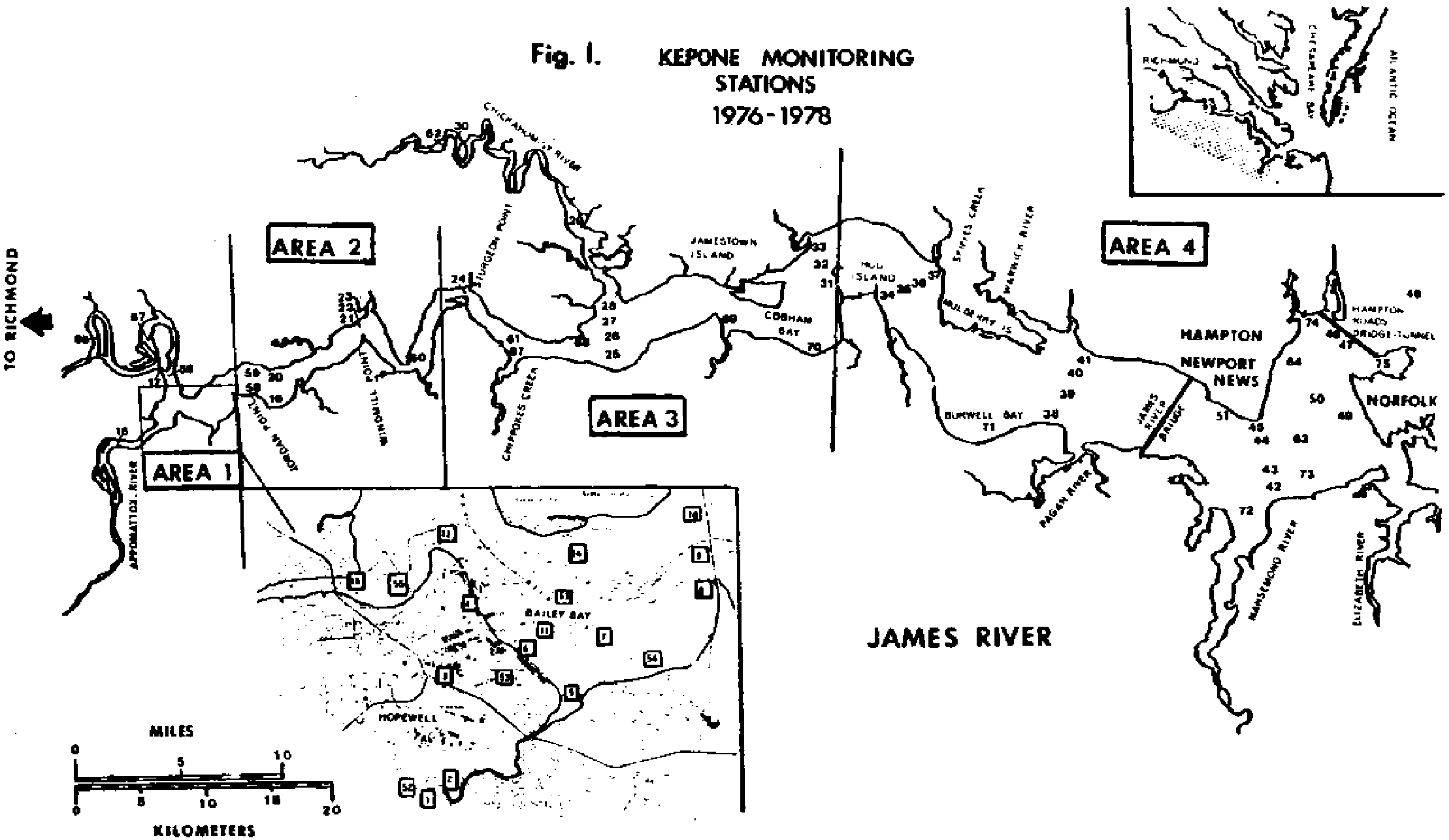
Samples were collected annually at 48 to 60 stations within the James River estuary. Station locations are shown in Fig. 1. A brief description of each station is given in Table 1.

Collections were made with a Phleger gravity corer, 4 to 5 cores were collected at each station. This sampler was fitted with buterate liners, 3.5 cm in diameter and 61 cm long. Core retention is accomplished by a simple positive displacement valve arrangement providing closure of the top of the sampler. Depending on the consolidation of the bottom material, various locking weights were attached to the sampler for proper penetration. Upon retrieval of the sampler after each collection, the core liner was removed and held in a vertical position. Each core sample was labelled according to station and substrate characteristics and frozen in a vertical position in a specially designed freezer box with dry ice. Samples were transported to the laboratory for processing.

The core samples were thawed, extruded from their core liners and sectioned into specified increments. Four of the five cores were sectioned and composited for each increment. The upper 16.51 cm of the fifth core was frozen and stored in the laboratory as a reference core.

Each composite sample was divided into two subsamples, each consisting of approximately 50 grams, wet weight. One sub-sample was submitted to the State Division of Consolidated Laboratory Services (DCLS) Richmond, Va. or Virginia Polytechnic Institute and State University (VPI & SU) Blacksburg, Va. for Kepone residue analysis.

Fig. I. KEPONE MONITORING STATIONS 1976-1978



SOURCE: STATE WATER CONTROL BOARD - DES

Table 1. Brief description of stations.

Station	Description
1	Hopewell STP, swampy area between the effluent and Bailey Creek.
2	Landfill beside Hopewell STP.
3	The upstream portion of Gravelly Run at the Pebble A&N plant.
4	Poythriss Creek at confluence of Bailey Bay. River Mile 64.5.
5	Mouth of Bailey Creek. River Mile 64.
6	Bailey Bay, in front of Continental Can complex. River Mile 64.
7	Mid-Bailey Bay. River Mile 63.
8	Bailey Bay area, west of Jordan Point. River Mile 63.
9	Mid-channel at Benjamin Harrison Bridge. River Mile 63.
10	North Shore of James River near Harrison Point. River Mile 63.
11	Mid-channel, off Continental Can complex. River Mile 64.
12	Mid-channel, off City Point. River Mile 65.
13	Spoil area in middle of Bailey Bay. River Mile 64.
14	Bailey Bay area, 850 yds. N.E. of Station 13. River Mile 64.
15	Mid-channel at Buzzard Island, mouth of Appomattox River.
16	Appomattox River at Point of Rocks.
17	Turkey Island cutoff, in the channel. River Mile 66.
18	James River at Deepwater Terminal, in channel. River Mile 78.
19	Middle of Tar Bay. River Mile 61.5.
20	Tar Bay area, mid-channel at buoys C99 and R100. RM 61.5.
21	Windmill Point Spoil area. River Mile 57.
22	James River at buoy C85. River Mile 57.
23	Windmill Point, north shore. River Mile 57.
24	James River, channel at Kennon Marsh. River Mile 48.
25	Mouth of Chickahominy River at buoy R68. River Mile 39.
26	Mouth of Chickahominy River, north of buoy R68. River Mile 39.
27	Mouth of Chickahominy, off Barrets Point. River Mile 39.
28	Mouth of Chickahominy, channel at buoy R6A. River Mile 39.
29	Chickahominy River, channel at buoy R10.
30	Chickahominy River below Walkers Dam.
31	James River, west side of Hog Island. River Mile 30.
32	James River, west side of Hog Island. River Mile 30.
33	James River, west side of Hog Island. River Mile 30.
34	Deepwater shoals transect. River Mile 25.
35	Deepwater shoals transect. River Mile 25.
36	Deepwater shoals transect. River Mile 25.
37	Deepwater shoals transect. River Mile 25.
38	Blunt Point transect through J11. River Mile 15.
39	Blunt Point trnasect through J11. River Mile 15.
40	Blunt Point transect through J11. River Mile 15.
41	Blunt Point transect through J11. River Mile 15.
42	Newport News transect, BW H11 to Pig Point. River Mile 6.
43	Newport News transect, BW H11 to Pig Point. River Mile 6.
44	Newport News transect, BW H11 to Pig Point. River Mile 6.
45	Newport News transect, BW H11 to Pig Point. River Mile 6.

Table 1. Cont.

46	North side of Hampton Roads Bridge Tunnel, at river mouth.
47	South side of Hampton Roads Bridge Tunnel, at river mouth.
48	Chesapeake Bay, buoy R10, Thimble Shoals channel.
49	Norfolk Harbor reach, between R12 and R14. Elizabeth River.
50	Newport News channel at R8. River Mile 3.5.
51	Newport News shopbuilding and Drydock, at pier 9. River Mile 7.5.
52	In Bailey Creek, 100 yds. above Station 2.
53	Gravelly Run at bridge - Continental Can.
54	Bailey Bay, 400 yards offshore. River Mile 63.
55	Bailey Bay, mudflat along City Point. River Mile 65.
56	Mudflat, 175 yards offshore at Shirley. River Mile 66.
57	Turkey Island Oxbow, opposite Curles Swamp Creek. River Mile 68.
58	West of Indian Point, 150 yards offshore. River Mile 62.
59	South of Charles Lake. River Mile 62.
60	Mudflat NE of Weyanoke Point due north of buoy 76. River Mile 52.
61	Mudflat off "The Row", 550 yards east of RN 72. River Mile 44.
62	Chickahominy Lake above Walkers Dam.
63	Newport News Middle Ground. River Mile 5.
64	Hampton Flats, middle area. River Mile 3.
65	Jones Neck. River Mile 72.
66	Above Richmond Yacht Basin at Hatcher Island. River Mile 73.
67	Mouth of Chippokes Creek off Chippokes Point. River Mile 44.
68	At Dancing Point, 300 yds. offshore. River Mile 40.
69	Tidal flat along Swanns Point. River Mile 35.
70	Middle of Cobham Bay, 400 yds. offshore. River Mile 32.
71	Burwell Bay, 400 yds. NE of buoy NJ16. River Mile 18.
72	Mouth of Nansemond River. River Mile 7.
73	West side of Craney Island disposal area. River Mile 5.
74	Flats at Hampton Roads Bridge Tunnel, at river mouth.
75	In Willoughby Bay Spit. River Mile .5.

The second sub-sample was used to characterize the sediment properties, which included percent moisture and particle size analysis and Inter-Society Color Council - National Bureau of Standards (ISCC-NBS) color classification. Each 50 gram sample was air dried and the percent moisture content was determined. Then each sample was color classified according to the ISCC-NBS color charts. Following the color classification, each sample was sieved either wet or dry through a series of sieves as described by Folk (1974), and the percentage of gravel, sand and silt-clay in each sample was determined.

Samples were dried at room temperature for 1-2 weeks. Samples were then ground with a mortar and pestle to break up clumped agglomerates and sieved through a # 18 US standard sieve to remove gravels and foreign objects. A 10 gram portion was weighed out for residue analysis. Sediment samples were soxhlet extracted with a stock solution of methanol/benzene according to the technical procedures of DCLS (1978). Prior to gas chromotography ECD or MC-GC, the extracts were cleaned by using activated florsil column chromatography. Residue concentrations were reported in ug/g (ppm) with a detection limit of 0.01, 0.02 ug/g for DCLS and 0.001 ug/g for VPI & SU.

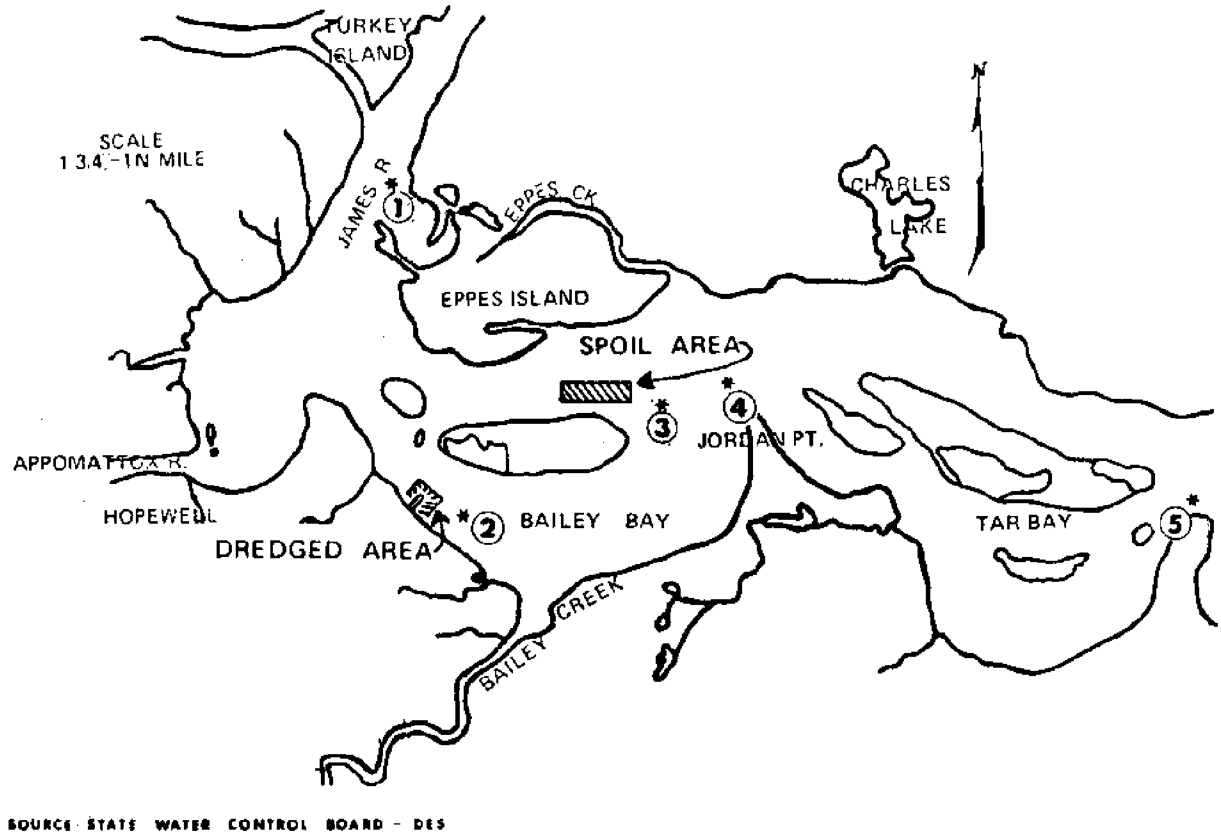
Dredging Study

Five stations were selected to monitor the dredging and disposal activities (Fig. 2). Station 1, the control station, was located 4.5 km upstream of the disposal site. Station 2 was downstream of the dredged area and station 3 was at the downstream side of the disposal site, outside of the mixing zone perimeter and silt curtain. The locations of stations 4 and 5 were chosen to determine the extent of downstream effects from the dredging and disposal activities. Station 4 was located at Jordan Pt., 1.7 km downstream of the disposal site and station 5 was at Coggins Pt., 6.7 km downstream.

Pre-dredged water samples and water quality measurements were taken at each of the 5 stations. Two water samples were collected at each station with a Kemmerer water sampler, one meter below the surface, and placed in clean 1.9 liter glass jars. Water quality measurements included water temperature (^{o}C), dissolved oxygen (mg/l), pH, conductivity (umhos/cm) and turbidity. Water temperature and conductivity determinations were made

Fig. 2.

Location of monitoring stations, dredged area and disposal site in James River, Hopewell, Virginia



with a salinity-conductivity-temperature meter (Yellow Springs Instruments, model # 33). Dissolved oxygen was measured with a DO meter (Yellow Springs Instruments, model # 57). The DO meter was calibrated by the Winkler-azide method. A Hach kit (model # 17-N) was used to measure pH. Turbidity was measured with a secchi disc and recorded in meters. Collections were arranged so that stations 2-5 were sampled on an ebb tide and station 1 was sampled on a flood tide. The above outlined procedures were followed during the monitoring of dredging and post-dredging.

Prior to the start of the dredging project, clams were collected with a modified oyster dredge from the Chickahominy River, tributary to the James River. A total of 3 background Kepone residue samples (10 clams per sample) were taken. The remaining clams were placed in submersible live boxes suspended one meter below the water surface at the 5 stations.

Dredging was completed in 12 days. Every 4th day during dredging and post dredging, clam and water samples were collected. Approximately 5 to 16 clams were removed from each live box and held, until sacrifice, in styrofoam containers containing river water collected at the sampling site.

Water samples remained refrigerated until delivery to DCLS. One of the two water samples was analyzed by DCLS as a routine, unfiltered sample. The second sample was filtered through a Whatman GFC glass fiber filter (1.2 um porosity) and the filtrate analyzed. In some cases the particulate matter removed by the filter was analyzed for Kepone residue. Determinations of Kepone in water was performed by extracting the Kepone from the water using benzene according to the technical procedures of DCLS (1979a). The extract was then injected into a gas chromatograph equipped with an electron capture detector. Residue concentrations were reported in ug/l (ppb) with a detection limit of 0.01 or 0.02 ug/l.

Clams were opened at the hinge, drained and shucked. The meat was blended to form a homogeneous mixture. Three replicate samples for each station were prepared, frozen and delivered to DCLS for residue analysis.

After thawing, the blended samples were extracted with 35% water/acetonitrile on an omnimixer according to the technical procedures of DCLS (1979b). The samples were then filtered through filter paper. The filtrate was transferred to a separatory funnel and extraction was carried out using 1:1 ethyl ether - petroleum ether. Extracts were concentrated by evaporation under vacuum and heat and cleaned by activated florisil column chromatography. Residue concentrations were reported as ug/g, wet weight with a detection limit of 0.02 ug/g.

RESULTS AND DISCUSSION

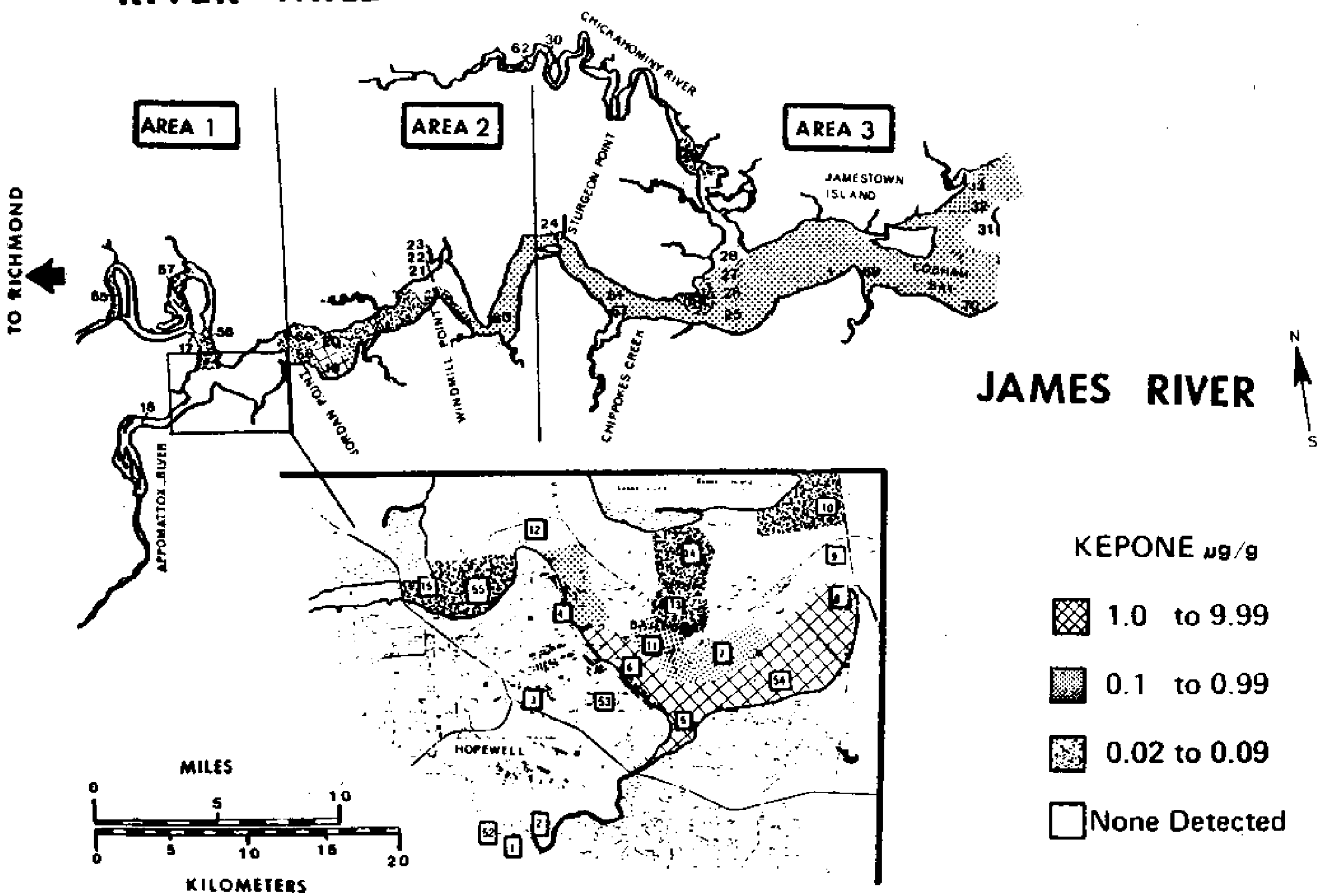
Sediment Monitoring

The results of the Kepone analysis of more than 800 samples that were collected in 1976-1978 have been mapped according to ranges of concentrations and are presented in Figs. 3-10. These ranges were defined by Chigges (1976) as: 1.0-9.99 ug/g, heavy contamination; 0.10-0.99 ug/g, moderate contamination; 0.02-0.09 ug/g, trace contamination and non-detectable levels. The area mapped includes approximately 72 river miles of the James River from Jones Neck oxbow, above Hopewell, to the Chesapeake Bay. The James River is divided into four areal reaches in Figs. 3-10 for comparative purposes. These areas are defined as Area 1, Hopewell area (river mile 66-63); Area 2, Jordan Pt. to Sturgeon Pt. (river mile 63-48); Area 3, Sturgeon Pt. to Jamestown Island (river mile 48-30, turbidity maximum zone) and Area 4, Jamestown Island to Chesapeake Bay (river mile 30-0).

When monitoring was initiated in 1976 spatial variations in the Kepone distribution in bed sediments of the James River estuary were prominent. Over the period 1976-1978 some temporal variations have been noted. The navigational capability of locating at the same sampling location each collection is limited and must be taken into consideration when considering temporal variations.

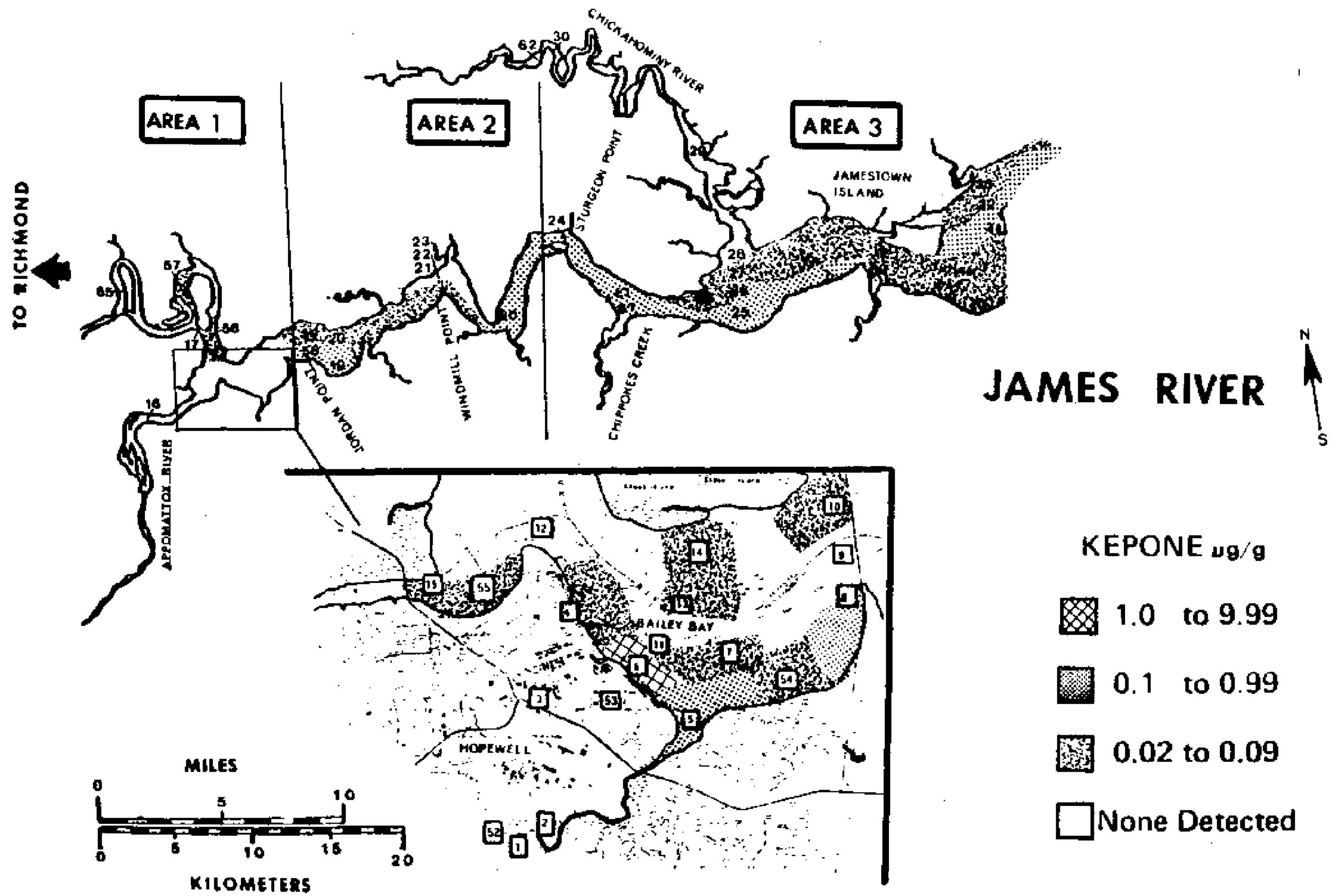
Above Hopewell, between river mile 78-66, 5 stations were sampled (18, 56-57 and 65-66). Kepone residues ranged from non-detectable to

