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WEIR DESIGN TO MAINTAIN EFFLUENT QUALITY FROM DREDGED MATERIAL --ETC(U)
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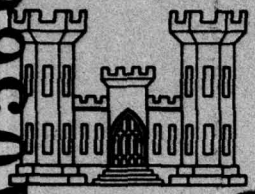

DREDGED MATERIAL RESEARCH PROGRAM

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WEIR DESIGN TO MAINTAIN EFFLUENT QUALITY FROM DREDGED MATERIAL CONTAINMENT AREAS.

by

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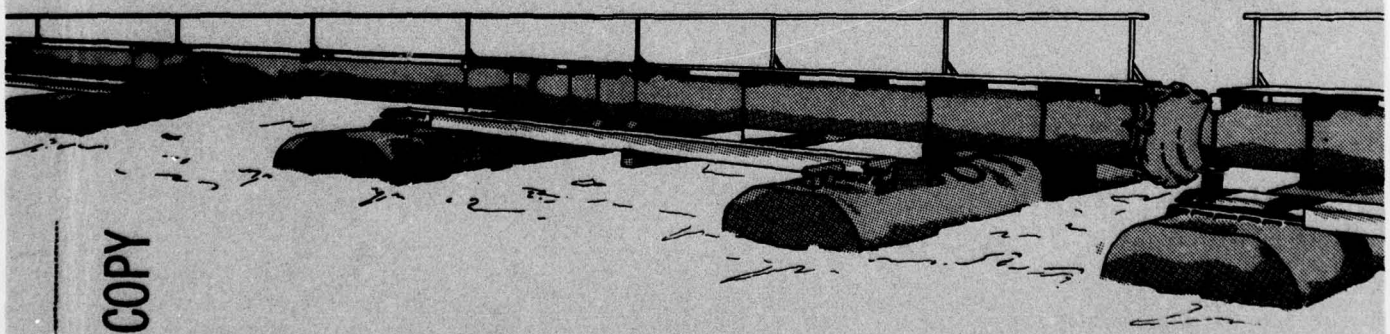

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VICKSBURG, MISSISSIPPI 39180

IN REPLY REFER TO: WESYV

30 June 1978

SUBJECT: Transmittal of Technical Report D-78-18

TO: All Report Recipients

1. The report transmitted herewith represents the results of one of the research efforts (work units) accomplished as part of Task 2C (Containment Area Operations) of the Corps of Engineers' Dredged Material Research Program (DMRP). Task 2C was a part of the Disposal Operations Project, which among other considerations included research into the various ways of improving the efficiency and acceptability of facilities for confining dredged material on land.

2. Practically no specific design or construction improvement investigations of confined dredged material disposal facilities had been undertaken prior to the DMRP. Being a form of waste product disposal, dredged material placement on land has seldom been evaluated on other than purely economic grounds with emphasis usually on lowest possible cost. There has been a dramatic increase within the last several years in the amount of land disposal necessitated by confining dredged material classified as polluted. Attention necessarily has been directed more and more to environmental consequences of this disposal alternative and methods for minimizing adverse environmental impacts.

3. Several DMRP work units were conducted to investigate and improve facility design and construction and to investigate concepts for increasing facility capacity and improved effluent quality. During these studies, it became apparent that no sound procedure existed for the design of weirs for containment areas. Proper design is necessary to prevent resuspension of settled material, particularly the fine material. Since practically all contaminants are associated with the fines, retention of the fines is essential in meeting water-quality standards. The investigation reported herein was accomplished by the Environmental Laboratory of the Waterways Experiment Station (WES) to eliminate this design deficiency.

4. Stratified-flow and sediment-transport models were investigated to describe the depth of withdrawal, velocity profile, and effluent suspended solids concentrations, given a concentration profile and flow.

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Field data on these parameters were collected at three sites: Yazoo River, Mississippi; Fowl River, Alabama; and Oyster Bay, Alabama. The WES selective-withdrawal model, modified to fit observed data, was chosen as the basis for the design procedure. Using this model, nomograms were developed for silt and saltwater clay and for freshwater clays. The nomograms relate the flow, weir length, ponding depth, and effluent suspended solids concentrations. The designer manipulates these four variables until a satisfactory balance between weir length and ponding depth is needed. In general, the weir crest should be maintained at as high an elevation as feasible during dredging operations. Guidance on operation of the weir for special applications is also presented.

5. It is believed that the procedures given herein will provide a rational method for a designer to determine the required weir length and ponding depth for specific sites. However, it should be noted that the nomograms are based on limited field data and further refinement is needed based on actual performance data.

JOHN L. CANNON
Colonel, Corps of Engineers
Commander and Director

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

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

Stratified-flow and sediment-transport models were investigated to describe the depth of withdrawal, velocity profile, and effluent suspended

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20. ABSTRACT (Continued).

solids concentration, given a concentration profile and flow. Field data on these parameters were collected at three sites--Yazoo River, MS, Fowl River, AL, and Oyster Bay, AL.

The Waterways Experiment Station's selective withdrawal model developed by Bohan and Grace, modified to fit observed data, was selected as the basis of the design procedure. Using this model, nomograms were developed for the design procedure for silt and saltwater clays and for freshwater clays. The nomogram relates the flow, weir length, ponding depth, and effluent suspended solids concentration. The designer manipulates these four variables until he reaches a satisfactory balance between weir length and ponding depth, based on his design flow and effluent goal. Sharp-crested rectangular or shaft-type weirs are recommended.

Proper operation of the weir can ameliorate the effects of short-circuiting or an undersized basin. In general, the weir crest should be maintained at as high an elevation as feasible during dredging operations. Guidance for operation of the weir for special applications is also presented.

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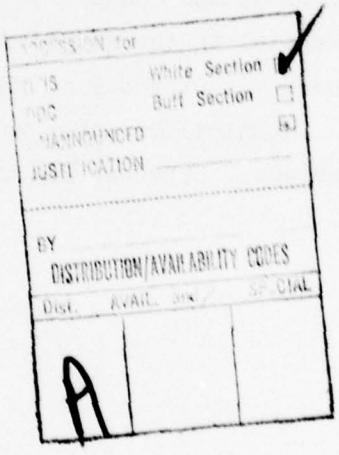

PREFACE

This study was conducted as part of the U. S. Army Corps of Engineers' Dredged Material Research Program (DMRP), which is sponsored by the Office, Chief of Engineers, as part of Task Area 2C, Containment Area Operations, of the Disposal Operations Project (DOP).

The work was performed during the period February-July 1977 by Mr. Paul R. Schroeder and Mr. Thomas M. Walski of the Design and Concept Development Branch (DCDB), Environmental Engineering Division (EED), Environmental Laboratory (EL), U. S. Army Engineer Waterways Experiment Station (WES). The investigation was conducted under the active supervision and guidance of Mr. Raymond L. Montgomery, Chief, DCDB. Manager of DOP was Mr. Charles C. Calhoun, Jr. Mr. Newton C. Baker was manager of Task 2C. Review and assistance were provided by Dr. William D. Barnard of DOP and Mr. Darrel G. Fontane and Mr. Marden B. Boyd of the Hydraulics Laboratory, WES. Instrumentation support was provided by Mr. Bobby E. Reed and Mr. John W. Beasley of the Instrumentation Services Division, WES. Assistance in planning and preparation for the field trips was provided by a large number of Corps district office personnel, especially in the Mobile and Vicksburg Districts.

The contribution of E-4 José L. Llopis of the DCDB in the organization, supervision, and conduct of the field trips and laboratory analyses was essential to the successful completion of this study.

Commander and Director of WES during this study was COL John L. Cannon, CE. Technical Director was Mr. F. R. Brown. Chief of EL was Dr. John Harrison. Chief of EED was Mr. A. J. Green.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
acres	4046.856	square metres
feet per second	0.3048	metres per second
cubic feet per second	0.02831685	cubic metres per second
feet per second per second	0.3048	metres per second per second
cubic feet per second per foot	0.0929	cubic metres per second per metre
million gallons per day per foot	0.1437	cubic metres per second per metre
gallons (U. S. liquid) per day	0.4381264	cubic centimetres per second
million gallons (U. S. liquid) per day	0.04381264	cubic metres per second
degrees (angular)	0.01745329	radians
miles (U. S. statute)	1.609344	kilometres

WEIR DESIGN TO MAINTAIN EFFLUENT QUALITY FROM DREDGED MATERIAL CONTAINMENT AREAS

PART I: INTRODUCTION

Background

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

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

3. The weir is only one component of containment area design. Its function is to withdraw the clarified water from the basin. The weir alone can not assure good effluent quality since effluent quality

is also dependent on the basin volume and hydraulics efficiency. The weir can be used to maintain good effluent quality from the properly designed basin.

Scope

4. The scope of this study was to assess the relationship between weir design and effluent quality and to develop a procedure for designing and operating weirs. The five major areas of work included (a) problem assessment; (b) review of weir design, density-stratified hydraulics, and sediment-transport models; (c) field data collection; (d) model selection and verification; and (e) design procedure development.

PART II: PROBLEM ASSESSMENT

Effluent Quality Goals

5. Effluent quality standards are often based on the quantity of material or sediment particles in the effluent. However, no uniform set of effluent guidelines presently exist for disposal sites since they are imposed by either state regulations, local ordinances, or district guidelines.[1] Current Federal legislation does not specify quantitative guidelines, but, qualitatively, Sec. 404 of PL 92-500 states that dredged material disposal must not be detrimental to the environment.[2]

6. Current effluent guidelines vary in both quantity and type from state to state and locality to locality. They are expressed in terms of suspended solids above ambient, turbidity, and settleable solids. Common guidelines are 8 to 13 g/ℓ above ambient for suspended solids, 5 to 50 JTU for turbidity, and 0.2 mℓ/ℓ for settleable solids.[1] These variable effluent quality guidelines required the design procedure to be flexible and the designer to be able to closely predict the effluent quality.

7. The purpose of the design procedure is to assure good effluent quality. For the purpose of this report, the effluent quality is expressed in terms of the suspended solids concentration in the effluent (effluent suspended solids concentration). The user is responsible for converting the values to turbidity units or settleable solids if effluent standards are expressed in these units. A low concentration indicates a good effluent quality and a high concentration indicates a poor effluent quality.

Concepts in Weir Design for Containment Areas

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

Containment areas

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

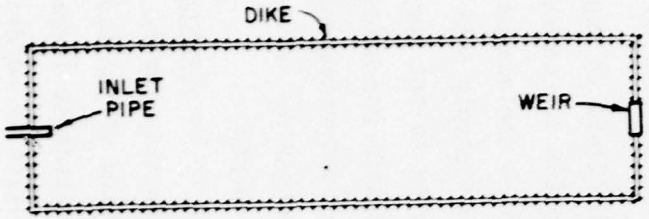
Suspended solids and density profiles

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

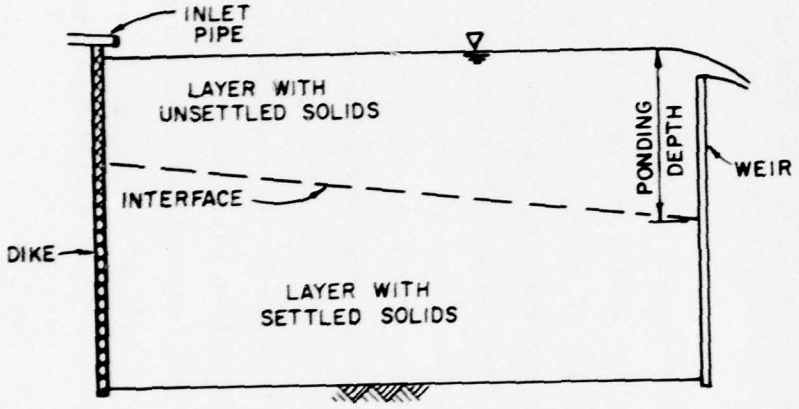
11. Since suspended solids are constantly moving downward, the suspended solids concentration is highest at the bottom of the basin and is lowest at the surface. A graph showing the change of concentration with depth is shown in Figure 1. This type of graph is referred to as suspended solids concentration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

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

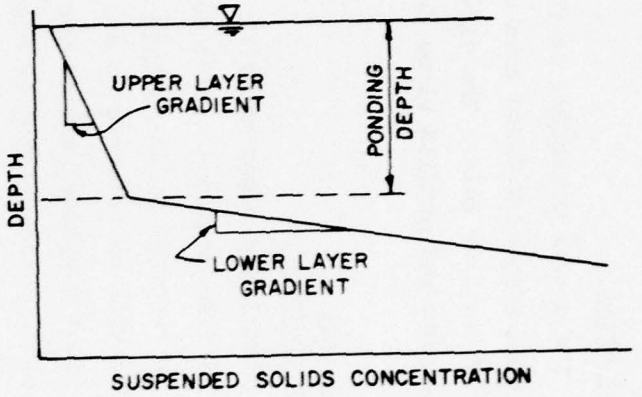
13. The fluid in the containment area is said to be stratified if the density increases with depth. (The term fluid in this report refers to all water and unconsolidated solids above the bottom of the basin.) The gradient is said to be strong if the density gradient is large (i.e., the difference in solids concentration with depth is large).



a. PLAN VIEW OF A CONTAINMENT AREA



b. PROFILE VIEW OF A CONTAINMENT AREA



c. TYPICAL SUSPENDED SOLIDS PROFILE

Figure 1. Basin and settling descriptions

Ponding depth

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

Weir concepts

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

$$Q = C_D BH^{3/2} \tag{1}$$

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

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

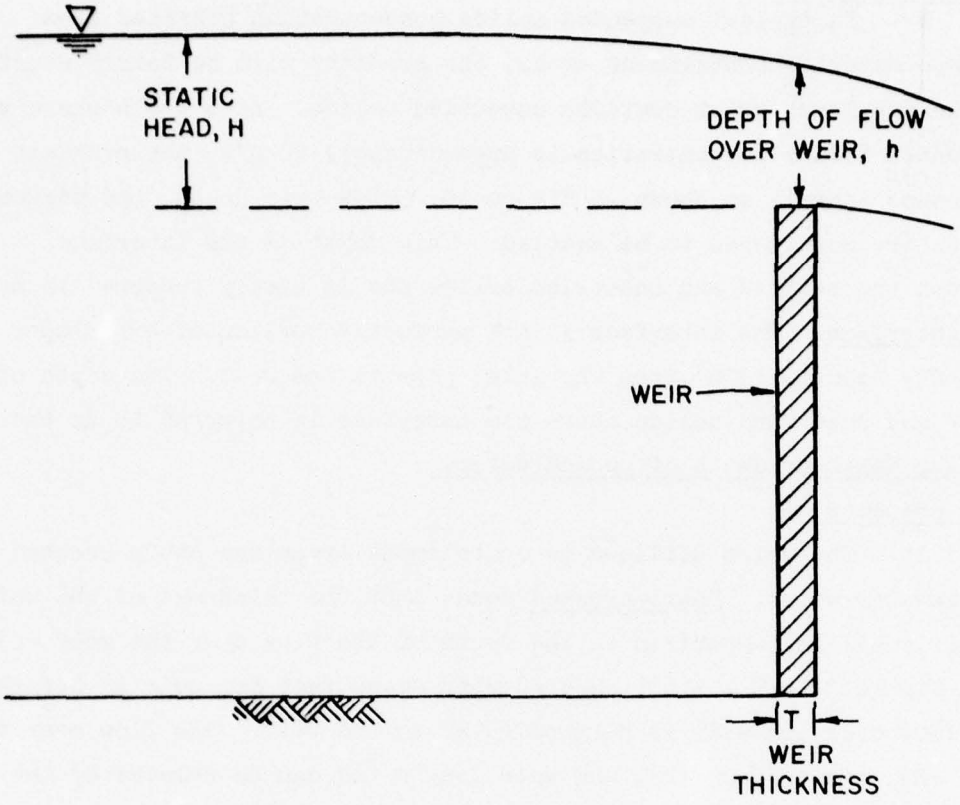


Figure 2. Weir characteristics

$$h = 0.85H \tag{2}$$

h must be measured directly above the weir crest.

Withdrawal zone

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

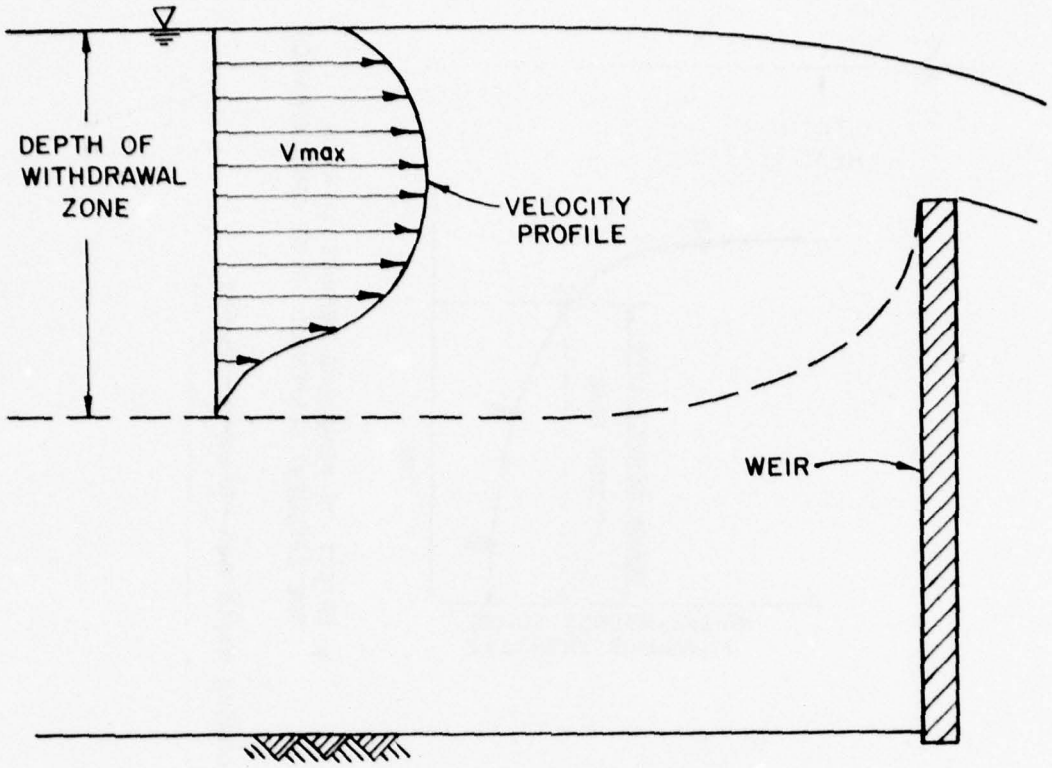


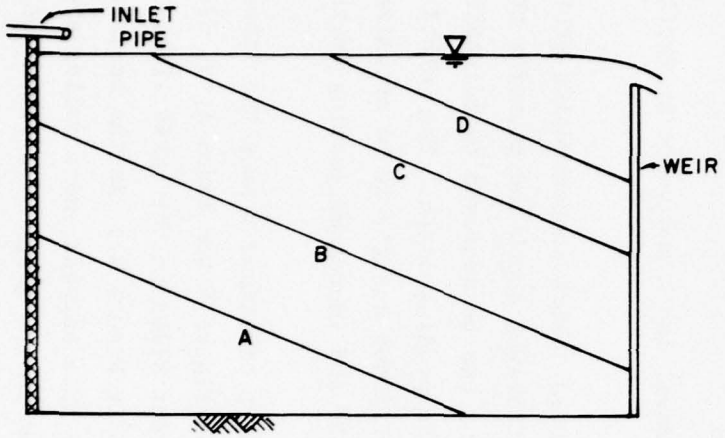
Figure 3. Withdrawal depth and velocity profile

Design description

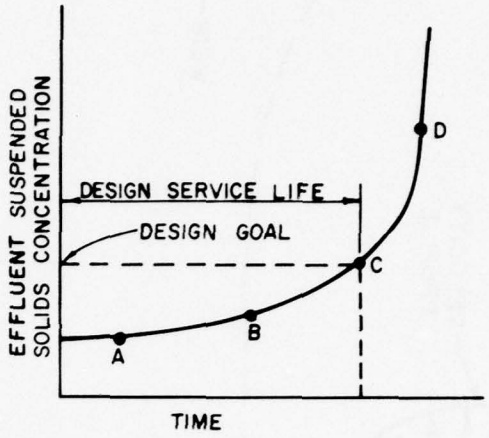
18. For a given suspended solids concentration profile and flow, a longer weir reduces the withdrawal depth and improves the effluent quality. The same improvement can be achieved by maintaining the same weir length and increasing the ponding depth. The method for designing weirs to maintain adequate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

Service life of basin

19. During the life of a containment area the interface moves upward and toward the weir. In Figure 4 the lines A, B, C, and D represent the interface at different times in the basin life. (The vertical scale is greatly exaggerated in Figure 4.) As the basin fills, the ponding depth decreases. As this happens, more solids are withdrawn over the weir. This is shown in the graph of effluent solids versus



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



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

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

time in Figure 4. Sufficient ponding depth must be provided so that the dredging job can be completed before the effluent quality deteriorates as it does between times C and D.

20. When a basin is used on a continuous basis the service life is defined as the time period from the start of disposal activities until the ponding depth is less than the design ponding depth. The service life can be extended by operating the dredge on an intermittent basis, allowing more time for sedimentation of solids. Assuming the weir elevation is constant, ponding depth becomes critical at the end of the service life. Beyond this time the withdrawal depth exceeds the ponding depth. Similarly, the predicted effluent suspended solids is for the end of the service life. It is this value for the ponding depth that is determined by the design procedure. The effluent suspended solids will be lower than the predicted value during the service life (e.g., times A and B).

21. Because of the sloping interface, the ponding depth is not constant throughout the basin but increases away from the inlet pipe. The ponding depth of concern in weir design is the final ponding depth immediately in front of the weir. (For the sake of this report "in front of the weir" refers to a distance one-half of a weir length (B/2) in front of the center of the weir.)

Factors Contributing to Effluent Solids

22. Given sufficient retention time in a containment area, non-colloidal suspended solids will settle. The bottom layer of fluid in the basin will contain a much higher concentration of solids and will therefore have greater bulk density. This type of profile will tend to prevent fluid from the bottom layer from being withdrawn over the weir.

23. Solids are discharged from containment areas because either they did not settle, they were resuspended by scour, or they were discharged with settled layers of fluid. The solids may not have settled due to an insufficient basin size and detention time, turbulence, or the particles' chemical and physical properties. The particles have been

resuspended by scour due to the drag and lift on the particles caused by the high localized velocity of turbulent eddies, often induced by flow contraction or wind. Finally, settled layers of solids are discharged over the weir due to inadequate weir length and ponding depth. Proper weir design and operation will control resuspension and withdrawal of settled material.

Weir Factors and Effects

24. The weir length, type, shape, and location have significant effects on the effluent quality from containment areas. First, the length of the weir controls the head over the weir for a given flow. The head, when considered with the density profile, controls the depth of the withdrawal zone. The deeper this withdrawal zone, the greater the effluent solids concentration will be for a given basin condition.

25. The weir type affects the effluent quality in several ways. Different weirs, such as broad- or sharp-crested, have different coefficients of discharge that change the required head over the weir for a given flow condition. Consequently, the depth of withdrawal and the velocity profile change. This affects the effluent quality by discharging different volumes of fluid from the layer with higher solids concentrations.

