CHAPTER 4

CONTAINMENT AREA DESIGN FOR RETENTION OF SOLIDS AND INITIAL STORAGE

4-1. General.

a. This chapter presents guidelines for designing a new containment area for suspended solids retention and for evaluating the suspended solids retention potential of an existing containment area. Intermittent dredging, with higher costs, may be required if dredging flow rates exceed the solids retention capacity of a disposal area. This condition can be avoided by following the design guidelines in this chapter. The focus in this section is on fine-grained dredged-material. Guidelines presented here will provide the necessary guidance for designing a containment area for adequate space and volume for retaining the solids within the containment area through settling and providing storage capacity of dredged solids for a single dredged material disposal operation. The major objective is to provide solids removal by the process of gravity settling to a level that permits discharge of the transporting water from the area. Although ponding is not feasible over the entire surface area of many sites, an adequate ponding depth must be maintained over the design surface area as determined by these design procedures to assure adequate retention of solids. Guidance is also presented in this chapter for the design of weirs for the release of ponded water and for chemical clarification systems for additional removal of suspended solids.

b. The design procedures presented here are for gravity settling of dredged solids. However, the process of gravity sedimentation will not completely remove the suspended solids from the containment area effluent since wind and other factors resuspend solids and increase effluent solids concentration. The settling process, with proper design and operation, will normally provide removal of fine-grained dredged material down to a level of 1 to 2 grams per litre in the effluent for freshwater conditions. The settling process will usually provide removal of fine-grained dredged material down to a level of several hundred milligrams per litre or lower for saltwater conditions. If the required effluent standard is not met by gravity settling, the designer must provide for additional treatment of the effluent, e.g., flocculation or filtration.

c. The generalized flowchart shown in Figure 4-1 illustrates the design procedures presented in the following paragraphs. These steps were adapted from procedures used in water and wastewater treatment and are based on field and laboratory investigations on sediments and dredged material at active dredged material containment areas. The procedures in this chapter are presented in the manner required to calculate the minimum required disposal area geometry for a given inflow rate (dredge size) and dredged volume. The same procedures would be used in reverse fashion to calculate a maximum flow rate (dredge size) allowable for a given disposal area geometry. Numerical examples of both approaches are presented in Appendix C. Procedures for computer-assisted design for sedimentation and initial storage are available as discussed in Chapter 8.