Fig. 3. SEDIMENT KEPONE CONCENTRATIONS 0-8.89 CM., RIVER MILE 72-30



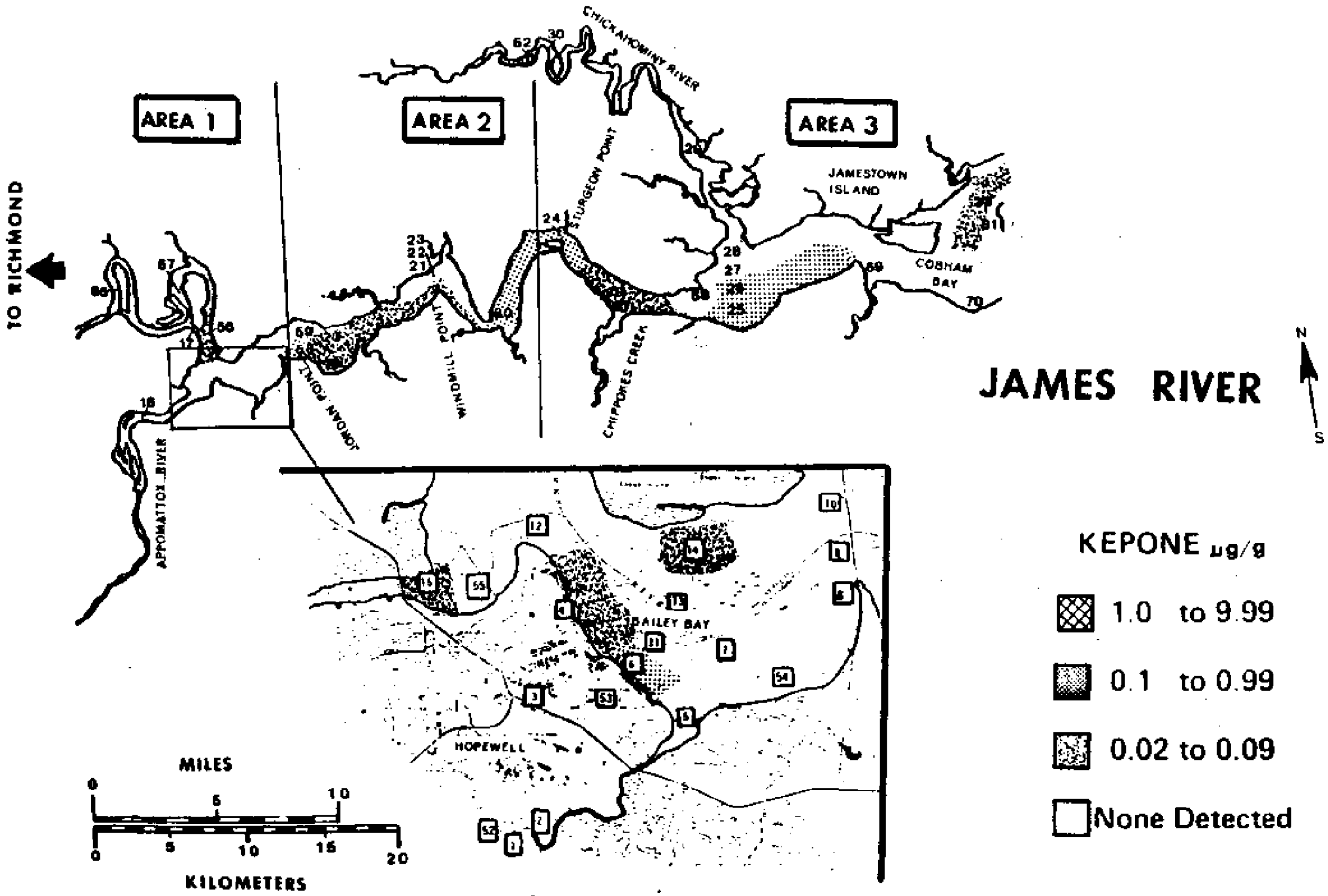
SOURCE: STATE WATER CONTROL BOARD - DES

Fig. 4. SEDIMENT KEPONE CONCENTRATIONS 8.89-16.51 CM., RIVER MILE 72-30



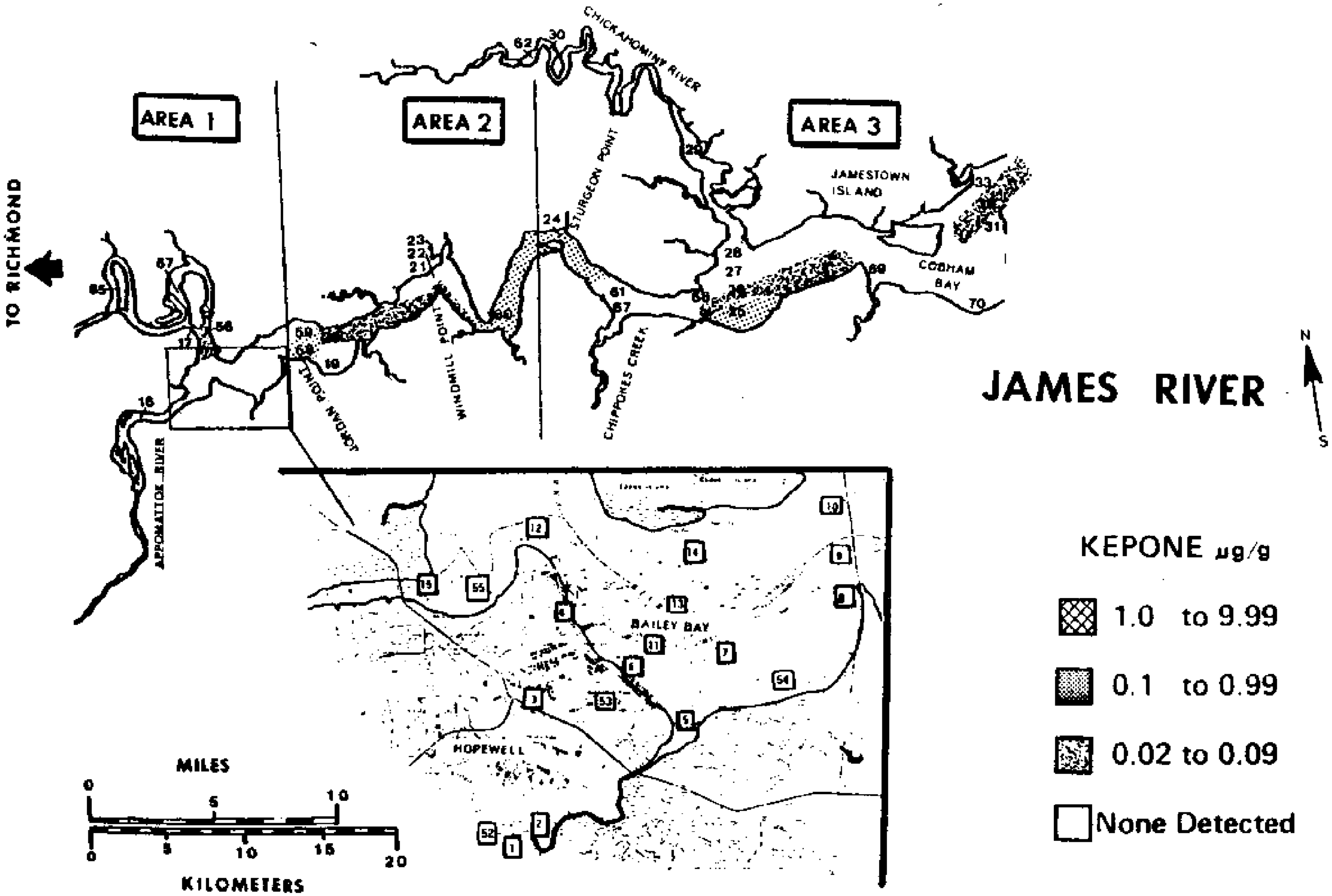
SOURCE: STATE WATER CONTROL BOARD-DES

Fig. 5. SEDIMENT KEPONE CONCENTRATIONS 16.51-31.75 CM., RIVER MILE 72-30



SOURCE: STATE WATER CONTROL BOARD - DES

Fig. 6. SEDIMENT KEPONE CONCENTRATIONS 31.75-46.99 CM., RIVER MILE 72-30



SOURCE: STATE WATER CONTROL BOARD - DES

Fig. 7. SEDIMENT KEPONE CONCENTRATIONS 0-8.89 CM., RIVER MILE 30-0

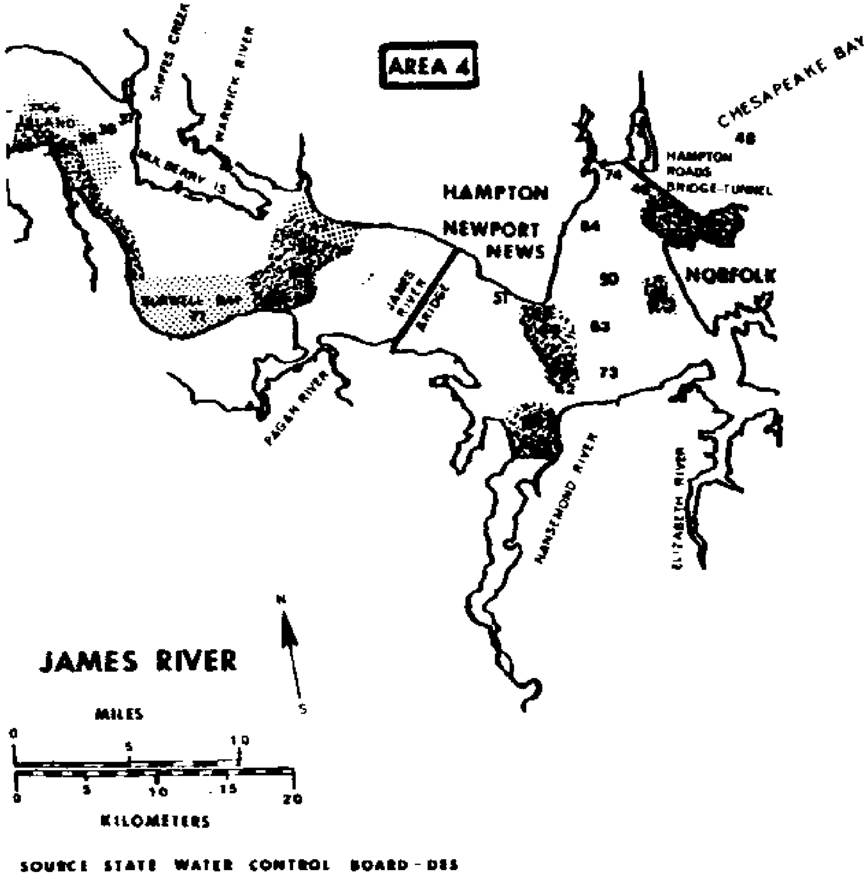


Fig. 8. SEDIMENT KEPONE CONCENTRATIONS 8.89-16.51 CM., RIVER MILE 30-0

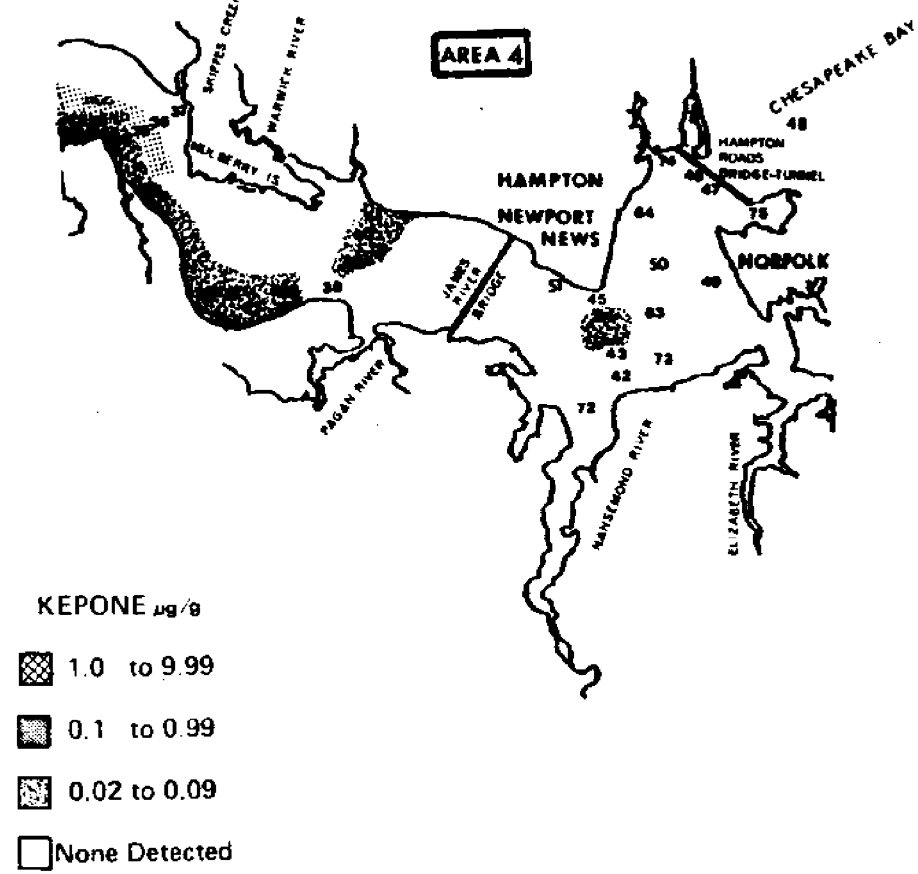


Fig. 9. SEDIMENT KEPONE CONCENTRATIONS 16.51-31.75 CM., RIVER MILE 30-0

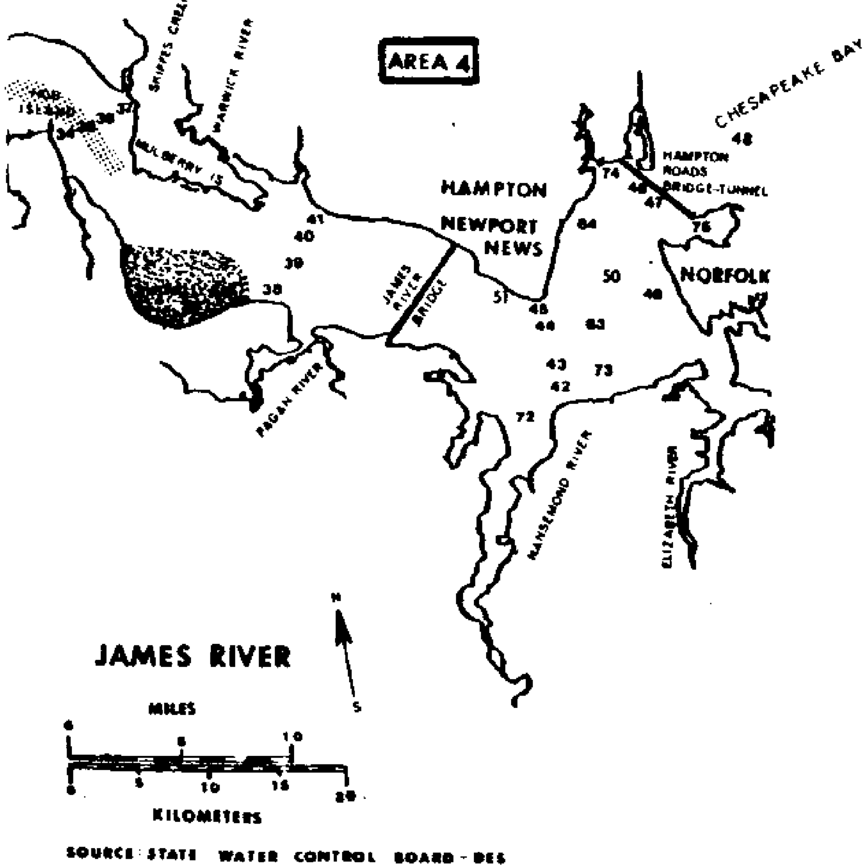
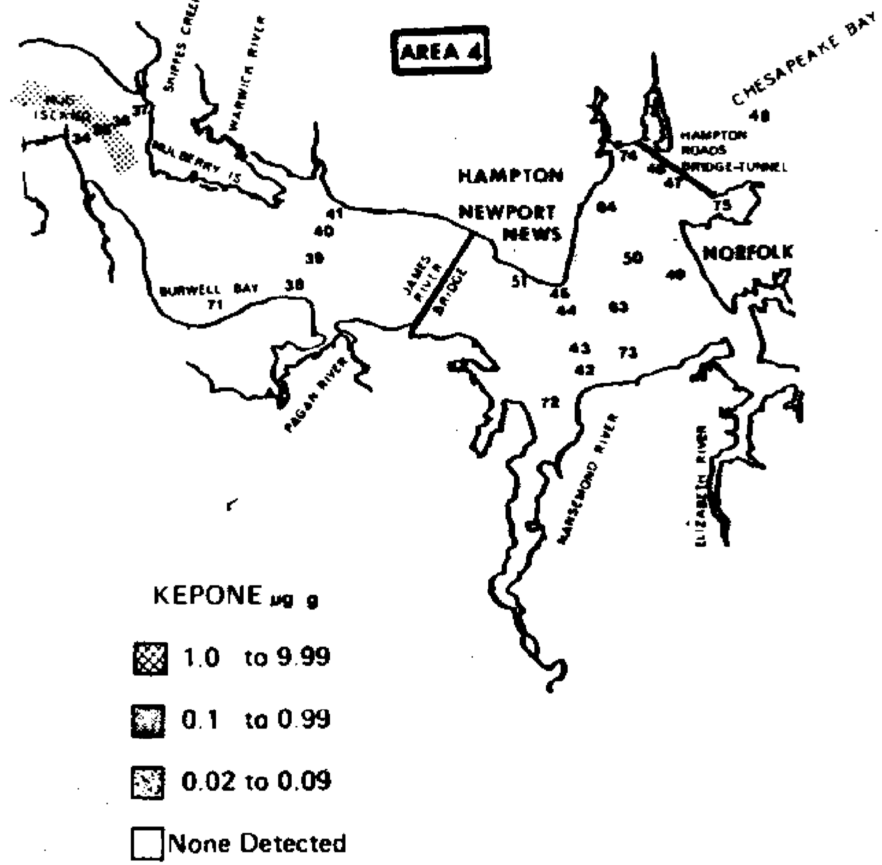


Fig. 10. SEDIMENT KEPONE CONCENTRATIONS 31.75-46.9 CM., RIVER MILE 30-0



moderate concentrations in the oxbow areas, whereas sediments in the shipping channel were uncontaminated. Kepone was probably introduced into these upriver reaches by aerial dispersion from Hopewell during the Kepone manufacturing period, 1966-1975, and/or tidal transport of contaminated sediments deposited in Bailey Bay.

In the Hopewell area, Area 1, 19 stations were sampled. These stations were located in Bailey Bay and its tidal tributaries. In Bailey Creek, stations 1 and 2 at the Hopewell wastewater treatment plant, from which Kepone was discharged during Life Science Products operations in 1974-1975, were heavily contaminated. Other tributaries, Gravelly Run (station 53) and an unnamed tributary (station 3), were contaminated at moderate to trace levels.

Within Bailey Bay, the bulk of the contamination was distributed in the top 16.51 cm. Below 16.51 cm Kepone concentrations decreased sharply. The contamination forms a "Y" shaped pattern with the base originating in Bailey Creek, and the arms extending along the western and eastern shoreline of Bailey Bay. Kepone was detected within the range of 1.0-9.99 ug/g at station 5, mouth of Bailey Creek, stations 8 and 54 along the eastern shoreline and stations 4 and 6 along the western shoreline. This deposition pattern coincides with the path of the channel through which Bailey Creek water passes and is influenced by tidal currents as reported by Battelle (1978).

Outside the "Y" distributional pattern, sediments in the middle of Bailey Bay (station 13) and the north shore (stations 10 and 14) were trace to moderately contaminated. Sediments in the shipping channel passing through Bailey Bay were relatively clean (stations 9, 11 and 12). Two stations (15 and 55) were sampled at the confluence of the Appomattox and

James Rivers. Kepone was detected at trace concentrations.

Downstream of Hopewell within Area 2, 8 stations were sampled. Concentrations in this reach ranged from ND to 1.23 ug/g. Station 58, soft muddy bottom along the south shore at Indian Pt., was moderately contaminated at all increments sampled. Residues at Tar Bay, station 19, were 1.0 ug/g in the upper 8.89 cm. This was the only station outside of Bailey Bay where contamination exceeded 1.0 ug/g. Sediments at station 20, in the shipping channel, contained trace to moderate concentrations of Kepone. Sediments at the Windmill Point transect (stations 21-23) were relatively clean. Sediments at station 60, shoal area at Weyanoke Pt. were moderately contaminated with Kepone concentrations increasing with depth.

Within the turbidity maximum zone, Area 3, 12 stations were sampled. The turbidity maximum zone is created by the mixing of freshwater moving seaward and saltwater moving up the estuary. Net velocity approaches zero at this interface. According to Nichols and Trotman (1977), the bulk of the river-borne sediment load in the James River is trapped within this reach. Pollutants such as Kepone, which have an affinity for particulate matter, will build up in this reach of the estuary. Among the 12 stations sampled, stations 24, 25 and 32 were in the shipping channel, stations 26 and 61 were in contiguous shoal areas outside the shipping channel in waters $>$3.5 meters in depth, and stations 27-28, 33 and 67-70 were located along shoal areas in water $<$2 meters.

Kepone concentrations within this reach ranged from ND to 0.51 ug/g, and sediments were predominately within the moderate range of contamination (0.10-0.99 ug/g). Moderate contamination was detected in the upper 18.89 cm. Below 18.89 cm, concentrations decreased to trace levels, except for the shipping channel where the contamination was moderate at all increments sampled. Sediments within this reach were predominately fine grained.

A comparison of the 1976-1978 sampling results reflects a build-up of Kepone contaminated sediments with time within the Jordan Pt. to Jamestown Island reach, which lends proof that the turbidity maximum zone is trapping contaminated sediments moving seaward. This build up has occured in the top 8.89 cm of bed sediments along shoal areas, whereas concentrations in the top 8.89 cm of the shipping channel have decreased. The pattern of contamination in the shipping channel may reflect the erosion or scouring of the Kepone - rich sediments from the surface layers by currents and ship traffic.

Downstream of the turbidity maximum zone, Area 4, Kepone concentrations decreased sharply. Within this reach 23 stations were sampled. Concentrations ranged from ND to 0.61 ug/g, and sediments were predominately contaminated at trace levels. Moderate contamination (0.10-0.99 ug/g) was detected at the Deepwater Shoals, Blunt Point, Newport News transects and Burwell Bay. Station 48 was the only location sampled in the Chesapeake Bay. Sediments at this station were uncontaminated.

Within Area 4 temporal variations are not as prominent. There appears to be little variation with time in the Kepone concentrations at stations comprising the Deepwater Shoals transect (34-36) and the Blunt Point transect (38-41). Concentrations at stations 43-45 at the Newport News transect showed an increase in 1978 over the 1976, 1977 study results. This increase has mainly been from non-detectable to trace concentrations.

There appears to be no build-up of Kepone contaminated sediments with time at the James River mouth (Hampton Roads area). Numerous samples have been collected in this area as part of the annual monitoring program and for Kepone clearance in issuance of 401 dredging certificates. Presently there is no apparent contamination of bed sediments in the Chesapeake Bay.

The distribution of Kepone in the bed sediments of the James River estuary is dependent on the distance downstream of the Hopewell area and such sediment properties as particle size and amount of organic matter present. The most contaminated sediments are generally fine grained and rich in organic matter, which are characteristic of Kepone deposition sites.

Heavy contamination persists with time in Bailey Bay. Downstream of Hopewell, Kepone deposition sites include Tar Bay, mouth of Chippokes Creek, James - Chickahominy confluence, Cobham Bay and Burwell Bay. Concentrations in the turbidity maximum zone appear to be increasing, but they do not equal levels detected in the Hopewell area. There are isolated areas, downstream of the turbidity maximum zone, where Kepone contamination is similar to levels detected within the turbidity maximum zone. Spot contamination in the Hampton Roads area indicates no significant deposition of contaminated sediments at the river mouth. There is no apparent contamination of bed sediments in the Chesapeake Bay.

Dredging Study

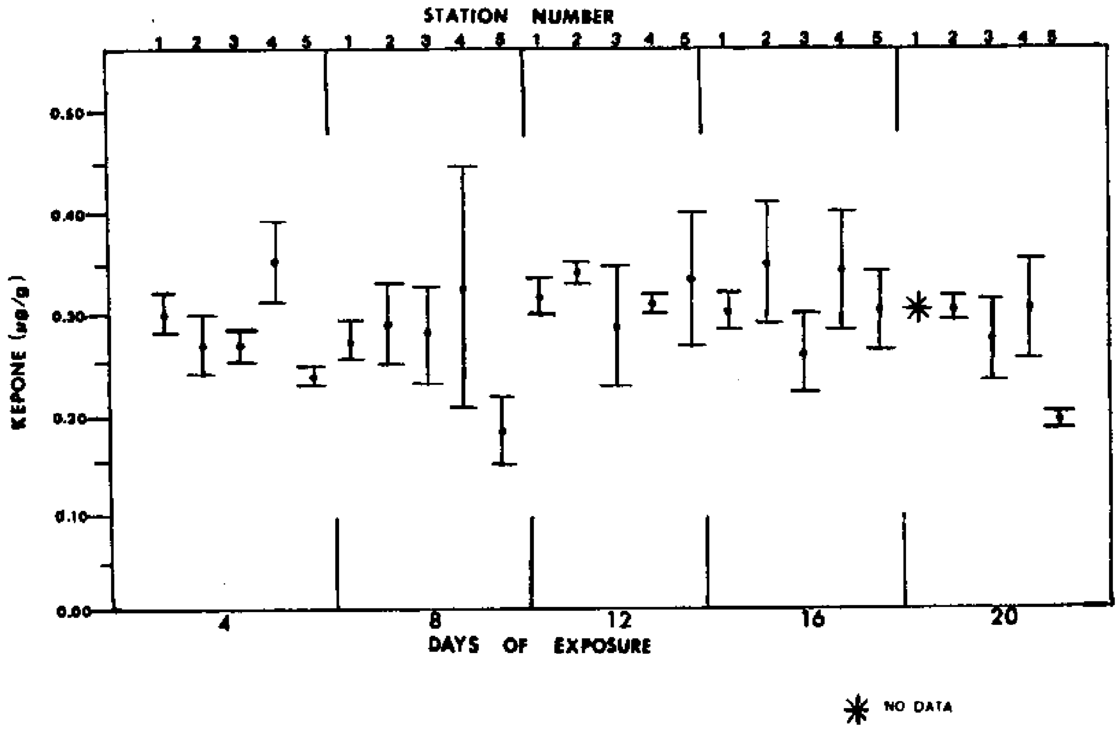
The background Kepone residue of the clams collected for the study averaged 0.30 ug/g. During the study period, Kepone residues in the clams ranged from 0.10 to 0.46 ug/g. The change in the Kepone residue at each station with time is shown in Table 2 and illustrated in Fig. 11.

Table 2. Kepone residues in clams (ug/g), wet weight, mean and standard deviations of the mean of replicate samples (3) per day.

Station #	DAYS OF EXPOSURE 4	8	12	16	20
1	0.30±0.02	0.26±0.02	0.32±0.02	0.30±0.02	NS[a]
2	0.27±0.06	0.28±0.04	0.35±0.01	0.36±0.06	0.30±0.01
3	0.27±0.03	0.27±0.05	0.28±0.06	0.24±0.04	0.26±0.04
4	0.37±0.04	0.33±0.12	0.31±0.01	0.35±0.06	0.30±0.05
5	0.22±0.01	0.18±0.07	0.34±0.07	0.30±0.04	0.19±0.01

[a]NS - No samples collected

Fig. 11. Comparison of Kepone uptake by clams (R. cuneata) at stations 1 through 5 during 12 days of dredging and 8 days of post-dredging. Mean ± standard deviation values are presented.



To detect whether differences in Kepone uptake between stations were significant a one-way analysis of variance was performed. Differences were significant at the 0.05% level. Duncan's new multiple test procedure was then used, and the difference was determined to be between stations 4 and 5. Residues at station 4 averaged higher than the other stations, whereas residues at station 5 averaged the lowest. The results indicated no significant difference in uptake between clams located at the dredging and disposal sites (stations 2 and 3) and those located at station 1, the control station.

Pre-dredging water column residues ranged from ND (stations 1-3) to 0.04 ug/l (station 4). During the study, water column residues ranged from ND to 0.10 ug/l. The change in the water column residues with time is presented in Table 3. The results of the unfiltered samples indicate total Kepone (dissolved and particulate) residues. The results of the filtered samples are separated into dissolved (filtrate) and particulate residues. Due to the large number of non-detectable water samples, the particulate matter removed by filtration was not analyzed by DCLS after the 8th day, and only the analysis of the filtrate is reported from that point on.

To detect whether significant differences existed in the Kepone water column residues between stations the Kruskal-Wallis one-way analysis of variance by ranks was performed. This is a nonparametric procedure and was used because Kepone residues in the water column are not normally distributed. Differences were not significant at the 0.05% level.

Residues at stations 4 and 5, downstream of the dredging and disposal area, averaged higher than those at stations 1-3 throughout the study. Both stations were located in areas which are influenced by tidal currents passing through Bailey Bay where bed sediments are heavily contaminated with Kepone (>1.0 ug/g).

The results demonstrated that ambient Kepone water column residues outside the mixing zone were not increased by the dredging and disposal activities. This has been predicted and reported by Williams (1978). Modeling with the VSWCB James River Water Quality Model, developed by the Virginia Institute of Marine Science, estimated that increases in Kepone water column residues as the result of the dredging would not be detectable at present detection limits (0.01, 0.02 ug/l). According to the model, the sediment Kepone desorbed to the water column would be quickly diluted to undetectable concentrations by the large volume of James River water.

Table 3. Kepone residues in water column (ug/l).

	STATION #	DAY	UNFILTERED SAMPLE	FILTERED SAMPLE FILTRATE	FILTERED SAMPLE PARTICULATE MATTER
PRE-DREDGING	1		ND[a]	ND	ND[b]
	2		ND	ND	ND
	3		ND	ND	ND
	4		.04	ND	ND
	5		.03	ND	ND
	1		ND	ND	ND
	2		ND	ND	ND
	3	4th	ND	ND	ND
	4		.02	ND	ND
	5		ND	ND	ND
DREDGING	1		ND	ND	ND
	2		ND	ND	ND
	3	8th	ND	ND	ND
	4		.02	ND	ND
	5		.05	.02	.02
	1		ND	ND	
	2		.03	ND	
	3	12th	.04	ND	
	4		.05	.06	
	5		.08	.05	
POST-DREDGING	1		ND	ND	
	2		.03	.02	
	3	16th	.03	.04	
	4		.04	.03	
	5		.06	.10	
	1		NS[c]	NS	
	2		ND	ND	
	3	20th	.02	.04	
	4		.03	.02	
	5		.09	.10	

[a]ND - Non detectable; <0.02 ug/l
[b]ND - Non detectable; <0.01 ug/l
[c]NS - No samples collected

The results of the water quality measurements taken during the study period are presented in Table 4. The data does not reflect any adverse changes in water quality as the results of the dredging and disposal activities.

From the study results, there was no indication of the acceleration of Kepone uptake by the test organism, *Rangia cuneata*, which was also concluded for the 1976 live box study by the State Water Control Board.

The 1976 study was conducted in July. The study reported here was conducted in January. Water temperatures were recorded between 1.5 - 5.7^{o}C, except at station 2 where water temperatures were as high as 11.9^{o}C due to an industrial thermal discharge. Such low temperatures had to affect the metabolism of *Rangia cuneata*, which would affect pumping rates and Kepone uptake. Studies are lacking on the effects of water temperature on the metabolism and feeding rates of *Rangia* (D. Haven, personal communication). Bender, Huggett and Hargis (1977) reported that Kepone residue levels in the oyster, *Crassostrea virginica*, decline during the colder months when the oysters are relatively inactive.

Due to this environmental factor, the change in the residue levels of *Rangia* may not have indicated the increase availability of Kepone as the result of the dredging and disposal activities. The sediments dredged from Bailey Bay were contaminated with Kepone at concentrations as high as 3.38 ug/g, whereas the sediments dredged in 1976 for the "test dredging" at Windmill Pt. were not as contaminated. The highest concentration detected was 0.87 ug/g as reported by the U.S. Army Corps of Engineers (1976). The Kepone desorbed to the water column should have been at greater concentrations during the Bailey Bay dredging.

Table 4. Values for physical and chemical water quality parameters.

	STATION #	DAY	D.O. (mg/l)	WATER TEMP. (°C)	pH	CONDUCTIVITY (umhos/cm)	TURBIDITY (meters)
PRE-DREDGING	1		7.8	4.8	7.3	100	.40
	2		6.2	10.2	7.3	430	.25
	3		8.4	5.0	7.3	98	.50
	4		8.6	5.0	7.3	115	.50
	5		8.2	4.9	7.3	118	.40
DREDGING	1	4th	8.6	3.2	7.3	60	.31
	2		10.8	3.7	7.0	70	.30
	3		9.2	3.3	7.0	58	.23
	4		9.3	3.9	7.0	75	.27
	5		10.2	3.8	7.0	69	.25
	1	8th	8.2	3.0	7.2	70	.53
	2		11.0	3.0	7.2	60	.41
	3		10.2	1.6	7.3	68	.35
	4		9.6	1.9	7.0	72	.33
	5		8.9	1.5	7.3	149	.27
	1	12th	10.0	2.8	7.2	64	.54
	2		10.1	2.5	7.0	58	.48
	3		10.2	2.0	7.0	64	.25
	4		9.8	2.0	7.0	75	.28
	5		10.2	2.2	7.0	88	.37
POST-DREDGING	1	16th	11.2	4.0	7.0	45	.23
	2		12.2	4.2	7.0	40	.22
	3		11.8	4.5	7.0	41	.16
	4		11.4	4.5	7.0	48	.22
	5		11.4	4.0	7.0	87	.31
	1	20th	NS[a]				
	2		8.8	11.9	7.0	320	.18
	3		9.8	5.7	7.3	42	.15
	4		10.2	5.0	7.4	40	.18
	5		10.5	5.0	7.4	50	.18

[a]NS - No samples collected

The results of this study substantiate the 1976 study by R. A. Gregory (unpublished data). Neither study was able to detect an increase in the Kepone uptake by test organisms during short-term dredging and disposal operations in the James River estuary.

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EFFECTS OF OPEN-WATER DISPOSAL OF DREDGED MATERIAL ON BOTTOM TOPOGRAPHY ALONG THE TEXAS GULF COAST

by

David Fenwick Bastian[1]

ABSTRACT

Five open-water disposal sites were monitored for a series of years to determine how ocean bottom topography was influenced by hopper dredging disposal operations along the coast of Texas. Hydrographic surveys done by the U.S. Army Engineer District, Galveston of dredged disposal areas were conducted before and after dredging and were used to determine the average elevation of each disposal area. The dredged material was mostly sand, and within a short period of time the dredged area reflected little evidence of the dredged material except at Matagorda.

The three hopper dredges used and their methods of material placement are described.

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INTRODUCTION

Interest in the effects on ocean-bottom topography of open-water dredged material disposal has resulted in extensive research. Of basic concern is what happens to the dredged material once it is dumped from the hopper dredges into the open water. Pertinent records are sparse. Because geometry, hydraulic forces, meteorological conditions, and dredging requirements vary according to disposal site and time, any changes in bottom site topography are highly variable.

This paper addresses site survey data analysis to yield information on the stability and growth rate of hopper dredge disposal areas along the Texas coast where, since the 1960's, the U. S. Army Engineer District, Galveston, has made hydrographic surveys of most of their hopper dredge disposal areas primarily to ensure sufficient draft for the hopper dredges. Generally, the surveys were made prior to dredging operations and were in conjunction with surveys made to determine the condition of navigation channels. Fathometers were used and readings were corrected for tides. The disposal areas discussed are at Brazos Island, Port Mansfield, Matagorda, Freeport, and Port Aransas (Figure 1) on the Texas Gulf Coast, within the Galveston District.

Contours of the five areas mentioned above were made at the Waterways Experiment Station from survey data furnished by the Galveston District. The Galveston District usually marked only the shore-side and channel-side boundaries of the disposal areas, the other two sides had to be selected to obtain a basis of comparison between surveys. The selection was an arbitrary one, determined by the extent of the hydrographic survey. An effort was made to establish a comparable disposal area that was as large as possible. Occasionally this required extrapolation of contours.

Comparisons were made of changes of the average elevation of the disposal area with time. These comparisons are more significant when "before" and "after" dredge surveys are compared rather than when "after" and "after" surveys are compared (which is the most common case) because of the time intervals involved.* One difficulty in working with this method of comparison was that there were no records of the hopper dredge route as it dumped the dredged material. Therefore, the possibility exists that some portion of the material was dumped in an area outside of the boundaries shown in this study, due to the limited size of the areas contoured and the distance of travel required by the hopper dredge to empty its bins.

The survey was made by soundings taken from a boat operating in zigzag patterns perpendicular to the channel. In the few cases where an additional sounding-survey path was run in a zigzag pattern normal to the previously mentioned survey pattern the intersecting soundings were found to vary as much as 2 ft but these additional survey patterns were not used.

DISPOSAL OPERATIONS AND SEDIMENT TRANSPORT

The two hopper dredges operating regularly in the Galveston District are the MacKENZIE and the McFARLAND. On occasion, the LANGFITT has also been operated in the District. Facts about these dredges pertinent to

* Normally hydrographic surveys were conducted soon after dredging was completed to ensure adequate channel depth as required by contract. These surveys are referred to as "after" surveys by the Galveston District. On occasion, hydrographic surveys were conducted just prior to the initiation of dredging. These surveys are referred to as "before" surveys.

this report are tabulated below.