26. The weir shape or configuration affects the dimensions of the withdrawal zone and, correspondingly, the coefficient of discharge. The width of the withdrawal zone expands as one moves further into the basin from the weir until the zone reaches a dead zone or the sides of the basin. The expansion of the withdrawal zone results in decreased velocity in the zone. The withdrawal zone for a weir to which water flows from several directions expands quicker because it discharges fluid in more than one direction. The expansion of the flow field and reduction of the approach velocity reduce the depth of withdrawal. Consequently, more fluid is discharged from the upper portion of the top layer and the effluent quality is improved. In the case of a polygonal weir with flow approaching from one direction, the width of withdrawal zone is smaller

than for a rectangular weir with the same crest length. This means that the velocities in the withdrawal zone and depth of the withdrawal zone are larger and hence the effluent quality is poorer. This will be explained in greater detail in Part VII.

27. Finally, the weir location is important for several reasons. First, it can help prevent short-circuiting and channeling. This provides the suspended particles more time to settle. Similarly, it spreads out the flow, reducing the velocities in the basin and thereby minimizing the scour and resuspension of the settled material. Other results concerning short-circuiting and weir design have been reported by Gallagher.[3]

28. For a given flow and material, the two most important parameters in the weir design for meeting effluent quality goals are weir length and ponding depth. A longer weir reduces the depth of the withdrawal zone for a given density profile. Similarly, increasing the ponding depth provides a larger layer of fluid with low solids concentrations to be discharged over the weir. Consequently, this will result in discharge of fluid from greater depths but will result in improved effluent quality. Therefore, a tradeoff develops between increasing the weir length or increasing the ponding depth to meet the effluent quality goals. This tradeoff is incorporated into the design procedure to give the designer the flexibility needed in the weir design due to site constraints and economic considerations.

29. Weir loading rates used in sanitary engineering are of little help in designing containment areas since they are for deeper basins with virtually no density gradient and are responsible for producing a much clearer effluent. The Ten States Standards[4] recommend weir loadings of 15,000 gpd* per foot of weir for plants larger than 1 mgd. Assuming that an 18-in. dredge will produce approximately 20 cfs (13 mgd), weirs of over 1000 ft in length would be required. It is possible to utilize much shorter weir lengths for dredged material

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 5.

containment areas if sufficient ponding depth is provided.

Other Factors Affecting Effluent Quality

30. Many other factors besides the weir affect the effluent quality. Among the most important are the basin design, the dredged material type, the weather, and the dredging operation itself. This report will focus on these factors only insofar as they affect weir design.

Basin design

31. The basin size is important because larger basins (a) allow the particles more time to settle, and (b) lower the velocities in the basin, thereby reducing scour. The basin inlet structure and shape can minimize short-circuiting and channeling. Furthermore, the basin shape can provide added service life for the basin and maintain good effluent quality longer. This occurs since basins fill near the inlet first and then progressively closer to the weirs, resulting in the interface sloping down toward the weir. Thus a longer distance between the basin inlet and the weir will maintain large ponding depths at the weir for a longer period of time, permitting more of the basin volume to be used for sediment storage before the effluent quality is lowered. Murphy and Zeigler[1] state that basin operation can be improved by properly vegetating the basin, which would reduce scour, resuspension of settled material, and creeping of the settled material toward the weir; furthermore, such vegetation can dissipate the energy of the incoming water and spread the flow more uniformly throughout the basin. However, spotty vegetation can produce dead zones and induce short-circuiting. The age of the basin in terms of the remaining capacity can alter the concentration profile; also, the time allowed for consolidation between uses can influence the amount of unconsolidated material available to be scoured.

Dredged material type

32. The dredged material type influences the effluent by several mechanisms. First, small-diameter particles (clays and silts) settle slower than large-diameter particles (sands and gravels). Clays do not

settle rapidly in fresh water due to their colloidal properties. These differential settling rates alter the concentration profile for different dredged material since different dredged material has different size distributions. Second, in saline water the soil particles are flocculated. Therefore, they settle much quicker and produce a sharp break in the density profile. Finally, dredged material with large proportions of sands rather than silts or clays also tends to produce a sharper break in the profile.

Weather

33. Temperature and wind can affect effluent quality. Cold temperatures increase the viscosity of the water, which affects the effluent in two ways. First, it decreases the settling rate, increasing the suspended solids at the weir. Second, it increases the shear in the fluid, which increases the depth of the withdrawal zone. Both actions slightly impair the effluent quality. Wind produces a shear on the water surface that can initiate waves and currents in the water creating scour and resuspending the settled material. Furthermore, the wind can produce turbulent eddies that disrupt the settling process.[3]

PART III: LITERATURE REVIEW

34. A literature review was conducted to determine which model would best predict the depth of the withdrawal zone or the required ponding depth and the velocity profile. Both stratified-flow selective withdrawal models and sediment-transport models were considered for predicting the withdrawal depth. The velocity profile models are based on the density profile, weir type, or boundary shear. While theoretical descriptions of flow conditions similar to those encountered in containment areas are available, no field data could be found in the literature for the withdrawal depth, or for the velocity profile in a dredged material basin.

Withdrawal Depth Models

35. Two model types existed for modeling the requirements for weir design. The first model type was based on selective withdrawal of a density-stratified fluid through a horizontal line sink (e.g., a weir, sluice gate, or linear orifice). These models correlated the characteristics of flow in the withdrawal zone with a densimetric Froude number. A densimetric Froude number is a ratio of gravity force between the different layers to the inertia forces in the withdrawal zone. The definition of the densimetric Froude number varied for the different selective withdrawal models. The point where the gravity forces exceeded the inertia forces determined the depth of the withdrawal zone. The shape of the velocity profile is a function of the density profile and the inertia forces controlled by the weir flow, weir shape, weir type, and flow contractions. The second model type was based on sediment-transport principles, in which the critical or threshold conditions required for sediment movement are not exceeded in the design. In these models, various soil types, grain diameters, and flow conditions have been evaluated to determine the extent to which scour will occur. All of these models can be used to predict the permissible basin velocities that can be achieved by adjusting the ponding depth and weir length.

Selective withdrawal models

36. The stratified-flow selective withdrawal hydraulic models varied in their approaches but can be classified in three classes based on their assumption for the density profile. The simplest theory was the two-layer model that assumed two layers of fluid with different densities. The depth of the withdrawal zone was then based on the density difference between the two layers and the unit flow rate or weir loading (flow per foot of weir length). The second class assumed a linearly stratified fluid (i.e. constant density gradient). Again, the withdrawal depth was based on the density gradient and the unit flow rate. The third type of model could use any type of density profile. The two-layered and linear stratification assumptions did not fit the typical density profile exactly (see Figure 1). The typical profile more closely represented two linearly strafified layers with a sharp break in the density gradient between the two layers. Only the third type of model closely represented the typical field situation but the other models deserved comparison and testing due to the simplicity of their approaches and usefulness in demonstrating applicability of the models. A large number of these stratified-flow models were investigated. Their characteristics are summarized below.

37. Bohan and Grace[5] developed a one-dimensional computerized model for selective withdrawal based on laboratory flume studies that correlated the depth of the withdrawal zone with the head over the weir and the local densimetric Froude number. This model is the WES selective withdrawal model. The program also calculates the velocity profile based on the weir type and density profile. It is capable of using any form of density stratification and predicting the effluent solids concentration. This model offered the best potential for use in the design procedure.

38. Wood and Lai[6] developed a two-layered stratified-flow model based on the Bernoulli energy equation. The approach considered the effects of different weir types by using the weir discharge equation with different coefficients of discharge for the different weirs. The model predicts the depth of the withdrawal zone but cannot predict a

velocity profile. The model was compared for model selection.

39. Yih[7] determined theoretically the critical densimetric Froude number at which the entire depth of linearly stratified fluid would be discharged to be $1/\Pi$. However, the critical densimetric Froude number is sufficient for movement but not necessary (i.e., the entire fluid can be discharged at lower Froude numbers). Furthermore, his approach did not consider withdrawal from fractions of the depth and therefore was not applicable to this study.

40. Debler[8] experimentally determined the critical densimetric Froude number at which fractions of the depth would be discharged for a linearly stratified fluid. His work was compared with the other promising models. His work was one-dimensional and did not predict a velocity profile.

41. Kao[9] extended Yih's work into a streamline analysis for a linearly stratified fluid with irrotational flow by employing stream functions. The results verified the works of Debler and Yih. Kao further stated that the depth of the withdrawal zone would be shallower if the viscosity of the stagnant layer was greater than that of the moving layer. His work cannot be easily applied in a design procedure.

42. Huber[10] assumed a fluid with two layers of equal thickness. He then used a relaxation technique to theoretically determine the critical value of his densimetric Froude number. His model was used for comparison though his assumptions appeared prohibitive.

43. Koh[11] developed a two-dimensional model for a viscous, diffusive, slightly stratified laminar flow. This model was not applicable since it was valid only for very small flow. Koh also proposed a similar model for turbulent flow but the model required information on the diffusion in the basin that was not available.

44. Gelhar[12] proposed a model for viscous, nondiffusive, linearly stratified laminar flow toward a line sink. His model was based on the Navier-Stokes equation but the solution was valid only near the weir. The model was not applicable because containment areas have turbulent flow at the weir while the model was for laminar flow.

45. Schlag[13] experimentally developed a relationship for

two-layered flow over a weir. The ratio of the head over the weir to the adjusted depth of withdrawal below the weir crest was linearly correlated with the density difference between the two layers. He further demonstrated the effect of the ratio of weir length to basin width on the effective length of the weir. He also concluded that the effect of the length to depth ratio of the weir was insignificant. This method required flume studies for each dredged material to determine the coefficients for his relationship. This was impractical for this design procedure.

46. Water Resources Engineers[14] developed a two-dimensional finite element computer model for stratified flow. It was based on the equations of continuity, motion, and mass transport or diffusion. This method can describe the flow but cannot be easily utilized in design.

Sediment-transport models

47. Several approaches based on sediment-transport principles existed. The first type utilized an entrainment function to determine the critical shear stress based on experimental results. The second type relied on empirical or semiempirical relationships to determine the critical shear stress. The third used empirical or semiempirical equations to determine the critical velocity. Finally, a model developed for dredged sediments existed that used the sediment concentrations and the initial rigidities of the deposited materials to determine the critical shear stress. All the models were sediment-type dependent using factors or added terms to account for the different sediment types. The sediment models must be incorporated with the continuity equation to determine the required ponding depth and a velocity profile to predict the effluent quality. The following sediment-transport models were reviewed.

48. Shields[15] proposed an entrainment function from experimental data in the form of a plot of the dimensionless critical shear stress versus the bed Reynolds number. The model required an iterative use of the model and, consequently, cannot be easily applied. The function is applicable to bed Reynolds numbers greater than 1.5 (i.e., laminar boundary layer).

49. Camp[16] derived an empirical expression for the critical velocity based on data for coarse granular particles with boundary Reynolds numbers greater than 1.5 (i.e., transition or turbulent boundary layer). His equation was sediment-type dependent by using the particles' specific gravity, grain size, and a "stickiness" factor. His model was not applicable for laminar bed shear.

50. Vanoni[17] and Laufer[18] both used the same approach. Their models are designed to predict scour by turulent eddies, using suspended sediment principles. They stated that the ratio of the settling velocity to shear velocity must be greater than 1.2 to 2.0 depending on sediment type. This correlated well with clarifier design theory where the ratio of the basin velocity to the settling velocity should be less than 9 to 15. The shear velocity can be used to calculate the basin velocity using the Darcy-Weisbach friction factor. The Vanoni and Laufer models used too many assumptions that could not be verified.

51. Migniot[19] correlated the initial rigidity of the sediment with the critical shear stress. This method was undesirable for a design procedure because the required laboratory equipment is not readily available to conduct the rigidity test on each sediment in the field. Furthermore, the test was intended to be performed on partially consolidated sediments. However, the top layer of sediment has usually not consolidated during the dredging operation. The model could not be easily applied and therefore was not tested.

52. White[20] derived a semiempirical expression for noncohesive sediments to determine the critical shear stress by balancing the lift, drag, and gravity forces on a particle. He included a coefficient based on experimental studies in a large flume. This coefficient varied for different flow conditions based on the boundary Reynolds number. His model appeared flexible and sound.

53. Ingersoll[21] verified the results of Camp and White in flume studies. He also proposed a method to calculate the critical bed shear stress using the average velocity and the surface velocity from these tests. This method would require flume studies for each type of sediment and, therefore, was not applicable.

54. Chepil[22] proposed an expression for the critical shear stress based on turbulent flow conditions in a wind tunnel. His work agreed with White's for the turbulent flow condition though his relationship included a special term for lift, which was much greater in the wind tunnel. His work supported the use of White's expression in the design model.

Velocity Profiles

55. The third phase of the literature review was an investigation of the possible velocity profiles. The velocity profile is required to determine the effluent solids concentration given the solids profile and depth of withdrawal zone. The following three models were available.

56. The WES selective withdrawal model by Bohan and Grace[5] presented the only velocity profile based on stratified flow. The model correlated the ratio of the velocity at any point to a maximum velocity with the ratio of the product of the distance between that point and the point of maximum velocity and the density difference between those two points to the product of the distance between the limit of the withdrawal zone and the point of maximum velocity and the density difference between the two points. This correlation was made for several weirs with different coefficients of discharge for both free and submerged weir flow to incorporate the effects of weir type into the velocity profile. This model appeared to be the most applicable model for velocity profiles near the weir.

57. The Prandtl-von Karman velocity deficiency law[23,24] assumes a logarithmic velocity profile with the shape based on the friction factor and the magnitude based on the shear velocity. The model assumes that the velocity profile is generated by the boundary friction for viscous flow. This model would be applicable in the basin away from the weir where a stable interface existed between the density-stratified layers.

58. Prandtl's one-seventh power law[24] states that the ratio of velocity at a point to the maximum velocity is equal to the ratio of the

distance of the point from the solid boundary to the distance of the point of maximum velocity from the solid boundary raised to a power, n (n = 1/7 for turbulent flow). The power, n, is a function of the flow condition. This profile is empirical and applicable only outside the zone of the weir's influence.

59. The Water Resources Engineers[14] finite element program was the only other model that determined velocity profiles. It, however, was not available for use. Furthermore, the approach was not easy to incorporate into a design procedure for modeling only the weir.

PART IV: DATA COLLECTION

60. Because there were insufficient data in the literature to verify the models, data were collected in field trips to three sites. These were on the Yazoo River near Yazoo City, Miss., on the Fowl River south of Mobile, Ala., and at Oyster Bay on the Gulf Intracoastal Waterway in Gulf Shores, Ala. (see Figure 5 for location map).

Purpose

61. There were four goals in the data collection portion of the study. The first was to determine the magnitude of the weir's effect on the effluent quality. In doing this, the effluent suspended solids concentration was measured for various weir flows obtained by adjusting the head over the weir by lowering the weir crest. Increasing the head resulted in increases in the unit flow (flow per unit width of weir), velocities, and depth of the withdrawal zone and, consequently, the effluent suspended solids also increased. The magnitude of the effect was demonstrated at the Fowl River disposal site where the ponding depth was approximately 15 in. With changes in the head over the weir, the effluent suspended solids varied from 3 to 60 g/ℓ . Therefore, it was clear that the weir loading or, similarly, the weir length had a strong influence on the effluent quality.

62. The second purpose of the data collection was to gather representative input data for the various models to be tested. The models required information on the velocity, concentration and density profiles, flow, depth, weir length, head over the weir, velocity of flow over the weir, and grain size, specific gravity, and angle of repose of the sediment material. Much of the information was available in the literature,[1,19,21,23,25,26] except for the concentration and density profiles. Concentration profiles for different dredged material and site conditions were determined for all three sites. Other data, including the flow velocity profile, weir length, depth of withdrawal zone,

Figure 5. Data collection site locations

head over the weir, velocity over the weir, and specific gravity of the dredged material, were also measured.

63. The third purpose of the data collection was to verify the models prior to the final selection of one for use in the design procedure. The needed verification data included measurements of the velocity profile, the effluent suspended solids concentration, the depth of the withdrawal zone, and the input data described previously for a variety of weir conditions. These data were collected at both the Fowl River and Oyster Bay disposal areas.

64. The final purpose of the data collection was to obtain data to be used in the design procedure. The procedure required concentration and density profiles for different types of dredged material and sites. Furthermore, it required classification of the dredged material. Classification was based on the Unified Soil Classification System.[27]

Equipment and Procedures

65. Specialized equipment and procedures were required to collect data on the following parameters: velocity, concentration of suspended solids, density, specific gravity of the sediment particles, particle size distribution, and sediment classification. Field equipment was modified to adapt to sampling and analyzing at dredged material sites. Standard or generally accepted equipment and procedures were used when available.

66. Velocities were measured at Yazoo City and Fowl River with a Marsh-McBirney Model 727 current meter. This probe was not optimal since it could not measure velocity within 9 in. of the surface. At Oyster Bay a Marsh-McBirney Model 711 meter was used that could measure to within 3 in. of the surface and weighed considerably less. These meters, using electromagnetic induction principles, were accurate to $\pm$0.07 fps according to the manufacturer. The velocity probes were mounted on a tripod at the weir (Figure 6) to keep them stationary during readings. Figure 7 shows the probe at the end of the stabilizing pole; Figure 8 shows the velocity readout devices.

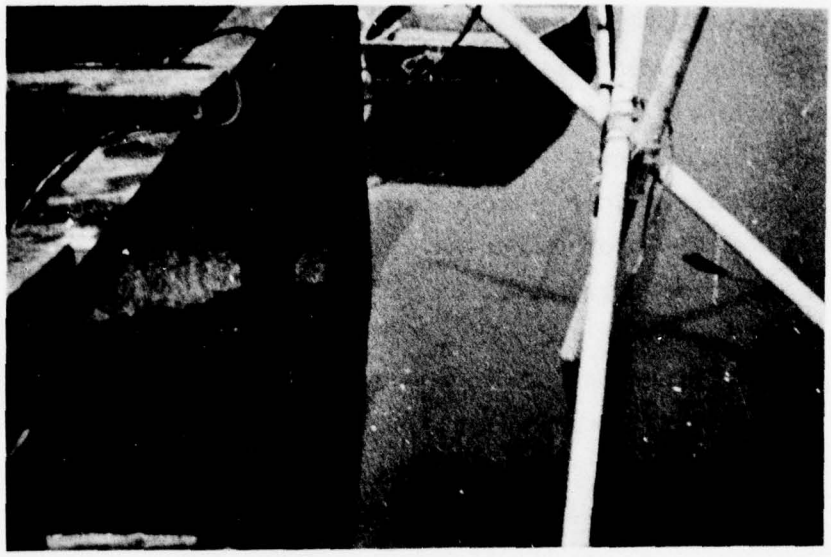


Figure 6. Tripod supporting velocity probe

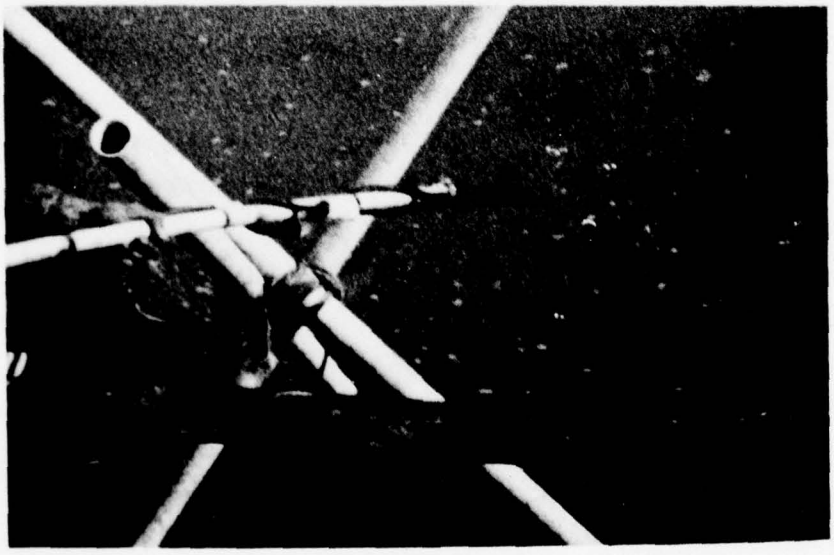


Figure 7. Velocity probe being mounted on tripod

Figure 8. Reading velocity meter

67. Solids concentrations were measured on samples collected by a suspended sediment sampler developed by WES (see Figure 9). The samples

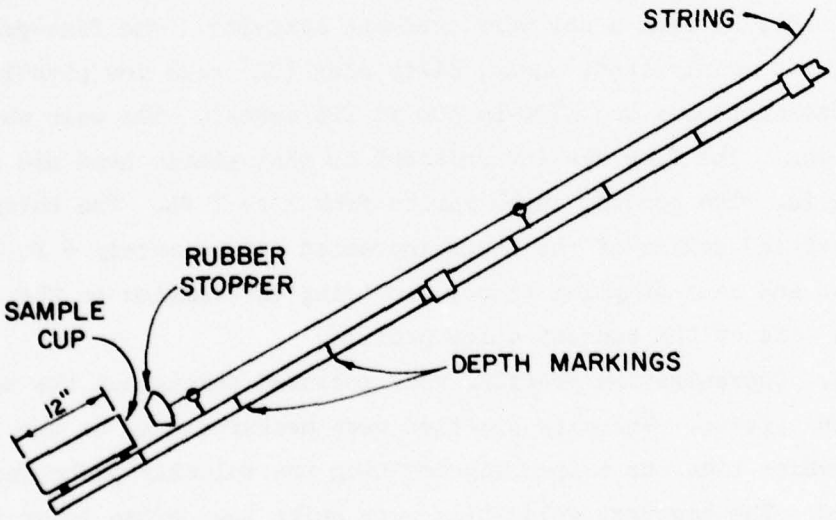


Figure 9. Sediment sampler

were collected by lowering the sampler to the desired depth and opening it. Samples were taken at specific depths starting from the surface to minimize disturbance of the settled material. The solids concentrations

were then measured according to the procedures outlined in EM 1110-2-1906, Laboratory Soils Testing,[28] and Standard Methods.[29] The suspended solids concentration, specific gravity, and dissolved solids concentration were used to determine the density of the sample using the formulas given in Appendix A. For some of the samples, the bulk density was measured directly by weighing a known volume of sample.

68. The sediment properties of the material to be dredged were determined from samples taken from the river bottom with a Peterson sampler. From the sediment samples, the particles' specific gravity, grain size distribution, and sediment classification were determined by the procedures outlined in EM 1110-2-1906.[28]

Field Site Descriptions

Yazoo City

69. Basin 5 at the Yazoo City, Miss., area was visited on four dates, 23 Feb 77, 7 Mar 77, 10 Mar 77, and 16 Mar 77. The Yazoo City site was a new containment area for fine- and coarse-grained freshwater dredged material from a new work dredging activity. The fine-grained material was mainly lean, sandy, silty clay (CL) with low plasticity. The containment area was 1700 by 500 ft (20 acres). The weir was 100 ft long. The flow was intermittent so that static head did not exceed 2 in. The ponding depth varied from 1 to 7 ft. The thickness of the settled solids of the layer increased approximately 6 ft between the first and last sampling trips, providing information on the effect of basin life on the concentration profile.

70. Concentration profiles were measured throughout the basin on all of the visits. Velocity profiles were measured only on the final trip at which time the tripod for mounting the velocity probe had been developed. The measured velocities were quite low, often lower than the accuracy of the meter.