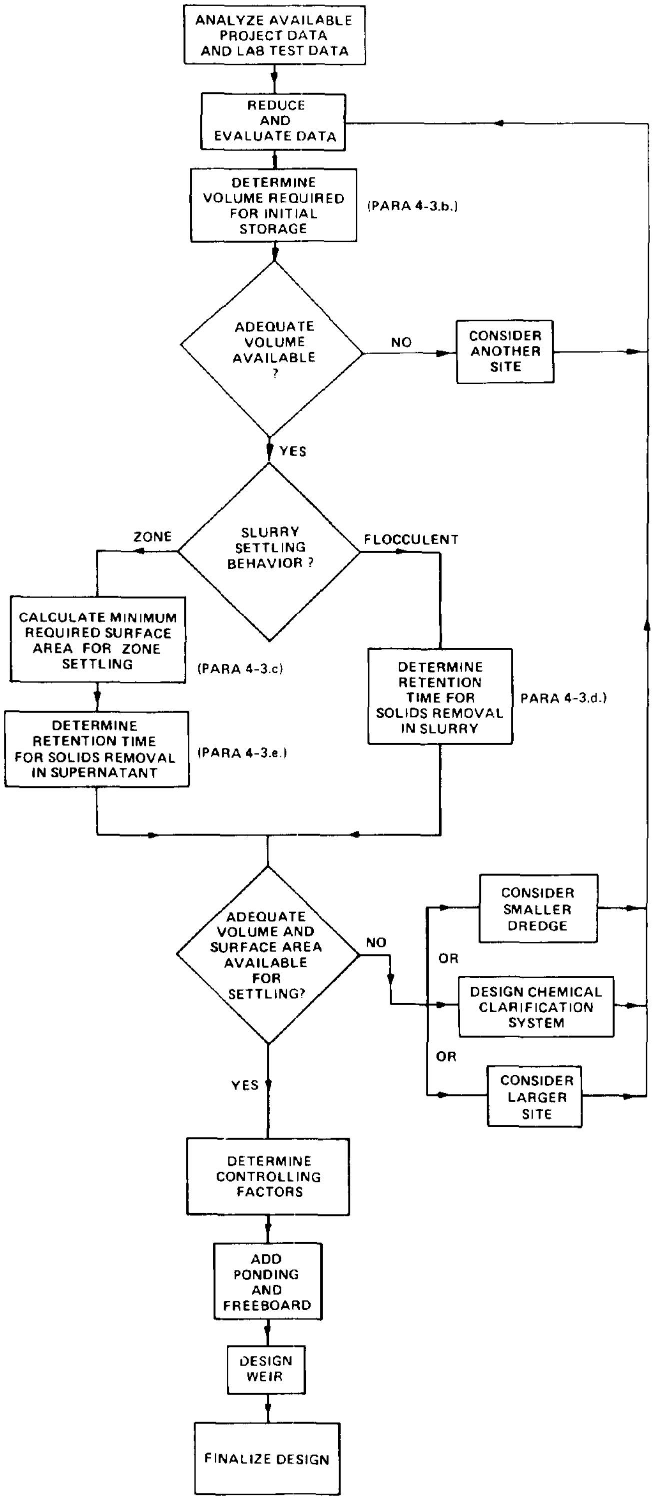


Figure 4-1. Flowchart of design procedure for settling and initial storage

4-2. Data Requirements.

a. General. The data required to use the design guidelines are obtained from field investigations (Chapter 2), laboratory testing (Chapter 3), project-specific operational constraints, and experience in dredging and disposal activities. The types of data required are described in the following paragraphs.

b. In Situ Sediment Volume. The initial step in any dredging activity is to estimate the total in situ channel volume of sediment to be dredged V_c . Sediment quantities are usually determined from routine channel surveys.

c. Physical Characteristics of Sediments. Field sampling and sediment characterization should be accomplished according to the laboratory tests described in Chapters 2 and 3 of this manual. Adequate sample coverage is required to provide representative samples of the sediment. Also required are in situ water contents of the fine-grained maintenance sediments. Care must be taken in sampling to ensure that the water contents are representative of the in situ conditions. Water contents of representative samples w are used to determine the in situ void ratios e_i as follows:

$$e_i = \frac{wG_s}{S_D} \tag{4-1}$$

where

e_i = in situ void ratio of sediment

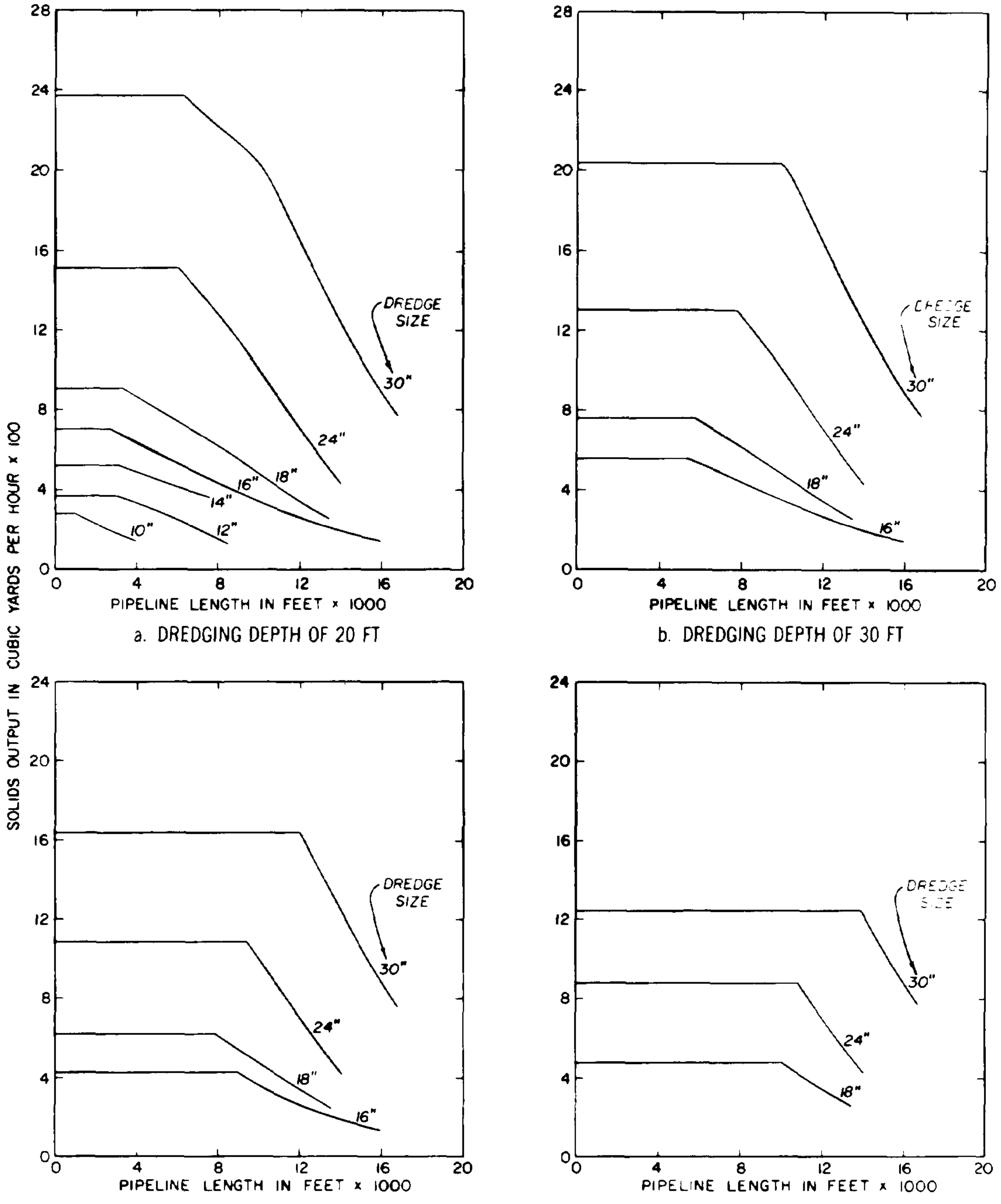
w = water content of the sample, percent