Name	Rated Speed fps Loaded	Rated Speed fps Light	Capacity cu yd	Draft Loaded ft
LANGFITT	17.6	23.2	--	--
MacKENZIE	17.6	19.5	1656	21
McFARLAND	20.4	22.2	3140	23

The dredges operate up and down the channel. When the bins are full or when the dredge has obtained an optimum load, the dredge navigates to the disposal area, the bottom doors of the hopper bins are opened, and the dredged material falls from the bins while the speed of dredge as it moves through the disposal area remains the same (i.e., the material is expelled from the hopper while the dredge is moving). Whether or not, or how much of, the material is dumped while the dredge is over the disposal area is unknown.

The path that a dredge takes while dumping is not recorded, but records are available from which the average length of the dumping path can be determined.

These records, entitled "Report of Operations - Hopper Dredges," were condensed and pertinent data are displayed in Table I along with dumping distance calculations. The author assumed that the speed of the dredge during dumping is equal to or less than the average speed of the dredge as it traveled to and from the dump. Therefore the distance that the dredge would travel while dumping can be expressed as

$$D_{max} = \overline{V} \times \overline{T}_d$$

where

D_{max} = maximum distance dredge travels while dumping, miles

$\bar{V}$ = average velocity of dredge to and from dump site, mph

$\bar{T}_d$ = average time dumping per load hauled, hr

This calculation for each project gives an idea of the size of disposal area needed to correspond to the operation of the hopper dredge. From Table 1 the average maximum distance the dredge would travel while dumping is 2.2 miles at Brazos, 1.4 miles at Port Mansfield, 1.9 miles at Matagorda, 1.9 miles at Corpus Christi, and 2.2 miles at Freeport. However, this tells nothing about the path of the dredge.

There are two basic considerations in attempting to determine the fate of the dredged material once it is dumped from the dredge. First, when and where does the material reach the sea bed. Second, once at the bed, how long and what percentage of the material reaching the bottom will remain in place.

Corps of Engineers records for dredging at the study areas in the Galveston District excluding Freeport show that over 80 percent of all the material handled by hopper dredges is sand (see Table 2). Upon release from the bin, this sand drops in bulk with an imparted horizontal velocity equal to that of the hopper dredge. How far the sand has to fall from the hopper dredge and through what currents are important factors relating to the initial lateral transport of the sand.

Due to turbulence and shear stresses, the sand falls in a combination of three forms: solid bulk, slurry, and particulate. As the sand falls, the bulk material moves fastest with the surrounding slurry moving as a distinct density current. Eddies are formed and particulate matter is sheared from the slurry. Particles fall the slowest. The fall velocity W of various sizes and shapes of sand particles is well documented. In the present situation, according to Graf, this is complicated since

> If more than a single sphere moves through an unbound fluid system, a mutual interaction will be noticed. It has been observed that the settling velocity increases if only a few closely spaced particles move, and that the fall velocity is reduced (i.e., the drag increased), if many particles are dispersed throughout the fluid.

If we assume that the ocean current is zero, the empirically-determined values for the fall velocity, W, can be used to describe the velocity of particulate matter. From sieve analysis of samples taken aboard hopper dredges while dredging, the extreme low value for 20 percent finer by weight, d_{20}, and the extreme high value for the 80 percent finer by weight, d_{80}, for all study areas except Freeport are 2.35×10^{-3} in. and 1.56×10^{-1} in., respectively. The following tabulation presents the fall velocities and the expected time for these particles to reach the ocean floor, assuming that the hopper dredge has a draft of 21 ft and the water depth is 36 ft.

	d_{20}	d_{80}
Particle size, in.	2.35×10^{-3}	1.56×10^{-1}
Fall velocity W, fps	0.008	0.2
Time of fall, sec	1888	75

The larger sand grains would reach the bottom in about 1 min, whereas the smaller sand would take up to 30 min.

Both the speed of the dredge and the velocity of the ocean currents impart a horizontal component to the falling sand, which prescribes a non-vertical path. Still, when turbulence is ignored, and when the vertical velocity distributions are not know, prediction of the final target is difficult to pinpoint.

The final problem in predicting when and where the sand will fall is turbulence. Turbulence counterbalances the settling tendency of the particles. Depending upon the turbulence, the sand can remain suspended and be transported great distances.

In summary, the material released from the dredge falls in a combination of three forms: particulate, slurry, and bulk. The larger particles and that material falling in bulk and slurry can be expected to hit the bottom within a few minutes.

As the sand hits the sea bed, turbulence and rebound occur. Some of the material is undoubtedly resuspended. Depending upon the concentration of the suspensions and local currents, this suspended material can be transported away from the disposal area.

The erosion material from a dredged material bank is dependent upon the type and degree of compaction of the material, bed form, water viscosity, and magnitude of local currents. As the sand size under consideration (for all sites except Freeport) is greater than 7.48×10^{-3} in., the effects of special properties of shape, packing, and cohesiveness are negligible (Sternberg). Thus, there is no armoring effect (Smith and Hopkins) as is observed with silts and clays. However, research has been able to show that in some situations benthos can stabilize the dredged material mound surface (Rhoads and Young).

By neglecting the stabilizing effect of benthos, the erosion of the sand can be studied from strictly a fluid velocity standpoint. Simply stated, critical shear velocity is required to move bottom particles. Sediment motion is of two forms: bed load and suspended load.

To help evaluate what effect known currents will have upon the stability of the mound of dredged material it is necessary to know the size of particles, water viscosity, and bottom currents. With this information various competency curves (i.e. Shields Diagram) exist which can be used to determine if the material will be transported and in what form. By knowing the duration of current equal to or greater than the critical shear velocity, the amount of time the material will move can be predicted. But this tells nothing about the volume of material which would be transported; this requires the use of equations of motion and continuity for both water and sand. Unless the bank is large enough to change the local current patterns, material will be eroded not only from the bank but also from the surrounding area. If the bank is composed of sands different from those of the surrounding area, then the bank will be subject to a different rate of attack. Some of the bank material will move and will be replaced simultaneously to some degree.

According to Morris and Wiggert, ordinary wave activity can move sand to depths of about 30 ft below the water surface. Since most of the disposal areas under consideration are in greater depths, ordinary wave activity should have little effect on the disposal area. Studies by Smith and Hopkins (p. 172) and by Sternberg and McManus show that storm-generated currents have a pronounced effect on sediment movement even in depths greater than 200 ft. A logical conclusion is that storms in the Gulf of Mexico generate enough energy to produce pronounced effects of the bottom topography. The critical erosion velocity measured about 3 ft above the bed for sand ranging from 2.76×10^{-3} to 1.57×10^{-2} in. is about 1.5 fps with the cessation of movement velocity being even less.

Littoral drift is the major classification of nearshore sediment movement. Disposed dredged material can be part of the littoral drift process, depending on location. If not directly, then it can be influenced by its proximity to the jetties and channel which interrupt the littoral process and result in topographical changes. Brunn and Lackey have concluded that up to about 400,000 to 500,000 cu yd a year pass a given point on part of the Texas shore.

SURVEYS OF DISPOSAL AREAS

Freeport

Hopper dredges operating at Freeport use the two disposal areas (A and B) shown in Figure 2. These areas are along both sides of the entrance channel and reach from sta -20+00 to sta -110+00. They parallel the channel 500 ft from the channel sides. There is no defined outside border (away from the channel). The northern disposal area, A, is used when the current is from south to north and the southern disposal area, B, is used when the current is from north to south.

The exterior boundary for both areas A and B was chosen such that the area of each measures 1,135,000 sq yd.* The results of only three surveys were available; these surveys cover the period 1964-1966.** Between the first and third surveys, a total of 1,771,889 cu yd of dredged material was dumped. Despite this dumping, the average elevation of both disposal areas decreased about 0.5 ft. Table 3 presents the dates of the surveys and the corresponding disposal area bottom elevations, as well as the dates of dredging and the corresponding volume of material dumped.

The areas chosen for study are 5,000 ft long and lie parallel to the channel. From records for 1968-1972 (see Table 1), the average length of travel required by the hopper dredge to empty its bins was calculated to

* The study areas had to be shortened to sta -70+000 because of the data available from the surveys.
**The disposal area contour maps were developed using Galveston District hopper dredge survey maps. The contouring was done at WES.

be about 1.1 miles, which corresponds well with the modified length of the disposal area.

The disposal areas do not reflect the amounts of dredged material being dumped but instead show a tendency for the disposal areas to be scoured.

Two possible reasons for the lack of buildup are (a) the dredged material was 80 percent silt according to Galveston District maps and (b) there was a long time period between dredging and surveys. Once dumped from the bins, any material that remained particulate would stay in suspension for relatively long periods. This material could easily come to rest outside the disposal area. The material that reached the disposal area is erodible, and the rate of erosion is related to the cohesion, compaction, and water content of the disposed material. The second reason for lack of buildup may be attributed to the 6-month to 1-yr lapse of time between dredging and subsequent surveys, during which time equilibrium of the bottom topography in the disposal area was probably achieved. However, neither reason explains the net deepening of the disposal area.

Port Aransas-Corpus Christi

At Port Aransas, hopper dredging is done in the jetty and outer bar channel. The dredged material is dumped in the designated disposal area bounded between sta 80+00 and sta 145+00 and 800 ft south of the south side of the outer bar channel (Figure 3). The width of the disposal area was chosen to be 1500 ft.

Data in Table 3 indicate a definite accretion trend in the area of study. From 1961 to 1973 the disposal area bottom elevation rose about 5 ft.

Brazos Island Harbor

Hopper dredges at Brazos Island Harbor utilize a dump area north of the sea bar channel. The District has defined the southern channel and western shore boundaries of the disposal area. The southern boundary remained constant at 800 ft from the center line of the channel; the western boundary was extended seaward from sta -10+200 to sta -13+000 in 1966 to sta -15+00 in 1967.

Because the western (shoreside) boundary was moved seaward twice during the period of study and because of the extent of soundings, two areas were defined for comparative study. The first area chosen was designated Brazos I (Figure 4). This area is 5,000 ft long starting at sta -10+200, and is 2,000 ft wide. Table 3 presents the dates of surveys and the corresponding disposal area bottom elevations and the dates of dredging and the corresponding volume of material dumped. This area remained relatively stable during the period of study and showed no positive trend toward buildup of the disposal area.

The second area chosen, designated Brazos II (Figure 5), covers the period of dredging from 1968 to 1970. Brazos II disposal area starts at sta -15+000 and extends 3,200 ft seaward and is 2,000 ft wide. Average hopper dredge dumping distances for the period 1969-1972 were calculated and found to be 1.2 miles, which is about twice the length of the chosen study area. Table 3 presents the dates of the survey and the corresponding disposal area bottom elevations and the dates of dredging and the corresponding volume of material dumped. The fathometer survey of 5 June 1968 yielded an average bottom elevation of -54.7 ft from 14-30 June; 228,103 cu yd of dredged material were dumped, but the 2 July fathometer survey shows an average bottom elevation of -55.4 ft. If the results of these surveys are correct, there was a net scour of 166,000 cu yd during this 18-day period. Subsequent surveys show a buildup trend.

Matagorda Ship Channel

The hopper dredge operating at Matagorda utilizes a disposal area south of the entrance channel. This area is bounded on three sides, 1,000 ft south of the south side of the channel and by sta -11+000 and sta -17+000. Since 1963, the disposal area shoreside limit has moved seaward from sta -8+000 because of shoaling off the ends of the jetties. During the 1960's, the project area was going through a period of adjustment because of the cut made through the Matagorda Peninsula for the Matagorda Ship Channel. Whether or not it has stabilized is not known. The comparative study area is 6,000 ft by 1,000 ft as shown in Figure 6. This agrees with the average computed required dumping distance of 1.2 miles.

The study area has had a buildup of about 7 ft between 1963 and 1971 as shown in Table 3. (Unexplainable is the survey of March 1968 immediately after disposal, which shows a net scour; a survey three months later, during which time there was no dredging, shows an accretion.)

Port Mansfield

At Port Mansfield the hopper dredge used two disposal areas until 1967, after which a single disposal area was used. As defined by the District, these areas are on either side of the entrance channel, 825 ft from its center line; they start at sta 3+200 and there is no seaward limit. Only five surveys were available. Of these, only two show the south disposal area. For purposes of the study a common area of comparison was defined in the north disposal area shown in Figure 7. This area is 2,000 by 1,500 ft. The average required dumping distance calculated from available records shown in Table 1 indicates that the chosen study area is too small. However, because of the limited extent of some of the surveys, the small size of area was necessary for comparison. The study area remained stable over the period of record from 1965 to 1970 with an average bottom elevation of -30 ft.

SUMMARY

Three of the six navigation project hopper dredge disposal areas studied showed accretion, two were stable, and one showed a slight scour tendency. The Matagorda area showed the greatest change with about a 7-ft bottom elevation rise during a 9-yr period. The rise is attributed not only to dredge disposal but to adjustment of the area to the cut through Matagorda Peninsula. Because the studies focused only on the disposal areas nothing can be said about the surrounding areas. In all probability the surrounding areas experienced the same net change in bottom elevation. It appears that net changes in the bottom elevations are strongly related to the dynamics of the area involved. In addition, disposal areas are not physically marked resulting in a low probability of coinciding repetitive dumping paths of the hopper dredges. Because the dredging process takes such a long time, hydrodynamic forces tend to smooth out each individual disposal. In terms of significance, dumping has had little effect on bottom topography for the sites surveyed. There does not appear to be definitive disposal banks of dredged material.

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Table 1

Determination of Required Hopper Dredge Dumping Distances

Dredge	Date of Operation	No. of Loads	Average Distance miles	Time to And from Dump hr	Total Distance miles	Average Speed mph	Dumping Time hr	Time per Dump hr	Dumping Distance miles	Average Load cu yd
				Freeport						
A. MACKENZIE	Oct-Nov 68	589	1.0	155	1178	8.0	67	0.11	0.88	1,205
	Apr-May 69	502	1.6	168	1606	9.6	43	0.09	0.82	1,315
	Dec 69	399	0.9	72	718	10.0	37	0.09	0.90	1,410
	May-Jun 70	399	2.2	73	1931	26.5	33	0.08	2.19	1,511
	Dec 70	430	1.5	102	1410	12.8	36	0.08	1.01	1,438
	Feb-Mar 71	154	3.7	140	1140	8.1	25	0.16	1.31	1,510
	Apr-May 71	539	1.3	135	1401	10.4	45	0.08	0.87	1,429
	Oct-Nov 71	348	0.6	64	418	6.5	32	0.09	0.60	983
	Apr-May 72	488	1.0	83	976	11.8	43	0.09	1.04	1,269
	Nov-Dec 72	310	1.1	63	682	10.8	31	0.10	1.08	1,329
				Port Aransas-Corpus Christi						
	May-Jul 69	509	1.4	164	1465	8.69	102	0.20	1.74	1,633
	Jul 70	368	1.3	73	957	13.11	50	0.10	1.29	1,540
	8-12 Feb 71	114	3.4	95	730	7.68	21	0.18	1.41	1,406
	May-Jul 71	802	1.4	291	2246	7.72	164	0.20	1.58	1,064
McFARLAND	1-22 Jul 72	175	1.5	48	525	10.94	30	0.17	1.88	1,465
A. MACKENZIE	Jul-Sep 72	686	0.5	153	686	4.98	148	0.22	0.97	1,332
McFARLAND	Sep-Dec 72	925	1.3	252	2405	9.54	78	0.08	0.80	1,421
				Brazos Island Harbor						
A. MACKENZIE	Jan 67	208	2.2	69	915	13.3	36	0.17	2.26	1,637
McFARLAND	Aug 68	107	2.3	55	501	9.1	11	0.10	0.91	1,488
A. MACKENZIE	Aug 70	426	2.7	195.2	2300	11.8	65.6	0.15	2.12	736
	Aug-Sep 71	360	1.1	195	792	7.3	65.5	0.18	0.74	981
	Jun-Jul 72	379	0.3	125	227	1.8	79.5	0.21	0.38	1,602

Table 1 (Continued)

Dredge	Date of Operation	No. of Loads	Average Distance miles	Time to and from Dump hr	Total Distance miles	Average Speed mph	Dumping Time hr	Time per Dump hr	Dumping Distance miles	Average Load cu yd
				Matagorda Ship Channel						
A. MACKENZIE	Sep-Oct 68	479	0.9	163.4	862	5.28	1.02	0.21	1.11	1,363
	Feb-Apr 69	414	1.9	198.55	1573	7.92	98.37	0.24	1.90	1,507
	Oct-Nov 69	710	0.7	130	994	7.65	128	0.18	1.38	1,559
	Apr-May 70	385	0.6	77.55	462	5.96	65.25	0.17	1.01	1,545
	Oct-Nov 70	571	1.7	233	1941	8.33	101.15	0.18	1.01	1,446
	Jul-Aug 71	179	0.9	57.15	322	5.63	32.55	0.18	1.01	1,360
	Mar-Apr 72	392	0.2	80.45	1568	1.94	67.35	0.17	0.33	1,429
				Port Mansfield						
	Jun 69	195	0.9	32.15	351	10.92	24.50	0.13	1.42	1,492
	Jun 70	255	0.6	44.05	306	6.95	24.05	0.09	0.63	1,157
	May-Jun 72	494	0.3	82.10	296	3.61	41.35	0.08	0.29	1,252

Table 2

Grain-Size Distribution of Dredged Material

Period of Sampling	Grain Size, mm d_{20}	d_{80}
Freeport		
7 Oct-11 Nov 68	0.007	0.05
14 Apr-11 May 69	0.001	0.08
1-30 Dec 69	0.001	0.07
18 May-7 Jun 70	0.001	0.067
30 Nov-27 Dec 70	0.001	0.08
22 Feb-7 Mar 71	0.001	0.026
19 Apr-16 May 71	0.001	0.072
14 May 71	0.001	0.64
29 Oct-28 Nov 71	0.001	0.061
15 Nov-10 Dec 72	0.001	0.058
Port Aransas-Corpus Christi		
12 May-4 Jun, 16 Jun-3 Jul 69	0.08	0.17
8-15 Jun, 30 Jun-19 Jul 70	0.08	0.17
8-21 Feb 71	0.048	0.18
17 May-25 Jul 71	0.11	0.18
1-22 Jul 72	0.09	0.17
24 Jul-24 Sep 72	0.09	0.15
Brazos Island Harbor		
6-17 Aug 68	0.09	0.17
4-31 Jul 69	0.067	0.17
27 Jul-30 Aug 70	0.085	0.158
9 Aug-19 Sep 71	0.088	0.17
26 Jun-17 Jul 72	0.09	0.388
Matagorda Ship Channel		
10 Feb-13 Apr 69	0.085	0.17
29 Jul-6 Oct 69	0.09	0.2
3 Oct-30 Nov 69	0.081	0.27
20 Apr-17 May 70	0.11	0.37
11 Oct-29 Nov 70	0.085	0.171
25 Jul-8 Aug 71	0.078	0.163
20 Mar-16 Apr 72	0.081	0.171
Port Mansfield		
5-15 Jun 69	0.06	0.22
16-29 Jun - 20-26 Jul 70	0.065	0.224
15 May-22 Jul 72	0.063	0.226

Table 3

Survey and Dredging Data

Freeport

Date of Survey	Date of Dredging	Volume of Dredged Material cu yd	Average Elevation of Disposal Area ft, mlt Area A	Area B
7 Jan 1964			-30.22	-29.31
	5 Oct-15 Nov 1964	806,689		
Oct 1965			-30.24	-29.66
	24 Oct-19 Nov 1965	965,200		
Jul 1966			-30.68	-30.07

Port Aransas-Corpus Christi

Date of Survey	Date of Dredging	Volume of Dredged Material cu yd	Average Elevation of Disposal Area ft, mlt
14 Aug 1961			-36.55
26 Sep 1961			-39.07
18 Dec 1962			
	Dec 1964-Jan 1965	266,041	
	Dec 1965	51,906	
	Feb 1966	398,478	
17-20 May 1966			-34.33
	Jun-Aug 1966	498,279	
	Aug-Dec 1967	1,264,972	
	Apr-Jun 1968	489,132	
10-21 Apr 1969			-33.94
	May-Jul 1969	898,568	
11-25 May 1970			-33.07
	Jun-Jul 1970	570,010	
	Feb 1971	157,500	
Apr 1971			-32.58
	May-Jun 1971	571,147	
Jun 1972			-32.60
Feb 1973			-31.96

Brazos I

Date of Survey	Date of Dredging	Volume of Dredged Material cu yd	Average Elevation of Disposal Area ft, mlt
Apr 1964			-46.72
Jan 1965			-47.76
	Feb 1965	112,089	
5 Mar 1965			-47.08
	Apr-Jun 1965	309,338	
11 Mar 1966			-47.69
	Apr-May 1966	247,903	
10 May 1967			-47.98
30 Sep 1967			-46.86
	Oct-Nov 1967	337,870	

Table 3 (Continued)

Date of Survey	Date of Dredging	Volume of Dredged Material cu yd	Average Elevation of Disposal Area ft, mlt
		Brazos II	
5 Jun 1968			-54.72
	14-30 Jun 1968	228,103	
2 Jul 1968			-55.40
	Jul 1969	217,940	
Jun 1970			-52.64
	Jul-Aug 1970	341,593	
Feb 1971			-52.58
		Matagorda	(msl)
7 Mar 1963			-33.2
8 Jul 1963			-35.9
12 Oct 1963			-33.3
11 Jun 1965			-32.9
	Aug-Oct 1965	1,712,285	
	Nov-Dec 1965		
	Feb 1966		
16 Feb 1966			
	Mar-Apr 1966	536,212	-33.0
28 May 1966			-30.8
	Jul, Sep-Oct, Dec 1966	728,300	
13 Feb 1967			-30.1
	Mar-Apr 1967	381,500	
27 Jan 1967			-31.0
	Jul-Aug 1967	537,100	
1 Oct 1967			-29.2
	Oct 1967	448,364	
23 Jun 1968			-29.5
	Jan-Mar 1968	661,000	
26 Mar 1968			-29.8
30 Jul 1968			-28.83
	Jul-Oct 1968	683,664	
	Feb-Apr 1969	711,000	
Jul 1969			-29.0
	Oct-Nov 1969	1,003,000	
12 Mar 1970			-27.8
	Apr-May 1970	492,087	
24 Aug 1970			-26.5
	Oct-Nov 1970	906,785	
Feb 1973			-26.82

Table 3 (Concluded)

Date of Survey	Date of Dredging	Volume of Dredged Material cu yd	Average Elevation of Disposal Area ft, mlt
	Port Mansfield		
20 Oct 1965			-30.4
22 Aug 1966			-31.4
	Jul 1968	261,461	
Apr 1969			-29.8
	Jun 1969	161,110	
May 1970			-30.6

ACKNOWLEDGEMENT

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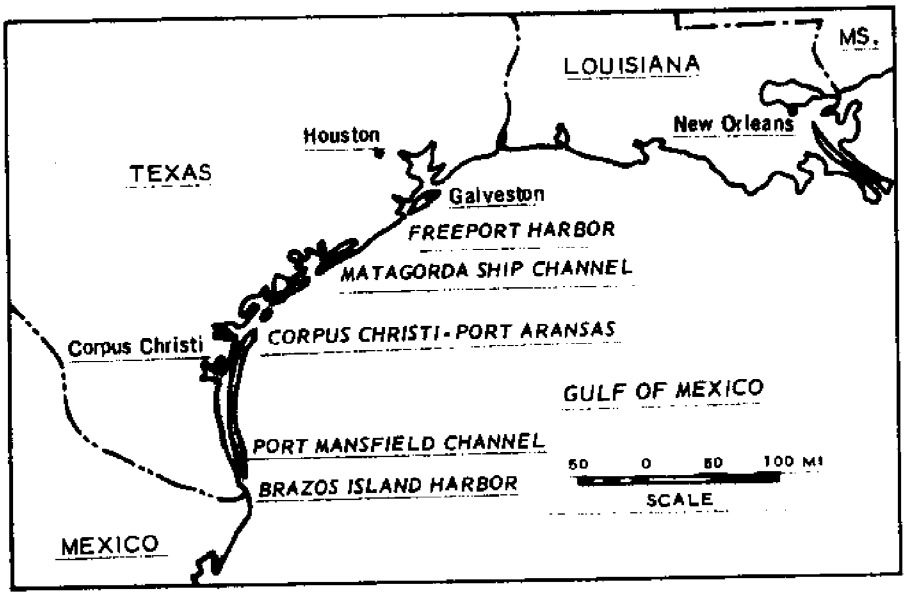