Fowl River

71. The Fowl River dredged material disposal site was visited on 21 and 22 Apr 77. At this time of the year the water was fresh (about

1 ppt of salinity), but during low flow periods the water is saline. The site was being used for disposal of fine-grained material from maintenance dredging. The dredged material was a CH clay with 8 percent organic matter. The basin was small, about 13 acres, and irregular in shape. The weir was approximately 10 ft, and the ponding depth was 15 in.

72. The trip to Fowl River produced several pieces of information. First, the weir elevation was varied to determine the effect of the weir on the effluent quality. Second, the velocity profiles and the depth of the withdrawal zone were measured at the weir to verify model results. Third, concentration profiles were measured. Finally, sediment properties were evaluated. The data were useful for problem evaluation, model input, model evaluation, and design data.

Oyster Bay

73. The Oyster Bay dredged material containment area was visited on 26 Jun 77. Maintenance dredging was being performed at the site. The water was brackish, about 7.5 ppt of salinity, which promoted flocculation of the dredged material in the basin. The dredged material was a fat clay (CH) with high plasticity. The basin was about 15 acres and rectangular, about 1100 by 600 ft (see Figure 10). In the center of the basin, there was a large stand of pine trees about 900 by 400 ft which produced short-circuiting around the vegetation and along the dike from the inlet pipe to the weir. The basin had a ponding depth of 12 to 15 in. The weir was rectangular, 20 ft long, with three sections of 2- by 10-in. boards.

74. The trip provided the data required for model verification and design data for saltwater dredged material. The concentration and velocity profiles at the weir, the depth of the withdrawal zone, and the flow velocity and head over the weir were measured for a variety of flows and density stratifications for model verification. Concentration profiles and dredged material samples were taken throughout the basin for design data. The sediment properties and salinity were measured. Finally, operational guideline information on weir operation, short-circuiting, and wind effects was gathered.

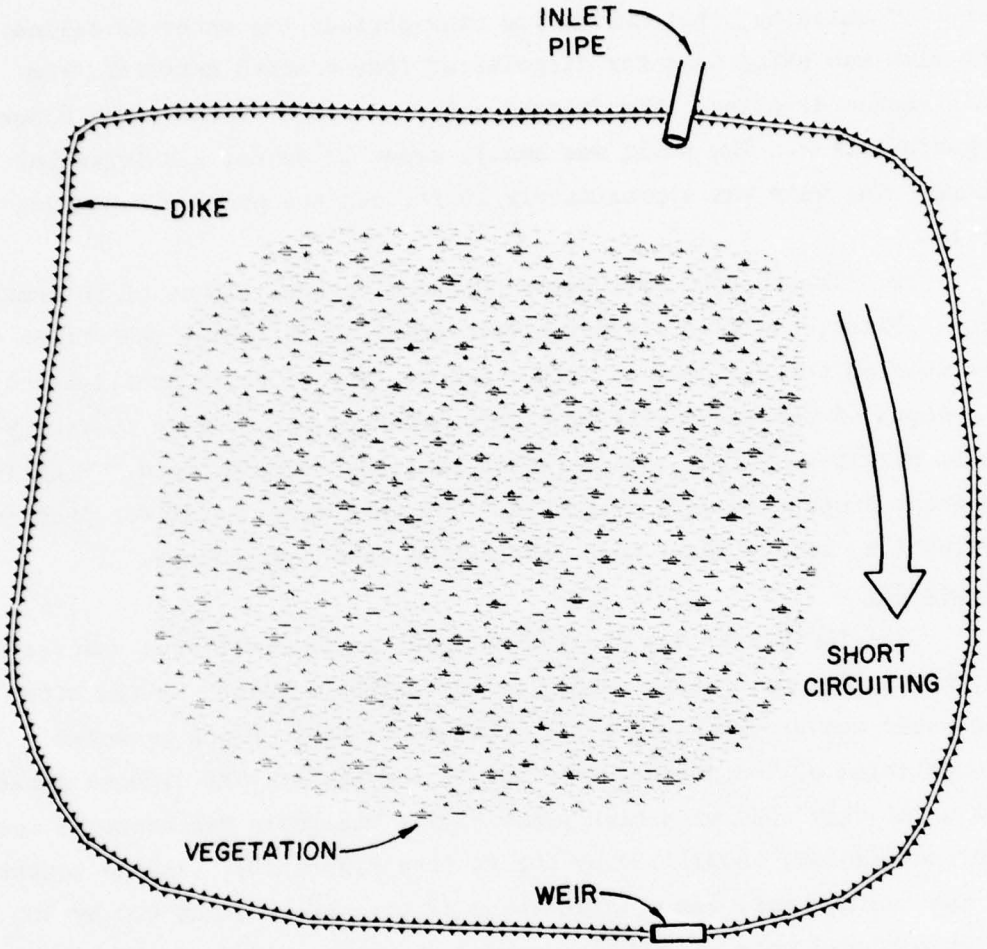


Figure 10. Oyster Bay containment area

Field Data

75. Field data were collected to determine the in situ concentration and density profiles, the velocity profile at the weir, the influent and effluent concentrations, and the soil properties of the dredged material.

Concentration profiles

76. The concentration profiles varied from site to site due to the differences in the dredged material. Typical profiles for the three sites are presented in Figure 11. The profiles are used to

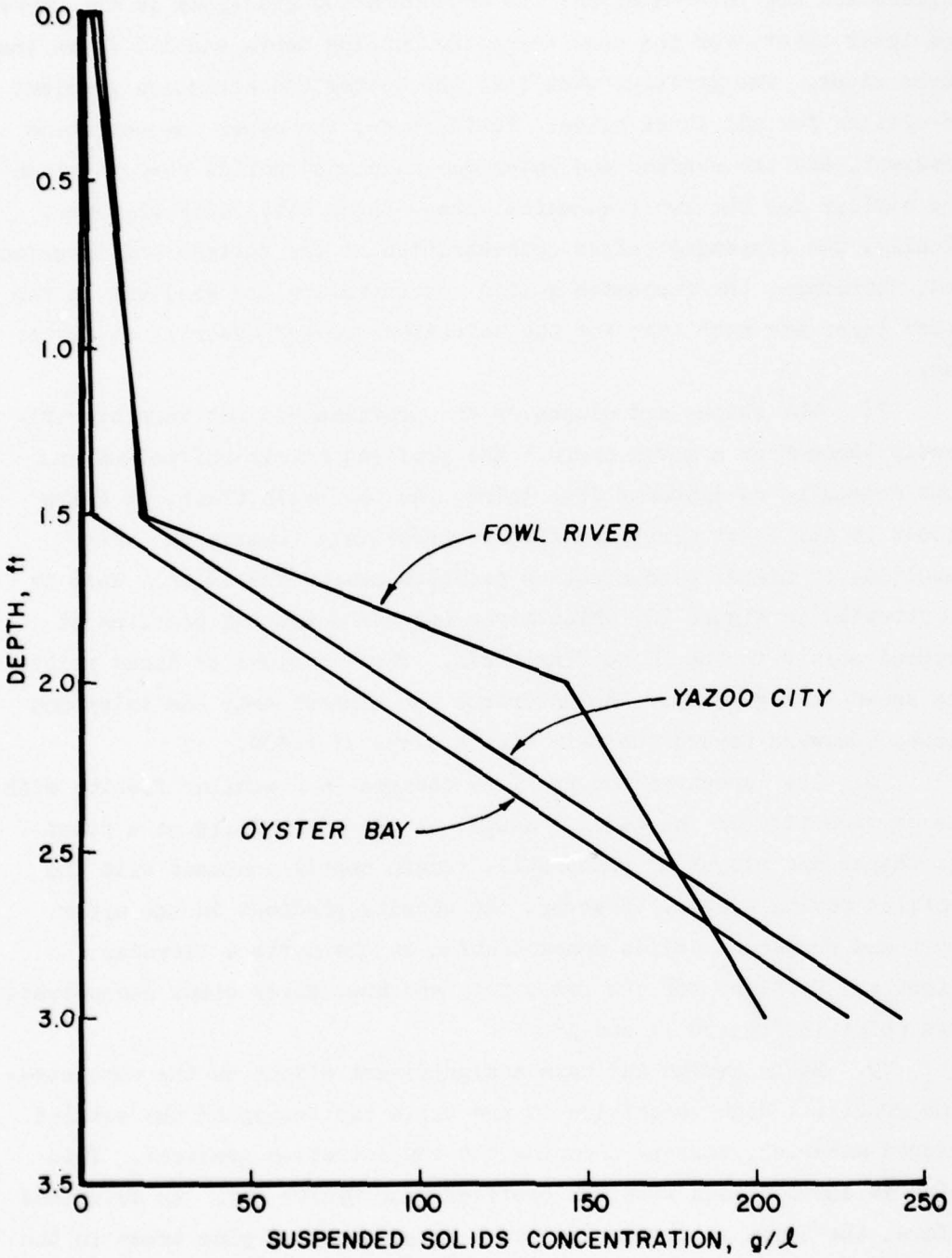


Figure 11. Typical suspended solids concentration profiles for 1.5 ft of ponding depth

demonstrate the differences in the suspended solids concentration at the surface and the interface, and the concentration gradients in the upper and lower layers for the case where the ponding depth was 1.5 ft at the three sites. The profiles show that the bottom concentration gradient is similar for all three sites. Furthermore, the upper concentration gradient, and the surface and interface suspended solids concentration are similar for the two freshwater sites--Yazoo City and Fowl River. Finally, the suspended solids concentration at the surface and interface and, therefore, the suspended solids concentration and gradient in the upper layer are much less for the saltwater dredged material at Oyster Bay.

77. The shapes and slopes of the profiles did not vary significantly throughout a given basin. The profiles merely shifted up and down depending on distance from inlet. As the basin fills, it fills closer to the inlet first and then progressively towards the weir, resulting in higher concentration profiles nearer the inlet. This is illustrated in Figure 12, which gives the concentration profiles at several points in the Yazoo City basin. The locations of these points are shown in Figure 13. The interface was highest near the inlet and sloped downward toward the weir with a slope of 1:500.

78. The concentration profiles changed in a similar fashion with the service life of the basin. Again, as the basin fills at a point, the shapes and slopes of the profile remain nearly constant with the profiles moving higher. However, the density gradient in the upper layer and suspended solids concentration at the surface increase slightly. Profiles for the Yazoo City and Fowl River sites demonstrate this point in Figures 14 and 15.

79. Basin design can have a significant effect on the concentration profile. High velocities in the basin can resuspend the settled dredged material, thereby changing the concentration gradient. This point is demonstrated with the profiles from Oyster Bay. As described before, the basin at Oyster Bay had a large stand of pine trees in the center of the basin. This caused short-circuiting with high velocities that prevented settling of the dredged material, changing the profiles

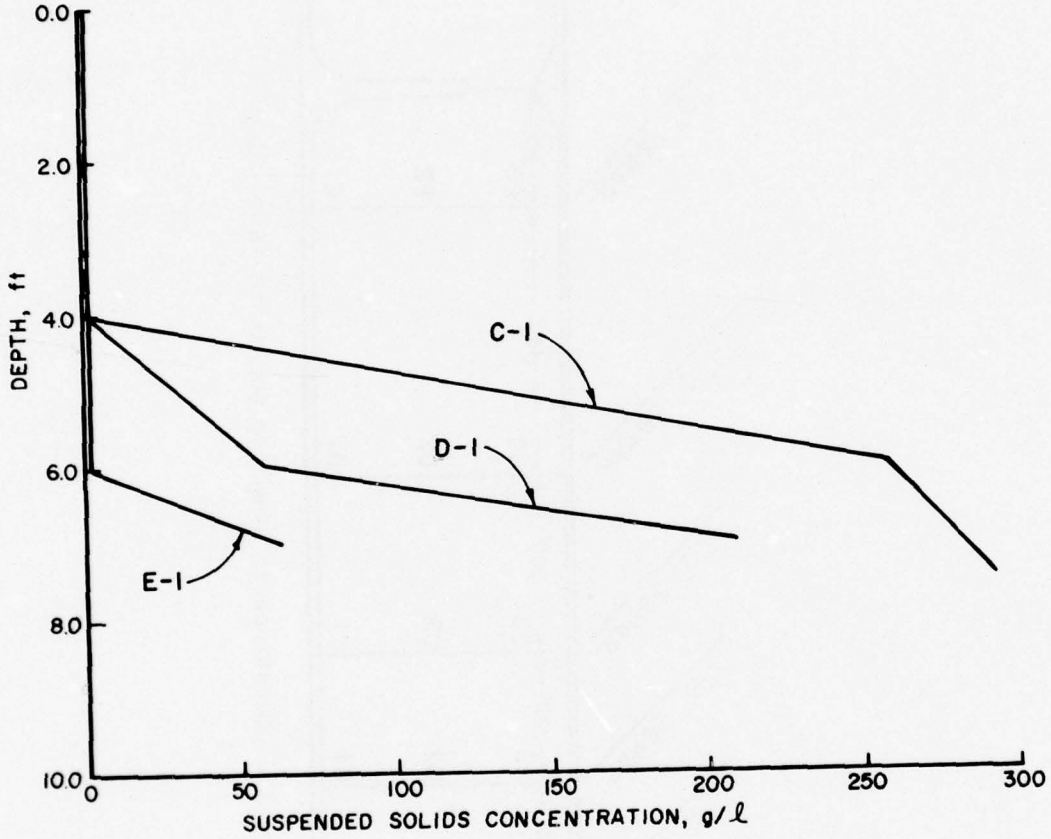


Figure 12. Effect of location on suspended solids profiles from Yazoo City on 23 Feb 77[30]

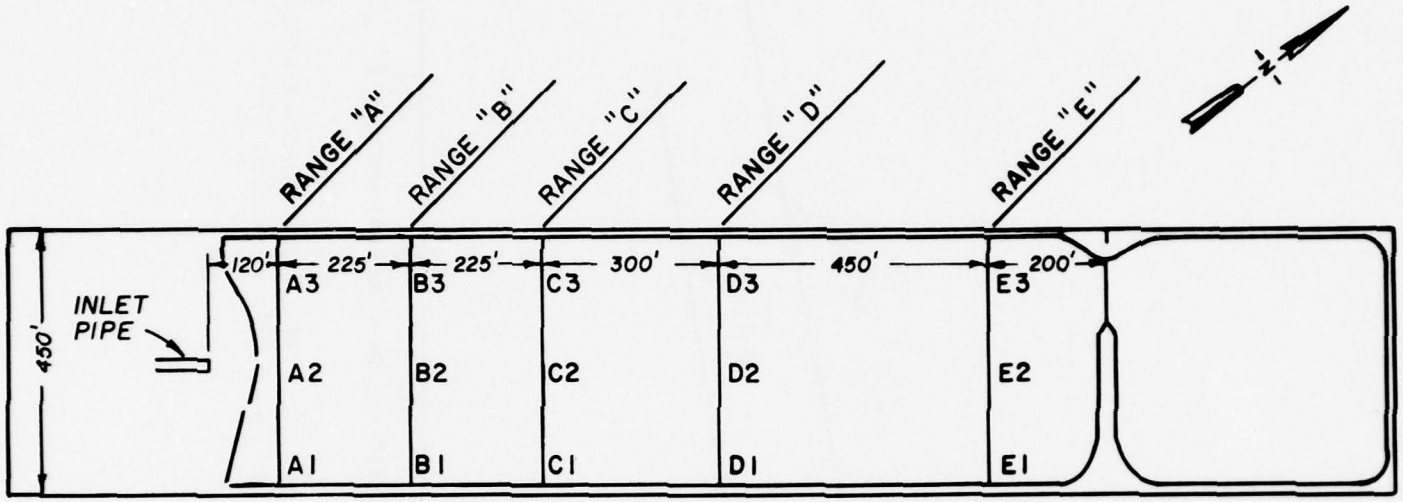


Figure 13. Locations of sampling points at Yazoo City[30]

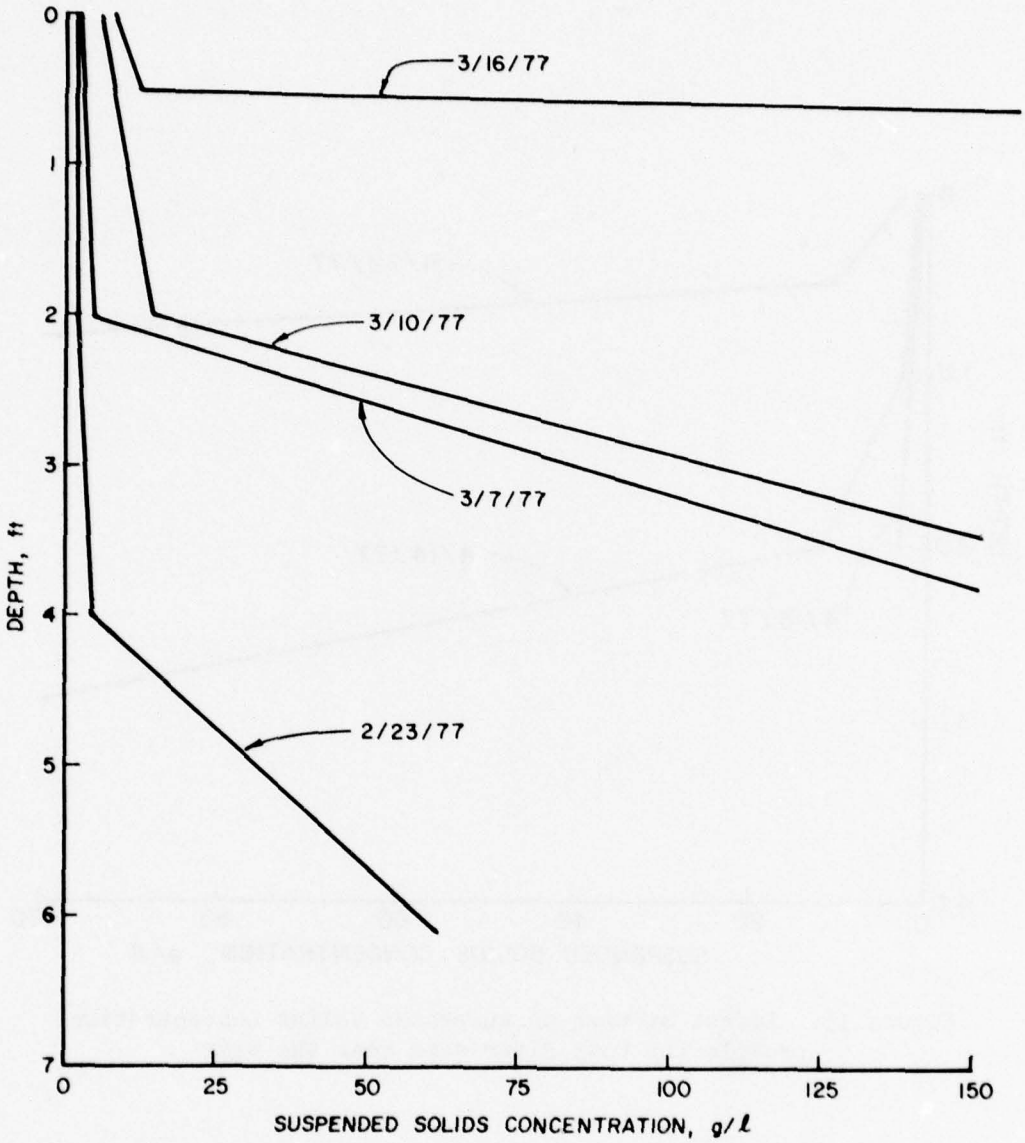


Figure 14. Effect of time on suspended solids concentration profiles for Yazoo City site at D-330

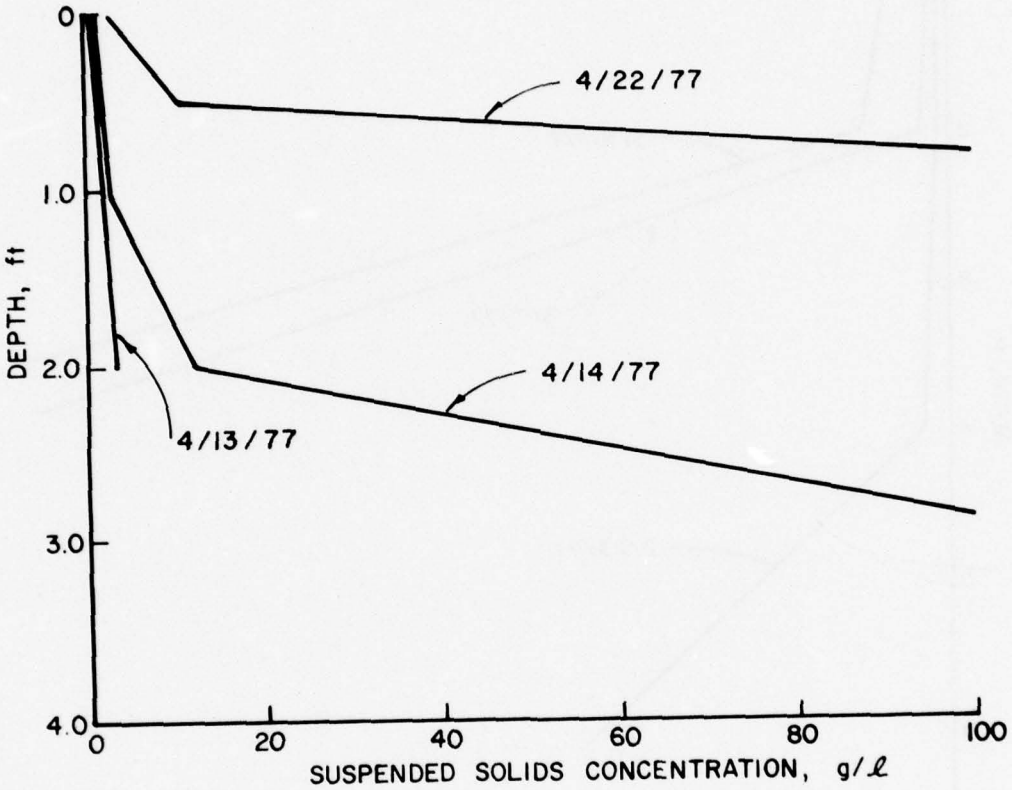


Figure 15. Effect of time on suspended solids concentration profile for Fowl River site near the weir

as shown in Figure 16. The profiles were measured with the dredge both operating and not operating. Profile A was measured after a shutdown

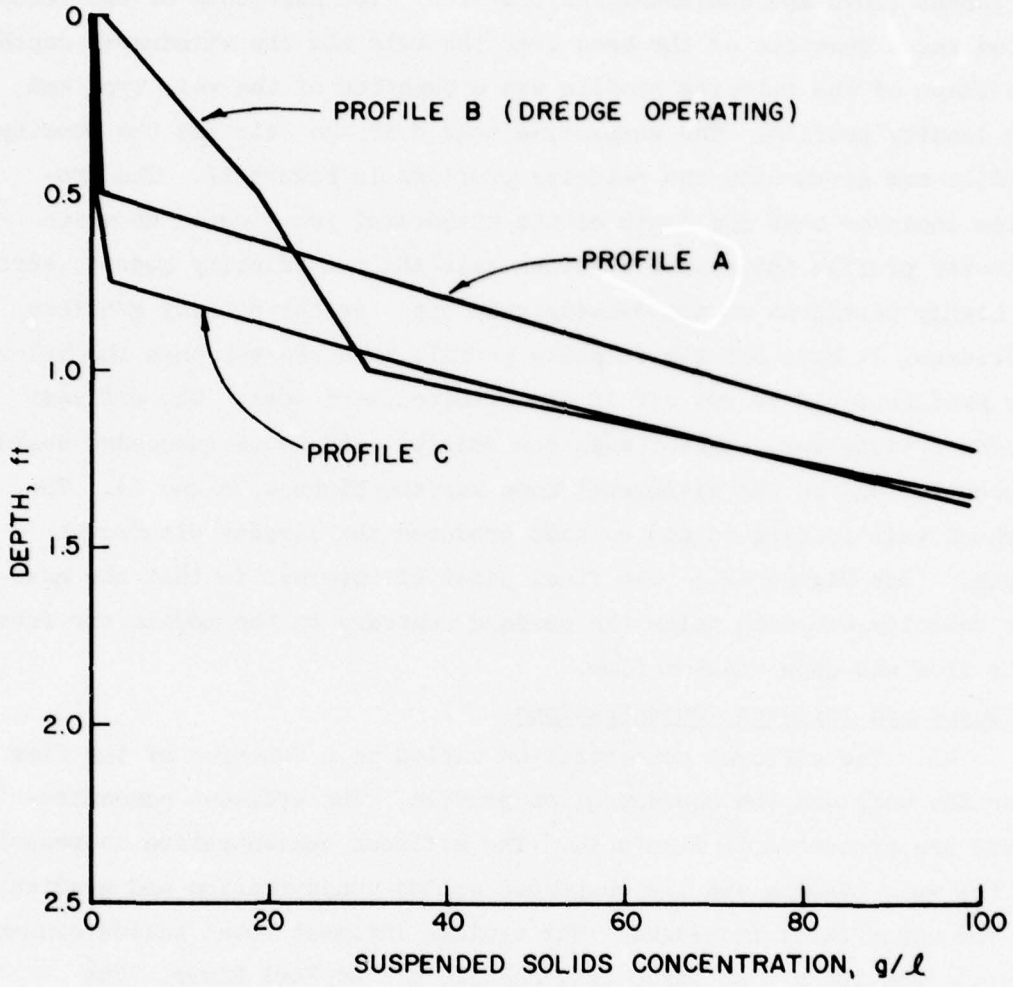


Figure 16. Effect of short-circuiting on suspended solids concentration profiles for Oyster Bay site near the weir

of several hours. Profile B was measured about 1 hr after the dredge started. Profile C was measured about 2 hr after the dredge stopped. Note the high suspended concentration in the ponded water when the dredge was operating. The profiles show that a poorly designed basin can fail to provide adequate detention time and thus override any efforts to control the effluent quality by the weir.