Figure 1. Hopper dredge disposal areas

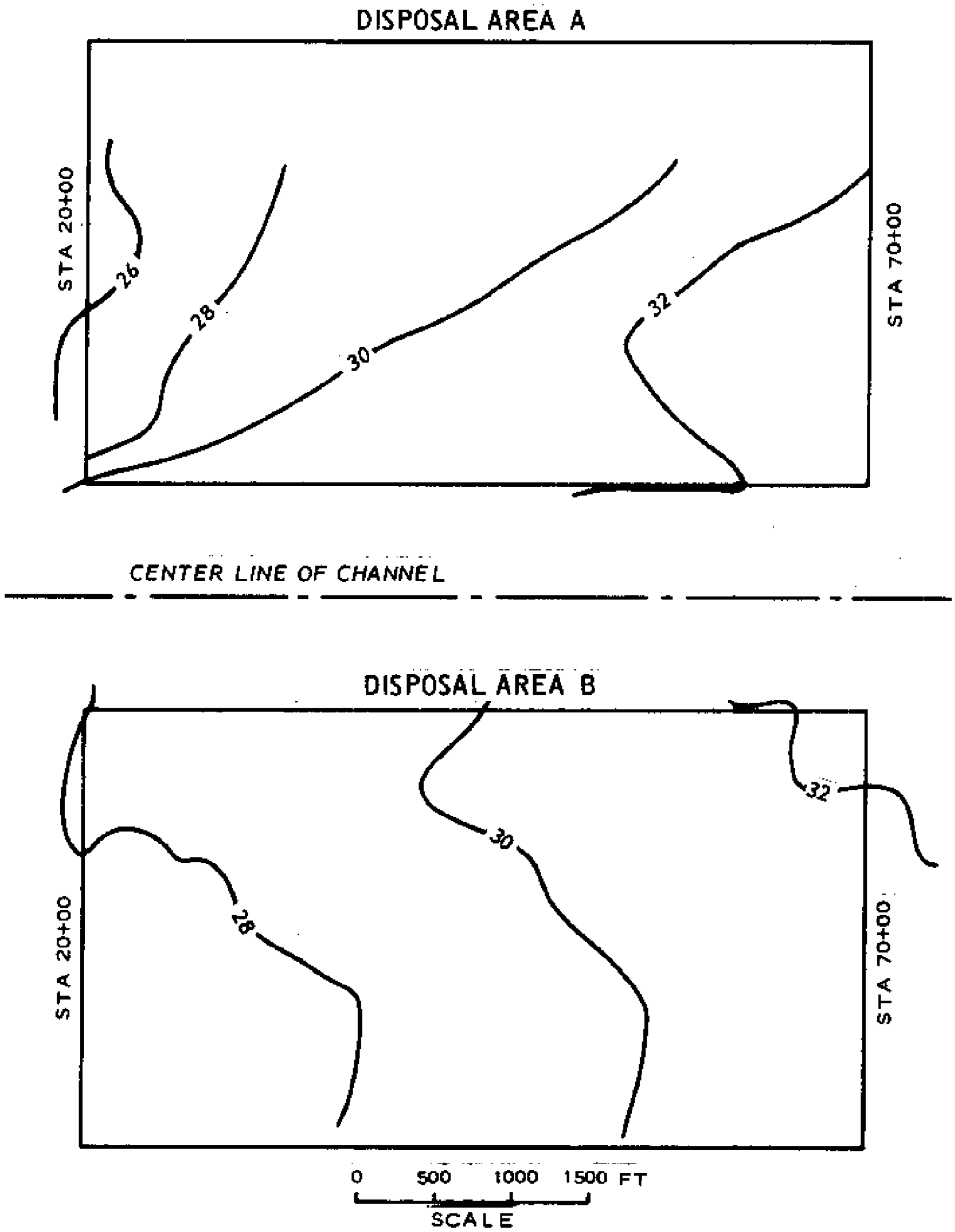


Figure 2. Contoured disposal areas at Freeport Harbor, Jan 1964

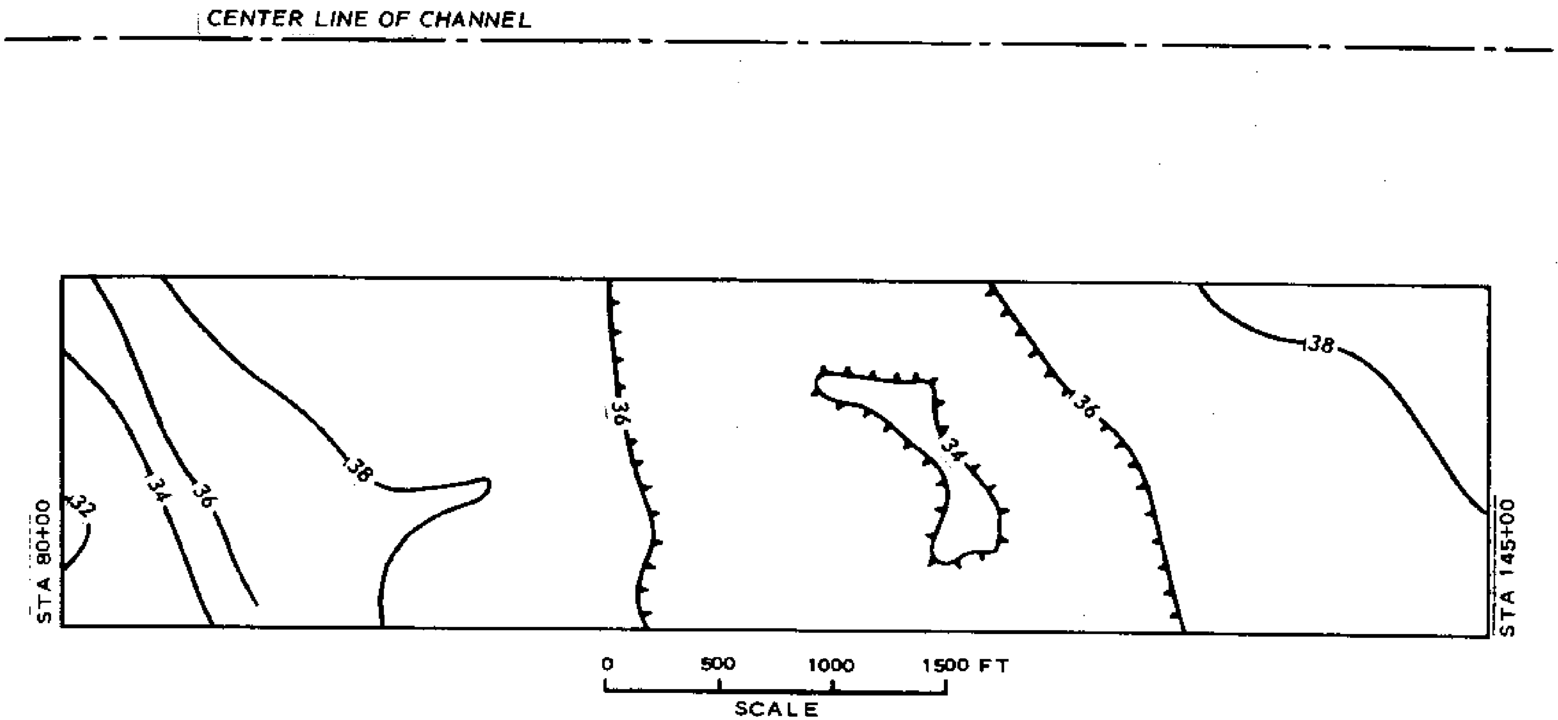


Figure 3. Contoured disposal areas at Port Aransas, Corpus Christi, Aug 1961

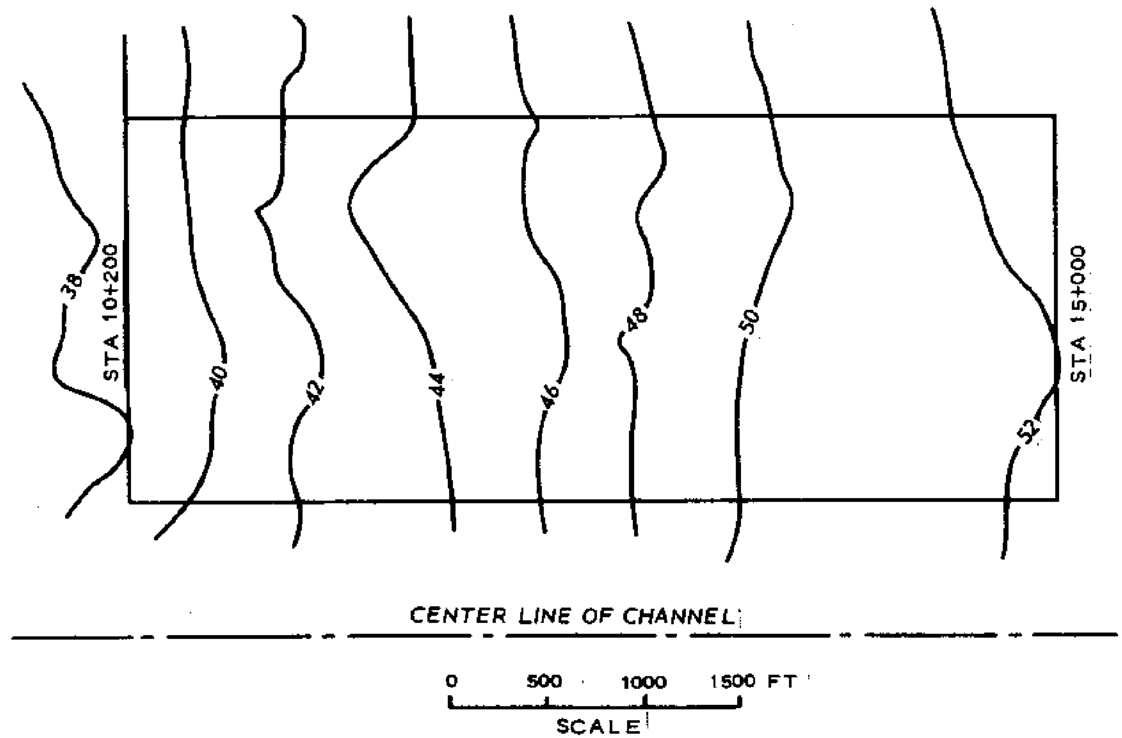


Figure 4. Contoured disposal areas at Brazos I, Apr 1964

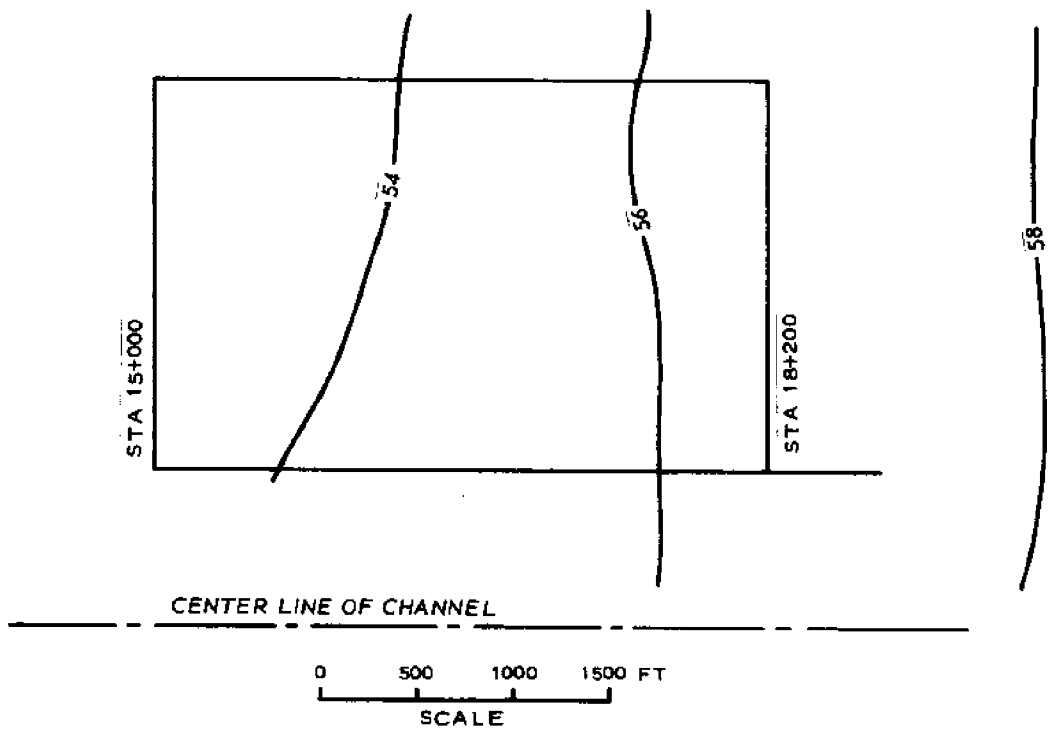
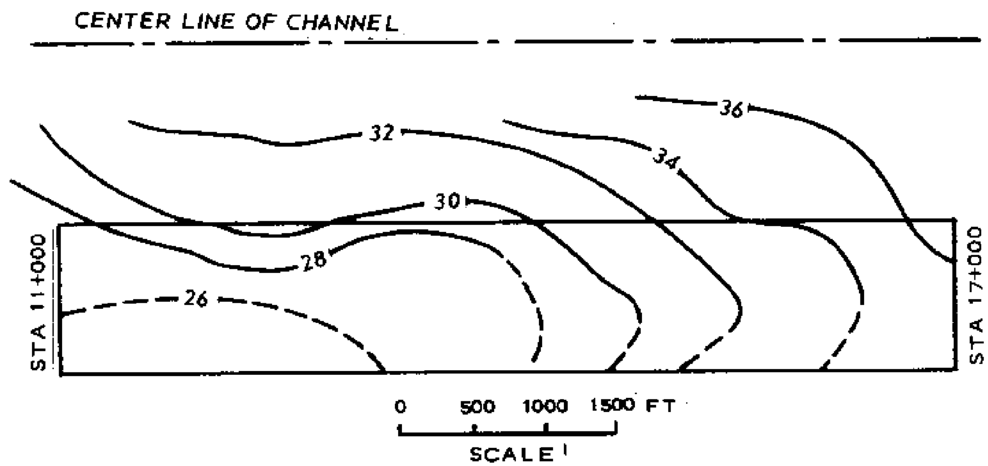


Figure 5. Contoured disposal areas at Brazos II, June 1968



NOTE: DASHED LINES INDICATE THAT CONTOURS WERE ESTIMATED.

Figure 6. Contoured disposal areas at Matagorda Ship Channel, Jan 1968

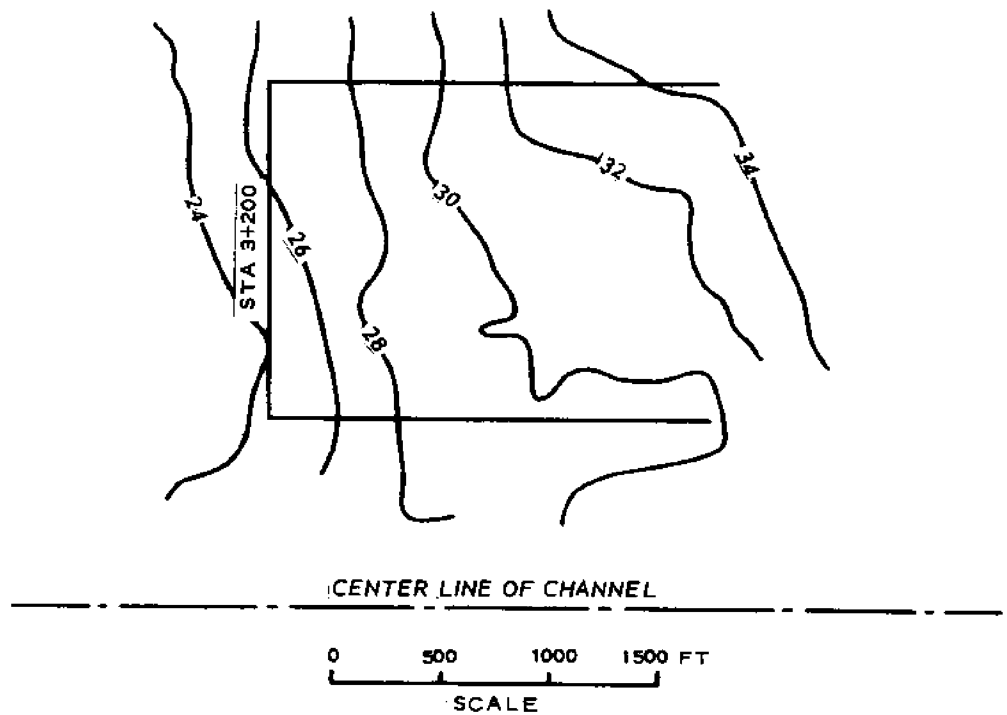


Figure 7. Contoured disposal areas at Port Mansfield, Oct 1965

PHYSICAL AND CHEMICAL CHARACTERIZATION OF DREDGED MATERIAL INFLUENTS AND EFFLUENTS IN CONFINED LAND DISPOSAL AREA

by

Ronald E. Hoeppel[1]

ABSTRACT

Nine dredged material land containment areas, located at upland, lowland, and island sites, were monitored during hydraulic dredging operations in fresh-and brackish-water riverine, lake and estuarine environments. Influent-effluent sampling at the diked disposal areas showed that, with proper retention of suspended solids, most chemical constituents could be removed to near or below background water levels.

Most heavy metals, oil and grease, chlorinated pesticides, and PCB's were almost totally associated with solids in both the influent and effluent samples. The parameters which appear to have the greatest potential impact as a result of land disposal of dredged material are ammonium, soluble manganese, total mercury, and dissolved oxygen. However, none of these should present serious problems after dilution of the effluent discharge in the receiving waters. Actively growing vegetation in disposal areas appeared to be efficient in removing ammonium to low levels and also for filtering out suspended solids. Dissolved oxygen ranged from 0.6 to 12.5 ppm. The lowest dissolved oxygen values were observed when suspended solids and/or nutrients were high in the effluents.

Geochemical phase partitioning of influent and effluent solids indicated that carbonate solids of several heavy metals tended to form during land disposal, along with noticeable increases in metal fixation with iron and manganese precipitates. Metal adsorption onto suspended

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particles (exchangeable phase) also increased slightly, along with a small increase in action exchange capability of effluent solids.

INTRODUCTION

Recent legislation has given the Corps of Engineers greater regulatory jurisdiction over lands adjacent to navigable waterways, including wetlands and drainage systems from upland areas. Since upland areas along navigable waterways are becoming scarce, there is also increasing economic pressure to create new upland areas by disposing dredged material on wetlands and marshlands. Additionally, there is increasing emphasis on the land containment of highly contaminated or toxic dredged material, instigated by growing concern about the pollution potential of open-water or near-shore disposal operations. Irrespective of any potential environmental impact created by the disposal of dredged material in aquatic systems, one must keep in mind that land disposal produces effluent and leachate discharges, which can irreversibly impact sensitive wetland or upland habitats.

There have been only limited studies concerning the pollution potential and physicochemical changes which are induced by the disposal of dredged material in land containment areas. Some research has suggested that the mobility or availability of many harmful chemical constituents in dredged material can be accentuated by changing environmental conditions.[19,20] The placement of reduced subaqueous sediments on aerobic upland soils certainly should not create stable environmental conditions. However, other studies[1,2,13,17] have failed to demonstrate significant releases of contaminants in disposal area effluent discharges, with the noted exceptions of ammonia[2,7,19,20] and orthophosphate[2,17,19]. Due to the paucity of information available and conflicting findings, a comprehensive field study concerning the impact of land disposal was warranted.*

SITE SELECTION

Nine different confined land disposal areas were monitored in different geographic settings, including three freshwater and six brackish water dredging locations. Descriptions of each site are given in Table 1. These sites were chosen on the basis of dredge site sediment and water variability, including suspected high concentrations of contaminants such as oil and grease, chlorinated pesticides, PCB's, nutrients, and heavy metals. Investigated dredged material characteristics that could greatly influence the mobility of contaminants include the sediment texture, oxidation-reduction status (Eh), pH, sulfide and organic matter contents, water salinity and alkalinity, and solids to water ratio of the dredged slurries. Disposal area characteristics were also considered, including the geographic location, effective size, potential slurry residence time, degree of ponding, extent of vegetation cover, and past history of each site.

TEST PROGRAM

The relationship between slurry residence time and effluent quality was evaluated by considering all of the sites but in particular the comparisons between cross-dike and final effluent samples collected concurrently at

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* This paper summarizes data and findings presented in Technical Report D-78-24 (June 1978), U. S. Army Engineer Waterways Experiment Station, Environmental Laboratory, Vicksburg, Mississippi 39180.

the Sayreville, N. J., and Seattle, Wash., disposal areas. The effect of increased residence time on ammonium and phosphate release, in conjunction with pH changes, was evaluated by continuous monitoring of effluents from the Vicksburg, Miss., disposal area during and after completion of the disposal operations. The heavily vegetated containment area at Southport, N. C., was monitored primarily to assess what influence actively growing vegetation might have on effluent quality; the influence of dormant winter vegetation in disposal areas on contaminant release was evaluated by monitoring the heavily vegetated Wilmington, N. C., site. Salinity effects were evaluated by comparing trends at all of the disposal areas because of the wide range of salinities encountered. However, other physicochemical variables, such as those prevalent in freshwater versus marine environments (e.g., variance in sulfide levels) were always considered in context with the salinity comparisons.

The mobility and toxicity of trace metal contaminants are regulated by the chemical compounds with which they become associated. Although the number of discrete compounds is immense, the association of a contaminant with a general group of chemical complexes can be determined by subjecting the sediment to different specific chemical extractions or treatments. These will be referred to as "geochemical phase partitioning." The association of metals with "geochemical phases" such as soluble, exchangeable, acetic acid extractable (carbonate), and manganese and amorphous iron oxide (easily reducible) sediment components was determined for influent and effluent solid phase samples from Wilmington, N. C., Richmond, Va., Lake Charles, La., and Seattle, Wash.; the hydrogen peroxide-oxidizable (organic-sulfide) component was also determined in the Seattle samples. The metals, which are mainly bound in very stable crystalline matrices, were included in the analysis of final total acid digests (residual phase) of samples from the four sites.

FIELD AND LABORATORY METHODS

More than 50 different physical and chemical parameters were determined in total samples, 0.45-μm filtrates, and greater than 0.45-μm suspended solids of disposal area influents and effluents, and surface background water samples. Influents were generally collected beneath the end of the dredge discharge pipe in the turbulent mixing pool; effluents were obtained either at the outfall pipe beneath the sluice or from the back side of a weir structure; surface background water samples were collected from the water body receiving the effluent discharge. Compositing of at least three subsamples was performed in most cases to obtain more representative samples. Three to four daily samples were collected from the monitoring stations at each site either consecutively or during separate trips; six samples were obtained at the Southport disposal area but in the vegetation interaction study these were divided into an initial and final set for comparative purposes.

Collapsible 4ℓ polyethylene containers were employed for collection of samples used for heavy metal, nutrient, and oil and grease analyses, after being prewashed with 0.1 M hydrochloric acid and rinsed twice with deionized-distilled water. Usually, four of these containers were used; 50 ml of chloroform was added to one container as a preservative for nutrients. Samples for chlorinated hydrocarbon analyses (pesticides, PCB's) were collected in 2ℓ glass wide-mouth jars. The containers were prewashed with hexane, rinsed twice with deionized-distilled water, and combusted at $350^{\circ}C$ for 30 minutes in a muffle furnace. All containers were completely filled with sample to exclude air, and the polyethylene containers were collapsed as aliquots were removed.

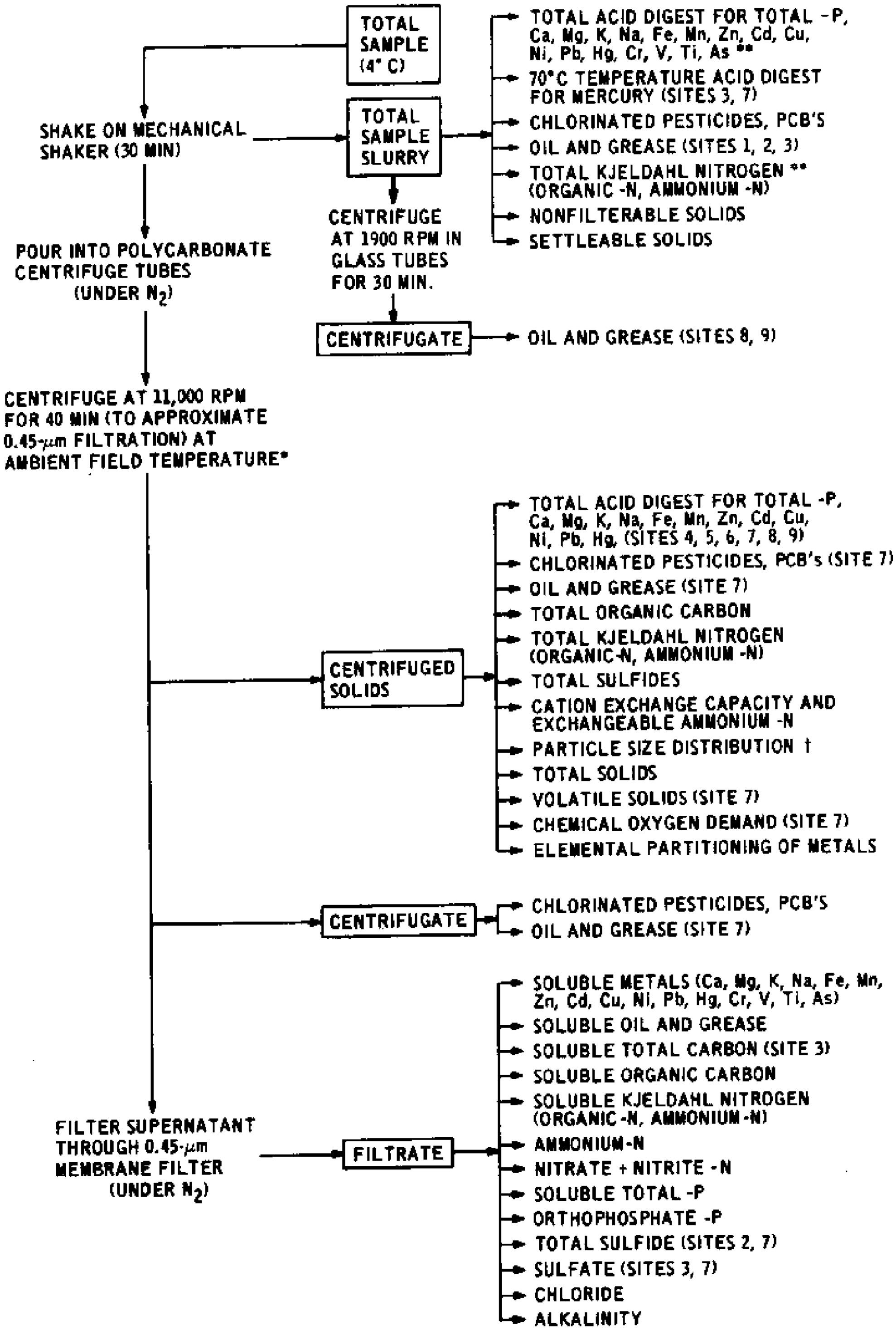
All samples were packed in ice immediately after collection and shipped by air freight to an analytical laboratory. Samples from sites 1-3 (see Table 1) were sent to the Environmental Engineering Laboratory at the University of Southern California in Los Angeles, while the samples from sites 4-6, 8, and 9 were shipped to the Environmental Laboratory at WES. Sample collection and preparation for site 7 were performed by the personnel of the Environmental Protection Agency (EPA) Region X Laboratory in Seattle, Wash. Samples were stored in environmental chambers maintained at $4^{\circ}C$.

Salinity, conductivity, dissolved oxygen (DO), slurry pH, and water temperature were measured in the field concurrently with influent, effluent, and background water subsampling. Disposal area sediment pH and oxidation-reduction potential (Eh) were obtained in fresh sediments in the disposal areas; depending on site conditions, 6 to 40 measurements were made at each site.

Upon arrival at the analytical laboratory, the field-collected samples were processed as soon as possible to separate the solid from the liquid phase and to prepare each phase for different chemical analysis. Sample phase separation (centrifugation and filtration), total and filterable sulfides, and geochemical phase partitioning extractions were performed in a glove bag that was purged continuously with nitrogen gas to preserve the anaerobic integrity of the samples. The sequential preparation scheme for influent samples is shown in Figure 1, while the slightly different scheme used for effluent and background water samples is shown in Figure 2. The modified preparatory procedures were necessary because of the often extreme variations in solids content between respective influent, effluent, and background water samples. Generally, if an influent sample was low in suspended solids, the total sample was subjected to total acid digestions for metals, phosphorus, and total Kjeldahl nitrogen (TKN).

The parameters analyzed in the less than 0.45-μm (soluble phase) fractions of samples are listed in Figures 1 and 2. The soluble phase separations for chlorinated pesticide, PCB, and oil and grease determinations involved only high-speed centrifugation in stainless steel centrifuge tubes to approximate 0.45-μm filtration. The remaining parameters, listed in Figures 1 and 2 as filtrates, were determined in liquid which was both centrifuged (in polycarbonate centrifuge tubes) and then filtered through 0.45-μm nitrocellulose membrane filters. The filters were previously washed twice with 1 M hydrochloric acid and deionized-distilled water to ensure the removal of acid-leachable chemical constituents present on or in the filters. Meticulous cleaning of all labware was routinely practiced, following precautions described elsewhere[6,7]. All centrifugation and filtration steps were performed in a nitrogen gas atmosphere.

Following the separation of the solid and soluble phases, different aliquots were preserved according to standard procedures[10,11]. The solid phase material was placed in small plastic specimen cups under a nitrogen gas atmosphere and tightly sealed. Soluble phase samples were placed in tightly capped polyethylene bottles. All of the prepared samples were stored at 4°C until further processing and analysis. Detailed descriptions of the analytical methods and instrumentation used in the study are cited elsewhere[5,12,15]. Most parameters were measured according to Standard Methods[3,4] or procedures recommended by the EPA[11].



* SITE 7: 12,500 RPM FOR 20 MINUTES AT 4°C; ALL CENTRIFUGATIONS WERE MADE IN A SORVALL GSA ROTOR.

** TOTAL SAMPLE WAS DIGESTED ONLY WHEN THE SOLIDS WERE LOW.

† THE <2-μm (CLAY) FRACTION WAS FURTHER ANALYZED WITH A COULTER COUNTER.

Figure 1. Influent sample preparation

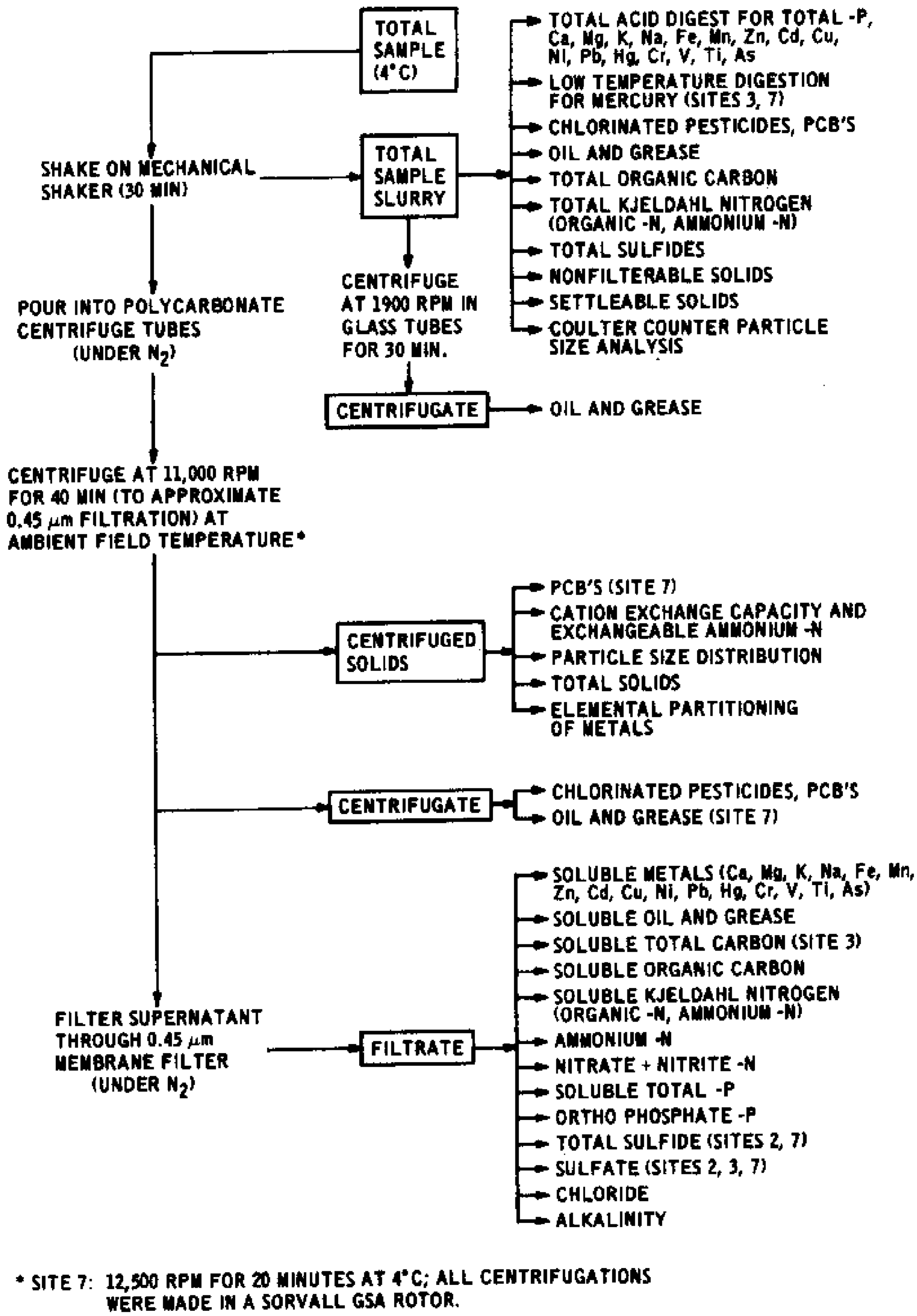


Figure 2. Effluent and background water sample preparation

Quality control within and between laboratories consisted of multiple digestions and analyses, standard addition, and interlaboratory correlation of select samples. About half of the samples were subjected to these control measures.

Geochemical Phase Partitioning Analysis

The geochemical phase partitioning analyses were performed for influent and effluent slurry solids from sites 4-7, according to the methods outlined by Chen et al[7]. They include sequential extractions with chemicals which selectively remove metal contaminants associated with certain major geochemical complexes or phases. A detailed description of the procedures is cited elsewhere,[12] but a generalized pairing of the extracted phases with the extractants, in sequential order, is as follows:

a. Exchangeable phase: 1.0 M ammonium acetate.

b. Acetic acid extractable (carbonate) phase: 1.0 M acetic acid.

c. Easily reducible (manganese-amorphous iron oxide) phase: 0.1 M hydroxylamine hydrochloride in 0.01 M nitric acid.

d. Organic and sulfide phase: 30% acidified hydrogen peroxide.

e. Remaining (Residual) metals: total hot acid digest.

Extractions were conducted by shaking the extractant with wet solids for 30 minutes on a mechanical shaker, followed by centrifugation and 0.45-μm membrane filtration under nitrogen gas.

RESULTS

Site Variability

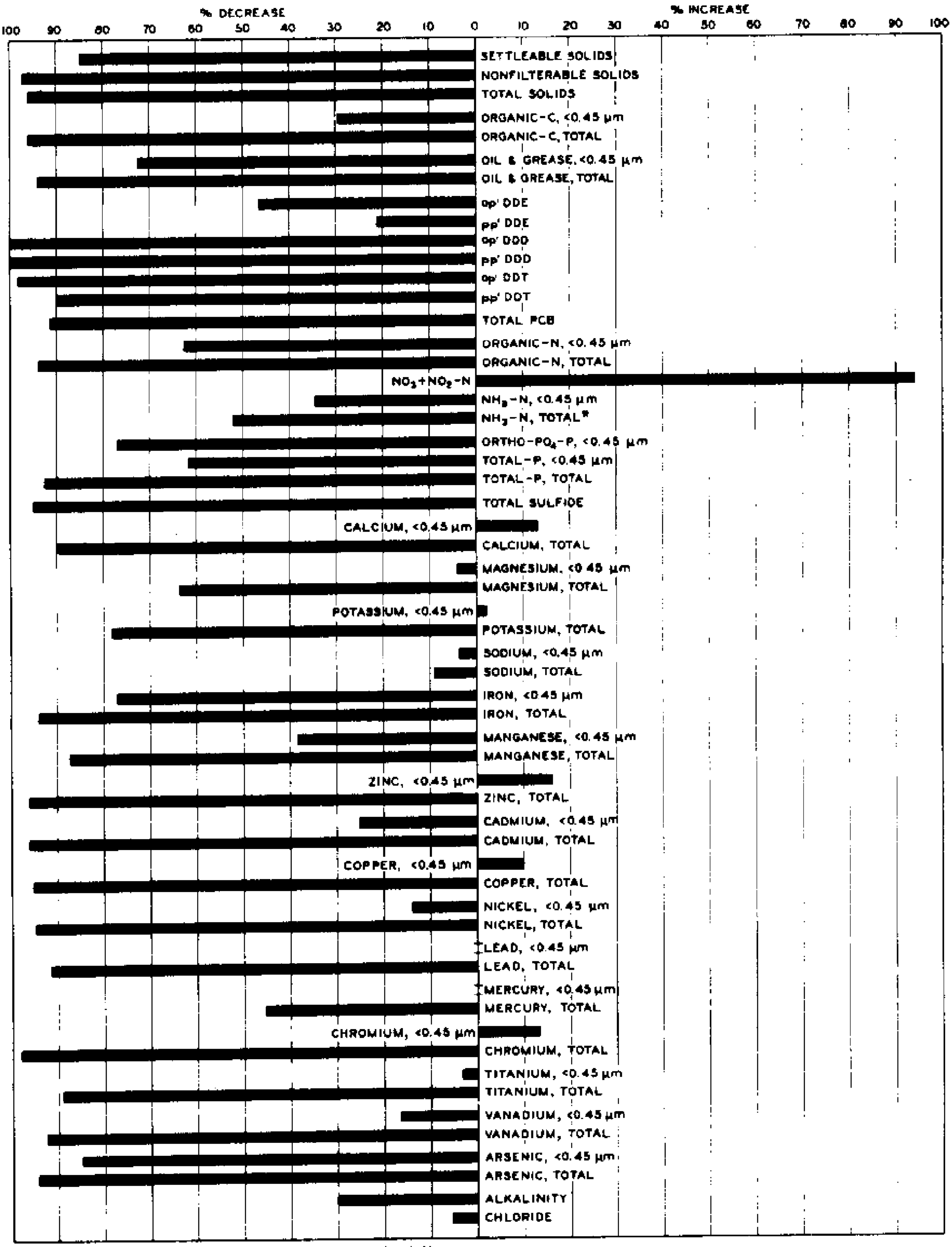
The monitoring of nine different land containment areas during dredged material disposal operations was intended to aid in an assessment of the environmental impact of this mode of disposal. The general site descriptions are given in Table 1. The values for many physicochemical parameters measured in waters and sediments at each site are listed in Table 2. Many of these parameters govern the mobility of contaminants in sediment-water systems as well as being contaminants in their own right. Data on each measured parameter, including the total number of samples, ranges, means, standard deviations, and analysis of variance F values for surface background water, influent, and effluent, are presented in Table 3. The F values show whether variations between data sets are significantly different from variations within each tripartite set. The nature of the variances can be roughly assessed by consulting the means, ranges, and standard deviations given for each sample source. However, wherever variations between two means within a data set appeared to be significant, a Student's t-test was used to determine their significance.

Generally, 77 to 98 percent by weight of disposal area influents consisted of sediment pore and bottom water from the dredged channel (Table 2), and at least 80 percent of this total water should be bottom water. Site salinities ranged from fresh to over 20 0/00, particle size distributions from 5 to 65% sand and 11 to 65% clay-sized particles, and cation exchange capacity from 10 to 80 meq/100g of dry weight dredged sediment. Although DO was usually near zero when measured directly at the influent discharge pipe, it averaged 3.8 ppm when monitored in the mixing pool beneath the pipe discharge (Table 3).

In order to determine dredged material contaminant mobility and fate in land disposal areas, contaminant levels in influent and effluent samples should be compared. Comparisons should also be made between effluents and background water to roughly assess the potential impact of the effluents on water quality near the discharge site. The information in Table 3 presents a general idea of the changes in contaminant levels which were encountered during influent, effluent, and background water monitoring.

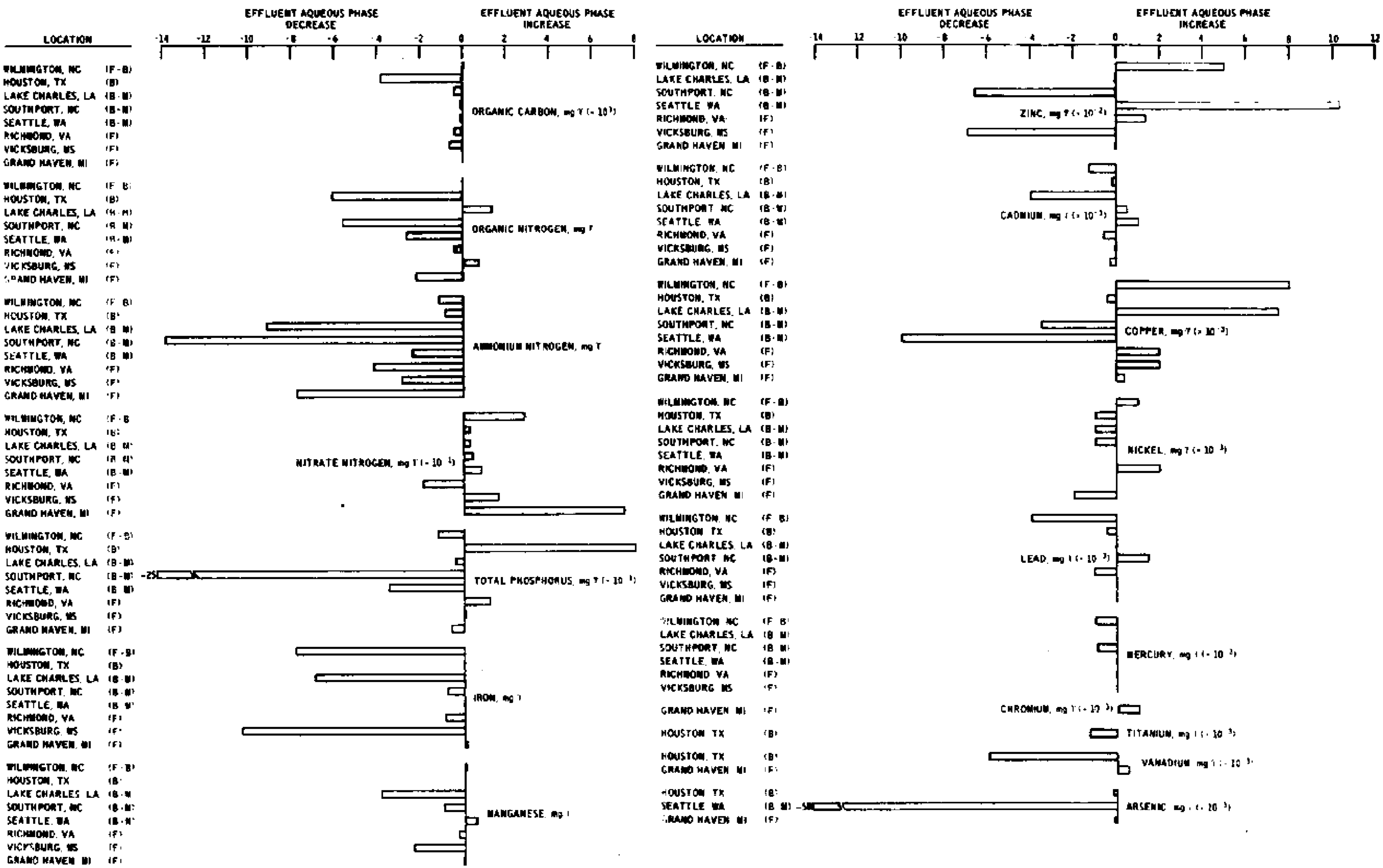
Influent-Effluent Comparisons

Comparisons between total influent and effluent digests showed prominent net decreases for all nutrients, oil and grease, PCB's, DDT, DDD, and most major elements and trace metals during land containment as shown in Figures 3 and 4. However, site variability was great for most contaminant levels in effluents, as shown in Figure 5. Despite the often very great variance in the solids content of the influent samples, most total contaminants showed highly significant differences between influent, effluent, and background water concentrations (Table 3). The statistical nonsignificance shown for solid phase sulfide, DDT, DDD, and possibly total mercury is due mainly to influent solids variability, which in reality has a minimal impact on the effluents because of attenuation of the rapidly fluctuating influents. Sodium, mercury, and DDE were the only parameters which did not seem to be greatly reduced in the total effluents. For most elements, the percent decrease in total concentration was very close to the respective decrease in suspended (nonfilterable) solids during containment, which averaged 97.2% for all sites (Figure 3). However, some variances in elemental versus solids removal efficiency seemed to be greater than analytical error. Despite overall significant decreases in total samples, organic nitrogen, phosphorus, potassium, sodium, manganese, zinc, and mercury showed major increases in the effluent solid phase (Table 3). These contaminants were probably mainly associated with very small or low density (e.g., organic) particles. Other studies have shown similar relationships in dredged material from land containment areas[15], marine sediments[10], and in ocean discharges of wastewater effluents[8].



*EXCHANGEABLE AMMONIUM-N + <0.45 µm AMMONIUM-N

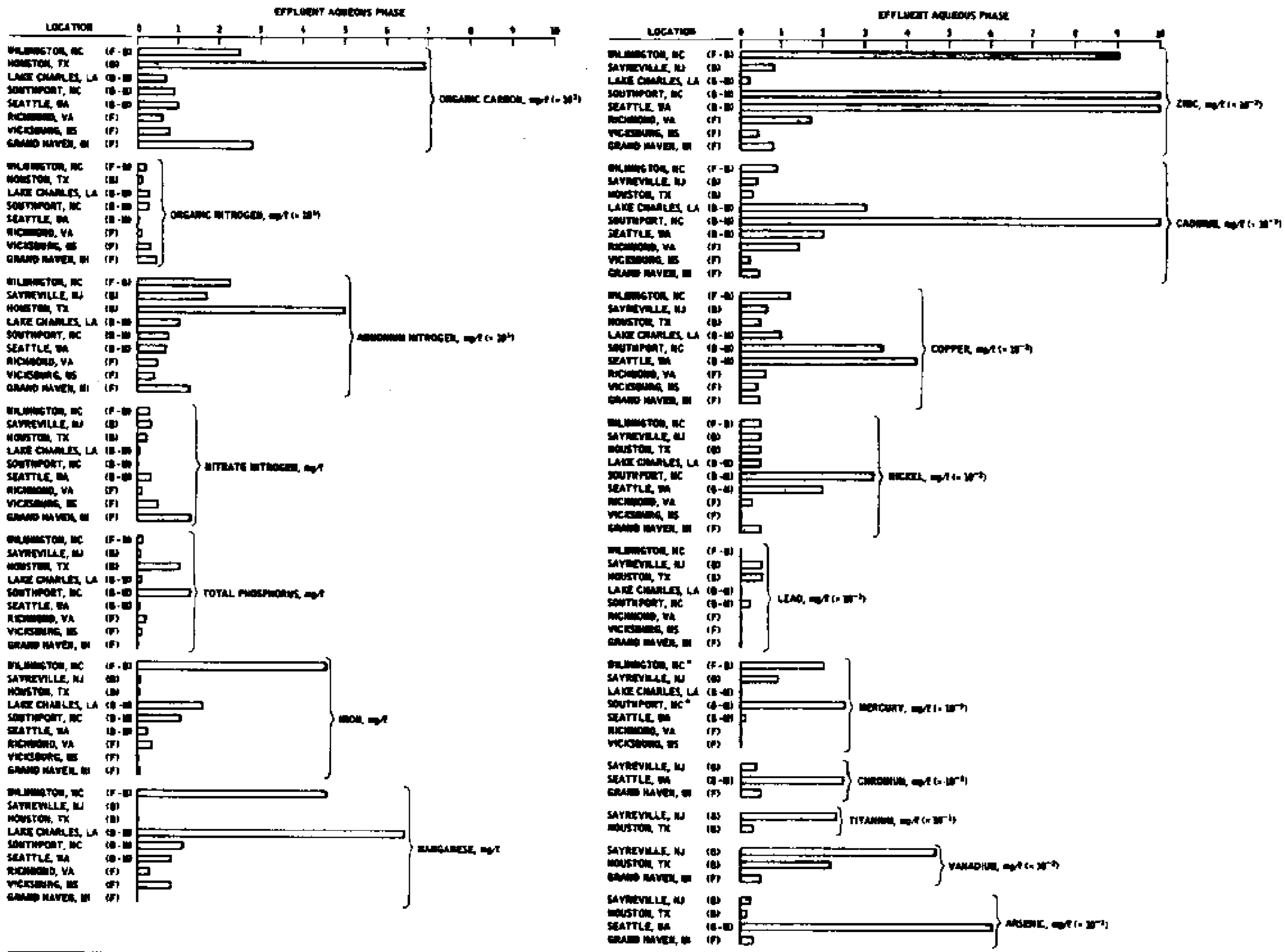
Figure 3. Percent increase or decrease of physical and chemical parameters in total and soluble phase effluent dredged material based on influent-effluent samples



(F) : FRESH WATER — INFLUENTS WITH < 0.1% SALINITY
(F-B) : FRESH BRACKISH — INFLUENTS WITH 0.1 TO 0.5% SALINITY
(B) : BRACKISH WATER — INFLUENTS WITH 0.5 TO 1.5% SALINITY
(B-M) : BRACKISH MARINE — INFLUENTS WITH > 1.5% SALINITY

NOTE: DISPOSAL AREA LOCATIONS ARE LISTED IN THE ORDER OF THEIR DECREASING DETENTION TIMES BASED ON SIZE, INFLUENT EFFLUENT FLOW RATES, AND TORTUOSITY

Figure 4. Changes in the nutrient and heavy metal concentrations in the aqueous phase of effluents from confined disposal areas



(F) = FRESH WATER: INFLUENTS WITH < 0.1% SALINITY
(F-B) = FRESH-BRACKISH: INFLUENTS WITH 0.1 TO 0.5% SALINITY
(B) = BRACKISH WATER: INFLUENTS WITH 0.5 TO 1.5% SALINITY
(B-M) = BRACKISH-MARINE: INFLUENTS WITH > 1.5% SALINITY

* ANALYSES WERE MADE WITH A ZEEMAN SPECTROPHOTOMETER, WHICH GAVE HIGHER VALUES THAN THE COLD VAPOR METHOD.

NOTE: DISPOSAL AREA LOCATIONS ARE LISTED IN THE ORDER OF THEIR DECREASING RETENTION TIMES, BASED ON SIZE, INFLUENT-EFFLUENT FLOW RATES, AND TORTUOSITY.

Figure 5. Concentrations of nutrients and heavy metals in the aqueous phase of effluents from the confined disposal areas

The only measured parameters which exhibited removal efficiencies of less than 90% in total sample preparations include: titanium (89%); manganese (88%); settleable solids (85%); potassium (78%); magnesium (64%); ammonium nitrogen (57%); mercury (46%); op´ DDE (46%); and pp´ DDE (21%). The chlorinated pesticide analogs of DDE were at similar concentrations in both the effluent and background water samples.

Nitrate-nitrite nitrogen was the only chemical parameter which showed a significant increase during land containment of dredged material (Figure 3). The concentration increased almost twofold, from 0.18 to 0.35 mg/ℓ. However, this increase should not create serious impact since the discharge concentration was generally below the surface background water average of 0.46 mg/ℓ (Table 3) and well below accepted water quality standards[9]. The greater than threefold increase in soluble chromium indicated by Table 3 is the result of high levels of chromium in effluents from the Seattle, Wash., disposal area; chromium was not measured in the influent samples from Seattle. Excluding the Seattle data, chromium showed only a very small increase, as shown in Figure 3. High concentrations of soluble ammonium nitrogen and manganese occurred in some influent and effluent filtrates. Soluble ammonium nitrogen in the influents averaged 20.8 mg/ℓ, with an upper range of over 70 mg/ℓ; effluent samples averaged 13.6 mg/ℓ, with maximum concentrations also around 70 mg/ℓ. Soluble manganese showed a comparable decrease during residence in the land containment areas; the influent and effluent means were 2.35 and 1.45 mg/ℓ, while maximum values were 14.4 and 8.0 mg/ℓ, respectively. Some total alkalinity values (as calcium carbonate) were also very high, giving average influent and effluent values of 412 and 287 mg/ℓ in comparison to a background water average concentration of 88 mg/ℓ (Figure 3 and Tables 2 and 3). Dissolved oxygen in effluents, based on multiple surface water measurements made inside of the disposal areas adjacent to the discharge weirs, fluctuated greatly. Ranges were from 0.6 to 12.5 mg/ℓ, with an average of 5.3 mg/ℓ. Thus low effluent DO can occasionally be a problem.

Average soluble phase concentrations of zinc and copper were slightly higher in effluents. However, as indicated in Figure 4, their increased mobility was site-specific and their levels (Figure 5) were below the present drinking water standard and criteria for aquatic life[9].

Geochemical Partitioning of Influent and Effluent Solids

The potential mobility or toxicity of an element is highly dependent on the geochemical phases with which it is associated in the solids fraction. The exchangeable ions are considered to be readily available to aquatic organisms as they are mainly weakly adsorbed on the surfaces of fine solids. The carbonate phase may also be readily affected by changes in the environment, especially as a result of interaction with or uptake by the biota. Also the carbonates of a given metal are generally more soluble than many other solid phase chemical precipitates, and thus a major shift of a metal to a carbonate complex may result in a similar increase in its soluble phase concentration. The easily reducible phase variations reflect the oxidized or reduced status of a containment area during disposal operations. An increase in this phase would suggest an overall increase in the oxidation of the dredged material slurry, and vice versa. If reducing conditions occur, increased mobility of ions from the easily reducible phase (iron-manganese oxide precipitates) is favored, thus presenting a potential water quality problem. In retrospect, greater mobility may occur from sulfide complexes (organic-sulfide phase) under oxidizing conditions since most metal sulfides are much less soluble than their more oxidized substituents. Thus with changing environmental conditions many contaminants change their mobility and form, and likewise their bioavailability.[18]

Figure 6 gives the amounts and percentages of each of 14 nutrient and trace metals that were solubilized during sequential chemical extractions of five influent and effluent solids. Some metals exhibited noticeable phase changes during migration of suspended solids in the dredged slurry across land containment areas, while other metals showed little change. About a third of the solids-bound calcium and sodium were removed from the influent solids during extraction of the exchangeable phase, with measurable increases in exchangeable calcium, sodium, copper, and arsenic noted in effluent solids. Exchangeable phase manganese, magnesium, and cadmium were similarly high ($\sim$ 10 percent) in both influent and effluent solids. Most metals showed increases in carbonate phase concentrations as a result of confined disposal. Influent solids generally showed high carbonate phase values for cadmium and manganese, while zinc, cadmium, manganese, lead, copper, and sodium showed major increases in effluent solids. Carbonate phase cadmium, zinc, and manganese composed 57, 33, and 20 percent of the effluent solids, respectively. Iron, manganese, cadmium, and copper increased in the easily reducible phase of the effluent solids, although only manganese showed a major increase. Upon total digestion of the remaining influent and effluent solids, most metals (except for iron, nickel, and chromium) showed noticeable decreases in the effluent digests. A limited amount of data on the organic-sulfide phase (Seattle site) suggests that the concentration decreases were mainly associated with reductions in solid phase organic and/or sulfide complexes during disposal area detention under oxidizing conditions.

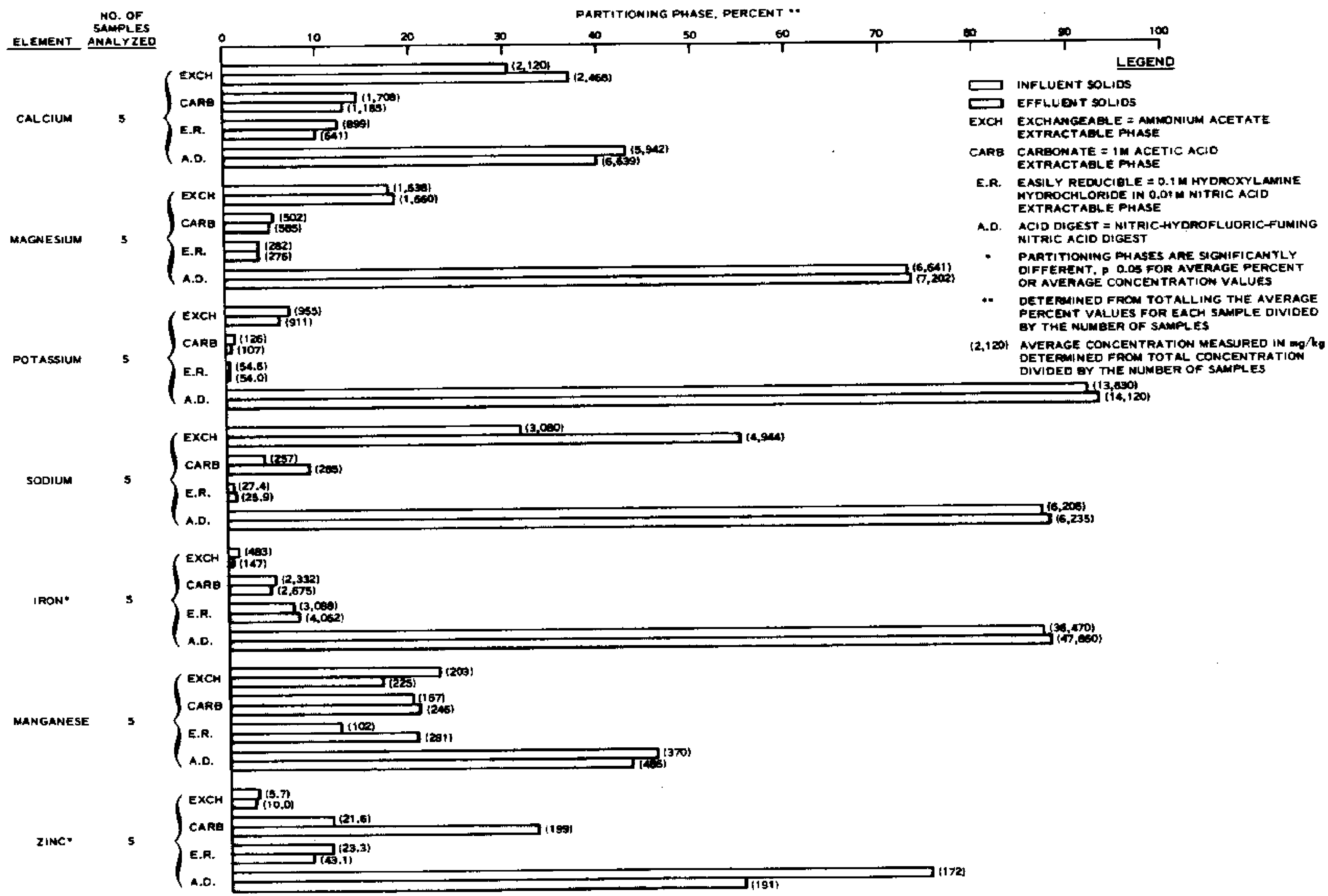


Figure 6. Geochemical phase partitioning of metals in influent and effluent solids from four confined land disposal areas (sheet 1 of 2)

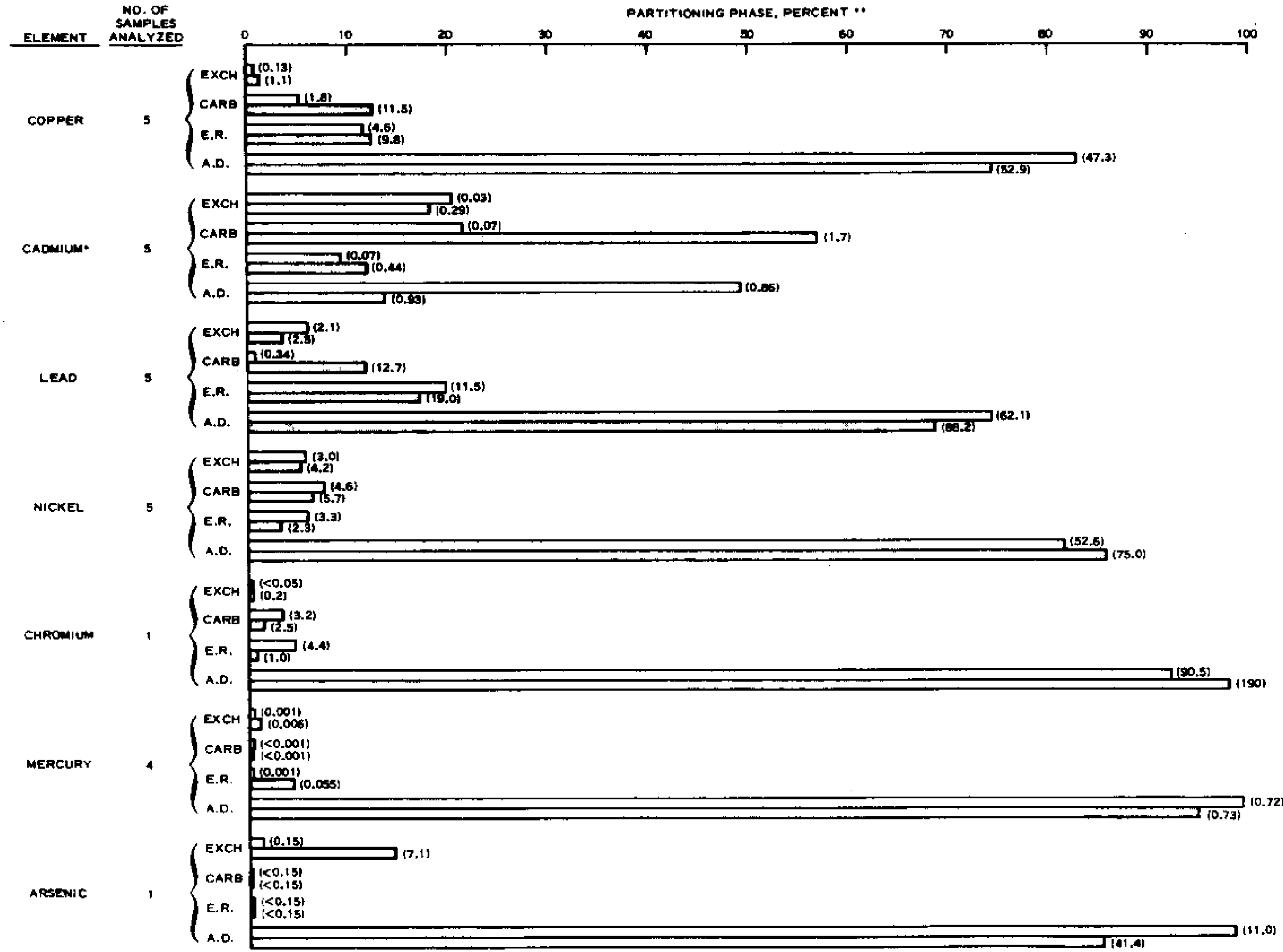


Figure 6. (sheet 2 of 2)

Metals showing major geochemical phase changes included manganese (easily reducible phase), cadmium (carbonate and easily reducible phases), zinc (carbonate phase), lead (carbonate phase), copper (carbonate phase), sodium (exchangeable phase), and calcium (exchangeable phase). Metals showing little change in phase during the solids detention times included chromium, nickel, mercury, potassium, and magnesium. Iron could not be properly evaluated because of its high total concentration. Arsenic showed a large increase in an exchangeable phase extract of an effluent sample from the Seattle site.

Effluent-Surface Background Water Comparisons

Suspended (nonfilterable) solids were 47 times higher in effluents than in adjacent surface background water (Table 3). Therefore, it is not surprising that most total sample contaminant levels are also correspondingly higher, as depicted in Figure 7. The only contaminants which were proportionately much higher than the suspended solids level in average effluents were total lead (125X) and total manganese (74X). Contaminants which had appreciably lower levels in total effluent samples than the respective suspended solids contribution included: PCB's, DDT, DDD, DDE, copper, chromium, sulfide, zinc, nickel, vanadium, calcium, magnesium, potassium, and sodium (Table 3). The major source for the total manganese, DDE, and major ions (Ca, Mg, Na, K) was the soluble phase.

The only measured soluble phase parameters which were appreciably higher than the background water levels include: ammonium nitrogen (50X), manganese (25X), alkalinity (3X), zinc 2.5X), copper (2.5X), and iron (2.5X) (Figure 7). Titanium and vanadium were significantly higher in effluents at the Houston site, but were at background levels at two other

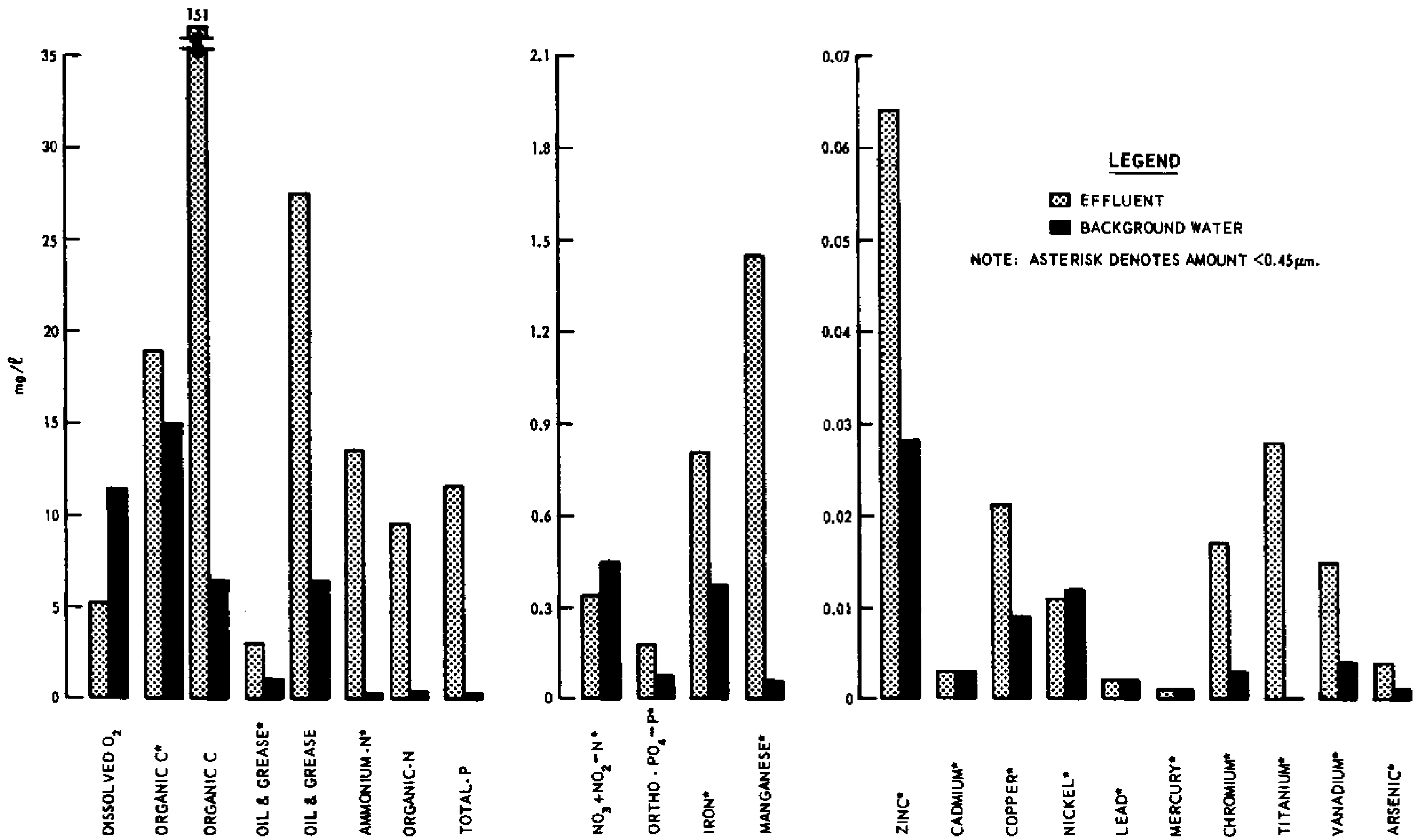


Figure 7. Average concentrations of nutrients, organic contaminants, and soluble phase heavy metals in effluents and surface background water samples representing the confined disposal areas

monitored sites. Despite the data given in Figure 7, chromium and arsenic were not shown to be higher in effluents since high effluent values for each at the Seattle site could not be matched with missing background water data. More information is required for those trace contaminants which are not commonly monitored.

The primary source for the DDE and nitrate levels in effluents seemed to be water incorporated with the dredged sediments since higher levels for each were present in surface background water samples (Table 3). The concentrations of major ions (Ca, Mg, Na, K, Cl) were significantly higher in effluents than in background water because of salinity stratification in brackish dredge site waters.

The association of trace contaminants with very fine particles was observed. This was particularly evident for mercury and potassium at most sites. Combined geochemical phase partitioning and particle-size fractionation of effluent solids show the association of potassium with very fine silicate mineral particles, e.g., illite clay. Fine clays, as well as manganese and iron oxide precipitates and low density organic detritus, may be important media for the transport of adsorbed contaminants. This is supported by high exchange capacity (Table 3) and increased levels of exchangeable cations (Figures 3 and 6) associated with effluent suspended solids.

DISCUSSION

The removal efficiency for most heavy metals closely paralleled the removal of the solids during dredged slurry containment in land disposal areas. However, different metals seemed to have varying affinities for different geochemical phases and particle sizes of suspended solids. Total mercury in effluent samples decreased by only 46 percent, which indicates that it was often associated with a fine particulate fraction and/or one of low specific gravity (e.g., organic suspended solids). Soluble phase titanium in the Sayreville samples mainly originated from the settling of very fine titanium oxide aerosol particles into the water near the dredging site. Most of the DDE also seemed to be associated with very fine particles present in dredged site waters. Thus, the filter size and the instrument employed for analyses are of great importance in determining "soluble phase" concentrations. The impact that fine particulate matter has on aquatic life is not well documented, but the findings of this study suggest that perhaps suspended solids should be included in any predictive test or effluent analysis; the methods used to separate solids and analyze samples should be well documented. However, bulk analyses (acid digests) of bottom sediments or influents are not recommended as a predictive tool as they generally show a poor relationship to contaminant mobility or availability.