Velocity profiles

80. The velocity profile at the weir was measured for several different flows and concentration profiles. The magnitude of the velocities was a function of the head over the weir and the withdrawal depth. The shape of the velocity profile was a function of the weir type and the density profile. The respective head over the weir and the density profile are given with the velocity profiles in Figure 17. The profiles indicate that the depth of the withdrawal zone (depth at which velocity profile intersects vertical axis and the velocity goes to zero) is highly dependent on the density gradient. As the density gradient increases, it cuts off the velocity profile much sharper than the velocity profile would be cut off if the gradient were weak. The effluent concentrations were highest when the density (and hence suspended solids concentration) in the withdrawal zone was the highest (b and f). The highest weir loading (d and e) also produced the largest withdrawal depth. (See Figure 17.) The final point of interest is that the maximum velocity occurred below the surface contrary to the models for free weir flow and open channel flow.

Effluent and influent concentrations

81. The effluent concentration varied as a function of the flow over the weir and the concentration profile. The effluent concentrations are presented in Figure 17. The effluent concentration increased as the weir loading and the suspended solids concentration and gradient in the upper layer increased. The typical influent total solids concentration was 100 g/ℓ at Yazoo City and 120 g/ℓ at Fowl River. The influent concentration was not measured at Oyster Bay.

Sediment properties

82. The dredged materials from the three sites were analyzed to determine their physical properties. The plastic limit and liquid limit were measured to calculate the plasticity index and to classify the material under the Unified Soil Classification System. The specific gravity and salinity were measured to determine the nature of the settling and to correlate the solids concentration with density. A summary of the data for each site is presented in Table 1. These

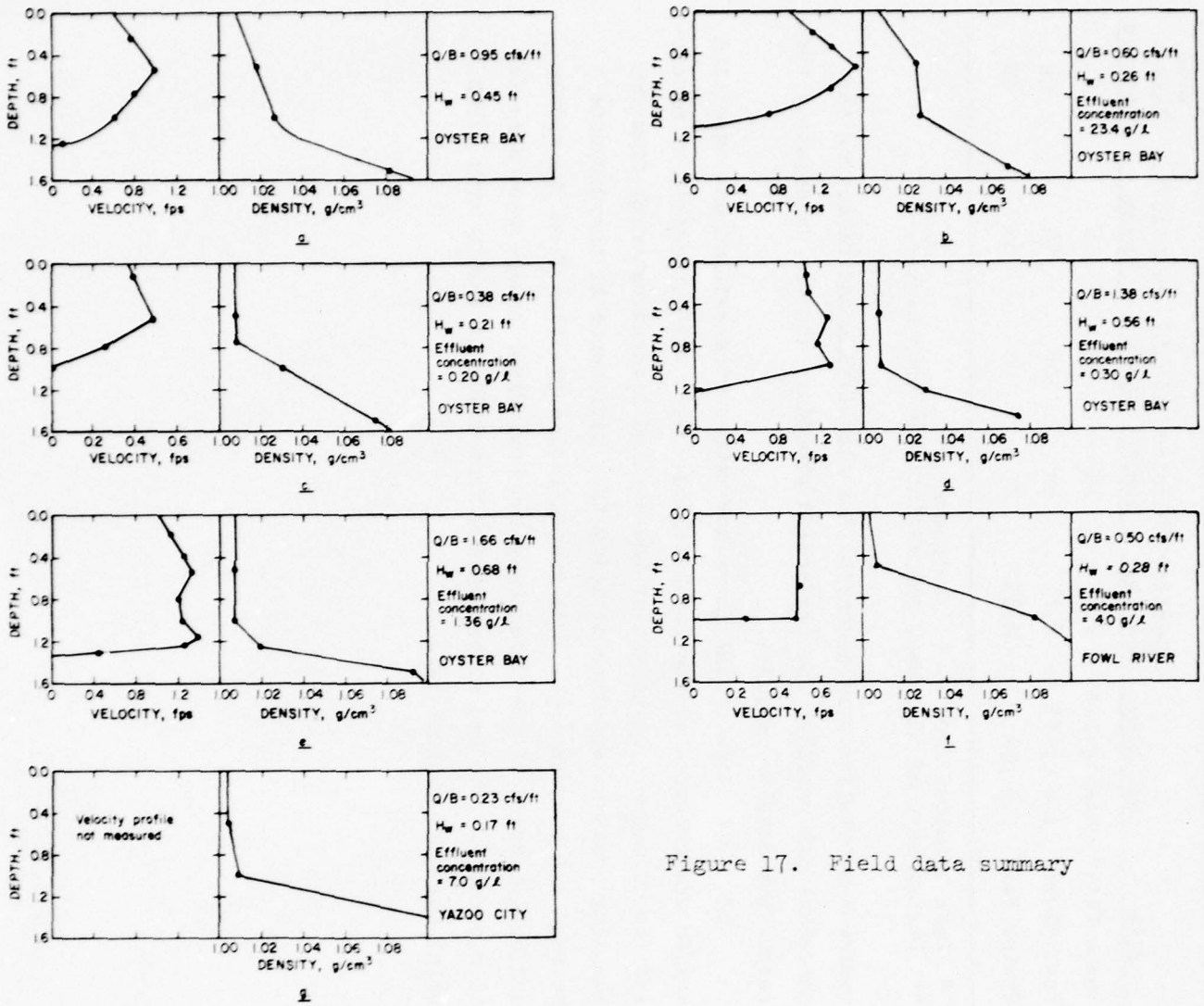


Figure 17. Field data summary

Table 1

Sediment Properties at Test Sites

Site	Salinity	Specific Gravity	Plastic Limit	Liquid Limit	Plasticity Index	Soil Classification
Yazoo City*	Negligible	2.67	18	33	15	CL
Fowl River	1.0 ppt**	2.71	30	104	74	CH
Oyster Bay	7.5 ppt	2.63	27	90	63	CH

* Data obtained from R. L. Montgomery.[30]
** During much of the year, the salinity is much higher.

sediment properties were essential for classifying and selecting the concentration profiles for the design procedure and for calculating the respective density profiles.

83. The slope of the interface between the settled and unsettled solids was measured as the basin filled. This was determined by measuring the difference in height of the settled dredged material from two sampling stations and then dividing this difference by the distance between the two stations. The concentration profiles from the middle and outlet areas of the Yazoo City and Fowl River basins were used for this purpose. The bed slopes varied from 1:200 to 1:1000 with 1:500 being the most typical value.

PART V: MODEL SELECTION AND VERIFICATION

Criteria for Selection

84. The model selection was based on three criteria. First, the model should be accurate for the range of flows and density profiles observed in containment areas. Second, the model should have a sound theoretical or experimental basis. Lastly, the model should be easy to develop into a simplified design procedure. That is, neither the model nor the design procedure should require large amounts of laboratory analysis, flume studies, or computer simulation. The design procedure should require only those laboratory analyses needed for classifying the dredged material. Finally, the design procedure should provide easy evaluation of design alternatives.

85. The applicable models discussed in the literature review are compared for accuracy in the following section. From these, the WES selective withdrawal model was chosen to predict the withdrawal depth and the velocity profile, and thereby the effluent suspended solids concentration. The model was then verified for use in the design procedure. The model's basis and ease of application are then discussed. The theory of the other models is presented in Appendix B.

Model Selection

Withdrawal depth models

86. The five models selected for comparison from the literature review are compared with each other in Figures 18, 19, 20, and 21. The graphs are intended to show the relative withdrawal depths predicted by each model, the effects of different density profiles, and the trend which exists between the weir loading rate (Q/B) and the withdrawal depth.

87. Most of the models assume that the withdrawal depth equals the ponding depth; therefore, the graphs presented in Figures 18, 19, 20, and 21 assume this case for comparison purposes. This approach

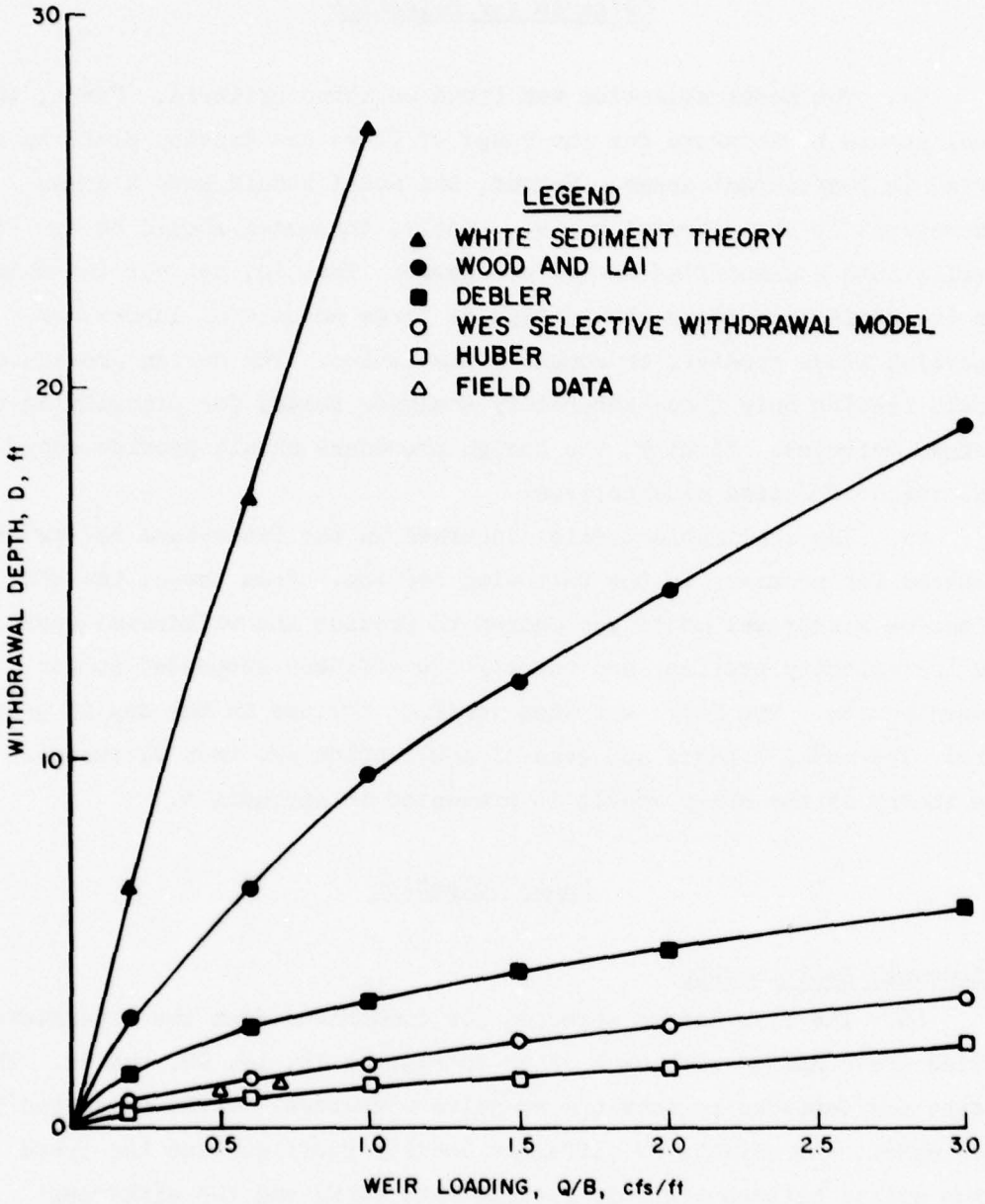


Figure 18. Comparison of withdrawal depth models for Fowl River dredged material

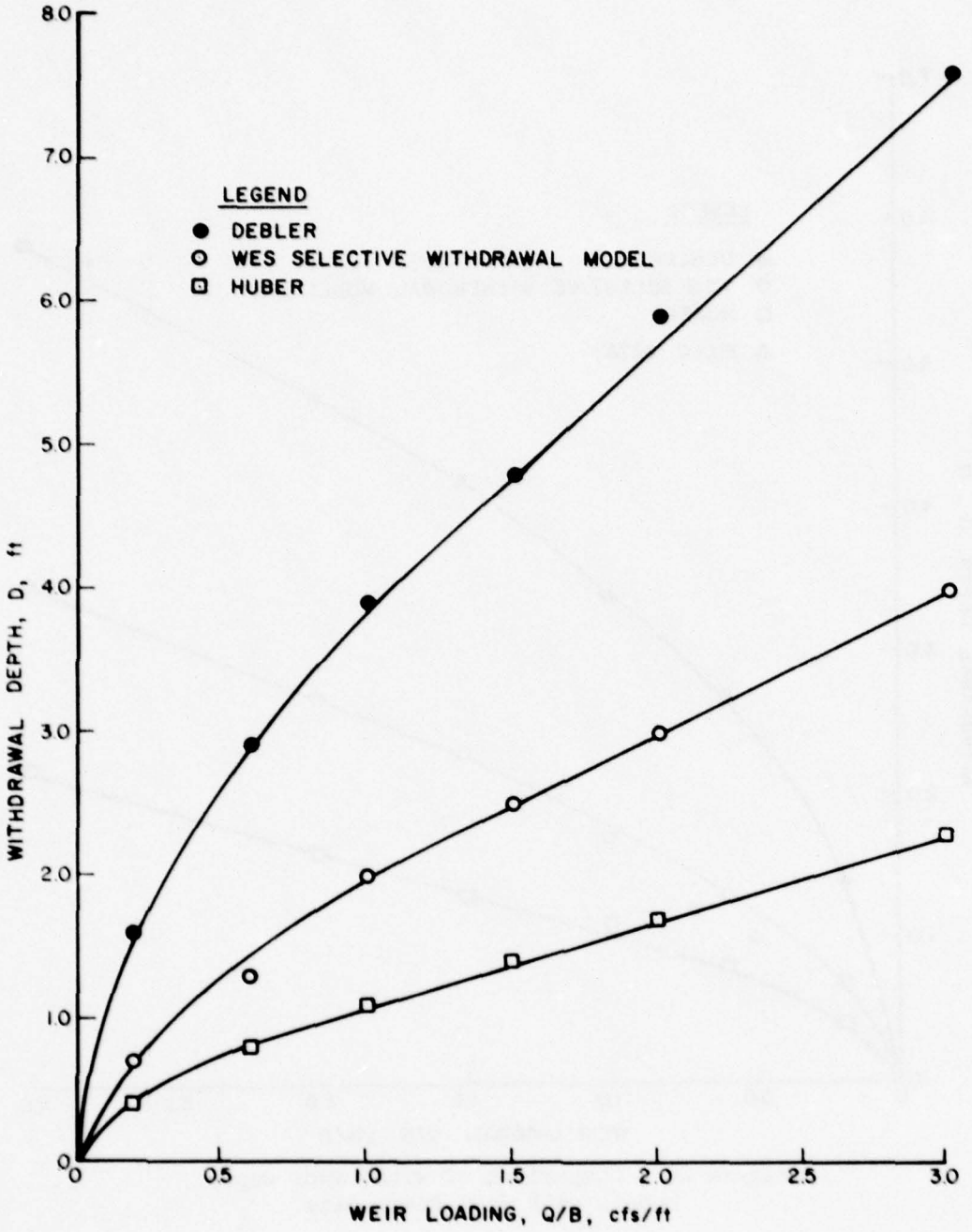


Figure 19. Comparison of withdrawal depth models with Yazoo City data

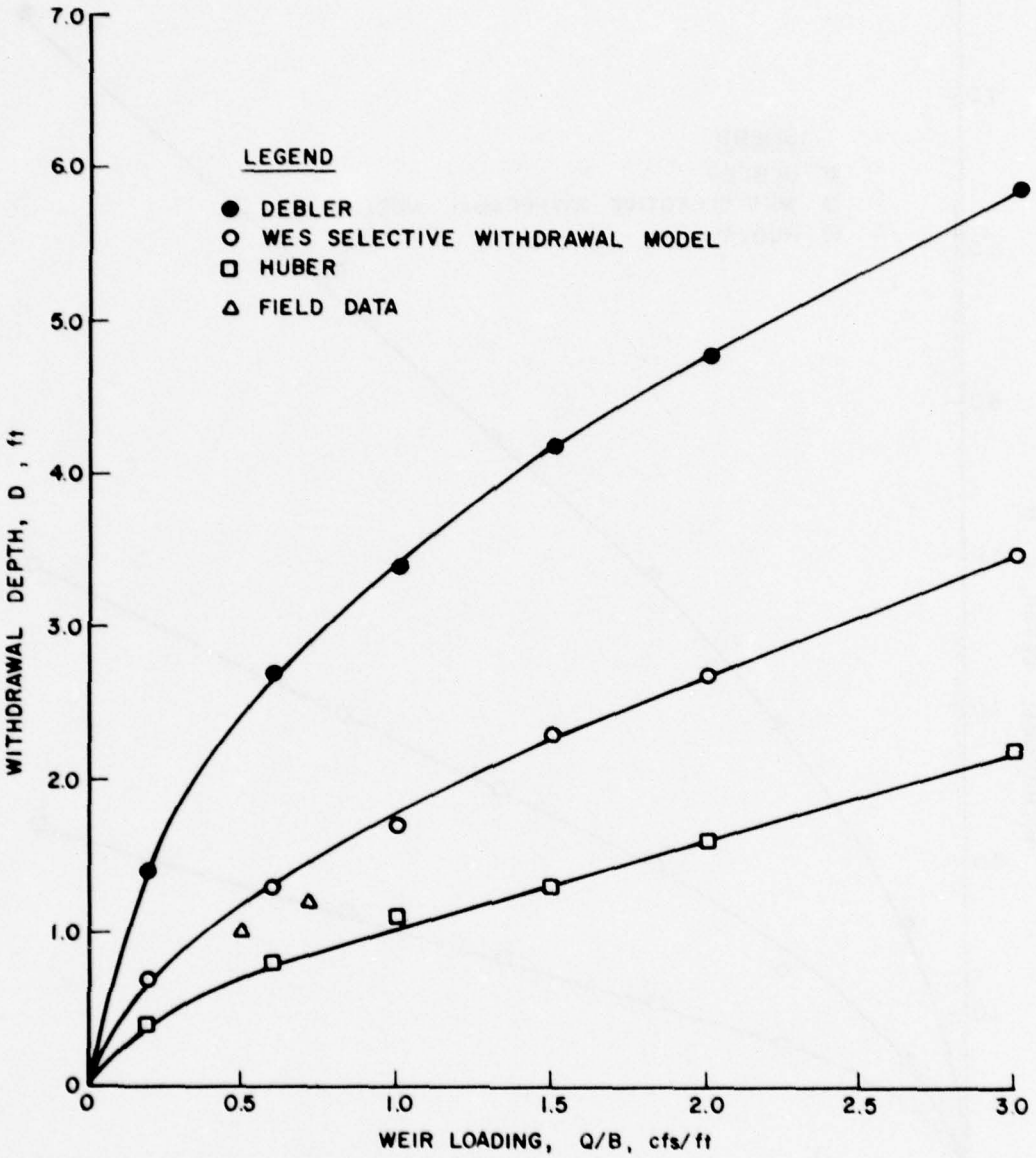


Figure 20. Comparison of withdrawal depth models with Fowl River data

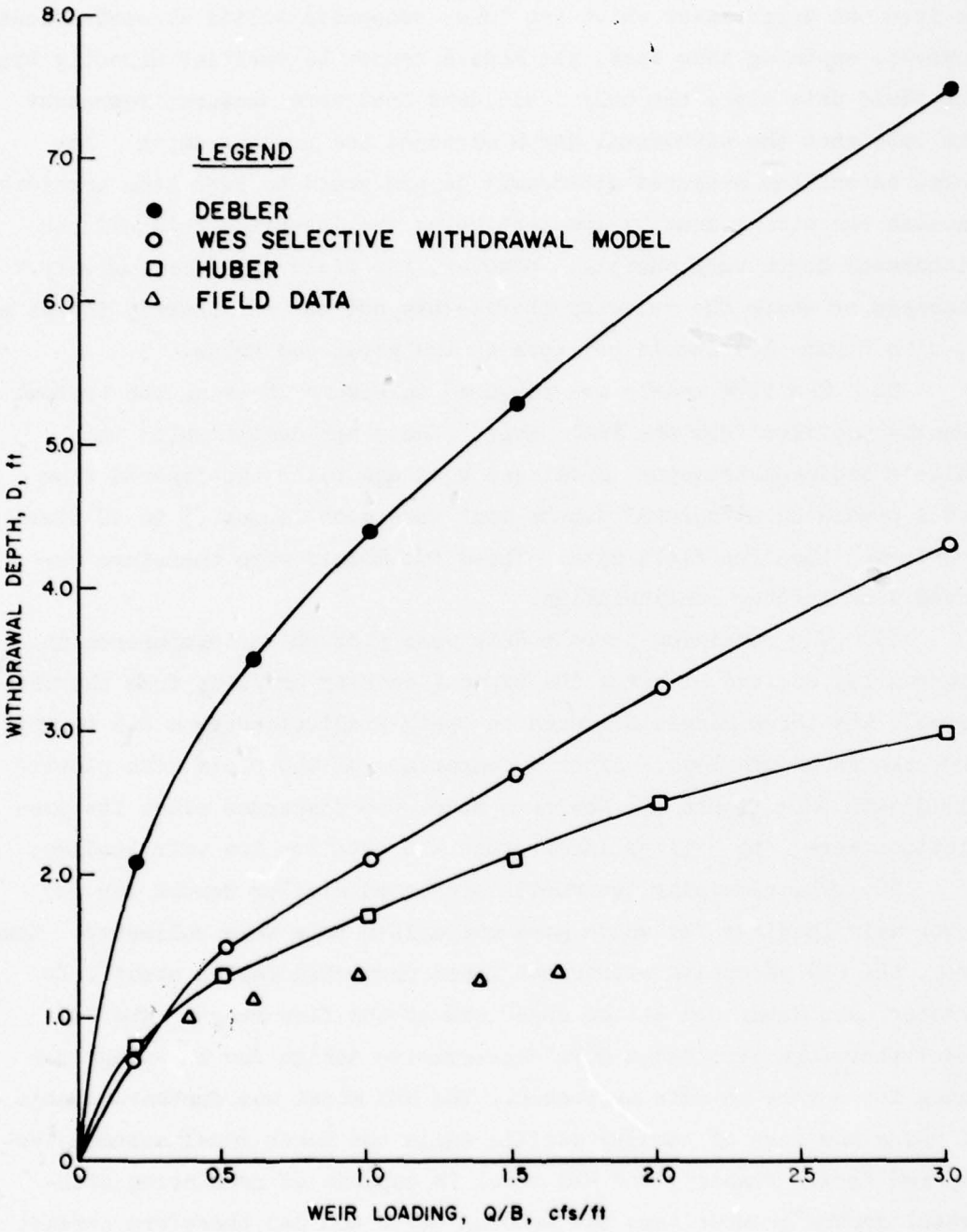


Figure 21. Comparison of withdrawal depth models with Oyster Bay data

would be acceptable for design purposes since all of the discharge would be from the upper layer which has lower suspended solids concentrations. However, assuming this case, the models cannot be verified directly with the field data since the only field data that were measured represent the case when the withdrawal depth exceeded the ponding depth. For these cases, the measured withdrawal depths would be less than predicted because the strong density gradient below the interface cuts off the withdrawal depth very sharply. However, the field data for low weir loadings or where the velocity profile was not cut off sharply (cases a, b, c in Figure 17) should be close to the predicted value.