Most of the chlorinated hydrocarbons (pesticides, PCB's), with the exception of DDE, showed very efficient removal when proper solids retention was maintained in confined disposal areas. The source for the DDE was probably not the dredged sediments since comparable DDE concentrations were observed in surface background water samples. Oil and grease were generally removed efficiently during dredged slurry containment. However, sediments containing high levels of petroleum residues seemed to settle more slowly, often creating highly fluid oil-water-sediment suspensions near the bottom of ponded areas. Poor site management may effect the release of these suspensions, resulting in a poor effluent quality.

Analytical data for influent and effluent sample filtrates showed that soluble phase ammonium nitrogen was released in high concentrations from some bottom sediments. Ammonium release was most frequently directly related to organic nitrogen and TKN concentrations in the bottom sediments. Soluble phase ammonium nitrogen concentrations in disposal area influent samples averaged 20.8 mg/ℓ with maximum levels of 70 mg/ℓ. Generally, an equivalent amount of ammonium nitrogen was exchangeable from the influent solids. A very rapid initial decrease in soluble phase ammonium was noted in most sites displaying a short slurry detention. This was attributed to sorption by disposal area solids in contact with the slurry and was most pronounced in the presence of fine-grained sediments. Although ammonium nitrogen was removed from the dredged slurry by 57 percent during residency, the observed effluent concentrations could warrant concern, especially if high pH conditions exist in the disposal area or discharge zone, promoting the formation of highly toxic un-ionized ammonia.

Soluble manganese was the only heavy metal that was consistently released at above ambient background water concentrations in effluent filtrates while zinc and copper were significantly higher in effluent filtrates from several sites. High soluble phase concentrations of manganese in effluents, which averaged 1.45 mg/ℓ (maximum: 8 mg/ℓ), resulted from high influent concentrations. Soluble phase iron was also high in influents but showed a high removal efficiency (77 percent). Although copper and zinc concentrations were higher in the soluble phase of many effluents, the levels encountered suggest that only rarely should these contaminants exceed present standards and criteria. The increased mobility of zinc and copper during containment could be promoted by their release from and complexation by organic compounds as well as the observed formation of moderately soluble carbonate minerals. Generally, the soluble phase concentrations of other heavy metals closely reflected the concentrations in comparable background water samples. Soluble phase cadmium showed no major change in concentration during confinement, but there was a major shift of cadmium into the carbonate phase of effluent solids. This suggests that cadmium associated with effluent suspended solids may have greater potential for bioavailability or mobility in receiving waters. However, solid phase cadmium seemed to be readily removed during short-term residency in land disposal areas.

There was a 60 percent increase in the cation exchange capacity of effluent solids. This increase was reflected by general increases of exchangeable ammonium and metals associated with the effluent solids. The dominance of organic detritus and fine particulate potassium (e.g., illite clay) in effluents was probably responsible for this. Increases in the easily reducible geochemical phase also suggest the importance of manganese and iron hydroxide precipitates in sorption phenomena. However, effluents having low solids contents should be negligibly affected by the increase in exchange capacity. Thus dredged slurry retention must be sufficient to allow for efficient suspended solids removal.

Although a direct relationship between carbonate phase shifts and influent-effluent alkalinity values was not consistently noted, high alkalinity undoubtedly played an important role. Alkalinity showed an overall decrease during containment, although the trends were site specific. Major shifts in alkalinity during dredged slurry containment seemed to be promoted by biological activity. The highest influent alkalinity values were noted when the dredged sediments contained a high total organic carbon concentration.

There was a small but significant increase in pH during the dredging and land disposal cycle (Table 3). Algal photosynthesis appeared to be an important source for the increase, with pH values in excess of 9 observed in effluents from a nonvegetated disposal area with a long retention time. High nutrient levels can increase algal growth in site waters, resulting in alkaline pH. The combination of high nutrient and pH levels may have both positive and negative effects; ammonia volatilization may lead to nitrogen depletion of site waters and increased ammonia toxicity could impact both the microbial pathogens in the dredged material and aquatic organisms in the receiving waters. High turbidity and vegetation in the disposal areas should prevent excessively high pH.

Dissolved oxygen averaged 5.3 mg/ℓ in effluents, and generally effluents containing higher solids contents were lower in DO. Some very clear effluents which contained high concentrations of soluble nutrients were also low in DO. Subsurface effluent discharge had lower DO than surface discharge. Low DO values were observed in vegetated overland flow treatment areas. This trend may be prompted by the turbulent mixing and greater contact of reduced sediments with the water in overland flow systems. Influents entering the vegetated overland flow areas were also unusually high in nutrients, which may have prompted low levels of DO by accentuating microbial growth.

In summary, trace contaminants are mainly associated with suspended particles, and turbid water will impact sensitive aquatic life. Recently disturbed bottom sediments, resulting from dredging, will also suppress DO levels in the immediate discharge waters. Therefore good disposal area management requires removal of suspended solids. However, long-term retention of the dredged material supernatant in disposal areas could produce a high pH problem. Although slightly alkaline pH generally favors the removal of soluble contaminants,[14,18] very high values are not desirable. If organic matter and TKN values are high, ammonia toxicity from high pH effluent discharges may create problems and will exceed present criteria[9]. Since the main source for the high pH is probably algal photosynthesis in disposal area water, algal blooms should be watched for and prevented. Such biological blooms could also create unstable physicochemical conditions during the rapid death phase, resulting perhaps in rapid decreases in DO, Eh, and pH. In turn, biochemical and geochemical phases of contaminants may shift, which could increase their mobility.

The increases of nutrient trace metals in soluble phase effluent samples from the Wilmington disposal area, where site water was subjected to long residency in dead vegetation, indicate that the mobility of especially zinc and copper may be favored under conditions of extensive retention time and presence of abundant organic debris (Figure 4). Other studies have shown similar trends[19,20]. However, the Southport disposal area, which contained actively growing thick vegetation, elicited the best removal of nutrients (ammonium and soluble phosphate) and suspended solids of any of the sites monitored, and no significant increases in soluble trace metals were found. Thus, there seems to be a seasonal pattern for best treatment. The presence of vegetation in disposal areas can accentuate slurry solids removal, but it is recommended that such dredging be scheduled for the spring or summer period for best effluent quality. In retrospect, disposal into nonvegetated areas may be best during cooler weather, to avoid algal blooms.

Proper planning and management of land disposal area operations should be a goal of dredging concerns and regulatory agencies. Based on present criteria and standards, the main goal for achieving acceptable effluent water quality should be the removal of suspended solids, even at the expense of increasing the concentrations of certain soluble phase trace contaminants. Most soluble phase trace contaminant levels in effluents were found to be within present water quality guidelines, except for ammonia, manganese, and iron, and longer residency under oxidizing conditions should favor their removal. However, it must be kept in mind that there are seldom close relationships between total contaminant levels and toxicity or biological accumulation[16,18-20]. Certainly much more research must be performed on such relationships before a clear understanding of impact of confined land disposal of dredged material can be formulated.

CONCLUSIONS AND RECOMMENDATIONS

1. Most contaminants (chlorinated hydrocarbons, oil and grease, nutrients, and trace metals) are primarily associated with the solid phase of dredged material. Ammonium, manganese, and possibly iron, copper, and zinc are the only contaminants which might occasionally exceed both background water levels and present water quality standards and criteria following efficient suspended solids removal. Other soluble phase contaminants in effluents were either below presently prescribed criteria or comparable to background water concentrations. Thus proper retention or treatment of site waters to limit suspended solids levels seems necessary to meet the present regulatory guidelines. However, background water levels should also be considered in properly evaluating and regulating excessive effluent levels.

2. Major shifts of several trace metals from organic-sulfide and residual to carbonate and exchangeable geochemical phases were noted during residency of dredged material solids in land containment areas. Despite a potential increase in mobility of most metal carbonate complexes, increased adsorption onto and coprecipitation with fine effluent solids probably prevented notable soluble phase increases. Only zinc and copper were noted to increase in effluent filtrates at several sites, but levels were below present regulatory guidelines.

3. Actively growing vegetation in land disposal areas elicited excellent removal of nutrients (ammonium and soluble phosphate) and suspended solids. However, seasonal use of such sites during active plant growth is suggested for best effluent quality.

4. Effluent monitoring and predictive testing methodologies should include suspended solids. Chemical analysis of settleable solids or bulk sediments should give a poor indication of environmental impact.

5. Confined land disposal of dredged material, under proper management, seems to be a viable method for the containment and treatment of most contaminated sediments. However, sites located adjacent to sensitive wetlands in confined water bodies are not recommended without extensive predictive testing.

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Table 1

Descriptions of the Dredging and Confined Land Disposal Areas and Operations

Site No.	Location of Dredging Site	Location of Disposal Area	Predominant Dredged Sediment	Effluent Discharge Site	Description of Disposal Area: Size of Diked Area acres (ha)	Size of Treatment Area acres (ha)	Configuration	Description of Treatment: Effective Length of Treatment Area yd (m)	Overland Flow Distance yd (m)	Size of Ponded Area acres (ha)	Site Vegetation
1	Sayreville, N. J. South channel of Raritan River, km 8 (brackish water)	National Lead Industries Disposal Area No. 4, adjacent to river	Dark grey silt	Raritan River (surface brackish water)	44 (17.8)	40 (16)	Roughly circular; divided into three equal rectangular compartments; sluice box discharge; vegetated	1400 (1280)	≈700 (640)	≈35 (14)	70 percent cover of half of site by common reed grass (*Phragmites communis*)
2	Houston, Tex. Houston Ship Channel at sta 1040+00 and in ship turning basin at sta 1080+ (brackish water)	East half of Clinton disposal area, about 1.5 km inland from channel	Fine reddish sand and silt, often heavily impregnated with weathered oil	Hunting Bayou-Houston Ship Channel, 3 km east of dredging site (surface fresh to brackish water)	280 (113)	225 (91)	Rectangular; large sluice box discharge	1300 (1190)	≈200 (185)	≈200 (81)	Sparse
3	Grand Haven, Mich. Grand River, main channel at sta 120+00 (freshwater)	Verplank's Coal & Dock Co., disposal area, Ferrysburg, Mich., adjacent to river	Fine sand with some oily fines	Grand River (surface freshwater)	6 (2.4)	6 (2.4)	Roughly rectangular; sluice box discharge	250 (230)	-- --	≈6 (2.4)	None
4	Wilmington, N. C. Anchorage Basin, Cape Fear River (fresh to brackish water)	Eagle Island disposal area, between Cape Fear and Brunswick Waterways	Dark grey silt and clay	Brunswick Waterway (surface fresh to brackish)	525 (212)	400 (162)	Discharge by one sluice box and two D-shaped weirs; circular; heavily vegetated	2000 (1830)	≈1800 (1650)	≈75 (30)	Approximately 80 percent cover by dead or dormant grasses and brush; *Phragmites communis* predominates
5	Richmond, Va. James River, main channel and dock area at Deepwater Terminal (freshwater)	Disposal area on east bank of James River	Coarse sand and gravel, some light brown to dark grey silt	James River (surface freshwater)	70 (28)	35 (14)	Long and narrow; divided into three equal square compartments; sluice box discharge	1100 (1010)	≈700 (640)	≈20 (8)	Approximately 20 percent low density cover by forest and dormant undergrowth
6	Lake Charles, La. Calcasieu River, main channel near northwest end of Lake Calcasieu (brackish water)	Disposal area No. 22, 16 river km south of Lake Charles, on dredged material islands between the ship channel and lake	Dark grey to reddish-brown mixed silt, clay, and fine sand; some oily sediments	Calcasieu River (surface fresh to brackish)	185 (75)	150 (61)	Roughly rectangular; pipe discharge,* and discharge over large rectangular weir**	300** (275) 1300** (1190)	≈300 (275)	≈125 (51)	Sparse; less than 10 percent cover by large bushes and dead grasses
7	Seattle, Wash. Duwamish Waterway, Slip No. 1 (brackish to marine)	Old wastewater treatment plant sludge lagoon area north of dredging site, 60 m from waterway	Black silt-clay	Duwamish Waterway (surface brackish water)	1.9 (0.77)	1.9 (0.77)	Rectangular; divided into two equal rectangular compartments, each 46 by 85 by 4.7 m, effluent pumped from second compartment	200 (185)	-- --	1.9 (0.77)	None
8	Vicksburg, Miss. Brown Lake, upper end of 9.5 ha lake (freshwater)	Adjacent to upper end of lake	Light grey silt with light brown crust	Durden Creek-Brown Lake (surface freshwater)	5 (2.0)	5 (2.0)	Rectangular; divided into two equal compartments; sluice box discharge	440 (400)	-- --	5 (2.0)	None
9	Southport, N. C. Elizabeth River, in open channel at confluence with the Cape Fear estuary and near the Coast Guard Boat Harbor (brackish to marine)	East end of Oak Island, with dikes adjacent to Intracoastal Waterway	Black silt-clay (both sampling trips)	Atlantic Intracoastal Waterway (surface brackish water)	48 (19.4)	45 (18)	Elongated; D-shaped weir discharge; heavily vegetated	1500 (1370)	≈700† (640) ≈400†† (370)	≈5† (2) ≈20†† (8)	6 ha of thick stand of trees and bushes in lower section; 6 ha of tall grass; 4 ha in southern section of virgin salt marsh vegetation

* Day 1.
** Days 2 and 3.
† Collection trip 1 (6-7 May).
†† Collection trip 2 (17-20 May).

Table 2

Average Values for Physicochemical Parameters of Influents, Effluents, and Background Water from the Sampled Confined Land Disposal Areas

Location	Water Temperature °C	Salinity °/oo	Conductivity mmho/cm	Dissolved O_2, mg/ℓ	pH	Mechanical Particle Size, percent: Clay (<2 μm)	Silt (2–50 μm)	Sand (>50 μm)	Coulter Counter Particle Size, μm: >50 percent	>80 percent	Nonfilterable Solids percent Weight	Settleable Solids ml/ℓ	Cation Exchange Capacity meq/100 g	Alkalinity mg/ℓ
Sayreville, N. J.														
Influent	--	12.0	15.3	--	--	--	--	--	--	--	--	--	--	680
Effluent	17.5	14.0	17.6	6.3	--	--	--	--	--	--	--	--	--	840
Background	--	9.0	10.5	--	--	--	--	--	--	--	--	--	--	500
Houston, Tex.														
Influent	21.0	7.95	--	--	7.1	--	--	--	--	--	--	--	--	524
Effluent	19.5	8.9	--	7.85	7.7	--	--	--	--	--	--	--	--	517
Background	--	1.7	3.25	--	6.9	--	--	--	--	--	--	--	--	92
Grand Haven, Mich.														
Influent	1.5	0.7	0.52	--	--	--	--	--	--	--	--	--	--	240
Effluent	0.75	0.6	0.41	12.5	7.5	--	--	--	--	--	--	--	--	218
Background	3.5	--	0.39	11.5	7.6	--	--	--	--	--	--	--	--	140
Wilmington, N. C.														
Influent	8.5	2.8	3.2	2.3	6.6	60	31	9	1.05	0.6	7.19	467	80.6	345.1
Effluent	9.9	2.6	4.05	2.2	7.4	61	35	4	2.0	1.2	0.901	417	120.6	480.3
Background	--	--	--	--	5.5	--	--	--	--	--	0.00448	<0.1	--	17.3
Richmond, Va.														
Influent	10.6	0.25	0.29	7.75	6.7	11	24	65	1.15	0.63	1.66	85	26.2	82.3
Effluent	9.8	1.25	1.61	8.6	7.2	--	--	--	3.75	1.9	0.0464	0.7	65.9	60.5
Background	--	0	0.18	--	6.7	--	--	--	7.6	3.8	0.0010	<0.1	--	40.8
Lake Charles, La.														
Influent	14.0	15.9	20.8	4.9	7.15	64.5	29.5	6	0.91	0.56	6.26	367	56.8	320.8
Effluent	17.3	18.6	25.3	3.7	7.25	58	30	12	3.1	1.65	1.218	120	66.2	199.2
Background	--	4.2	6.8	--	6.3	--	--	--	5.4	2.82	0.0036	<0.1	--	29.9
Seattle, Wash.														
Influent	--	--	--	--	--	--	--	--	--	--	--	350	70.0	348
Effluent, Pond 1	7.1	18.5	30.1	5.4	5.8	--	--	--	--	--	0.02	0.6	--	183
Effluent, Pond 2	11.2	23.0	36.2	6.5	7.7	--	--	--	--	--	<0.01	0.25	88.0	199
Background	7.8	8.0	14.1	8.3	7.3	--	--	--	--	--	--	--	--	--
Vicksburg, Miss.														
Influent	18.5	0.15	0.65	3.0	7.0	22.5	72.5	5	0.89	0.56	23.30	950	10.3	285.5
Effluent	26.1	0.1	0.70	2.4	7.7	--	--	--	8.9	3.4	0.248	13.5	--	237.3
Background	--	--	--	--	--	--	--	--	5.5	2.85	0.00204	<0.1	--	290.0
Southport, N. C. (1)														
Influent	21.2	20.0	30.0	2.75	7.9	49	26	25	--	--	18.50	500	55.0	512
Effluent	24.0	21.3	32.0	3.3	7.65	--	--	--	--	--	0.0152	<0.1	--	178
Background	--	21.0	31.5	--	--	--	--	--	--	--	0.0094	<0.1	--	101
Southport, N. C. (2)														
Influent	22.4	15.2	23.7	2.8	7.55	41.5	50	8.5	--	--	17.10	693	57.9	1024
Effluent	30.4	20.9	38.05	6.1	7.9	--	--	--	--	--	0.1715	1.9	--	631
Background	--	--	--	--	--	--	--	--	--	--	0.02697	<0.1	--	112

Table 3

Statistical Character of Background Water, Influent, and Effluent

Samples From the Confined Disposal Areas

Parameter	Number of Samples			Range			Mean			Standard Deviation			Probability
	Background Water	Influent	Effluent	Background Water	Influent	Effluent	Background Water	Influent	Effluent	Background Water	Influent	Effluent	F Value Exceeded
Water Temp, °C	1	24	26	3.5-3.5	1.0-23.7	0.0-34.0	3.5	14.9	16.8	0.0	7.299	10.16	0.308
Salinity, 0/00	4	23	23	0.0-21.0	0.0-21.5	0.0-22.0	6.7	8.2	9.6	10.50	7.800	9.202	0.862
Conductivity, mmhos/cm	5	21	22	0.39-31.5	0.0-32.1	0.0-39.2	8.4	11.3	14.0	14.25	12.39	15.62	0.788
Dissolved O_2, mg/ℓ	1	17	23	11.5-11.5	0.25-9.3	0.6-12.5	11.5	3.8	5.3	0.0	2.885	3.402	0.048*
Slurry pH	5	16	16	5.5-7.6	6.25-8.3	6.9-8.1	6.6	7.16	7.51	0.775	0.516	0.316	0.0025**
Particle size, Clay, %	0	17	2	- -	4 -68	58 -61	- -	43.2	59.5	- -	20.15	2.121	0.280
Silt, %	0	17	2	- -	17 -78	30 -35	- -	38.1	32.5	- -	16.68	3.536	0.649
Sand, %	0	16	2	- -	0.0-79.0	4.0-12.0	- -	19.9	8.0	- -	22.50	5.657	0.479
Coulter Counter >50%, μm	3	9	10	5.4-7.6	0.77-1.2	0.2-10.5	6.2	0.97	4.75	1.242	0.131	3.545	0.0035**
>80%, μm	3	9	10	2.82-3.8	0.5-0.65	0.56-3.8	3.15	0.58	2.22	0.557	0.043	1.171	0.0001**
Total solids, % wt.	3	12	17	0.009-0.043	1.94-32.0	0.005-2.85	0.022	8.64	0.345	0.018	8.416	0.785	0.0006**

(Continued)

* Influent, effluent, and background water values are significantly different at $p \leq 0.05$.
** Influent, effluent, and background water values are significantly different at $p \leq 0.01$.

(Sheet 1 of 8)

Table 3 (Continued)

Parameter	Number of Samples: Background Water	Number of Samples: Influent	Number of Samples: Effluent	Range: Background Water	Range: Influent	Range: Effluent	Mean: Background Water	Mean: Influent	Mean: Effluent	Standard Deviation: Background Water	Standard Deviation: Influent	Standard Deviation: Effluent	Probability F Value Exceeded
Nonfilterable solids, % wt.	8	17	24	0.001–0.027	0.58–32.0	0.004–3.27	0.007	11.70	0.329	0.009	10.61	0.760	0.0000**
Settleable solids ml/ℓ	7	22	24	0.1–0.1	45 –999	0.1–950	0.1	452	69	0.0	288.8	205.6	0.0000**
Total C, <0.45 μm mg/ℓ	1	3	3	58 –58	30 –85	60 –70	58	57	65	0.0	27.54	5.000	0.872
Organic C, Total, mg/ℓ	13	16	13	2 –14	35.0–11500	4 –1060	6.5	3880	151	4.521	3405	303.1	0.0000**
Solid, mg/kg	1	20	7	10 –10	974 –53400	11 –53200	10	25100	20800	0.0	16026	23677	0.385
<0.45 μm, mg/ℓ	10	29	35	4 –43	3 –185	3 – 120	15	27	19	12.89	43.32	22.31	0.456
Oil & Grease, Total, mg/ℓ	11	12	18	0.1–47.2	25.0–1497	2.4–196	6.5	458	27.5	14.32	433.0	50.99	0.0000**
Solid, mg/kg	0	5	0	- -	2928 –8492	- -	- -	6060.	- -	- -	2121	- -	- -
<0.45 μm, mg/ℓ	2	11	9	1.1–1.1	1.8–48.0	0.32–13.0	1.1	11.2	3.1	0.0	16.88	3.997	0.300
Total Chlorinated Pesticides op'DDE, mg/ℓ	5	14	13	<0.01–0.28	<0.01–0.53	<0.01–0.50	0.08	0.13	0.07	0.117	0.139	0.136	0.555
pp'DDE, mg/ℓ	5	14	13	<0.01–1.57	<0.01–1.72	<0.01–2.87	0.37	0.47	0.37	0.679	0.527	0.773	0.915

* Influent, effluent and background water values are significantly different at $p \leq 0.05$.
** Influent, effluent and background water values are significantly different at $p \leq 0.01$.

(Sheet 2 of 8)

Table 3 (Continued)

Parameter	Number of Samples: Background Water	Number of Samples: Influent	Number of Samples: Effluent	Range: Background Water	Range: Influent	Range: Effluent	Mean: Background Water	Mean: Influent	Mean: Effluent	Standard Deviation: Background Water	Standard Deviation: Influent	Standard Deviation: Effluent	Probability F Value Exceeded
op'DDD, mg/ℓ	1	14	13	<0.01–<0.01	<0.01–1.28	<0.01–<0.01	<0.01	0.19	<0.01	0.0	0.368	0.0	0.183
pp'DDD, mg/ℓ	0	14	13	- -	<0.01–1.04	<0.01–<0.01	- -	0.21	<0.01	- -	0.368	0.0	0.050*
op'DDT, mg/ℓ	1	14	13	<0.01–<0.01	<0.01–5.4	<0.01–0.23	<0.01	1.1	0.02	0.0	1.993	0.061	0.152
pp'DDT, mg/ℓ	1	14	13	<0.01–<0.0	<0.01–5.94	<0.01–0.96	<0.01	0.68	0.07	0.0	1.627	0.263	0.416
Total PCB, mg/ℓ	1	20	23	<0.01–0.3	<0.1–21	<0.1–7.66	0.058	5.81	0.50	0.090	5.451	1.607	0.0000**
Organic N, Total, mg/ℓ	17	28	26	<0.01–2.35	3.6–839	0.1–74.5	0.376	168	9.7	0.556	205.8	18.017	0.0000**
Solid, mg/kg	0	21	3	- -	532 –3870	906 –3042	- -	1820	2220	- -	1087	1150	0.562
<0.45 um, mg/ℓ	10	28	34	0.1–1.35	0.1–27.6	0.1–6.7	0.55	4.3	1.6	0.397	5.871	1.627	0.0083**
NH_3-N, Total, mg/ℓ	16	7	21	0.01–1.54	7.3–86.0	0.82–80.3	0.53	45.6	19.6	0.459	28.46	22.93	0.0000**
Exch, mg/kg	0	18	7	- -	2.4–339	58.5–458	- -	110	196	- -	94.12	167.8	0.115
<0.45 um, mg/ℓ	10	29	35	0.01–0.82	0.66–71.7	0.74–70.9	0.27	20.8	13.6	0.270	19.03	15.73	0.0034**
NO_3-NO_2-N, mg/ℓ	16	26	32	0.01–1.98	0.01–0.82	0.01–1.83	0.46	0.18	0.35	0.523	0.234	0.396	0.066
Total P, Total, mg/ℓ	17	28	34	0.07–0.86	12.8–496	0.11–82.1	0.26	155	11.7	0.236	133.6	21.85	0.0000**
Solid, mg/kg	0	21	3	- -	639 –4400	1400 –4000	- -	1850	3070	- -	1179	1447	0.117

(Continued)

* Influent, effluent and background water values are significantly different at $p \leq 0.05$.
** Influent, effluent and background water values are significantly different at $p \leq 0.01$.

(Sheet 3 of 8)

Table 3 (Continued)

Parameter	Number of Samples: Background Water	Number of Samples: Influent	Number of Samples: Effluent	Range: Background Water	Range: Influent	Range: Effluent	Mean: Background Water	Mean: Influent	Mean: Effluent	Standard Deviation: Background Water	Standard Deviation: Influent	Standard Deviation: Effluent	Probability F Value Exceeded
<0.45 μm, mg/ℓ	10	29	32	0.02–0.5	0.03–9.47	0.01–1.53	0.18	0.86	0.33	0.175	2.047	0.499	0.221
Orthophosphate P,													
<0.45 μm, mg/ℓ	8	22	28	0.03–0.16	0.04–5.89	0.01–1.04	0.08	0.79	0.18	0.051	1.585	0.294	0.074
Alkalinity, mg/ℓ as $CaCO_3$	10	29	35	16.27–290	51.38–1520	29.75–670	88.10	412	287	83.08	217.4	184.5	0.0015**
Chloride, mg/ℓ	10	29	34	-20600	5.0–19200	5.0–16400	4150	8290	7810	7909	7326	6365	0.261
Total Sulfide, Solids, mg/kg	0	21	3	- -	17.8–3090	94.1–327	- -	493	208	- -	679.5	116.6	0.484
Cation Exch. Cap, meq./100g	0	19	6	- -	2.37–88.2	65.9–120.6	- -	50.9	82.5	- -	25.16	21.55	0.011*
Calcium, Total, mg/ℓ	10	24	25	4.8–390	45.7–11500	16.8–560	98.7	2450	250	142.3	3420	163.6	0.0013**
Solids, mg/kg	0	22	7	- -	1150 -37900	1190–26100	- -	16300	9930	- -	13722	10844	0.274
<0.45 μm, mg/ℓ	10	24	25	4.1–390	8.0–416	13.0–532	95.4	181.1	204.8	141.6	140.7	166.3	0.165
Magnesium, Total, mg/ℓ	10	24	25	2.5–1300	26.5–1320	3.15–1200	279.0	1270	464	514.4	1057	416.8	0.0003*
Solids, mg/kg	0	22	8	- -	933 -37800	4700–16300	- -	11200	10200	- -	8239	4853	0.746
<0.45 μm, mg/ℓ	10	24	25	1.5–1300	2.6–1300	2.6–1200	265	415	398	497.1	432.8	447.9	0.661

(Continued)

* Influent, effluent, and background water values are significantly different at $p \leq 0.05$.
** Influent, effluent, and background water values are significantly different at $p \leq 0.01$.

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Table 3 (Continued)

Parameter	Number of Samples: Background Water	Number of Samples: Influent	Number of Samples: Effluent	Range: Background Water	Range: Influent	Range: Effluent	Mean: Background Water	Mean: Influent	Mean: Effluent	Standard Deviation: Background Water	Standard Deviation: Influent	Standard Deviation: Effluent	Probability F Value Exceeded
Potassium,													
Total, mg/ℓ	10	24	25	2.2-338	128 -6360	4.6-1145	121	1770	390	148.3	1644	362.3	0.0000**
Solids, mg/kg	0	20	8	- -	3100 -43500	8330-18200	- -	13200	14200	- -	8655	3647	0.762
<0.45 μm, mg/ℓ	10	24	25	1.2-380	1.6-450	1.5-440	98.6	163	166	151.6	137.0	144.9	0.418
Sodium,													
Total, mg/ℓ	8	19	23	6.5-9800	85.0-9900	6.5-11300	2470	3900	3540	4026	3422	3963	0.772
Solids, mg/kg	0	11	5	- -	2394 -13100	240-43200	- -	9430	23600	- -	3583	18176	0.021*
<0.45 μm, mg/ℓ	9	24	25	6.1-9700	6.2-9500	6.0-9400	2183	3620	3490	3737	3806	3798	0.607
Iron,													
Total, mg/ℓ	17	29	34	0.38-63.6	46.1-12600	1.14-1290	5.19	3400	193	15.13	3565.	343.0	0.0000**
Solids, mg/kg	0	22	8	- -	24300-81600	24100-48300	- -	42300	38300	- -	13117	7068	0.421
<0.45 μm, mg/ℓ	10	29	35	0.001-1.43	0.043-15.9	0.01-10.1	0.378	3.52	0.814	0.457	5.009	1.816	0.0037**
Manganese,													
Total, mg/ℓ	17	29	35	0.04-0.40	0.8-310	0.21-48.5	0.107	63.1	7.9	0.095	73.00	13.61	0.0000**
Solids, mg/kg	0	22	8	- -	250-2110	683-2170	- -	682	1160	- -	383.9	470.9	0.0081**
<0.45 μm, mg/ℓ	10	29	35	0.002-0.184	0.004-14.4	0.002-7.95	0.059	2.35	1.45	0.057	3.485	2.090	0.057*

(Continued)

* Influent, effluent, and background water values are significantly different at $p \leq 0.05$.
** Influent, effluent, and background water values are significantly different at $p \leq 0.01$.