88. The five models are compared in Figure 18 using the typical density profiles from the Fowl River. The graph demonstrates that White's sediment-transport model and Wood and Lai's two-layered flow model predicted withdrawal depths that were much larger (5 to 10 times too large) than the field data. These two models were therefore removed from further consideration.

89. The remaining three models were plotted for comparison in Figures 19, 20, and 21 using the typical density profiles from the three sites. The three models differed in their predictions from 0.5 to 3 ft over the range of flows. After a comparison of the plots with plotted field data from Figure 17, Debler's model was discarded since its predictions were 2 to 3 times larger than the data for low weir loadings.

90. The remaining two models predicted similar depths for the lower weir loadings for which good comparison data were collected. However, the WES selective withdrawal model predicted depths about 1 ft greater than Huber did at the upper end of the flow range. The WES model therefore provided a more conservative design for the range for which there were no data available. The WES model was further capable of using any form of density profile while the Huber model assumed two-layered flow. Finally, the WES model is capable of predicting withdrawal depths greater than the ponding depth and can therefore predict the effluent suspended solids concentration for any flow situation. Consequently, it is very flexible and can be verified with any field data, not just data for when the withdrawal depth equals the ponding

depth as is the case with Huber's model. For these reasons, the WES selective withdrawal model was selected for verification.

Velocity profile

91. The velocity profiles are presented in their dimensionless form in this section so that they can be compared. The dimensionless depth is the ratio of the depth of the point where the velocity, v , was measured or calculated (y) to the withdrawal depth (D). The dimensionless velocity is the ratio of the measured or calculated velocity (v) at the depth, y, to the maximum velocity (V_{max}). The von Karman-Prandtl velocity deficiency law and the Prandtl one-seventh power law are plotted with the dimensionless field data points in Figure 22. The plot shows that neither profile fits the data very well. The data show that the point of maximum velocity is located below the

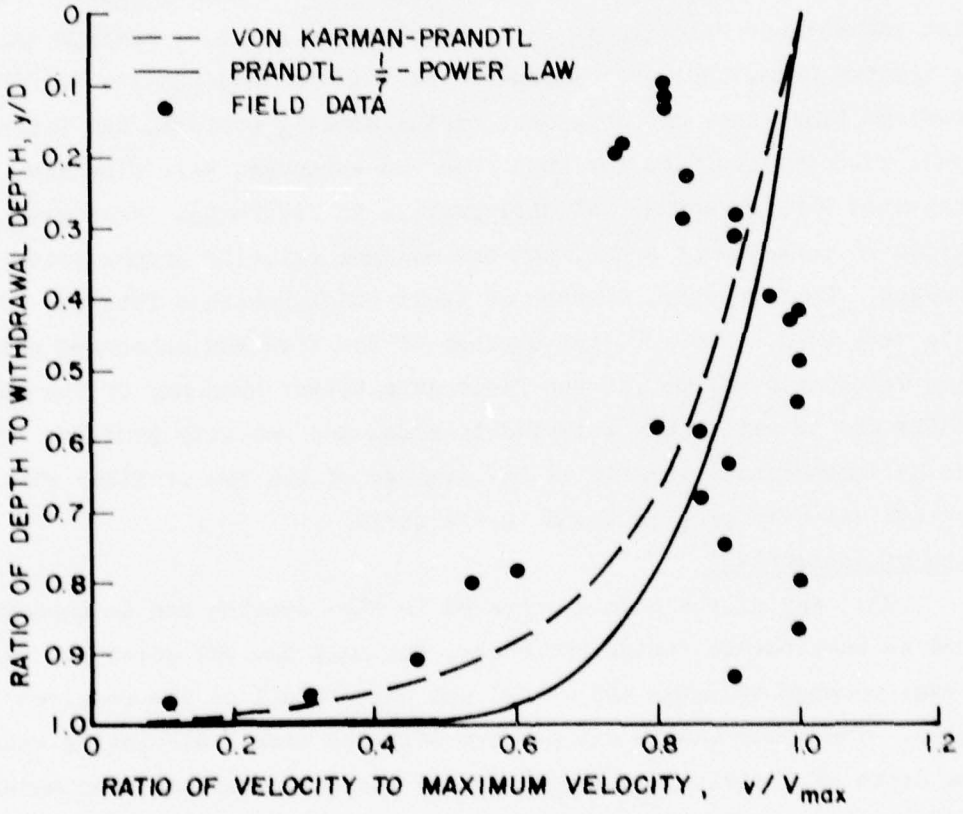


Figure 22. Dimensionless comparison of velocity profile models with field data

water surface. These models assume that it occurs at the water surface. Furthermore, the scatter in the data shows that the velocity profile is a function of the density profile and not only the depth as assumed in these two velocity profiles.

92. The WES selective withdrawal model free and submerged weir flow velocity profiles were the only other models investigated. Free weir flow exists when the water falls freely from the weir thereby not restricting the flow over the weir. Submerged weir flow exists when standing water downstream of the weir restricts the free flow over the weir. In other words, submerged weir flow occurs when a condition downstream from the weir controls the flow over the weir. These are the only profiles that incorporate the effects of both depth and density. The free weir flow velocity profile assumes that the maximum velocity occurs at the surface. The submerged weir flow velocity profile assumes that the surface velocity is zero and that the point of maximum velocity is located below the water surface. No uniform shape exists for the profiles since they are dependent on the density profile, but typical model velocity profiles for both free and submerged weir flow are presented with an actual velocity profile in Figure 23. Free weir flow exists in containment areas, but the maximum velocity occurs below the surface. Consequently, neither of these model profiles fits the actual data very well. However, the average of the free and submerged weir flow velocity profiles fit the field data better than any of the other models and therefore was selected to model the velocity profile. Figure 23 demonstrates the fit of the average of the two profiles with a typical velocity profile found in the field.

Ease of application

93. Any of the models compared in this section can be employed to form an easy-to-use design procedure, but only the WES selective withdrawal program by Bohan and Grace[5] can perform all of the required tasks. The other models can perform only one task, calculating either the depth of the withdrawal zone or the velocity profile. Two models would have to be combined with the concentration profile in a support program, which would then integrate the velocity and concentration

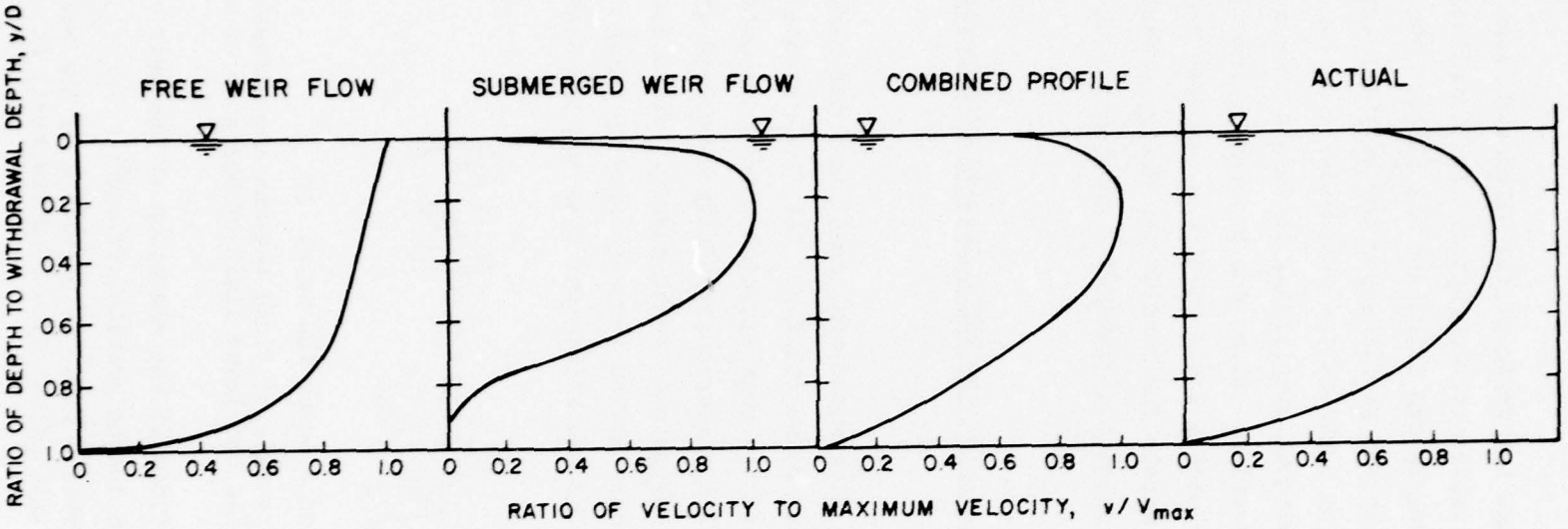


Figure 23. Dimensionless comparison of typical WES selective withdrawal velocity profiles

profiles together through the depth of the withdrawal zone to determine the effluent suspended solids concentration. As stated before, the WES selective withdrawal program can calculate all three: the depth of the withdrawal zone, the velocity profile, and the effluent concentration. Furthermore, this program contains the only model that can use the field density data directly without simplifying it into a two-layered or a linear density stratification. Also, the program is readily available for use while the other models would require a computer program to be written to support the models for forming the design procedure. For these reasons, the WES selective withdrawal program was used for developing a design procedure.

Equations and Theory of the WES Selective Withdrawal Model

Withdrawal depth

94. The WES selective withdrawal model is a one-dimensional model developed from laboratory flume studies. The flume studies were conducted for the case where the weir extended across the entire width of the flume. The depth of a dimensionless fully developed withdrawal zone was correlated with a densimetric Froude number. The following equation was developed from the correlation for weir flow by using dimensionless variables for the depth of the withdrawal zone and the density profile.[5]

$$\frac{V_w}{\sqrt{\frac{\Delta\rho_w}{\rho_w}(gZ_o)}} = 0.60\left(\frac{Z_o + H_w}{H_w}\right) \tag{3}$$

V_w = average velocity over the weir, fps

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

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

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

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

H_w = head on the weir for free flow or depth of flow over the weir for submerged flow, ft

The equation must be solved iteratively for Z_o. The model placed no restriction on the nature of the density profile but required that the density be specified at several depths. The model is empirical but takes into account most of the important variables. Furthermore, the model used a sound experimental base and has been verified for flow in reservoirs that were density-stratified by temperature.

95. During the verification phase, the coefficient, 0.60, in the above equation was ajusted from 0.32 as proposed in the model by Bohan and Grace[5] in order to account for the change in the viscosity in the lower layers due to the suspended solids concentration and to better fit the field data. Migniot[19] determined that the viscosity is proportional to concentration of suspended solids raised to the fourth power. Debler,[8] Kao,[9] and Koh[11] reported that an increase in the viscosity in the lower layers would decrease the depth of withdrawal in the upper layers. Increasing the coefficient to 0.60 accounted for this decrease in the withdrawal depth.

Velocity profile

96. The WES selective withdrawal model also predicts the velocity distribution for both free and submerged weir flow. The equations are empirical, based on laboratory flume studies.[5] The equations account for both weir type and density stratification. The equations for free weir flow are of the following form:

$$\frac{v_1}{V} = 1 - \left(\frac{y_1 \Delta\rho_1}{Y_1 \Delta\rho_{1m}}\right)^n \qquad (4)$$

where

v_1 = local velocity in the zone of withdrawal at a distance y_1 below the elevation of the maximum velocity V, fps

V = maximum velocity in the zone of withdrawal, fps

y_1 = vertical distance from the elevation of the maximum velocity V to that of the corresponding local velocity v_1, ft

$\Delta\rho_1$ = density difference of fluid between the elevations of the maximum velocity V and the corresponding local velocity v_1, g/cm^3

Y_1 = vertical distance from the elevation of the maximum velocity V to the lower limit of the zone of withdrawal, ft

$\Delta\rho_{1m}$ = density difference of fluid between the elevations of the maximum velocity V and the lower limit of the zone of withdrawal, g/cm^3

n = empirical coefficient that varies with coefficient of discharge (n = 1/2 for sharp-crested weirs)

The maximum velocity is located at the free surface for free weir flow. The submerged weir flow velocity distribution is broken into two parts, above and below the point of maximum velocity. The profile above the point of maximum velocity takes the following form:

$$\frac{v_1}{V} = \left(1 - \frac{y_1 \Delta\rho_1}{Y_1 \Delta\rho_{1m}}\right)^3 \tag{5}$$

The profile below the point of maximum velocity assumes the following form:

$$\frac{v_2}{V} = 1 - \left(\frac{y_2 \Delta\rho_2}{Y_2 \Delta\rho_{2m}}\right)^2 \tag{6}$$

where

v_2 = local velocity in the zone of withdrawal at a distance y_2 above the elevation of the maximum velocity V, fps

y_2 = vertical distance from the elevation of the maximum velocity V to that of the corresponding local velocity v_2, ft

$\Delta\rho_2$ = density difference of fluid between the elevations of the maximum velocity V and the corresponding local velocity v_2, g/cm^3

Y_2 = vertical distance from the elevation of the maximum velocity V to the upper limit of the zone of withdrawal, ft

$\Delta\rho_{2m}$ = density difference of fluid between the elevations of the maximum velocity V and the upper limit of the zone of withdrawal, g/cm^3

The point of maximum velocity is determined by the following equation:

$$\frac{Y_1}{H_w + Z_o} = \left[\sin\left(\frac{1.57Z_o}{H_w + Z_o}\right)\right]^2 \qquad (7)$$

where

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

H_w = head on the weir for free flow or the depth of flow over the weir for submerged flow, ft

97. The equations were developed experimentally to include the effects of the major variables--weir type and density stratification. The experiments were conducted with a fluid that was density-stratified by salinity.

Effluent suspended solids concentration

98. The effluent suspended solids concentration is predicted by numerically integrating the product of the velocity and concentration profiles across the withdrawal depth.

$$SS = \frac{\int_o^D c(y)v(y)\,dy}{Q/B} \qquad (8)$$

where

SS = effluent suspended solids concentration, g/ℓ

D = withdrawal depth, $H_w + Z_o$, ft

c(y) = suspended solids concentration profile, g/ℓ

v(y) = velocity profile, fps

y = depth from surface, ft

Q/B = weir loading, cfs/ft

Model Verification

99. The WES model was verified with the field data presented in Figure 17. The model was verified in two parts. First, the predicted

depth of withdrawal was compared with the actual depth of withdrawal. The actual depth of withdrawal was determined from the velocity profile. It is the depth where the velocity went to zero. Second, the predicted effluent suspended solids concentration was compared with the actual concentration to verify the velocity profile.

100. The results obtained with the WES model using the adjusted coefficient are presented with the actual data in Table 2. The

Table 2

Model Verification Comparisons

Trial No.	Depth of Withdrawal, ft		Effluent Concentration g/ℓ		Data Profiles Plotted in Figure No.
	Predicted	Actual	Predicted	Actual	
1	1.3	1.3	19.2	--	17a
2	1.1	1.1	19.5	23.4	17b
3	0.9	1.0	0.9	0.2	17c
4	1.4	1.3	2.5	0.3	17d
5	1.6	1.4	4.8	1.4	17e
6	0.8	1.0	12.6	7.0	17f
7	0.8	--	7.8	7.3	17g

withdrawal depth predictions are slightly low for the smaller flows and slightly high for the larger flows. The predictions and data agreed well for the average flows and the larger density gradients. It was concluded that the model is acceptable for predicting the withdrawal depth in the design procedure.

101. The predicted effluent suspended solids concentrations approximate the field data in most cases. Generally, the model predictions were higher than the actual effluent concentrations. The predicted effluent concentration would have been lower if the model predicted the depth of withdrawal exactly. Finally, some error may have been introduced in measuring the concentration profile. The depth at which the samples were taken could be in error by $\pm$0.1 ft, which could make a significant difference in the results. The WES model was selected to model the velocity profile since it is more suitable than any model available at this time and will provide a conservative design.

PART VI: DESIGN NOMOGRAM DEVELOPMENT

102. The most important component of the weir design procedure is a set of design nomograms which relate flow, weir length, ponding depth, and effluent suspended solids concentration for a particular dredged material type. There are a large number of parameters considered in these design nomograms. They can be divided into those explicitly considered in the nomograms (i.e., values which the designer can manipulate) and those implicitly considered in the nomogram (i.e., values which were utilized in the nomogram development).

Implicit Parameters

103. There are two types of implicit parameters, those pertaining to the type of weir and those pertaining to the suspended solids and density profiles.

Weir considerations

104. Since a sharp-crested weir has shallower withdrawal zones and is commonly used in the field, it was used for the design nomogram development (i.e., the discharge coefficient used in the WES selective withdrawal program was 3.33). Similarly, since a rectangular weir is less expensive to build and more commonly used in the field, it was used for design nomogram development. The concepts of approach velocity and width of withdrawal zone must be used to extend the nomograms to shaft-type weirs (drains) or polygonal (labyrinth) weirs.

105. The WES selective withdrawal model assumes that the length of the weir is the same as the length of the side of the basin in which the weir is located. In practice the weir extends across only a fraction of the side of the basin. This will tend to reduce the actual approach velocities in comparison with those predicted by the model. Since this effect is small and will yield more conservative results, the WES selective withdrawal model was not modified to account for this.

Density profile considerations

106. A single nomogram cannot be presented to cover all types of

dredged material under all conditions. This is because different dredged materials will develop different concentration profiles as they settle. The greatest accuracy can be achieved by developing a separate nomogram for each dredged material. This is not desirable since it would result in a very large number of nomograms and is not necessary since most fine-grained dredged materials can be classified into a small number of categories based on the type of density profile they produce.

107. The two most important parameters controlling the type of gradient are the soil classification and salinity of the material. Clays in fresh water (salinity less than 1 ppt) do not settle well due to their fine particle size and physicochemical properties. Therefore, dredged material consisting of clays in fresh water will be considered in a separate design nomogram.

108. Clays in salt water tend to flocculate, which causes them to settle much better, producing a significantly different density profile than in fresh water. Clays in salt water will be considered in a design nomogram for silts and saltwater clays.

109. Silty material settles better than freshwater clays because of its particle size. From the available data, it was not possible to determine the density profiles for silts. However, data from sites in which silty dredged material was being disposed indicated that the effluent concentration is similar to the effluent concentration from sites involving saltwater clays. It is therefore reasonable to believe that the density profiles will also be similar since flocculated clays have grain size distributions which are similar to those of silts. Consequently, the same nomogram will also be used for these materials.

110. In order to utilize the WES selective withdrawal model a consistent set of density and suspended solids profiles must be used in the model for each class of nomogram. The suspended solids or density profile can be described by four pieces of information (see Figure 1)--the suspended solids concentration or density at the surface, the suspended solids concentration or density gradient in the upper and lower layers, and the ponding depth (the depth to the interface between the two layers). The characteristics of the two sets of profiles (one for

freshwater clays and one for silts and saltwater clays) are presented in Table 3. The characteristics were developed for the average conditions

Table 3

Concentration Profiles Utilized in Design Procedure Development

Ponding Depth, ft	Surface Suspended Solids Concentration g/ℓ	Concentration Gradient, g/ℓ/ft Upper Layer	Concentration Gradient, g/ℓ/ft Lower Layer
	Freshwater Clays		
0.5	7.0	15	150
1.0	4.5	10	150
1.5	2.5	8	150
2.0	2.0	6	150
3.0	1.8	4	150
4.0	1.5	3	150
	Silts and Saltwater Clays		
For all ponding depths	0.0	0.5	150

found in the basin from the field data presented in Figures 12, 14, 15, 16, and 17.

111. The freshwater clay profiles were developed from the Yazoo City and Fowl River field data. This data demonstrated that the suspended solids concentrations at the surface increased as the ponding depth decreased (see Figures 13, 14, 15). Similarly, the suspended solids concentration gradient in the upper layer (the ponded water layer) increased as the ponding depth decreased. The suspended solids concentration gradient in the lower layer remained constant for all ponding depths (see Table 3).

112. The silts and saltwater clays profiles were developed from the Oyster Bay field data. There was insufficient field data to determine the trend in the suspended solids concentration at the surface and in the suspended solids concentration gradient in the ponded water. However, the surface concentration and upper layer gradient that were measured for a ponding depth of 1 ft (see Figures 16 and 17) were so

small that it was reasonable to use the same surface suspended solids concentration and upper layer suspended solids concentration gradient for all ponding depths. The same lower layer suspended solids concentration gradient was also used for all ponding depths.

Explicit Parameters

113. The parameters which the user can directly manipulate in the design nomogram are flow, weir length, ponding depth at the end of the service life, and effluent suspended solids concentration. In developing the nomogram, the flow, weir length, and ponding depth were chosen, and then the WES selective withdrawal program was run to predict the effluent suspended solids concentration, thus generating one point for the lower half of the nomogram. The flow was then varied, producing a different weir loading for the same ponding depth, and the program was run again to generate another point. Once several points were determined, they were plotted to generate a curve on the nomogram. The ponding depth was then varied and another set of weir loadings were evaluated to produce the remaining curves in the lower half of the nomogram. The curves in the top half of the nomograms are straight lines which pass through the origin and have a slope equal to the weir length in feet.

114. The ponding depth was varied from 0.5 to 4.0 ft and the weir loading was varied from 0.1 to 3.0 cfs/ft. These ranges should sufficiently bracket the desirable values for ponding depth and weir loading.

115. The design nomograms were developed to provide quick design alternatives without long or tedious computations. The procedure was designed to be complete, requiring only that the designer classify the material and be familiar with the site and equipment constraints. One nomogram was developed for each class of material. The two design nomograms are presented in Figures 24 and 25.

116. In summary, the design nomograms are comprised of two quadrants. The top half of the nomogram graphically divides the flow, Q, by the weir length, B, to obtain the weir loading Q/B. The

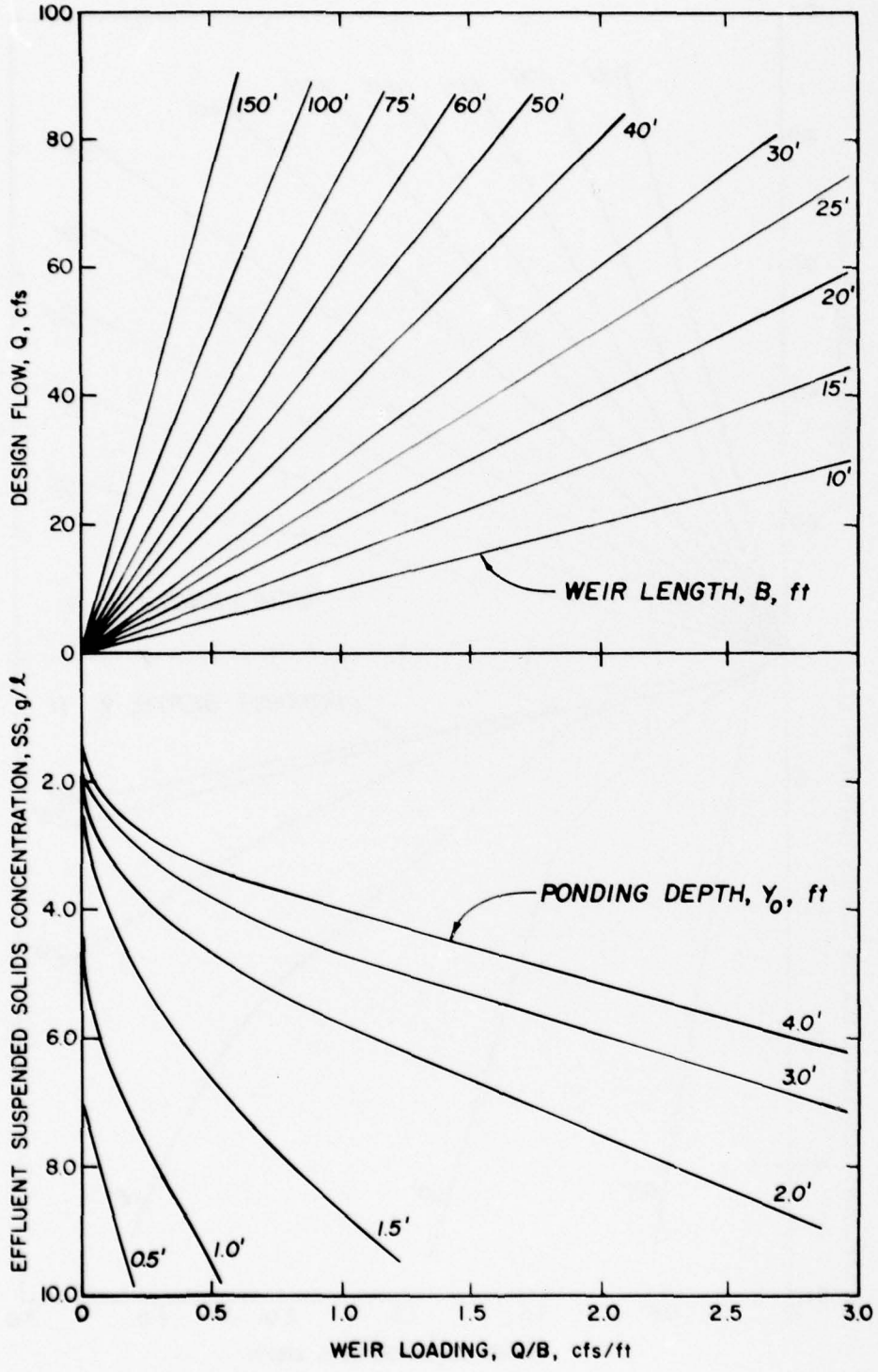


Figure 24. Relationships among design flow, weir length, effluent suspended solids concentration, ponding depth, and weir loading for freshwater clays

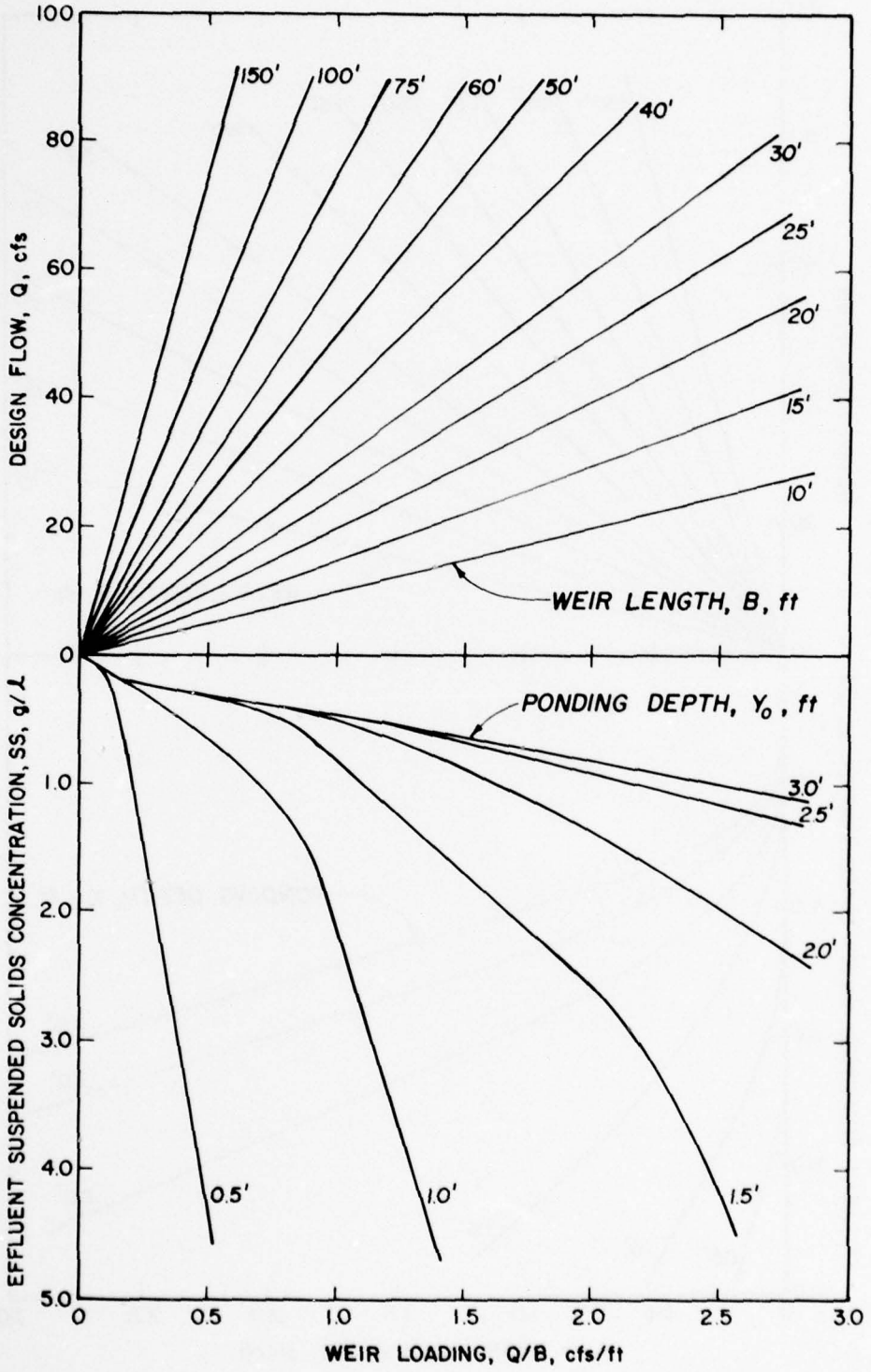


Figure 25. Relationships among design flow, weir length, effluent suspended solids concentration, ponding depth, and weir loading for silts and saltwater clays

bottom half of the nomogram was formed by running the computerized WES selective withdrawal model for a matrix of weir loadings and ponding depths. From each computer run, the effluent suspended solids concentration was obtained. These were plotted against the weir loading for each ponding depth in the lower half of the nomogram. The nomogram relates the flow, the weir length, the ponding depth, and the effluent concentration with each other in a single diagram.

PART VII: WEIR DESIGN AND OPERATIONAL PROCEDURES

117. Sufficient weir length and ponding depth near the weir must be provided in a containment area to prevent water with high suspended solids concentrations from flowing out of the basin. The following section provides a design procedure that uses nomograms for selecting weir length and ponding depth at the weir to maintain effluent quality, given the material type and design flows. The design procedure is based on the principles of selective withdrawal of stratified fluids by Bohan and Grace[5] as modified in the earlier sections of this report. The procedure is applicable for fine-grained dredged material containment areas. The performance of a basin for dredged material that is exclusively sands and gravels will not be significantly influenced by the weir design.

Design Procedure

Data required

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

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

the observed settling of a dredged material is significantly different from the settling of its recommended classification, as determined by the suspended solids concentration profiles in Table 3, then adjust its classification to fit the classification presented in Table 3 for the observed concentration profile.

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

121. In estuarine areas, the salinity may vary through the year due to differences in the freshwater flow and the location of the saltwater wedge. Therefore, the lowest probable salinity of the near-bottom water in the area to be dredged during the projected dredging operation should be used since this provides the most conservative design.

122. Knowing the salinity and the soil type, the designer can select the correct nomogram from Table 4. The nomogram in Figure 24 is for freshwater clays. The nomogram in Figure 25 is for silts and all

Table 4

Nomogram Selection

	Clays	Silts
Salinity <1 ppt	Figure 24	Figure 25
Salinity 1-3 ppt	Transition Range	Figure 25
Salinity >3 ppt	Figure 25	

saltwater fine-grained dredged material. The nomogram in Figure 24 is for dredged material that settles slowly, and the nomogram in Figure 25 is for dredged material that settles more rapidly.

123. The design flow refers to the peak flow over the weir during the design life of the basin. Typical discharge rates for different size dredges are shown in Table 5. If the dredge is not operating for a

Table 5

Flow Ranges from Various Sized Dredges

Discharge Pipeline Diameter, in.	Range of Discharge Rates (for Flow Velocity of 12 to 18 fps),* cfs
8	4.2-6.3
10	6.5-9.7
12	9.4-14.1
14	12.8-19.3
16	16.5-24.8
18	21.2-31.8
20	26.2-39.3
24	37.3-56.6
27	47.6-71.5
28	51.3-77.0
30	58.9-88.4
36	84.9-127.3

* To obtain discharge rates for other velocities, multiply the lower discharge rate shown in this tabulation by the velocity and divide by 12.

considerable period of time, the flow rate over the weir may be less than the inflows shown in Table 5. Therefore, a value on the low end of the flow range for a given dredge size in Table 5 should be sufficiently conservative. The actual flow rate will be a function of the dredge, the head loss in the pipe, and the elevation of the discharge pipe at the basin.

124. The designer must determine the appropriate effluent suspended solids limit for his dredging operation based on effluent standards, the water quality of the receiving stream, and environmental concerns. The effluent suspended solids concentrations predicted by the nomograms are average values. If the designer wants to design for

worst conditions, he must assume a value for the ratio of the maximum to average effluent suspended solids concentration for a given weir loading (Q/B) and ponding depth. A ratio of 1.5 to 2.0 was observed in the field data.

Use of nomogram

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

126. The weir loading (Q/B , the flow in cfs per ft of weir length) is the principal design parameter. If the designer wishes to use a low ponding depth, the weir loading must be kept small. Lower weir loadings will produce better effluent quality at the cost of a longer weir. Weir loading is related to static head (H) above the weir for a sharp-crested weir by:

$$\frac{Q}{B} = 3.3H^{1.5} \tag{9}$$

The weir loading should be kept between 0.1 and 3.0 cfs/ft to maintain good effluent quality without requiring excessively long weirs or deep basins. This corresponds to a range of static heads of 1 to 12 in. or a range of depths of flow over the weir of 0.8 to 10 in.

127. The ponding depth also provides the designer with a parameter through which he can control effluent quality. The optimal range for this parameter is from 1 to 3 ft. Ponding depths of greater than 3 ft will result in high and hence expensive dikes, while not considerably improving the effluent quality. Depths of less than 1 ft will

result in poor effluent quality. Ideally, the ponding depth and depth of withdrawal zone will be equal at the end of the basin's service life.

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

Example designs

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

130. In Problem 1, a weir is to be designed for a freshwater dredging site. The dredged material is classified as a CL clay. The design flow is 30 cfs and the effluent standard is 8 g/ℓ .

131. The designer first selects the proper nomogram from Table 4. Since the material is a freshwater clay, the nomogram in Figure 24 should be used. The designer then decides to maintain an average effluent suspended solids concentration of 5 g/ℓ at the end of the basin's service life in order to insure that the maximum effluent suspended solids concentration will not exceed the 8-g/ℓ effluent standard, despite fluctuations in conditions. Variable effluent quality can be caused by fluctuations in the influent dredged material slurry, suspended solids concentration, the wind disrupting settling, and many other factors. The designer is now ready to use the nomogram.

132. The designer draws horizontal lines on the nomogram at his design flow, 30 cfs, and his effluent suspended solids concentration, 5 g/ℓ. These parameters are shown as solid lines (A) and (B) on Figure 26. The designer can now select an infinite number of combinations of weir length, B , and ponding depth, y_o , to meet his design parameters, 30 cfs and 5 g/ℓ . A possible combination is determined by drawing a vertical line connecting the horizontal lines at 30 cfs and 5 g/ℓ . Six combinations that cover the range of feasible alternatives

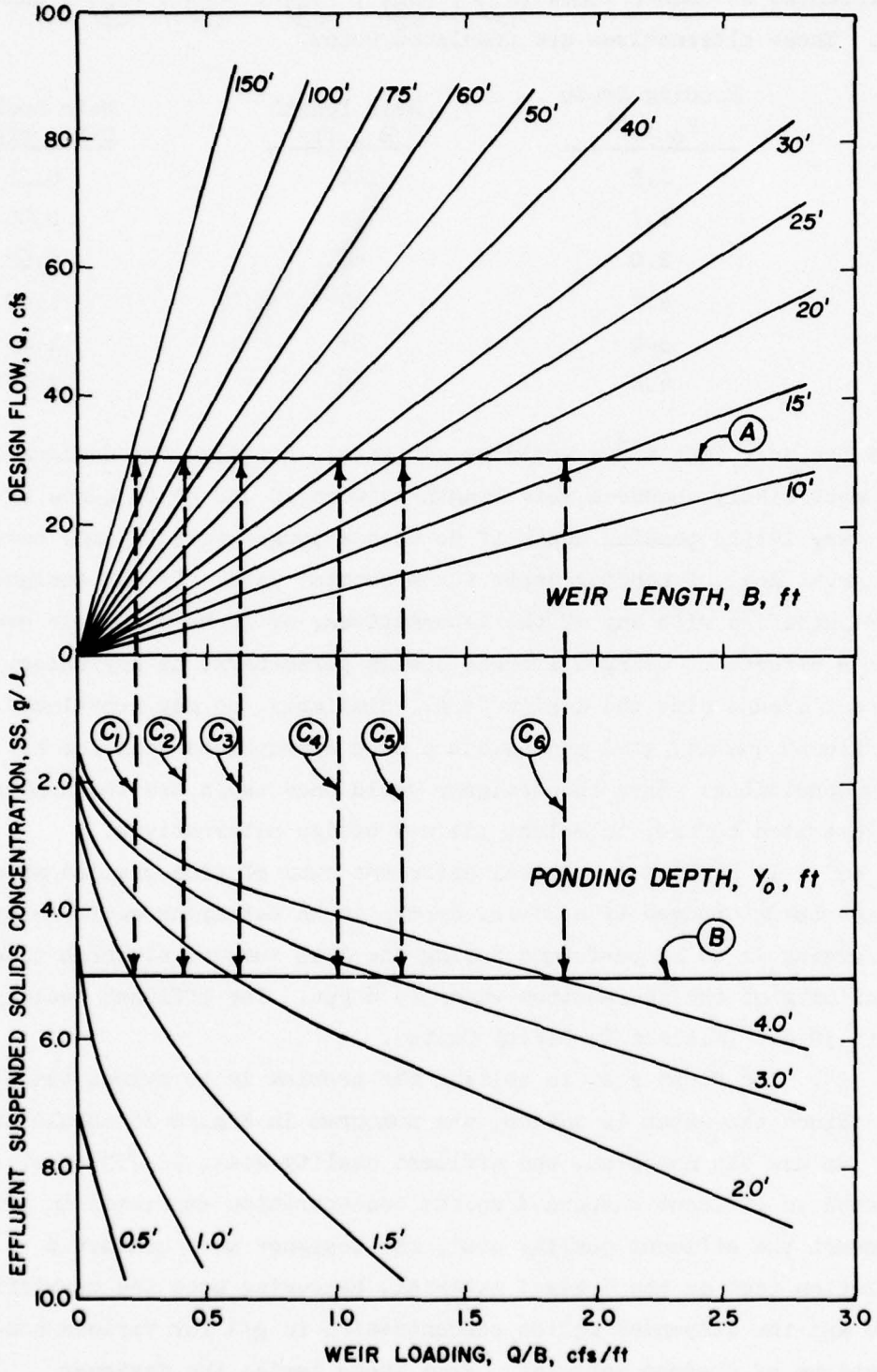


Figure 26. Example Problem 1

are presented as dashed lines (C1), (C2), (C3), (C4), (C5), and (C6). These alternatives are tabulated below.

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

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

133. In Problem 2, several different cuts of fine-grained material are being dredged by a 27-in. dredge in an estuarine environment. The dredging is to be performed during the late summer, at which time the salinity of the near-bottom water is 6 ppt. The effluent quality goal is 50 JTU (Jackson Turbidity Units).

134. The first step in solving the problem is to select the nomogram. Since the water is saline, the nomogram in Figure 25 should be used. To use the nomogram, the effluent quality goal, 50 JTU, must be converted to effluent suspended solids concentration expressed in g/ℓ. To convert the effluent quality goal, the designer must conduct a correlation test on the dredged material, measuring both the turbidity in JTU and the suspended solids concentration in g/ℓ for various concentrations of dredged material. From these tests, the designer

determines that 50 JTU corresponds to 1.2 g/ℓ of suspended solids. (Note: Any correlation between suspended solids and turbidity is highly dependent on the dredged material. A general correlation cannot be given.)

135. The range of expected flows from a 27-in. dredge is 47.6 to 71.5 cfs. The dredging site is fairly far from the disposal area and the down time is expected to be considerable due to the location of the cuts; therefore, a design flow of 50 cfs will be used in the design procedure.

136. The designer now draws horizontal lines at 50 cfs and 1.2 g/ℓ on the nomogram, lines (A) and (B) in Figure 27. The designer evaluates the alternatives and chooses to use a 30-ft weir length. He then draws a vertical line (line (C)) down from the intercept of the horizontal line at 50 cfs (line (A)) and the 30-ft weir length line to the horizontal line at 1.2 g/ℓ (line (B)) (see Figure 27). The designer reads from the nomogram at the intersection of lines (B) and (C) that he must maintain a ponding depth of 1.9 ft at the end of the basin's service life.

137. Now, suppose that the dredging is scheduled to be performed in the spring during the high flow period for a nearby river. During this period, it is possible that the salinity may be too low for the material to flocculate well. Therefore, the design must be checked using the freshwater classifications. To select the proper nomogram for freshwater material, each cut of dredged material is classified according to the Unified Soil Classification System. The dredged material is found to be composed of lean silts (ML), organic silts (OL), and lean clays (CL); therefore, it is classified as a clay (the slowest settling case). Table 4 indicates that the nomogram in Figure 24 should be used for freshwater clays.