(Sheet 5 of 8)

Table 3 (Continued)

Parameter	Number of Samples			Range			Mean			Standard Deviation			Probability F Value Exceeded
	Background Water	Influent	Effluent	Background Water	Influent	Effluent	Background Water	Influent	Effluent	Background Water	Influent	Effluent	
Zinc,													
Total, mg/ℓ	17	25	30	0.006–1.28	0.6–206	0.026–5.49	0.238	27.5	1.03	0.422	40.28	1.481	0.0003**
Solids, mg/kg	0	22	8	- -	55.8–1960	31.7–3660	- -	323	621	- -	435	1232	0.325
<0.45 μm, mg/ℓ	10	25	30	0.001–0.121	0.001–0.496	0.002–0.228	0.028	0.055	0.064	0.042	0.10	0.069	0.472
Cadmium,													
Total, mg/ℓ	13	26	32	<0.0002–0.01	0.002–7.17	0.001–0.37	0.003	1.39	0.051	0.003	2.036	0.102	0.0002**
Solid, mg/kg	0	17	4	- -	0.048–45.3	0.045–4.87	- -	7.1	1.62	- -	10.44	2.216	0.319
<0.45 μm, mg/ℓ	9	29	35	<0.002–0.012	0.0002–0.015	0.001–0.011	0.003	0.004	0.003	0.004	0.004	0.003	0.824
Copper,													
Total, mg/ℓ	15	29	35	0.003–0.16	0.1–18.2	0.02–1.59	0.038	6.09	0.28	0.042	5.327	0.414	0.0000**
Solid, mg/kg	0	22	8	- -	6.0–165	26.0–131	- -	52.2	46.3	- -	48.43	35.00	0.753
<0.45 μm, mg/ℓ	10	29	35	0.001–0.028	0.001–0.106	0.001–0.1	0.009	0.019	0.021	0.010	0.026	0.022	0.323
Nickel,													
Total, mg/ℓ	14	29	30	<0.01–1.5	0.21–18.2	<0.01–1.70	0.120	5.8	0.30	0.397	4.555	0.414	0.0000**
Solid, mg/kg	0	22	8	- -	15.4–124	25.3–74.6	- -	47.3	47.1	- -	26.06	15.18	0.982
<0.45 μm, mg/ℓ	10	29	30	0.003–0.036	0.003–0.047	0.002–0.043	0.011	0.014	0.012	0.013	0.013	0.013	0.820

(Continued)

** Influent, effluent, and background water values are significantly different at $p \leq 0.01$.

(Sheet 6 of 8)

Table 3 (Continued)

Parameter	Number of Samples: Background Water	Number of Samples: Influent	Number of Samples: Effluent	Range: Background Water	Range: Influent	Range: Effluent	Mean: Background Water	Mean: Influent	Mean: Effluent	Standard Deviation: Background Water	Standard Deviation: Influent	Standard Deviation: Effluent	Probability F Value Exceeded
Lead,													
Total, mg/ℓ	9	28	23	0.001–0.049	0.24–86.5	0.001–7.57	0.011	16.2	1.38	0.015	23.88	2.437	0.0033**
Solid, mg/kg	0	21	8	- -	5.7–327	1.0–142	- -	81.5	43.7	- -	88.65	43.58	0.262
<0.45 μm, mg/ℓ	10	24	25	<0.001–0.005	<0.001-0.012	<0.001–0.007	0.002	0.002	0.002	0.002	0.003	0.002	0.725
Mercury,													
Total, mg/ℓ	14	18	24	<0.0002–0.009	0.001–0.243	<0.0002–0.367	0.001	0.044	0.024	0.002	0.059	0.080	0.161
Solid, mg/kg	0	18	6	- -	0.07–1.66	0.08–3.2	- -	0.46	0.79	- -	0.438	1.234	0.334
<0.45 μm, mg/ℓ	8	19	28	<0.0002–0.004	0.0002–0.008	<0.0002–0.006	0.001	0.001	0.001	0.001	0.002	0.002	0.895
Chromium,													
Total, mg/ℓ	8	3	8	0.009–0.026	56.7–76.6	0.024–0.58	0.018	63.8	0.12	0.005	11.11	0.190	0.0000**
<0.45 μm, mg/ℓ	1	3	8	0.003–0.003	0.003–0.005	0.004–0.033†	0.003	0.004	0.017†	0.0	0.001	0.011	0.132
Titanium,													
Total, mg/ℓ	1	4	4	0.01–0.01	2.1–4.35	0.255–0.50	0.010	3.31	0.36	0.0	0.956	0.117	0.0017**
<0.45 μm, mg/ℓ	1	4	4	0.0001–0.0001	0.025–0.038	0.020–0.036	0.0001	0.029	0.028	0.0	0.006	0.009	0.030*

(Continued)

* Influent, effluent and background water values are significantly different at $p \leq 0.05$.
** Influent, effluent and background water values are significantly different at $p \leq 0.01$.
† High effluent concentrations resulted from inclusion of high effluent values from the Seattle, Washington site, which lacked comparable influent data.

(Sheet 7 of 8)

Table 3 (Concluded)

Parameter	Number of Samples			Range			Mean			Standard Deviation			Probability F Value Exceeded
	Background Water	Influent	Effluent	Background Water	Influent	Effluent	Background Water	Influent	Effluent	Background Water	Influent	Effluent	
Vanadium													
Total, mg/ℓ	2	7	7	0.029–0.32	2.29–5.23	0.076–0.47	0.175	3.52	0.26	0.206	1.321	0.134	0.0000**
<0.45 μm, mg/ℓ	2	7	7	0.004–0.004	0.004–0.039	0.003–0.027	0.004	0.018	0.015	0.0	0.013	0.010	0.317
Arsenic,													
Total, mg/ℓ	9	12	17	0.001–0.013	0.181–6.02	0.003–0.41	0.004	1.73	0.096	0.004	1.979	0.126	0.0006**
<0.45 μm, mg/ℓ	2	12	17	0.0003–0.001	0.0001–0.117	0.0001–0.021	0.001	0.027	0.004	0.000	0.043	0.006	0.077

** Influent, effluent and background water values are significantly differnt at $p \leq 0.01$.

(Sheet 8 of 8)

IMPROVING THE EFFICIENCY OF DREDGING SEVERAL FEET OF CONTAMINATED SEDIMENT OFF THE TOP OF UNCONTAMINATED SEDIMENT

by

T. J. Tofflemire[1]

ABSTRACT

Most dredging has had the goal of removing sediment to some final elevation. For this dredging, overcutting and mixing of sediment layers is common. Most contaminated silts and sands would assume a 25-45% angle of repose. Thus, sediments cave into dredging cuts rapidly. Dredging technology is oriented towards this elevation purpose. When a contaminated layer must be removed, a different orientation is needed. The limited experience of single pass dredging to remove a two or three foot contaminated layer has not been good. At Foundry Cove on the Hudson a two foot contaminated layer of cadmium laden sediment was dredged with very poor efficiency and limited benefit to the cove. For the Duamish River estuary where PCB contaminated the top several feet of sediment, double pass dredging achieved only 90% PCB removal. A laboratory study of dredging a contaminated muck layer off of the top of uncontaminated sand indicated that only 70 - 75% of the contaminated layer was removed by hydraulic or by clamshell dredging. Improved techniques could increase that percentage to 85 - 90%.

Mathematical calculations of clamshell volumes, overlap spacings and losses indicated that with no clamshell overlap biting or with overcutting, the loss of a two to three foot contaminated layer would be in the 25 - 50% range. By optimizing the overlap spacing and minimizing the overcutting, this loss can be reduced by 10 - 15% for single pass (cut) dredging. To achieve this low level of loss, dredge head modifications and a precise grid pattern for the dredge head must be followed. Precise radar surveying systems are used to monitor the dredge head location. An alternate way to achieve 85 - 90% removal is to use routine double pass dredging. However, this doubles the dredging cost and the yardage of sediment to be disposed of.

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Introduction

There are a number of water bodies and areas that require dredging to remove a contaminated surface layer. One purpose is to remove nutrient laden organic sediments in shallow lakes or ponds where excessive growth of aquatic vegetation is a problem. A world wide survey of lake rehabilitation techniques in 1974 revealed 17 lakes cited for nutrient removal dredging and 7 lakes for which the dredging had been completed.(1) In NY State there are about 10 Lakes that have been dredged for nutrient removal or deepening to lessen eutrophication. Another purpose is to remove toxic substances in sediments that might contaminate water and fish. These toxics vary from PCB spilled in the Duwamish Estuary in Washington to cadmium discharged to Foundry Cove, NY, to kepone dis-discharged to the James River in Virginia. For many of these cases the contaminant is found primarily in the top several feet of the sediment. A review of PCB contaminated sediments(2) indicated a number of major cases including the Upper Mississippi River near St. Paul, Minnesota, the Sheboygan River and Harbor in Wisconsin, the Waukegan Harbor in Illinois, the Duwamish Estuary and the Hudson River. Pollutants that are relatively nonbiodegradable, slightly soluble in water, bioaccumulative in fish and toxic are of most concern.

Most dredging has had the goal of removing sediment to some specified elevation. For this dredging overcutting and mixing of sediment layers is common. Sediments cave into dredging cuts rapidly. This is an anticipated part of conventional dredging. A paid 2 ft overcut is common.(3) When a contaminated layer must be removed a different orientation is needed in order to remove a high percentage of the contaminated layer and a low percentage of the unwanted sediment. Improved dredging techniques and proper equipment are needed. Certain dredges that take a wide sweep or bite of a large area and small thickness are ideal. Accurate location of the dredge grabs or sweeps is also essential.

Some Relevant Literature and Studies

The tendency for a sediment to cave in or flow into a dredging cut is dependent on a number of factors including the cohesiveness of the sediment, the particle size and shape, the history of compaction, the pore pressures, and the depth of cut. Coulomb's Law describes the relationships:[4]

$$S = P \tan \phi + C$$

where:
S = Shear strength
P = The normal force or pressure
ϕ = The angle of internal friction
C = The unit cohesion (cohesive strength)

The angle of internal friction is an important property. For granular or non-cohesive soils, this is determined by laboratory shear tests in which the normal force is varied and the shear strength measured. The slope of the shear strength versus normal force line is the angle, ϕ. Some typical values are given in Table 1.[4] The prior history of compaction of the soil affects the peak strength. The ultimate strength is measured after the particles are sheared and loosened and is some what less than the peak strength. The angle of repose or slope of a pile is approximated by the ultimate strength angle. For granular sands C (the unit cohesion) is zero, but for clays and muck soils with roots most of the shear strength of the soil is due to C. For some clays $\phi \cong 0°$ and S = C. Vane shear tests are useful to test the strength of river sediments in situ.

TABLE 1. TYPICAL VALUES OF FRICTION ANGLES, GRANULAR SOIL.

Classification	Slope at Angle of Repose		Friction Angles: At Ultimate Strength	At Peak Strength: Med. Dense	At Peak Strength: Dense
	i deg.	Slope vert. to hor.	ϕ deg.	ϕ_p deg.	ϕ_p deg.
SILT (Nonplastic)	26 to 30	1 on 2 / 1 on 1.75	26 to 30	28 to 32	30 to 34
Uniform fine to medium SAND	26 to 30	1 on 2 / 1 on 1.75	26 to 30	30 to 34	32 to 36
Well-graded SAND	30 to 34	1 on 1.75 / 1 on 1.50	30 to 34	34 to 40	38 to 46
SAND & GRAVEL	32 to 36	1 on 1.60 / 1 on 1.40	32 to 36	36 to 42	40 to 48

For a number of dredged sediments[5] the angles of internal friction varied from 25-40° and paralleled the values in Table 1. The unit cohesion varied from 100-1000 lb/ft^2 and in the situ vane shear strengths from 20-400 lb/ft^2. For Hudson River sediments the vane shear of the top several feet of silts and mucks were in the 20-450 lb/ft^2 range[6] while vane tests to 20 ft into the sediment gave values as high as 500 lb/ft^2.[7] The amount of compaction of the sediment increased with depth. Scuba diving was done along the Hudson River bottom and holes dug by hand and by a small shovel. The angle of repose of the sandy sediments was in the 25-40° range. The sediments in marshes (typically silty sediments in <5 ft of water) exhibited angles of repose as high as 60°. The roots held the sediment together and gave it considerable cohesion. In addition there appears to be a parallel between the tendency of sediment to erode and to cave in. The fine sands and coarse silts are most vulnerable while the gravels and clays are more stable.[8,9] The more organic sediments also appear less stable unless they are in shallow water where rooted aquatic plants hold the sediment together.[6]

There is relatively little literature on the efficiency and net results of contaminant layer dredging. Many ponds have been deepened and eutrophication temporarily lessened. It is not clear if this was due to removal of most of the nutrient-laden organic sediment or due to deepening or to physical removal of the rooted vegetation.[1] For the dredging of Collins Lake in Scotia, NY, a Mudcat dredge was used. It made 8 ft wide cuts 1.5 ft thick. It was noted that windrows of sediment were left from the cave-in at the edge of the cut. A one foot cut overlap was used.

Goodier[10] noted the following in comparing mechanical, hydraulic and pneumatic dredges: The clamshell or dragline looses sediment during the raising of the bucket and during the lowering of the bucket back into the water. In greater water depths more sediment is lost out of the clamshell. The clam is generally allowed to free-fall and the depth of bite is dependent on its momentum and penetration. Without location equipment it is difficult for the operator to overlap each bite evenly and a bottom with mounds and holes results. Periodically the clam is swung from side to side underwater to level the bottom. Other common mechanical dredges include dipper and dragline dredges.[10]

Cutterhead hydraulic dredges use walking spuds on the back and cable to make swings cuts in arcs back and forth. The rotating cutterhead loosens the sediments and mixes them with the overlying water. In soft unconsolidated sediments the dredge can operate satisfactorily with the cutterhead stopped or removed.[10] The dustpan dredge has a suction head shaped like a dustpan and has scarification water jets along the leading edge to losen the sediment. The suction head is pulled into the sediment by winches on two cables that generally run upstream to ancors set above the cut area. The dustpan can be accurately raised and lowered by cables to take a specified depth or thickness of cut. Some dustpans are 36 ft wide and cut this wide of a swath thru the sediments. This dredge usually employs a low head, high capacity pump designed to pump only a few feet above the water surface. It would be possible to equip a conventional cutterhead hydraulic dredge with a 12-15 ft wide dustpan and operate in the dustpan mode.[10] The Mudcat dredge operates similarily to the dustpan in that it is wenched into the sediment. However, it uses two augers that loosen and feed the sediment to the suction pipe. The head is 8 ft wide and can take as small as a 1.5 ft thick cut. The unit is small with an 8 inch pipeline, and is only capable of dredging to a 15 ft depth. As for other hydraulic dredges, booster pumps can greatly increase the pumping distance or head if needed. Goodier[10] implied that the dustpan or Mudcat type dredges would be theoretically most efficient in removing contanminated layers. A new dredge called the Mud Master* is similar to the Mudcat but has only one scarifing auger to loosen the sediment and catapiller tracks to propell it overland like a bulldozer. It also floats and can be winched by cables.

Airlift dredges are generally fabricated for a specific purpose and are not considered stock or off the shelf dredging units. The units are more efficient in deep water and use the principle of air injection to cause density reduction in the upper pipe which causes suction in the lower pipe section. The Pneuma Dredge is a patented Italian dredge that uses the air lift principle. The unit was employed to dredge PCB contaminated sediments from the Duwamish Estuary. The Oozer Pump system is a patented Japanese dredge that uses alternate vacuum and pressure air cycles to suck sediment into a tank and pump it out. There are two tanks and suction heads operating on alternate cycles, so the discharge can be nearly continuous. The Oozer system has automated sediment pressure sensors to prevent the suction of clear water. These suction

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* Dredge Masters International, Nashville, Tenessee.

and air lift systems can be added to the ladder of a conventional cutterhead type hydraulic dredge which makes swings. Goodier[10] noted that these suction dredges cause relatively little turbidity loss relative to cutterhead hydraulic dredges or clamshell dredges.

Tofflemire et al.[11] studied the various dredge losses of solids and PCB and noted the following types of losses in Table 2.

TABLE 2. EXPECTED PERCENT LOSSES IN HOT SPOT DREDGING

	Hydraulic		Clamshell	
	PCB	Solids	PCB	Solids
Missed during accurate dredging (not picked up)	13	13	13	13
Lost by suspention at the dredge head	<1	<.5	<2*	<1*
Lost in return flow with 2 hrs settling	1	.5	.2	.1

* Shrouding of the clamshell could reduce these loses.

For hydraulic dredging, treatment of the return water to achieve good solids separation is essential and practical.[11] Several studies involving detailed water monitoring around clamshell hydraulic cutterhead dredges have noted that the increased solids in the water column within 1500 ft of the dredge accounted for a loss of no greater than 2.5 percent of the solids dredged.[12,13,14,15] The suspended loss of the clam shell was twice the loss of the hydraulic dredge head in one study.[14] However, one is struck by the 13 percent or greater loss of sediment not picked up in the dredging operation.

This large potential loss of sediment not picked up was explored in a small laboratory tank study in which clamshell and hydraulic dredging were performed.[15] A carefully leveled layer of muck soil (95 percent passing No. 100 sieve) was placed over a measured layer of sand (3 percent passing No. 100 sieve) in the tank. All volumes and weights of materials added to or removed from the tank were carefully measured. A small clamshell set up shown in Figure 1 and 2 was used to do test dredging at precisely located overlap intervals. A variable speed Cole Palmer pump attached to a copper pipe shown in Figure 2 and 3 was used to achieve the hydraulic dredging. The purpose was to determine what percentage of the contaminated muck could be removed from the tank.

All the sediment removed was sieved to separate it into the sand and muck fractions and weighed. Corrections were made for the non-retrievable sediment near the walls. The following was concluded:

1. Accurate single pass clamshell dredging with a overlap could achieve 85-90 percent contaminated layer removal.

2. Accurate single pass hydraulic dredging could achieve 85 percent contaminated layer removal when uniform solids solids concentrations of 5 percent or less are pumped.

Some other observations and projections were made.

1. Routine clamshell dredging without accurate location positioning could achieve contaminated layer removal as low as 65 percent. For clamshell dredging the bite overlap spacing, depth of bite and accurate positioning of the bites was essential.

2. Routine hydraulic dredging that was not optimized for contaminated layer removal could also achieve as low as 65 percent recovery. Here the boom swing speed and pumping rate were critical, along with accurate positioning. If the boom is swung to fast solids are missed and plowed sideways. If the boom swing speed is to slow a lot of water is pumped but sediment recovery is more efficient. In the lababoratory tank as in the field for large hydraulic dredges, keeping just the right speed is a difficult task. The pump discharge frequently changes from dark to clear (high to low percent solids). The more water that is pumped, the lower the dredged yardage production rate, and the more expense added to the return water treatment system and dredge operation. Accurate positioning of sweeps is again important.

Several case studies of toxic sediment dredging will now be described. In the Duwamish Estuary 265 gallons of Aroclor 1242 were spilled. Divers with suction hoses removed some of the most concentrated PCB laden sediments and filled 215, 55 gallon drums. Then a Pneuma dredge removed 6,900 yd^3 (5300 m^3) of the lesser contaminated sediment. The entire area recieved a cut of about 2 ft (.6 m) and the central spill area a cut of 12 ft (3.7 m). The turbidity disturbance in the area was minimal and 90 percent of the PCB was recovered by the dredging.[18]

In a second case, the discharge of nickel and cadmium to a marsh occurred for 17 years and resulted in sediment cadmium concentration above 10,000 g/g. A Mudcat dredge was used to dredge 5,000 yd^3 of sediment and 4-6 tons of cadmium out of the marsh. The sediment was contaminated to 4 ft depth in the outfall area and to 1 ft depth in the outer cover area. Although several dredge cuts were made, there are still sediments in the outfall area with more than 10,000 $\mu g/g$ of Cd and broad areas with over 1,000 $\mu g/g$ of Cd. Scientists sampling the area after the dredging concluded that the dredging did little to reduce the effect of Cd surface concentrations and as much as 50 tons of Cd still remain in the cove.[19]

Fill in Volumes for Clamshell Dredges

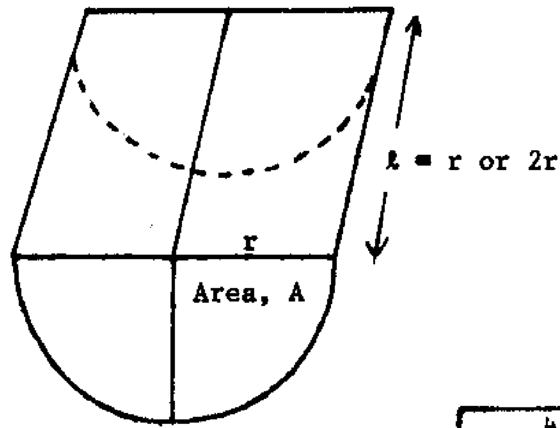
On the following three (3) pages, calculations of the fill in volume relative to the full clam volume are indicated. A sediment angle of repose of 45° was used. Most sediment would fill in at an angle lower than 45° making the fill in volumes greater. On the calculation sheets, the fill in volume is the missed volume V_m and the empty clam volume is V_e. The assumption was made that the contaminated layer to be dredged was of depth r, the clam radius. The conclusions for a semicircular clam are on the bottom of page 9.

The additional complicating factors of overcut depth, clam radius and clam shape were considered for dredging a 2 ft layer on page 3. Here another volume unwanted (U.W.) was also calculated. The assumption was also made that the sediments most likely to cave in are the top 2 ft of sediments which are the least consolidated. Deeper sediments would have a slightly higher compression and shear strength. For semicircular bites, a clam radius between 2 and 3 ft would be optimum. For large clams the unwanted sediment volume increases rapidly. A radius of greater than 3 ft would yield unwanted sediment volumes of > 25%. To the unwanted volumes, the empty clam volume of 20-30% should be added. It is not certain just what kind of cut a semicircular clam would make in the field. If it made a box shaped cut instead of a semicircle, then the efficiencies would go up. The calculations on the box cut are noted on page 10. In making a box cut, a clam would have to be opened to nearly vertical to minimize push away volume. The laboratory study clam which was of similar shape to a large American Dredging Company clam created approximately 18% push away volume as it was lowered into the sediments. It is assumed that about half of this push away volume is pushed into rows at the end of the clam. Clam end overlap would be needed to pick it up. These results do fit with the laboratory scale sediment mass removal experiment where the clam had a r/2 bitting and r/4 end overlap.

In summary the following clams could potentially give the greatest efficiency in removing a 2 ft contaminated top layer.

1. A 2.5' r semicircular clam with an r/2 bitting and end overlap.
2. An 8' r ¼ circle clam with a 2.8' bitting overlap.
3. A box cut clam with a 2' overlap.

FILL IN VOLUMES FOR SEMICIRCULAR CLAM with 40-45° Angle of Repose of Sediment

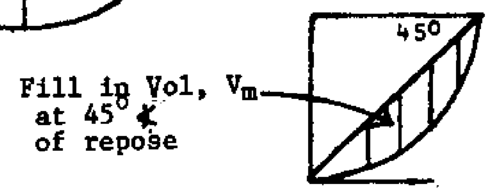


First Clam Bite

Full Clam = $2A = \frac{\pi r^2}{2} = 1.57\ r^2$

$A = .785\ r^2$

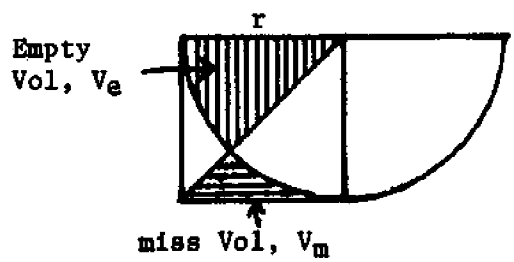
45° Fill Area = $\frac{\pi r^2}{4} - \frac{r^2}{2} = .285\ r^2$



2nd Clam Bite at 1/2 clam overlap (r)

$Vol_m = \left[\frac{r^2 - \frac{\pi r^2}{4}}{2}\right] \ell = .107\ r^2 \ell = 6.8\%$

$Vol_e = \frac{\pi r^2}{8} \ell = .392\ r^2 \ell = 25\%$

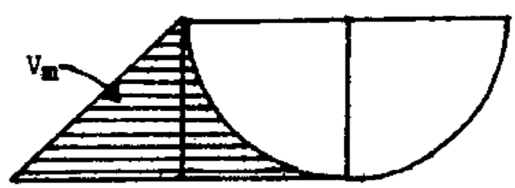


2nd clam bite with no overlap

$Vol_m = \frac{r^2}{2} \ell + .215\ r^2 \ell = .715\ r$

% miss = $\frac{.715\ r^2}{1.57\ r^2} = 45.5\%$

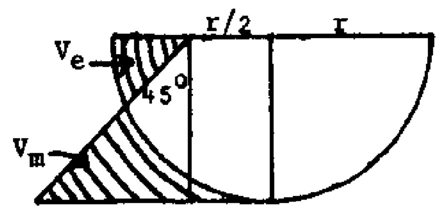
empty Vol = 0



2nd clam bite with r/2 overlap (1/4 clam)

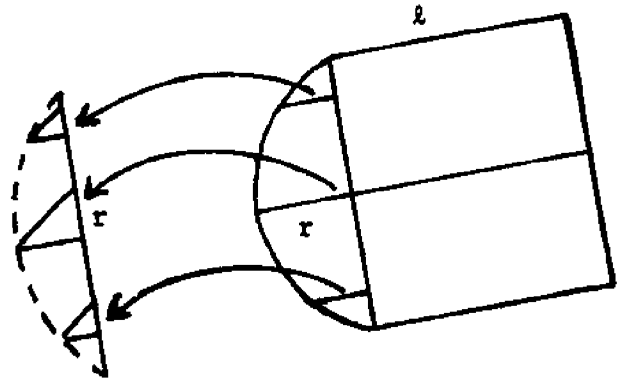
missed vol = $.0804\ r^2 \ell = 16.1\%$

empty vol = $.0325\ r^2 \ell = 6.5\%$



Ends overlaps only

	ℓ	
	r	2r
No overlap		
missed vol =	11.4%	5.7%
empty vol =	10%	5%
at r overlap		
missed vol =	0%	0%
empty vol =	89%	44%
at r/2 overlap		
missed vol =	8.2%	4.1%
empty vol =	47%	23%



Combined Row & Column Losses for No End Overlap

	ℓ = 2r			ℓ = r		
bitting overlap	0	r/2	r	0	r/2	r
missed vol %	51.4	21.8	12.5	56.9	27.5	18.2
empty vol %	5	11.5	30	10	16.5	35

Combined Row & Column Losses for r/2 End Overlap

	ℓ = 2r			ℓ = r		
bitting overlap	0	r/2	r	0	r/2	r
missed vol % A*	49.6	20.2	10.9	53.7	24.3	15.0
missed vol % B#	46.5	17.1	7.8	47.5	18.1	8.8
empty vol % A	23	29.1	48	47	53.1	72
empty vol % B	14	16.5	36	20	26.5	45

* Calc. Method A
\# Calc. Method B

Conclusions:
1. Optimum bite overlap is close to r/2 assuming a semicircular bite
2. Optimum end overlap between 0 and r/2
3. Efficiency improves in going from ℓ = r to ℓ = 2r

Good operation: missed or left sediment = 8-12%
empty clam volume = 30-40%

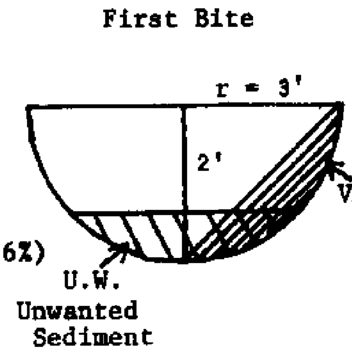
Poor operation: missed or left sediment = 25-50%
empty clam volume = 5-10%

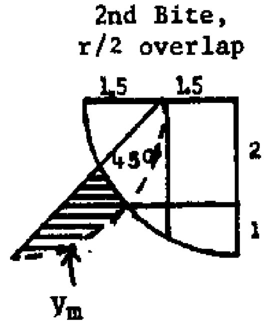
Typical when only elevation controls

Various Clam Sizes and Shapes Relative to Removing a 2 ft Layer

3 ft radius semicircle

	First Bite		Second Bite
length, ℓ =	1r	2r	
Total vol = yd^3	1.57	3.14	1.57
Unwanted(U.W.) = yd^3	.34	.69	.34
% U.W.	22	22	22
$V_m = .285\ r^2\ell = yd^3 =$	.285	.57	.15 (9.6%)

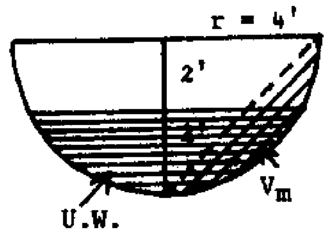


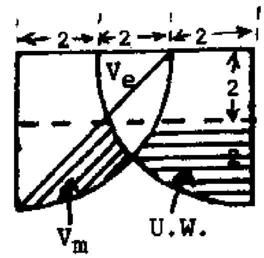


4 ft radius semicircle

	First Bite		Second Bite
length, ℓ =	1r	2r	
Total vol = yd^3	3.72	7.45	3.72
U.W. vol = yd^3	1.46	2.91	1.40
% U.W.	39	39	36
$V_m = .285\ r^2\ell = yd^3 =$	.675	1.35	.37 (10%)

Note: if a square cut could be made the unwanted sediment could be reduced.

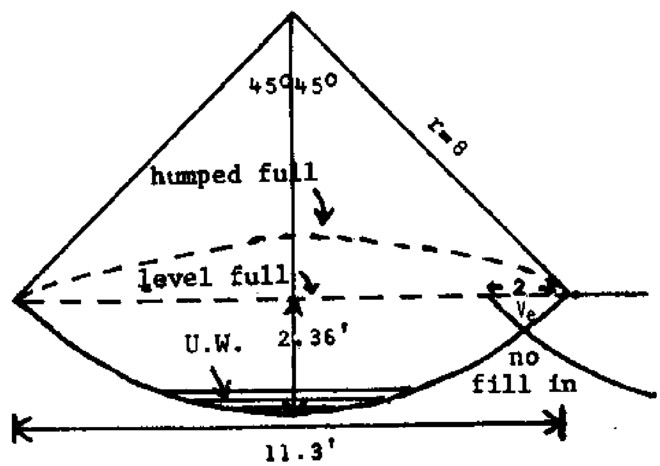




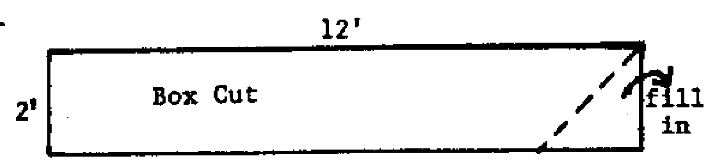
8 ft radius ¼ circle

yd^3 Capacity at lengths of				
5.66 ft		11.3 ft		2 x 12 + 11.3
Level	Humped	Level	Humped	Box
3.82	5.0	7.65	10.2	10.0

Note: This clam could be very efficient in removing a 2 ft contaminated layer if the clam could be lowered 2 ft into the sediments while open and then raised as the clam was shut. Then there would be almost no overcutting. On the other hand, if this clam made a circular bite, then it would leave about 17 % of the top 2 ft of contaminated sediment at a 2.8' overlap and pick up 7% unwanted.



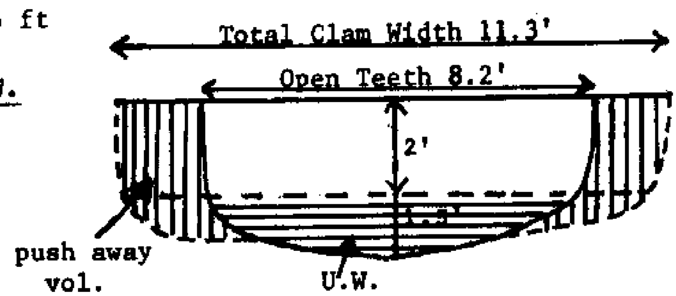
Box Cut	Size of Box Cut Clam			
	12' square		6' square	
Bitting overlap	0	2'	0	2'
Missed vol %	8%	0	16	0
Empty clam vol %	0	8	0	16
U.W. vol %	0	0	0	0



Lab clam vol = 125 cc or 125 ft^3 scaled up.
COE clam was of similar shape - transform to ft
Approx. square 8' x 3' hole cut:

	Missed	Empty	U.W.
with 2' bitting and 1' end overlap	14	20	25
with no overlap	36	0	37

Push away volume on one side = 18%



Several authors[20,21] discussed methods of reducing turbidity around hydraulic dredges.

Yagi[16,19] made a thorough study of hydraulic dredge pickup and residual solids left. He gave the formula:

$$X_c = \frac{1}{15} \frac{l \cdot t \cdot s}{D^2 \cdot V}$$

where: l = the spud advance spacing
t = the thickness of the attemped soil cut.
s = the rate of swing in m/min
D = diameter of the removal pipe
V = flow velocity in the pipe in m/s

here $X_c = \frac{\text{Vol Sediment Contacted}}{\text{Vol mixture pumped}}$

the wt. % ratio (w) $= \frac{\text{wt. \% of Solids in situ}}{\text{wt. \% of Solids pumped}}$

and $\frac{1}{wX_c} = \frac{\text{Vol \% Solids pumped}}{\text{Vol \% Solids contacted}}$, and $1 - \frac{1}{wX_c} = R$

R is the percent residual sediment volume missed per dredge swing. Yagi[17] noted that R increased with increasing dredge swing speeds and that R decreased with lower percent solids pumped. For the dredge studied[17] pumped about 105 cfs ($3m^3/s$). The cutter was 5.7 ft (1.75 m) in diameter and rotated at 18 rpm. The swing speed was in the 20-33 ft/min (6-10 m/min) range and the average cut of 1.75 m. The residual solids percent per swing, R, was in the 10-25 percent range. Turbidity around the dredge head increased with increased R and solids contents pumped. The turbidity on the side of the cutter entering the sediment was higher than for the other side. The additional loss of solids due to caving in of sediments from the top of the hole or due to poor alignment of swings was not considered. In their study one piece of sediment was as good as another. This is not the case for contaminated layer dredging.