138. The designer checks his design on the freshwater nomogram. Starting on the design flow axis at 50 cfs, a horizontal line (line (A') on Figure 28) is drawn to the 30-ft weir length line. Line (B') is then drawn vertically down through a weir loading of 1.67 cfs/ft to a ponding depth of 1.9 ft and then line (C') is drawn horizontally to the effluent

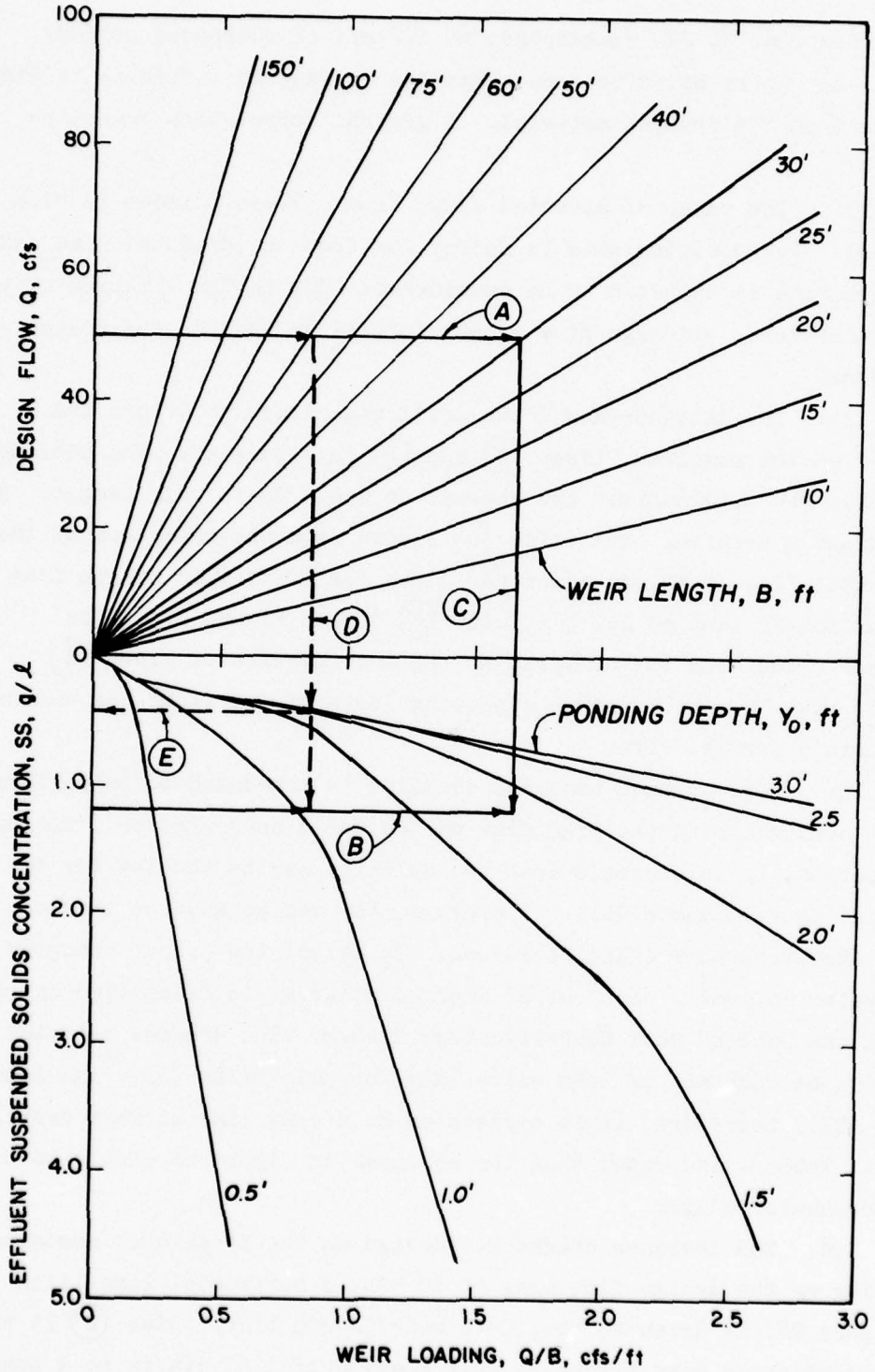


Figure 27. Example Problem 2 (part 1)

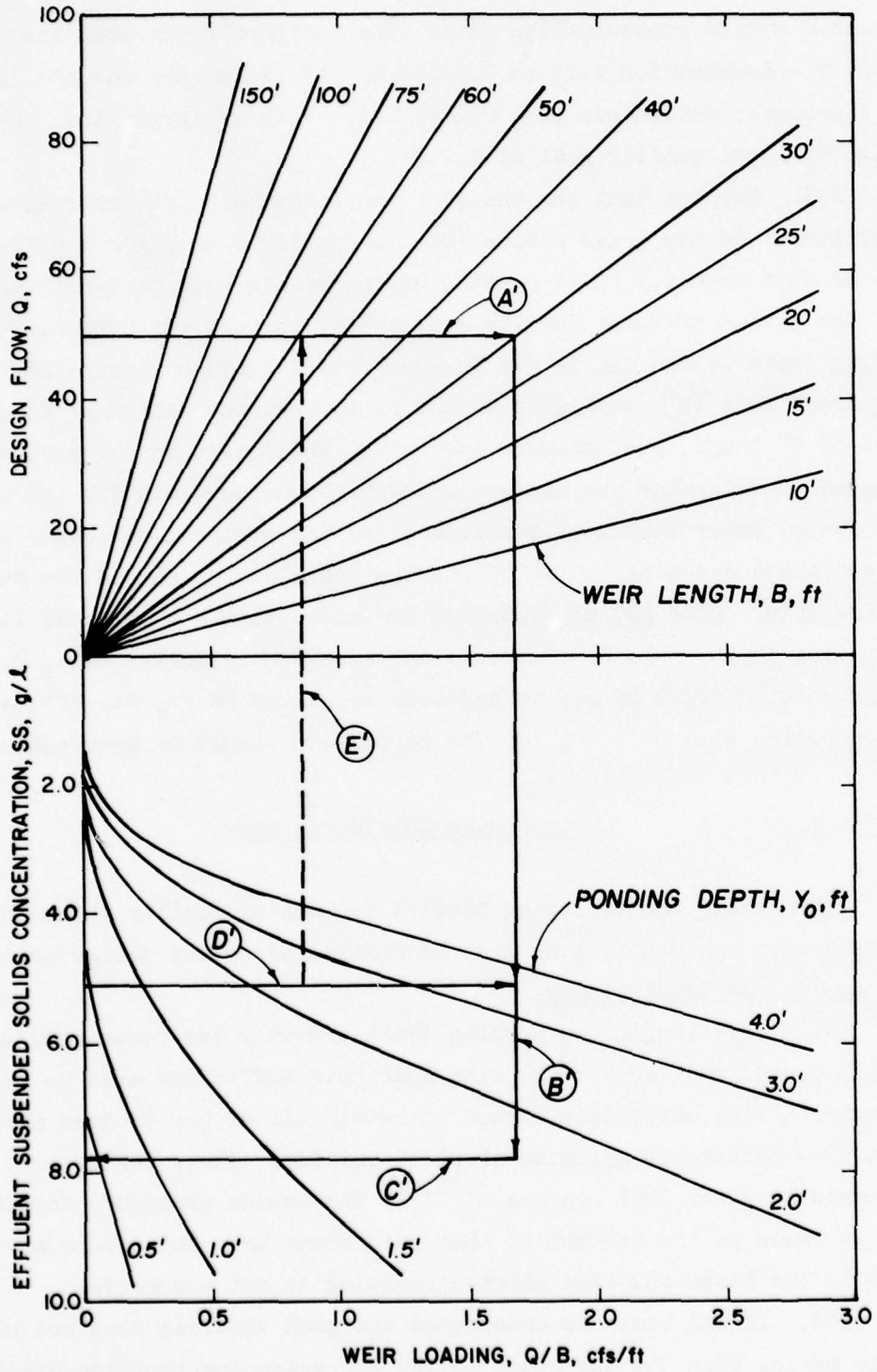


Figure 28. Example Problem 2 (part 2)

suspended solids concentration axis. The designer reads that the effluent concentration will be 7.7 g/ℓ at the end of the basin's life for freshwater conditions (see Figure 28). This is larger than the design effluent quality goal of 1.2 g/ℓ.

139. Suppose that the designer can tolerate 5 g/ℓ for freshwater conditions. He now draws a line (D') on Figure 28 at 5 g/ℓ and finds that he must have 3.7 ft of ponding depth for his weir design to maintain the design effluent quality goal. Now, suppose that the maximum ponding depth he can use is 2.5 ft due to basin volume constraints. He then draws line (E') on Figure 28 and finds that the weir must be at least 59 ft long. The designer now checks the design on the saltwater nomogram to determine the minimum allowable ponding depth for the new weir design under saltwater conditions and the effluent suspended solids concentration assuming 2.5 ft of ponding depth at the end of the basin's service life. Line (D) of Figure 27 indicates that a minimum of 1.1 ft of ponding depth would be tolerable and line (E) indicates that, for the design ponding depth of 2.5 ft and weir length of 60 ft, the effluent concentration will be 0.4 g/ℓ. The 60-ft weir should be acceptable.

Other Design Considerations

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

Weir design and basin sizing

141. Weir length and ponding depth are only two parameters in the overall containment area. The site must have sufficient area to permit proper settling, sufficient volume to retain all of the dredged material, and a flow pattern to minimize short-circuiting. These topics are addressed in other DMRP reports.[3,30,31] The design procedure developed here is based on the assumption that sufficient area and volume are provided in the basin and that short-circuiting is not excessive.

142. If the basin is undersized and good settling does not occur in the basin, then the suspended solids concentration profiles utilized in development of the nomograms will no longer be representative of the

concentration profiles found in the basin. The suspended solids concentration at the surface and the concentration gradient in the upper layer will be larger and, consequently, effluent suspended solids will be higher than predicted by the nomograms.

143. If the area is large (>200 acres), the weir design will not be as critical as in smaller areas because the solids concentration in the withdrawal zone will not be as high as in smaller areas, provided adequate ponding depth is maintained.

144. If sufficient volume is not provided in the containment area for all the material, then the design ponding depth for the end of the service life of the basin cannot be maintained. Consequently, the design effluent quality goal cannot be maintained. Therefore, adequate volume is quite critical.

Safety factors

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

Sharp-crested weirs

146. Sharp-crested weirs should be used in dredged material confinement basins whenever possible. They require a smaller ponding depth because the depth of their withdrawal zone is smaller. Consequently, the effluent quality will also be better. A weir is considered sharp-crested if the thickness of the weir is less than two-thirds of the depth of flow over the weir.[32] Except for very low flows, a weir made up of 2-in.-thick boards can be treated as a sharp-crested weir.

Shaft-type weirs

147. In some cases the outflow structure is a four-sided drop inlet or shaft located in the basin. The weir length (B) determined from the nomograms is for a rectangular weir. In converting the values to make them applicable to shaft-type weirs, the approach velocity of the fluid is the key consideration. To minimize the approach velocity and

hence the withdrawal depth, the shaft weir should not be placed too near the dike. In Figure 29, location A is the most desirable since flow can approach it from all four sides (four effective sides). Location B is less desirable since flow can only approach from three directions (three effective sides). Location C is the least desirable since it has only two effective sides.

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

$$S = \frac{B}{n} \tag{10}$$

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

149. When the shaft weir is installed and operated, all of the sides must be kept at the same elevation. If not, the weir will have a weir loading (Q/B) of the lowest side.

150. Since effluent pipes must run from the shaft weir under the dike to the receiving stream, a location such as A in Figure 29 may not be optimal since it is far from the dike and will require a longer pipe than B , which is easier to operate.

Polygonal (labyrinth) weirs

151. Polygonal (labyrinth) weirs have been used to reduce the head over the weir. Such weirs have very little impact on effluent quality since the controlling factor for the depth of withdrawal and consequently the effluent suspended solids concentration is not the head but the approach velocity. For a given flow, even though the depth of flow and velocity over the weir crest are less for a polygonal weir, the approach velocities, and therefore also the depth of withdrawal and effluent quality will be essentially the same as those for a rectangular weir of equal horizontal length along the dike, L , as shown in

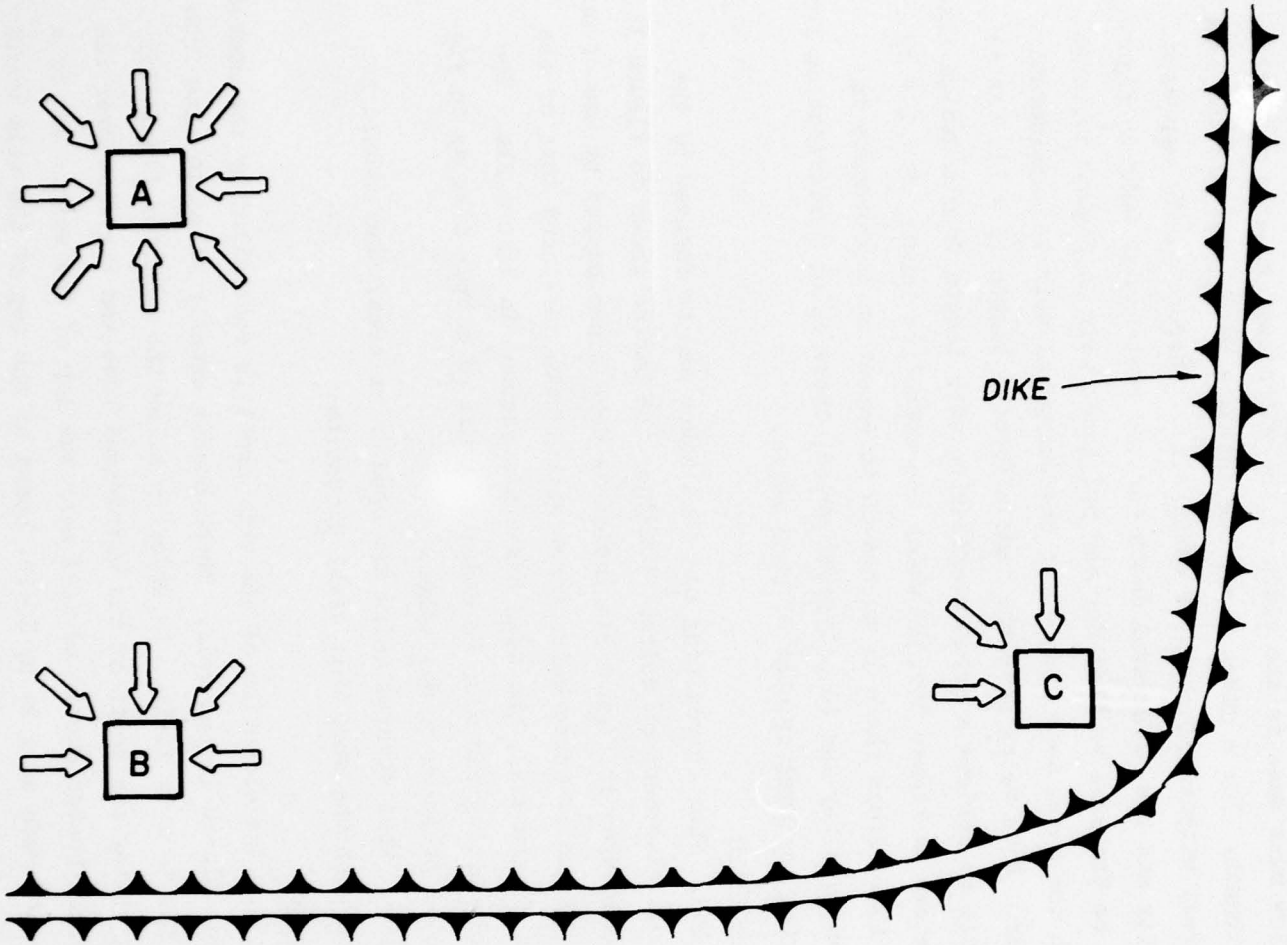


Figure 29. Possible locations for shaft-type weirs

Figure 30. Figure 30 illustrates the width of the withdrawal zone or effective weir length (B) for three types of weirs. The arrows indicate the approaching flow towards the weir. The minimum width through which the flow must pass is the width of the withdrawal zone or the effective weir length. For a given flow, the approach velocities are the same for different withdrawal zones of equal size. Therefore, the approach velocity and the withdrawal depth for the rectangular weir in Figure 30a would be the same as that for the polygonal weir in Figure 30b even though the total weir length for the polygonal weir is considerably greater. Both weirs have the same effective length (B = L). It is possible to achieve a greater effective weir length from a design like that shown in Figure 30c, in which the effective length $B = L + 2M$.

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

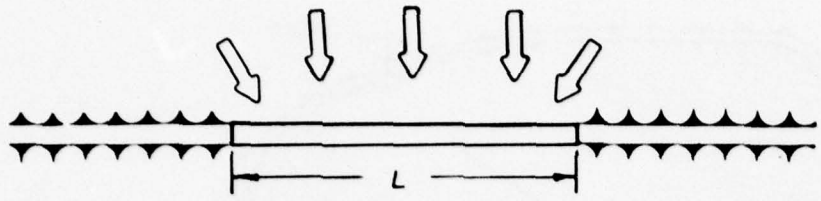
Weir location

153. Short-circuiting and dead zones can be reduced by the judicious placement of weirs. Consider the basins shown in Figure 31. The shaded area in Figure 31a indicates dead zones caused by use of one weir. By use of three weirs (each with length one-third that of the weir in Figure 31a), the dead zones are reduced in Figure 31b. The short-circuiting can also be reduced by use of a spur dike as in Figure 31c as proposed by Gallagher.[3]

154. When several weirs are used in an area, they should be operated with the same weir crest elevation.

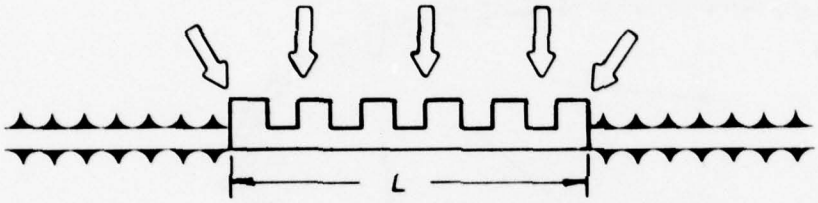
Board size

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



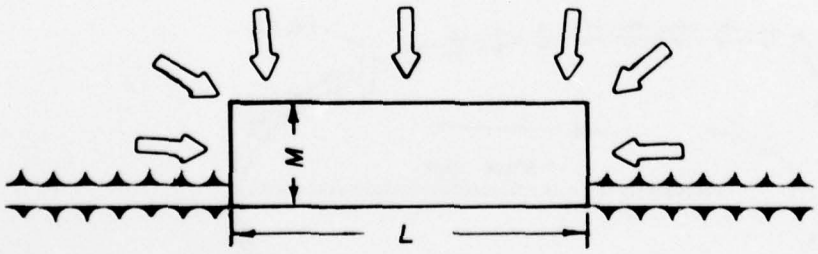
EFFECTIVE WEIR LENGTH = L

a. RECTANGULAR WEIR



EFFECTIVE WEIR LENGTH = L

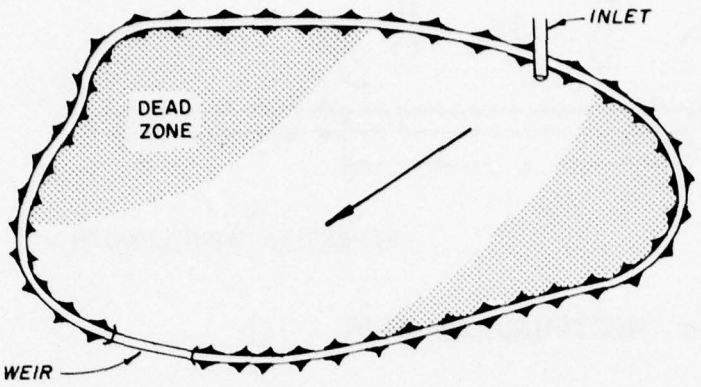
b. POLYGONAL WEIR



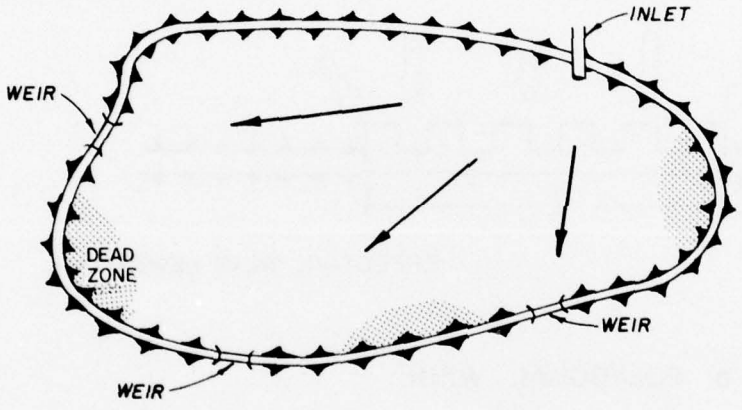
EFFECTIVE WEIR LENGTH = L + 2M

c. JUTTING RECTANGULAR WEIR

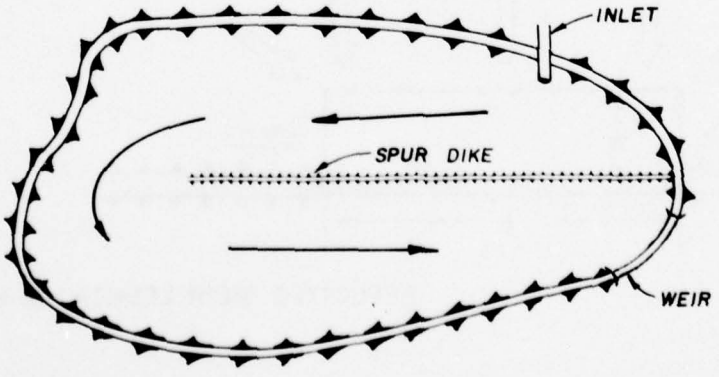
Figure 30. Effective lengths of weirs (plan view)



a.



b.



c.

Figure 31. Short-circuiting and dead zones

significant deterioration in the effluent quality by the removal of a small board.

156. Since some water with high solids concentration may leak between the boards, a small number of larger boards may be preferable to a large number of small boards near the bottom of the weir. Figure 32

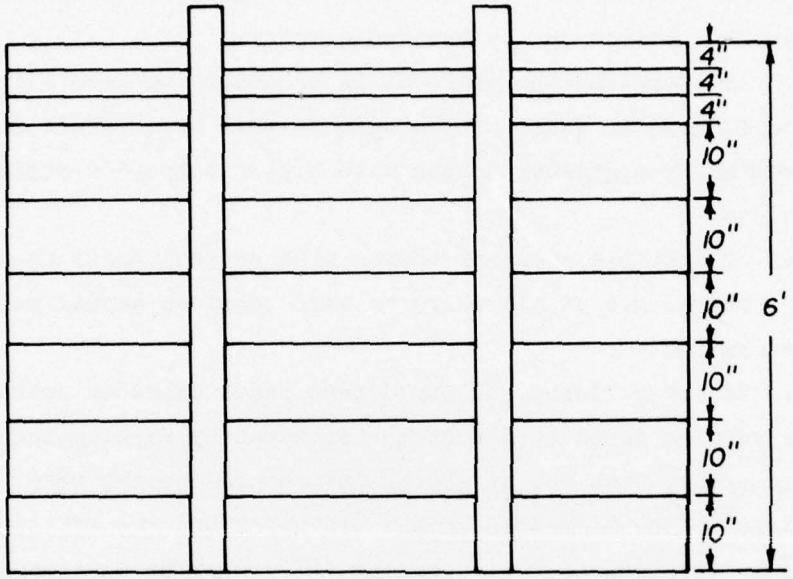


Figure 32. Suggested weir boarding

shows a weir that will be boarded up to 6 ft but will be operated between 5 and 6 ft. Ten-in. boards are used for the bottom layers with 4-in. boards for the higher zone.

Operational Guidelines

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

General guidelines

158. The best effluent quality in a dredged material containment area can be achieved if the weir crest is maintained at the highest

feasible elevation. This provides the maximum ponding depth at any given time.

159. The weir elevation may need to be lowered to provide the necessary freeboard or to protect the integrity of the dikes. In such a case, the preservation of the dikes is more important than effluent quality, and the boards may be removed quickly.

160. In operating the weir, it is necessary to keep floating debris from lodging in front of the weir as this will result in more of the flow coming from greater depths with higher suspended solids concentrations.

161. If multiple weirs or a weir with several sections are used in a basin, the crests of all weirs or weir sections should be kept at the same elevation.

162. If the effluent quality deteriorates below an acceptable limit, the ponding depth (y_o) must be increased by raising the elevation of the weir crest, that is, by adding more boards to the weir. If the weir crest is at the highest possible elevation and the effluent quality is still unacceptable, the weir loading (Q/B) must be decreased by lowering the flow into the basin and over the weir. The flow may be lowered by using a smaller dredge or by operating the existing dredge intermittently. The new weir loading may be selected by using the nomograms or by measuring the effluent quality for various weir loadings. The weir loading is controlled in the field by using the head over the weir as an operational parameter since the flow over the weir (Q) cannot easily be measured.