As indicated in Figures 5, 6 and 7 theoretical calculations of hydraulic dredge cuts and overlaps were made for a 4 ft diameter cutter. A 45° angle of repose of the sediment was again assumed. Cuts deeper than 3 ft result in removal of substantial amounts of unwanted sediment when the contaminants are only in the top 2 ft layer. The 15 inch hydraulic dredge used by NYS Dept. of Transportation, is approximately 100 ft long including the ladder, and 25 ft wide. Therefore a 100 ft radius of sweep in a 100 ft arc about the spuds was assumed in Figure 7. Dredging about alternate spuds for 3 arcs (Spud A, Spud B, then spud A) etc., results in roughly a 20 percent sediment missed just due to swinging form the alternate spud. This loss could be reduced to less than a couple percent if dredging swings were always made about one spud. In summary, conventional hydraulic dredge operation about an alternate 7 ft spud spacing, with a 4 ft deep cut, could result in much poorer efficiency than the same dredge operating at a 3 ft cut, a 5 ft single spud operation as noted below.

	Hyd. Dredge Operation Modes %	
	Conventional	Improved
Individual sweep loss, V_m* (function of swing speed and % solids)	20	10
Cut overlap loss V_m (7 vs 5 ft spacing)	20.2	3.7
Alternate spuding loss, V_m	20.0	2
Total, V_m	50 - 60	14 - 16
Unwanted Sediment (4 ft vs 3ft cut)	31	18
Duplicated Areas empty Vol V_e	.6 + 33	10 + 2

* V_m = % volume missed.

These losses are very approximate theoretical estimates and need field confirmation. Nevertheless, the figures do indicate that there is a great potential for improving contaminated layer dredging.

Accurate Location and Monitoring Techniques

To insure a high percent recovery of a contaminated layer accurate dredge bite location techniques are needed. A one foot horizontal control accuracy and a .3 ft vertial control accuracy are needed. What is needed is a system that measures, records, and displays the dredge head location and compares this location with a preplanned ideal grid dredging pattern. The dredge operator must be able to easily see if his dredge head is following the preplanned dredge grab or sweep pattern. The depth of the dredge head also needs to be measured and displayed to the operator in relation to the depth of cut desired. Most dredges have some means of measuring depth and depth monitoring equipment is less critical. For hydraulic dredges a swing speed indicator and a solids content meter on the pipeline is needed to optimize the removal per swing. The data should be stored on tape or by other means so the engineer managing the project c n review it when time permits.

Although relatively few dredges employ accurate location equipment, various models are available. Inter Oceans Systems, Inc. offers a SPANS and LONGBASE positioning system.[23] They employ underwater transponders that return signals to a command transducer and transmitter. Range indicators, digital printers, computers, and plotters are also available. The transponder are attached to buoys or fixed structures and the underwater distances to the dredge from the several surveyed transponders are monitored.

Another location system is the Motorola Mini-Ranger III System.[24] It employs several above-ground transponders that return pulsed radar signals to the receiver-transmitter unit that indicates the distances to the two or more reference transponders. The system has optional equipment including a data processer, a tape recorder, keyboard terminals with either paper output or a TV screen type output, track indicators, depth sounders and a series of packaged programs to calculate and compare many quantities of interest. Cable length (pay out) indicators[25] are also available to aid in measuring the depth of the dredge head. This depth measurement can then be electronically fed to the Mini Ranger III data processor computer for calculations. Figure 8 shows some of the mini ranger equipment. CUBIC AUTOTAPE[26] and DEL NORTE[27] are other similar type above-ground locating systems. These systems require line of sight and some read to only 1 or 2 meters accuracy which may prove a limitation. The equipment is in the $50,000 to $100,000 cost range. In contaminant dredging programs in which the cost of sediment dredging, treatment and secure land burial may run $5/yd^3$ the system would pay for itself if 20,000 yd^3 of unwanted sediments could be left or 20,000 yd^3 of additional contaminated sediment recovered by the sophisticated instrumentation. For dustpan or Mudcat type dredges electronic navigation would not be as essential as for cutterhead hydraulic or clamshell type dredges. Depth of cut monitoring is probably more critical for the hydraulic dredges than for the clamshell which is dependent on a controlled height of free fall for penetration. A trial program to test the benefits of improving the efficiency of contaminated layer recovery with the sophisticated navigation hardware is needed. Conventional locating systems including transits, rangefinders, sextants, and line-of-sight positioning would probably be too time consuming and cumbersome for monitoring each bite of a clamshell dredge or sweep of a cutterhead hydraulic dredge. A transponder could be mounted on the top of a clamshell boom. This would put it 50 ft or so above the water. As long as the reference transponders were more than 1000 ft away the correction would be less than 1 ft. On a hydraulic cutterhead a transponder could be mounted on the spud holder. This would place it at the center of the dredge arc. An angle indicator would also then be needed.

Summary

In summary, it has been observed and documented that the sediment caves in and around dredging cuts and that overcutting and leveling of sediments to achieve some final elevation is common in the dredging industry. It has also been demonstrated from laboratory experiments, theoretical calculations and very limited field dredging data that efficiency of conventional dredging techniques (cutterhead hydraulic and clamshell) in removing a contaminated layer would be in the range of 65 percent. With improved dredging operation to follow preplanned overlapping dredge cuts the removal efficiency could be increased to 85-90 percent and the volume of unwanted sediment dredged reduced from 30 to 50 percent down to about 20 percent. Suggested clamshell and hydraulic overlaps and cuts were stated, but require field trials with accurate location equipment for confirmation. The accurate location monitoring equipment appears essential and economical for large projects involving dredging of a contaminated layer by cutterhead hydraulic or clamshell methods. Certain types of specialized dredges are on the market that could provide efficent contaminated layer removal with conventional location equipment such as transits, rangefinders, sextants, etc. These dredges include the dustpan and Mudcat type dredges. However, there are situations and sediments for which these dredges are not well suited.

Another way to achieve 85-90 percent contaminated layer recover is to practice conventional double pass dredging. However this doubles the yardage of sediment to be disposed of.

First pass dredging = 65 % recovery.

Second pass dredging = 65 %(35 %) + 65 % = 87.7 % recovery.

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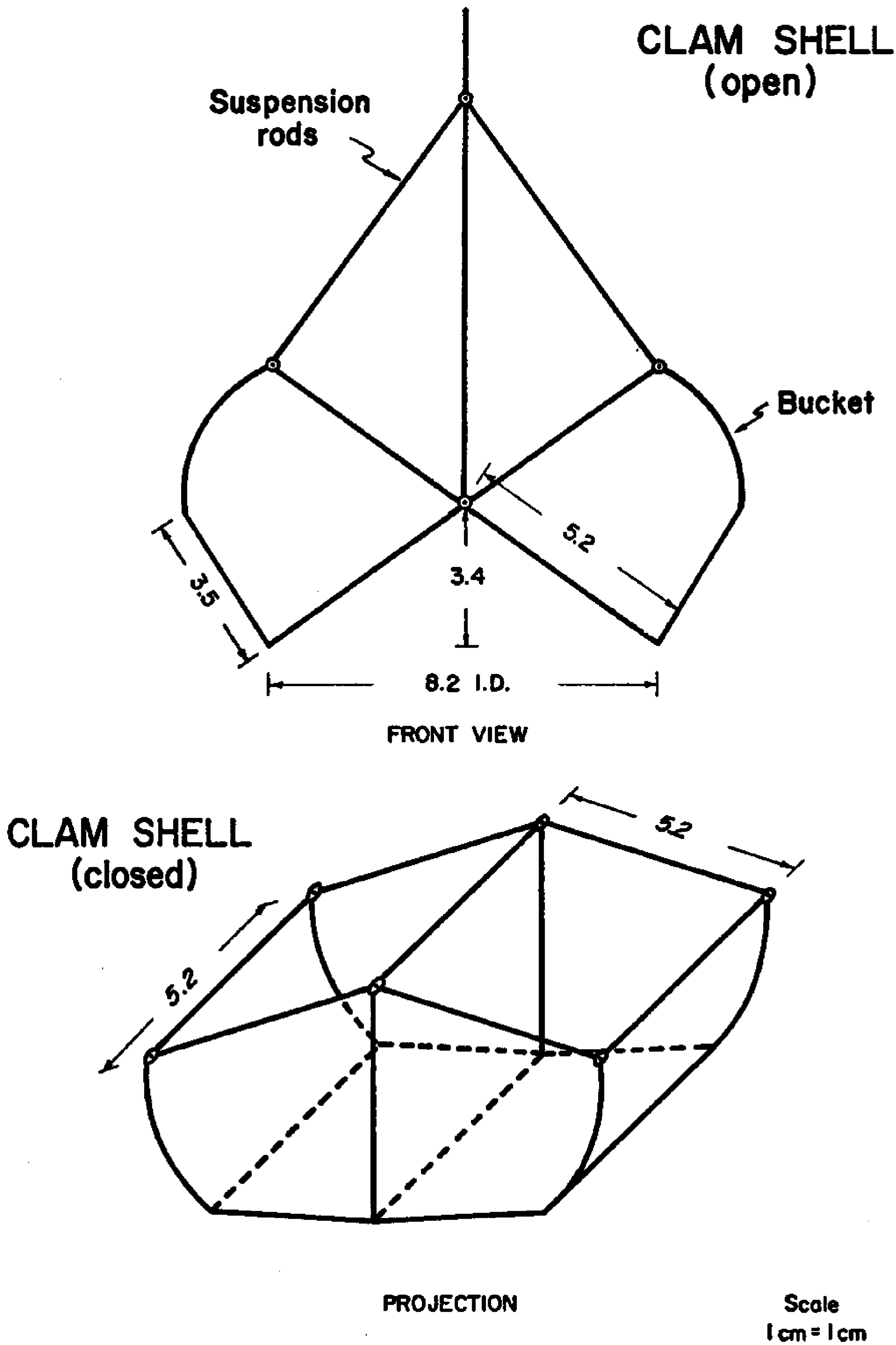
Figure 1



CLAM SHELL
(open)
Suspension rods
Bucket
5.2
3.4
3.5
8.2 I.D.
FRONT VIEW
CLAM SHELL
(closed)
5.2
5.2
PROJECTION
Scale
1 cm = 1 cm

FIGURE 2. CLAMSHELL TANK SET UP

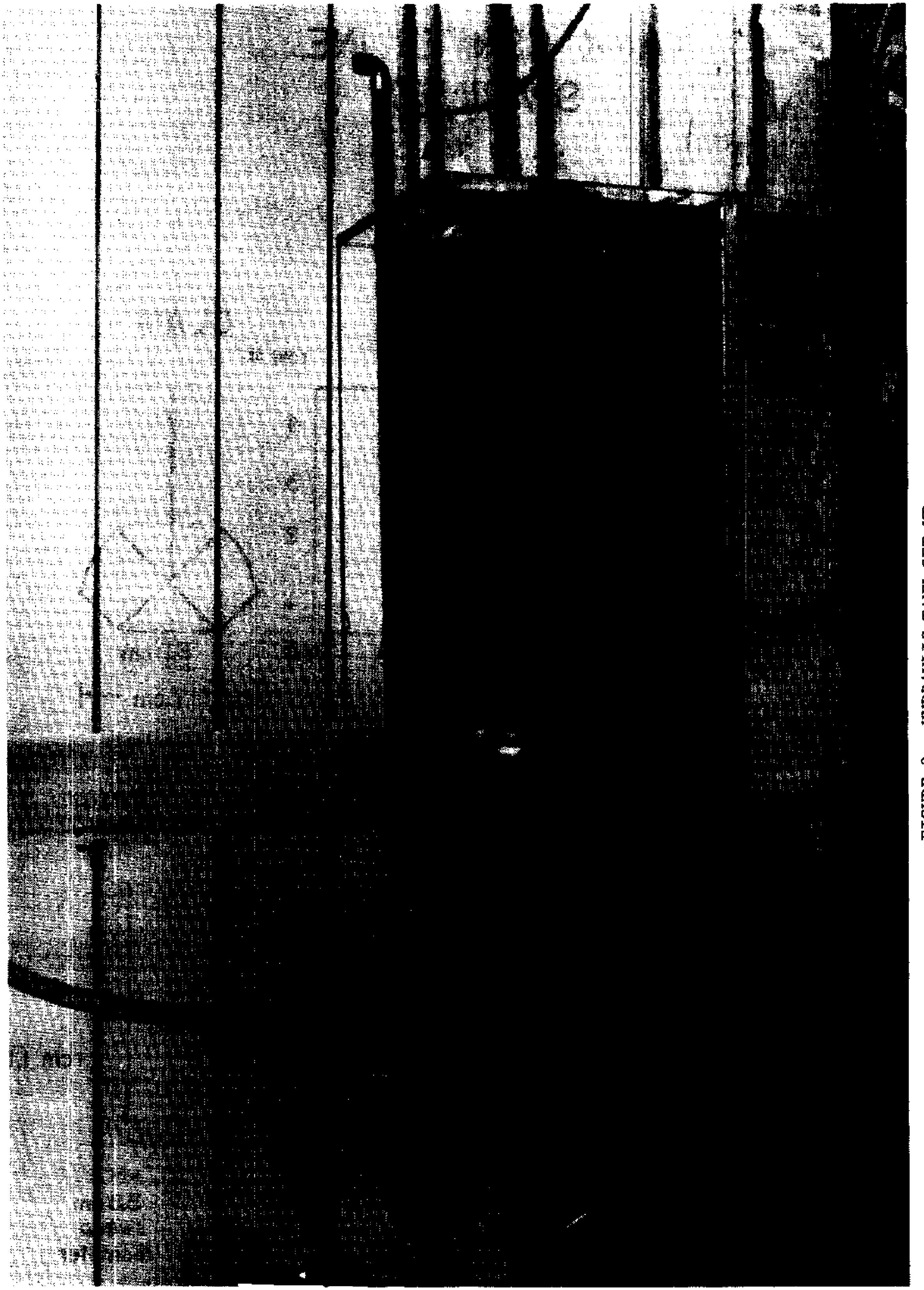


FIGURE 3. HYDRAULIC TANK SET UP

Figure 4

TANK CENTERLINE SPACINGS

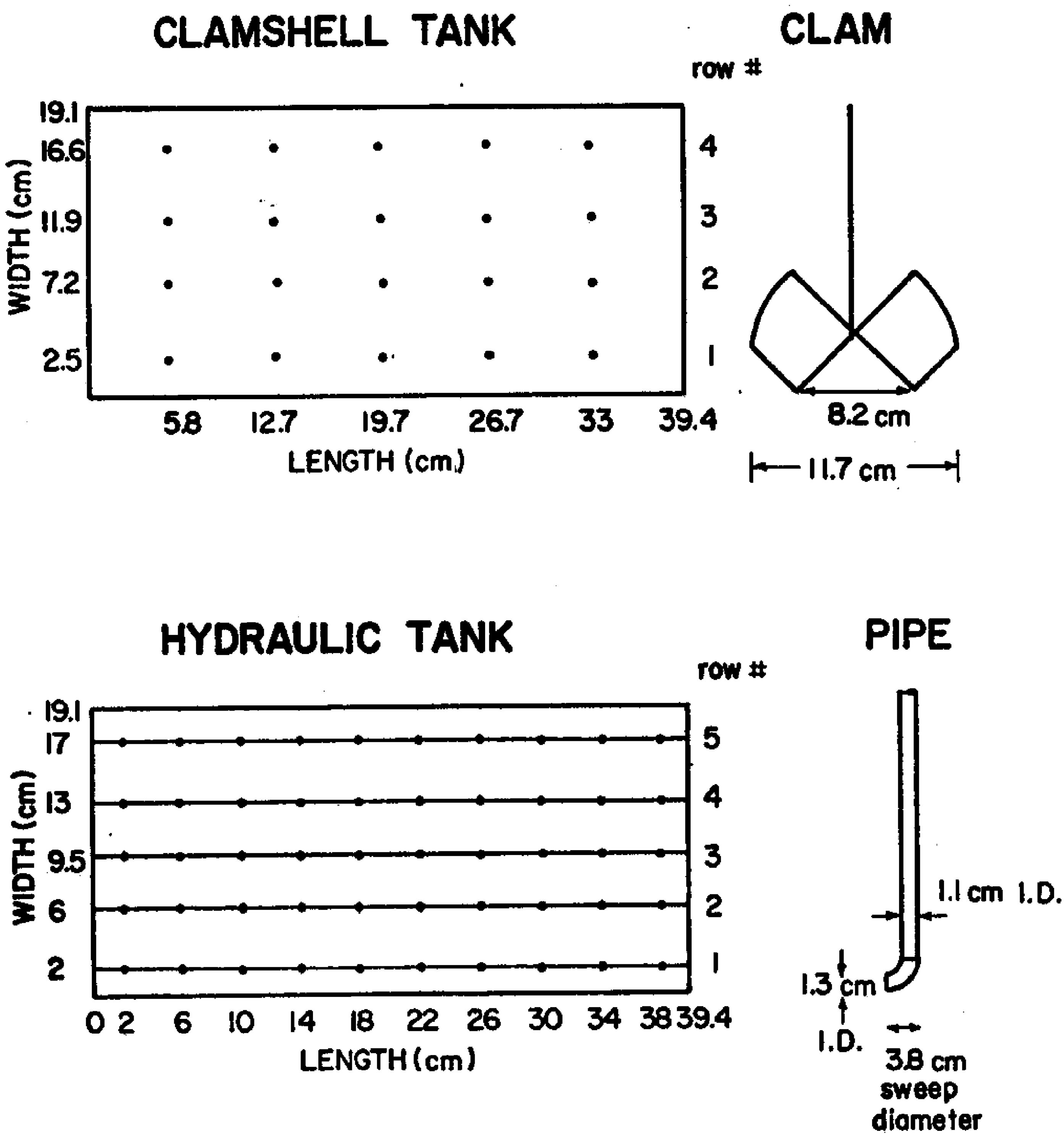
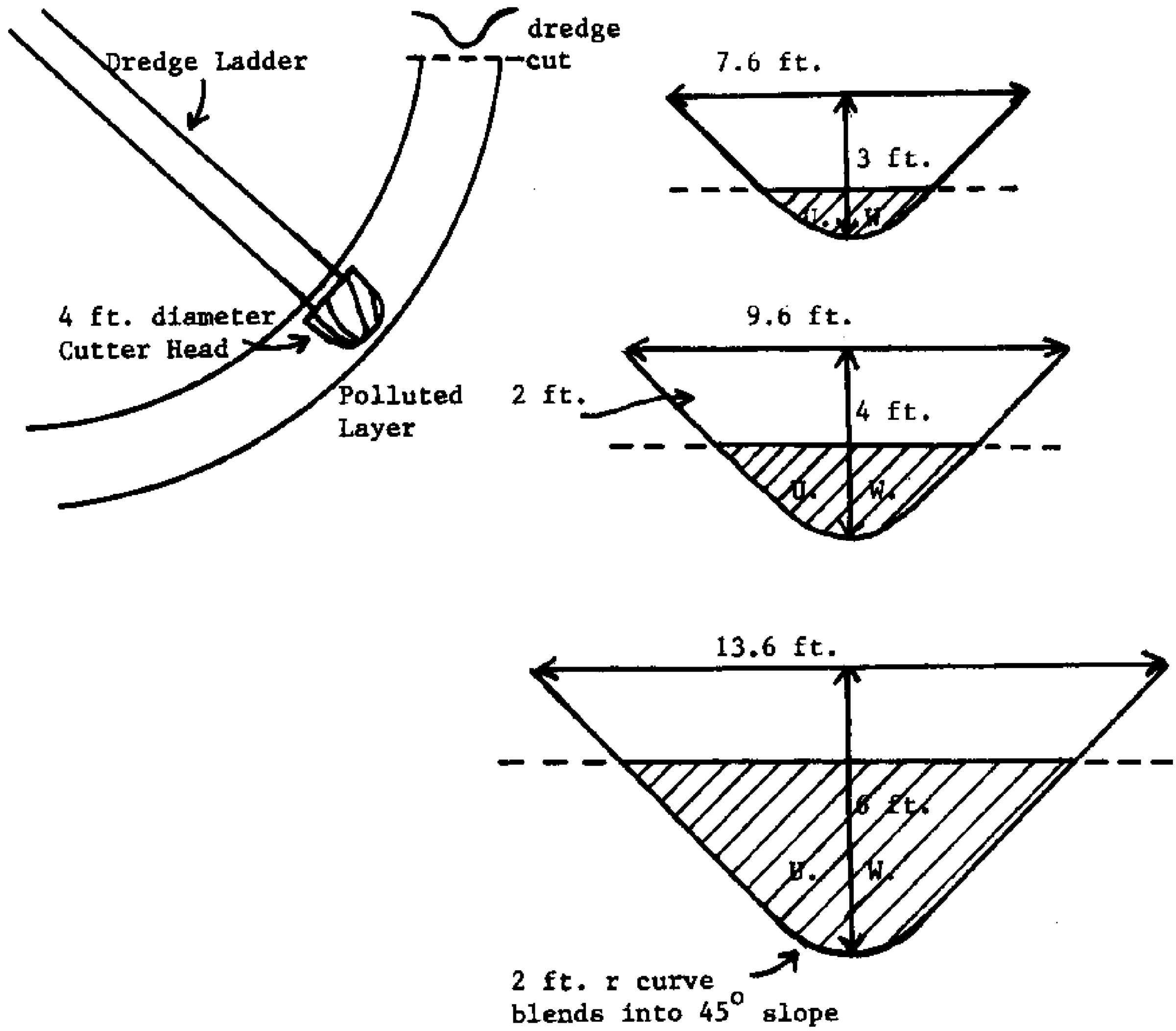
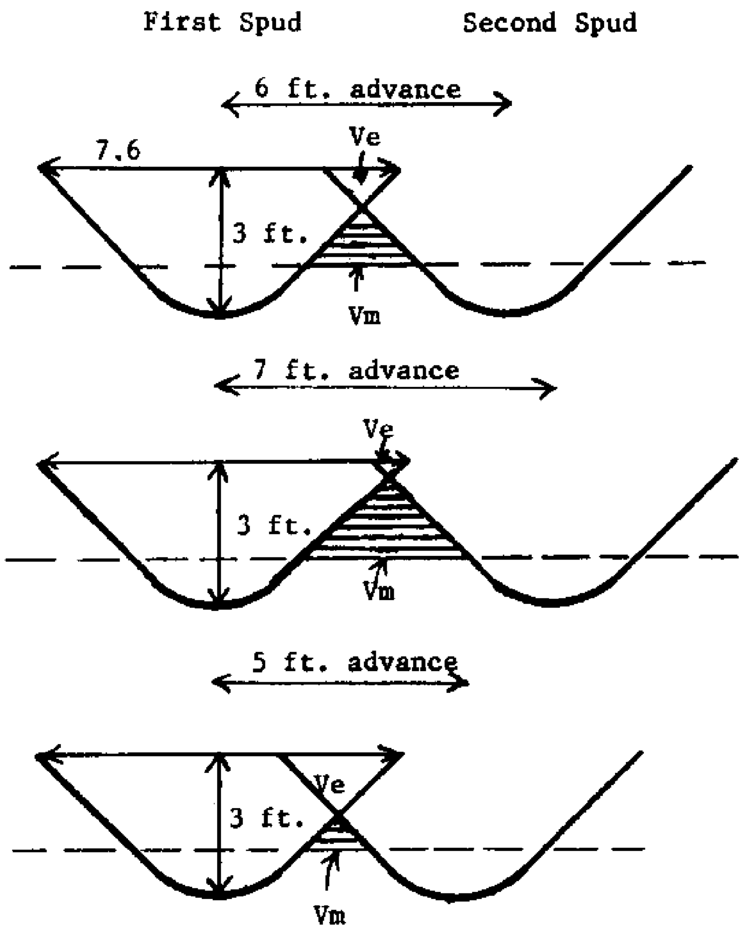


FIGURE 5. HYDRAULIC DREDGE CUTS - 45° CAVE IN



Areas	
Total ft^2	U.W. %
13.8	18.2
21.5	30.9
44.4	48.4

FIGURE 6. HYDRAULIC CUT OVERLAP



Areas-2nd Spud

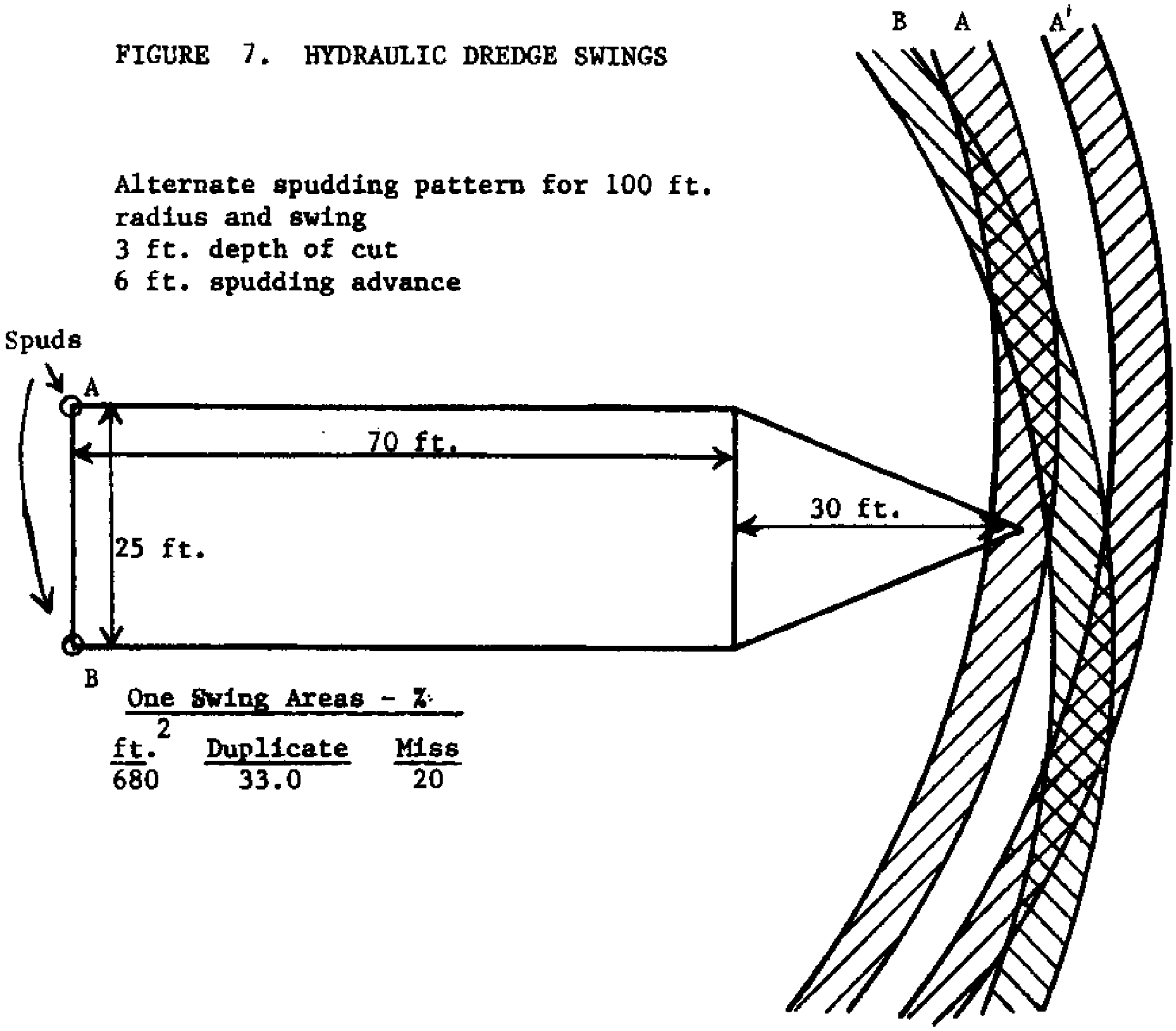
Total ft^2	Ve %	Vm %
13.8	4.0	10.1

Total	Ve	Vm
13.8	.6	20.2

Total	Ve	Vm
13.8	10.1	3.7

FIGURE 7. HYDRAULIC DREDGE SWINGS

Alternate spudding pattern for 100 ft.
radius and swing
3 ft. depth of cut
6 ft. spudding advance



One Swing Areas - %

$ft.^2$	Duplicate	Miss
680	33.0	20

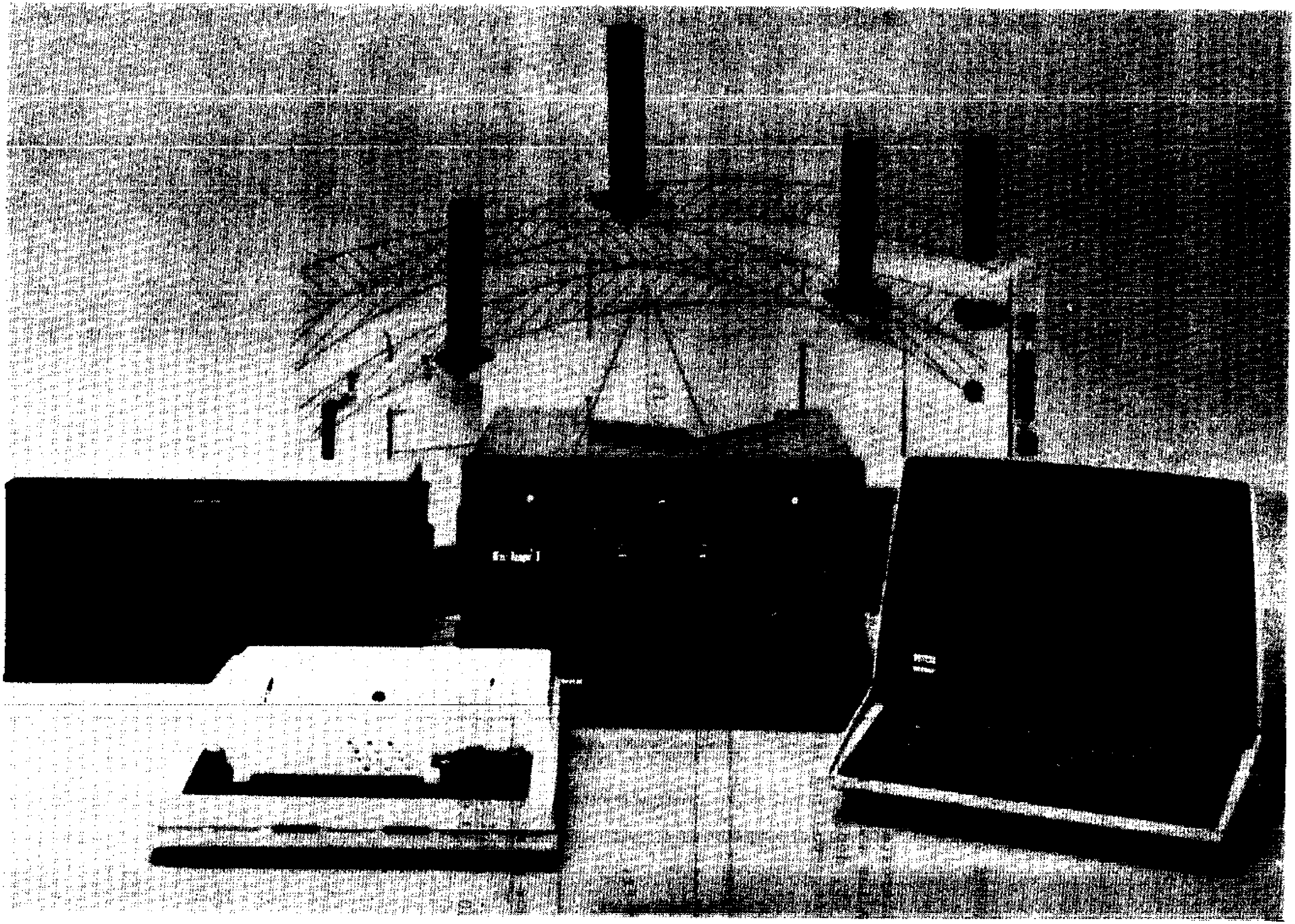


FIGURE 8. MOTOROLA MINI-RANGER III SYSTEM

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