Operating head

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

$$H = \left(0.3 \frac{Q}{B}\right)^{2/3} \qquad (11)$$

where

H = static head over the weir, ft

Q = flow over the weir, cfs

B = weir length, ft

Q/B = weir loading, cfs/ft

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

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

165. The static head can also be determined indirectly by measuring the depth of flow over the weir, h (see Figure 2). According to Rehbock,[32] the ratio of depth of flow over the weir to static head (h/H) equals 0.85 for sharp-crested weirs. This ratio approaches 0.67 for broad-crested weirs. Since the depth of flow over the weir is directly proportional to the static head, it may be used directly as an operating parameter. In this case, the weir loading can be controlled by the

depth of flow over the weir by using the following equation for sharp-crested weirs.

$$h = 0.85H = 0.85\left(0.3\,\frac{Q}{B}\right)^{2/3} \qquad (12)$$

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

166. The previous equations for the weir loading, static head, and depth of flow over the weir are valid only for sharp-crested weirs. If a different type of weir is used, the above equation must be modified to account for the differences in the coefficient of discharge and the ratio of depth of flow over the weir to static head. Information on polygonal weirs has been documented by Hay and Taylor[33] and Indelkofer and Rouvé.[34]

167. The head over the weir or depth of flow over the weir would be used as an operating parameter when the basin conditions or dredging operation exceeded the design limits. To illustrate how to use these parameters, suppose that an existing basin with a sharp-crested weir was designed to operate with a weir loading of 1.5 cfs/ft with an 18-in. dredge; however, due to breakdown of the 18-in. dredge, the contractor decides to use a 24-in. dredge. The 24-in. dredge produces enough flow to maintain a weir loading of 2.5 cfs/ft. Therefore, the dredge must operate intermittently after the effluent quality starts to deteriorate. At this time, the maximum allowable static head or depth of flow over the weir for the design weir loading (Q/B) of 1.5 cfs/ft must no longer be exceeded. Therefore, the operator calculates the maximum allowable for the static head (H) and the depth of flow over the weir (h) as follows:

$$H = \left[0.3\left(\frac{Q}{B}\right)\right]^{2/3} \tag{13}$$

$$= \left[0.3(1.5)\right]^{2/3}$$

$$= 0.59 \text{ ft} = 7.0 \text{ in.}$$

and

$$h = 0.85H \tag{14}$$

$$= 0.85(7.0 \text{ in.})$$

$$= 6.0 \text{ in.}$$

Therefore, the dredging should be performed intermittently so that the static head and the depth of flow over the weir do not exceed 7 in. and 6 in., respectively.

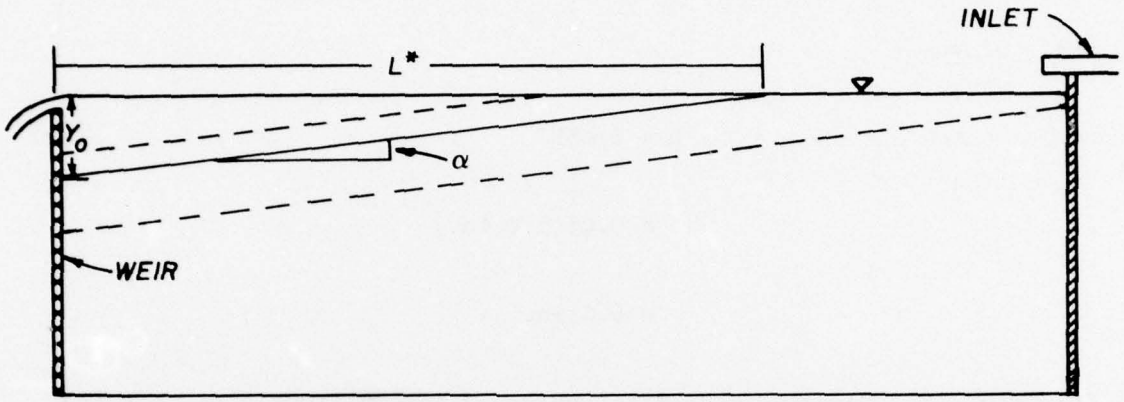
Undersized basin

168. If the basin is undersized and/or slow settling is occurring in the basin, added retention time is needed to achieve better settling. Added retention time can be obtained by first raising the weir crest to its highest elevation to maximize the ponding depth, and then if necessary, by operating the dredge intermittently or using a smaller dredge. The retention time with intermittent dredging can be controlled by setting a maximum allowable static head or depth of flow over the weir based on the effluent quality achieved at those heads. The operating procedure is analogous to the weir loading example since the weir loading and retention time are directly related for a given basin, ponding depth, and weir.

Critical effective basin length

169. The length of basin from the weir to the inlet over which water is ponded, hereafter termed the effective basin length (L), can serve as a means for estimating the ponding depth at the weir near the

end of the basin's service life. In a basin, the dredged material first settles closer to the inlet and then farther and farther from the inlet. This forms a sloping interface in the basin (see Figure 4). For a given basin with interfacial slope (α) and effective basin length (L), the ponding depth at the weir would be determined by the following equation. (See Figure 33.)



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

Figure 33. Effective basin length

$$y_o = \alpha L \tag{15}$$

A typical value for α is 0.002 ft/ft.

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

171. In a similar manner, the equation can be used to solve for the approximate effective basin length needed to maintain the design ponding depth, hereafter termed the critical effective basin length (L^*). (See Figure 33.)

$$L^* = \frac{y_o}{\alpha} \qquad (16)$$

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

Basin drawdown

172. Similarly, once the dredging operation is completed, the ponded water must be removed so that drying can occur. To drain the basin, the weir boards should be removed one row at a time. Preferably, 2- by 4-in. boards should be used in order to minimize the withdrawal of settled solids. The next row of boards should not be removed until the water level is drawn down to the weir crest and the outflow is low. This process should be continued until the interface is reached. It is desirable to eventually remove the boards below the interface so that rainwater can drain from the area.[35] These boards can be removed only after the material has consolidated sufficiently so that it will not flow from the basin. If it begins to do so, the boards should be replaced.

PART VIII: CONCLUSIONS

173. The flow over the weir from an upland containment area can be adequately described by a modified version of the WES selective withdrawal model for density-stratified flow. The withdrawal depth is a function of the density profile and the weir loading. The model predicts that a stronger density gradient in front of the weir reduces the withdrawal depth. The effluent quality is highly dependent on the suspended solids concentration in the ponded water. The model further predicts that higher weir loadings will increase the withdrawal depth and, consequently, the effluent suspended solids concentration.

174. Field data indicated that larger ponding depths reduce the suspended solids concentrations and gradient in the upper layer. Thus, larger ponding depths produce better effluent quality. To insure a large ponding depth during filling of the area, the weir crest should therefore be maintained at the highest feasible elevation to minimize the effluent suspended solids concentration. The model further showed that sharp-crested weirs have shallower withdrawal depths than broad-crested weirs. Also, polygonal weirs produce only a negligible improvement in the effluent quality as compared with rectangular weirs.

175. The model was used to develop two nomograms for designing weirs, one for clays in fresh water and the other for silts and clays in salt water. Two nomograms are needed since dredged materials form different suspended solids concentration profiles depending on the soil classifications and water salinity. Saltwater clays settle much better than freshwater clays. The suspended solids concentration profiles found in the field for freshwater clays and saltwater clays were used to develop the two nomograms. If the suspended solids concentration profile observed in the field for a given dredged material does not agree with the profile used for its classification in the nomogram development, the designer must use judgement in selecting and using the presented nomograms.

176. For a given dredged material type, a nomogram relates the flow, weir length, ponding depth, and effluent suspended solids

concentration with each other. This allows the designer to develop tradeoffs between them to select a weir design. For design purposes, the weir loading (Q/B), expressed in flow per length of weir, is the best parameter for relating the flow over the weir with the ponding depth and effluent quality. The weir loading should be designed in the range of 0.1 to 3.0 cfs/ft to maintain good effluent quality without using extremely long weirs or large ponding depths. While the weir loading in cfs/ft is useful in design, for operating purposes the flow over the weir is best managed by controlling the static head over the weir (H) or depth of flow over the weir (h). For sharp-crested weirs operating in the above range of weir loadings, the static head and depth of flow over the weir should be operated in the range of 1 to 12 in. and 0.8 to 10 in., respectively.

177. The weir can only maintain the quality of the water ponded in front of the weir; it cannot improve it. The quality of the ponded water in front of the weir is dependent on basin design. Proper weir design can prevent an increase in the effluent solids by preventing discharge of settled solids with the ponded water. Proper weir placement can reduce short-circuiting in the basin.

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WEIR DESIGN TO MAINTAIN EFFLUENT QUALITY FROM DREDGED MATERIAL --ETC(U)
MAY 78 T M WALSKI, P R SCHROEDER

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APPENDIX A: EQUATIONS TO RELATE DENSITY AND SOLIDS CONCENTRATION

1. Several equations are required to convert the density or concentration data obtained from the laboratory analyses into the required form for use in the model. The model needs the density measurements expressed in grams per cubic centimetre and the suspended solids concentrations expressed in grams per litre. Conversion of the various forms of solids concentration required a measurement of both the specific gravity of the soil particle and the salinity of the water plus the concentration or density measurement. The specific gravity test is outlined in EM 1110-2-1906.[28*] The salinity may be measured with a salinity probe or by the method outlined in Standard Methods[29] for total dissolved solids. The salinity is used to calculate the density of the filtered fluid as follows.

$$\rho_F = 1 + \frac{Sal}{1000 - Sal} \tag{A1}$$

where

ρ_F = density of the filtered fluid, g/cm^3 (ρ_F = 1.00 for fresh water, ρ_F = 1.03 for ocean water)

Sal = salinity of total dissolved solids, ppt

2. The normal laboratory procedure for analyzing samples with high suspended solids concentration (greater than 1 percent by weight) is outlined in EM 1110-2-1906.[28] In this procedure, a quantity of sample is weighed and then dried and weighed again. From this procedure, the total weight of the sample, the weight of water (evaporate), and the weight of the solids are obtained. These measurements are used in the following set of equations to calculate the percent solids and suspended solids by weight in the samples.

$$\%S = \frac{\text{Wt. Solids} \times 100}{\text{Wt. Total}} \tag{A2}$$

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* Raised numbers refer to similarly numbered items in the References at the end of the main text.

$$\%SS = \frac{\left[\text{Wt. Solids} - \frac{\text{Sal}}{(1000 - \text{Sal})} \text{Wt. } H_2O\right] \times 100}{\text{Wt. Total}} \tag{A3}$$

%S = total solids concentration, percent by weight

%SS = suspended solids concentration, percent by weight

Wt. Solids = weight of solids in the sample, g

Wt. Total = total weight of the sample, g

Wt. H_2O = weight of water in the sample, g

Sal = salinity, ppt

3. The percent solids and percent suspended solids by weight are used in the next set of equations to calculate the density and the suspended solids concentration of the sample.

$$\rho_s = \frac{\rho_F}{1 + \frac{\%SS}{100}\left(\frac{\rho_F}{S.G.} - 1\right)} \tag{A4}$$

$$\rho_s = \frac{S.G.}{(1 - \rho_F + S.G.) - \frac{\%S}{100}(S.G. - \rho_F)} \tag{A5}$$

$$\text{S.S. Conc.} = \frac{10\rho_F \%SS}{1 - \frac{\%SS}{100}\left(1 - \frac{\rho_F}{S.G.}\right)} \tag{A6}$$

$$\text{S.S. Conc.} = \frac{1000\left(S.G. - 1 - \frac{\%S}{100}\rho_F S.G.\right)}{\left(1 - \frac{\%S}{100}\right)(S.G. - \rho_F) + 1} \tag{A7}$$

where

ρ_s = density of the sample, g/cm^3

S.G. = specific gravity of the soil particles

S.S. Conc. = suspended solids concentration, g/ℓ

The rest of the variables are as defined before.

4. For samples with low suspended solids concentrations (less than 1 percent by weight), the suspended solids concentration may be measured directly by the procedure outlined in Standard Methods.[29] The values can be converted to density by the following equation.

$$\rho_s = \rho_F + \frac{\text{S.S. Conc.}}{1000}\left(1 - \frac{\rho_F}{\text{S.G.}}\right) \tag{A8}$$

5. The density can also be measured directly. These density measurements can be converted to suspended solids concentration by the following equation.

$$\text{S.S. Conc.} = \frac{1000(\rho_s - \rho_F)}{1 - \frac{\rho_F}{\text{S.G.}}} \tag{A9}$$

APPENDIX B: WITHDRAWAL DEPTH AND VELOCIY PROFILE MODELS

1. Wood and Lai used a theoretical approach to evaluate the flow of a two-layered fluid over a broad-crested weir with contracting. They incorporated the Bernoulli equation for each layer with the continuity equation and the simple broad-crested weir theory to form the following equation for one-dimensional flow.[6*]

$$(1 + \alpha) \frac{4}{27} \frac{Y_t^3}{y_1^2} + y_1 = Y_1 \tag{B1}$$

where

$$\alpha = \frac{\rho_1}{\Delta\rho} \tag{B2}$$

$$Y_t = \left(\frac{Q}{3.3B}\right)^{2/3} \tag{B3}$$

for sharp-crested weirs. The variables are defined as follows.

α = dimensionless density difference ratio

Y_t = static head on the weir, ft

y_1 = height of flow over the weir, ft

Y_1 = thickness of the top layer, ft

ρ_1 = density of the top layer, g/cm^3

$\Delta\rho$ = density difference between the bottom and top layers, g/cm^3

Q = discharge rate, cfs

B = weir length, ft

2. According to Rehbock,[32] y_1 and Y_t are interrelated by the following equation for sharp-crested weirs.

$$y_1 = 0.85Y_t \tag{B4}$$

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* Raised numbers refer to similarly numbered items in the References at the end of the main text.

The coefficient approaches 0.67 as the flow approaches critical depth for narrow- and broad-crested weirs.[30] By use of this relationship, the equation can be solved to determine the thickness of the top layer or ponding depth that will prevent discharge of the bottom layer for any given density difference and weir flow. The equation was verified in a small laboratory flume using a layer of fresh water and a colored layer of salt water. The model has not been verified in the field or with fluids that were density-stratified by suspended solids.

3. Debler[8] experimentally determined the critical densimetric Froude number for various ratios of withdrawal depth to total depth. In his experiments, he discharged the bottom layers through a line sink at the bottom of the laboratory flume. His approach assumed a linear density stratification,

$$\varepsilon = \frac{1}{\rho} \frac{d\rho}{dy} \tag{B5}$$

and defined his densimetric Froude number as

$$F = \frac{q}{h^2 \sqrt{g\varepsilon}} \tag{B6}$$

His results were as follows:

h/d	F (critical)
1.00	0.28
0.76	0.26
0.73	0.23
0.63	0.26
0.39	0.24
0.29	0.20

The critical densimetric Froude number varied as a function of h/d due to the viscous effects of the boundaries. The model is an approximation for the one-dimensional, inviscid, nondiffusive flow case. His work has been theoretically supported by a streamline analysis performed by Kao.[9] His model has not been verified in the field or with suspended solids.

4. Huber analytically solved the case for one-dimensional,

density-stratified flow in a system with two layers of equal thickness using a relaxation technique.[10] In his system, he assumed a discharge from the bottom layer through a line sink at the channel bed, such as a sluice gate. For this case, he found that the critical densimetric Froude number was

$$F = \frac{q_c}{h^2\sqrt{g\,\frac{\Delta\rho}{\rho h}}} \tag{B7}$$

where

q_c = critical unit flow, cfs/ft
h = thickness of each layer, ft
g = gravitational constant, 32.174 ft/sec^2
$\Delta\rho$ = density difference between the two layers, g/cm^3
ρ = fluid density, g/cm^3

The model has not been verified experimentally.

5. White proposed an equation to determine the critical bed shear stress by balancing the moments on a particle due to drag, lift, and its immersed weight.[20] The equation is:

$$\tau_c = C(\gamma_s - \gamma)\, d \tan\theta \tag{B8}$$

where C equals a coefficient that varies from 0.18 for a laminar boundary layer to 0.045 for a turbulent boundary layer.[20] The boundary layer is laminar if the boundary Reynolds number $\left(u^*d/\nu\right)$ is less 1.5, and is turbulent if the boundary Reynolds number is greater than 3.5. The shear velocity, u^*, equals $u^* = \sqrt{\tau/\rho}$. The variables are defined as follows:

τ_c = critical bed shear stress, psf
C = bed shear stress coefficient
γ_s = specific weight of the particle, lb/ft^3
γ = specific weight of the fluid, lb/ft^3
d = grain diameter of the sediment particle, ft
θ = angle of repose, degrees

u^* = shear velocity, fps

ν = kinematic viscosity, cm^2/sec

ρ = fluid density, g/cm^3

The angle of repose typically varies from 24° to 45° depending on the sediment type.

6. Equation B8 is applicable for noncohesive sediments and should be similarly applicable for unconsolidated sediment deposits. The equation can be incorporated into a model for determining the required ponding depth by relating the critical shear stress to a critical mean velocity by

$$\bar{u}_c = \sqrt{\frac{8}{f}} \sqrt{\frac{\tau_c}{\rho}} \tag{B9}$$

where

$\bar{u}_c$ = critical mean velocity, fps

f = Darcy-Weisbach friction factor

τ_c = critical shear stress, psf

ρ = fluid density, g/cm^3

The friction factor, f, is a function of the boundary roughness and the depth. It generally ranges from 0.02 to 0.05. The depth is then iteratively determined by using the continuity equation and the mean velocity equation.

$$\bar{u} = \frac{Q}{BD} = \sqrt{\frac{8}{f}} \sqrt{\frac{\tau}{\rho}} \tag{B10}$$

where

$\bar{u}$ = mean velocity, fps

Q = flow rate, cfs

B = length of weir, ft

D = depth, ft

τ = bed shear stress, psf

The equation has been verified experimentally for sediments, primarily sands, but never on dredged material that may become cohesive during consolidation. This model gives criteria for scour prevention throughout

the basin. It does not consider any effects of stratification and assumes that the velocity distribution is generated by boundary shear.

7. The second velocity distribution available was the von Karman-Prandtl velocity deficiency law. This is the most commonly applied velocity profile in the field of sediment transport. The velocity distribution equation is:

$$\frac{u}{u^*} = 5.75 \log\left(\frac{y}{k}\right) + C \qquad \text{(B11)}$$

C equals 5.5 for laminar flow and 8.5 for turbulent flow. Integrating over the depth, the mean velocity equals

$$\bar{u} = 5.75\, u^* \log\left(\frac{D}{k}\right) + 6u^* \qquad \text{(B12)}$$

and therefore by subtracting the mean velocity from the velocity distribution equation, the profile equals

$$u = 2.5u^*\left(1 + \log\frac{y}{D}\right) + \bar{u} \qquad \text{(B13)}$$

As described earlier, the mean velocity $\bar{u}$ equals $Q/BD = u^*\sqrt{8/f}$ and the shear velocity u^* equals $\sqrt{\tau/\rho}$. The profile is based on boundary-generated shear and does not consider the convective inertia effects generated by the weir or the density-stratification effects. The profile has been widely used in open channel flow but never for weir flow. The equation can be incorporated with any of the models for ponding depth to determine the effluent suspended solids concentration.

8. The final velocity distribution investigated was Prandtl's one-seventh power law for turbulent flow. The profile is an empirical fit of the following form:

$$\frac{u}{u_{max}} = (1 - y/D)^{1/7} \qquad \text{(B14)}$$

where

u = velocity at depth y, fps

u_{max} = maximum velocity located at the surface, fps

y = depth of velocity u , ft

D = total depth, ft

This equation is empirical and could be fitted with a different exponent to match the data. Similarly, it could be modified to account for the case where the maximum velocity is located below the surface. The equation is widely accepted for turbulent flow in pipes or over flat plates.

APPENDIX C: NOTATION

B	Effective weir length, ft
c(y)	Suspended solids concentration profile, g/ℓ
C	Bed shear stress coefficient
C_D	Weir discharge coefficient
d	Depth in Debler's model, ft; also grain diameter of sediment particle, ft
D	Withdrawal depth, ft
f	Darcy-Weisbach friction factor
F	Densimetric Froude number
g	Gravitational constant, 32.2 ft/sec^2
h	Depth of withdrawal in Huber's and Debler's models, ft; also, depth of flow over the weir, ft
H	Static head, ft
H_w	Static head for free flow or depth of flow over the weir for submerged flow, ft; also, static head in WES selective withdrawal model, ft
k	Roughness height of the bed, ft
L	Available effective basin length, ft
L*	Critical effective basin length, ft
n	Coefficient for weir type in WES selective withdrawal model; also, number of effective sides for shaft-type weir
q	Unit flow rate, cfs/ft
q_c	Critical unit flow, cfs/ft
Q	Flow rate or weir discharge, cfs
Q/B	Weir loading or unit flow rate, cfs/ft
S	Length of a side of a shaft-type weir, ft
SS	Effluent suspended solids concentration, g/ℓ
Sal	Salinity of the water, ppt
S.G.	Specific gravity of the dredged material
S.S. Conc.	Suspended solids concentration of a given location or sample, g/ℓ
T	Thickness of weir, ft
u	Velocity at any point, y , in the profile, fps
u_{max}	Maximum velocity, fps

$\bar{u}$	Mean velocity, fps
$\bar{u}_c$	Critical mean velocity, fps
u^*	Shear velocity, fps
v	Velocity at any point in the profile, fps
$v(y)$	Velocity profile, fps
v_1	Local velocity in the zone of withdrawal at a distance y_1 below the elevation of the maximum velocity V, fps
v_2	Local velocity in the zone of withdrawal at a distance y_2 above the elevation of the maximum velocity V, fps
V	Maximum velocity in the zone of withdrawal, fps
V_{max}	Maximum velocity in the velocity profile, fps
V_w	Average velocity over the weir, fps
Wt. H_2O	Weight of water in a sample, g
Wt. Solids	Weight of solids in a sample, g
Wt. Total	Total weight of a sample, g
y	Distance of any point from the surface corresponding to point of velocity, u or v, in the velocity profile, ft
y_o	Ponding depth, ft
y_1	Depth of flow of the upper layer over the weir in the Wood and Lai model, ft; also, vertical distance from the elevation of the maximum velocity V to that of the corresponding local velocity v_1 in WES selective withdrawal model, ft
y_2	Vertical distance from the elevation of the maximum velocity V to that of the corresponding local velocity v_2, ft
Y_t	Static head on the weir for Wood and Lai model, ft
Y_1	Thickness of the upper layer in the Wood and Lai model, ft; also, vertical distance from the elevation of the maximum velocity V to the lower limit of the zone of withdrawal in WES selective withdrawal model, ft
Y_2	Vertical distance from the elevation of the maximum velocity V to the upper limit of the zone of withdrawal in WES selective withdrawal model, ft
Z_o	Vertical distance from the elevation of the weir crest to the lower limit of withdrawal, ft
α	Dimensionless density difference ratio in Wood and Lai's model; also, slope of the interface of the settled dredged material

γ	Specific weight of water, lb/ft^3; also, specific weight of the dredged material, lb/ft^3
γ_s	Specific weight of the particle, lb/ft^3
$\Delta\rho$	Density difference between two fluid layers, g/cm^3
$\Delta\rho_1$	Density difference of fluid between the elevations of the maximum velocity V and the corresponding local velocity v_1, g/cm^3
$\Delta\rho_2$	Density difference of fluid between the elevations of the maximum velocity V and the corresponding local velocity v_2, g/cm^3
$\Delta\rho_w$	Density difference of fluid between the elevations of the weir crest and the lower limit of the zone of withdrawal, g/cm^3
$\Delta\rho_{1m}$	Density difference of fluid between the elevations of the maximum velocity V and the lower limit of the zone of withdrawal, g/cm^3
$\Delta\rho_{2m}$	Density difference of fluid between the elevations of the maximum velocity V and the upper limit of the zone of withdrawal, g/cm^3
ε	Linear density stratification slope, ft^{-1}
θ	Angle of repose, deg
ν	Kinematic viscosity, cm^2/sec
ρ	Density of fluid, g/cm^3
ρ_1	Density of fluid in the upper layer, g/cm^3
ρ_s	Density of sample, g/cm^3
ρ_w	Density of fluid at weir crest, g/cm^3
ρ_F	Density of filtered fluid, g/cm^3
τ	Bed shear stress, psf
τ_c	Critical bed shear stress, psf
%S	Percent solids by weight
%SS	Percent suspended solids by weight

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Walski, Thomas M
Weir design to maintain effluent quality from dredged material containment areas / by Thomas M. Walski, Paul R. Schroeder. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1978.
94, [12] p. : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; D-78-18)
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