G = specific gravity of sediment solids

S_D = degree of saturation, percent (equal to 100 percent for sediments)

A representative value for in situ void ratios is used later to estimate volume for the containment area. Grain size analyses are used to estimate the quantities of coarse- and fine-grained material in the sediment to be dredged. The volume of sand V_{sd} can be estimated as a percentage of the total volume V_c to be dredged by using the percent coarser than No. 200 sieve. The in situ volume of fine-grained sediment V_i is equal to $V_c - V_{sd}$.

d. Proposed Dredging and Disposal Data. The designer must obtain and analyze data concerning the dredged material disposal rate. For hydraulic pipeline dredges, the type and size of dredge(s) to be used, average distance to containment area from dredging activity, depth of dredging, and average solids concentration of dredged material when discharged into the containment area must be considered. If the size of the dredge to be used is not known, the largest dredge size that might be expected to perform the dredging should be assumed. The time required for the dredging can be estimated, based on experience. If no data on past experience are available, Figure 4-2, which shows the relationship among solids output, dredge size, and pipeline length for various dredging depths, should be used. It was developed from data provided for Ellicott dredges for dredging in sand (item 32). Additional guidance on dredge production rates is found in ER 1110-2-1300. For hopper dredge



a. DREDGING DEPTH OF 20 FT

b. DREDGING DEPTH OF 30 FT

c. DREDGING DEPTH OF 40 FT

d. DREDGING DEPTH OF 50 FT

Figure 4-2. Relationships among solids output, dredge size, and pipeline length for various dredging depths or barge pump-out operations, an equivalent disposal rate must be estimated based on hopper or barge pump-out rate and travel time involved. Based on these data, the designer must estimate or determine containment area influent rate, influent suspended solids concentration, effluent rate (for weir sizing), and time required to complete the disposal activity. For hydraulic pipeline dredges,-if no other data are available, an influent suspended solids concentration C_i of 150 grams per litre (14 percent by weight) should be used for design purposes. The influent flow rate Q_i can be estimated using the following tabulation or from other available data:

Discharge Pipeline Diameter, in.	Discharge Rate (for Flow Velocity of 15 ft/sec)*	
	cfs	gal/min
8	5.3	2,350
10	8.1	3,640
12	11.8	5,260
14	16.0	7,160
16	20.6	9,230
18	26.5	11,860
20	32.7	14,660
24	47.1	21,090
27	59.5	26,630
28	64.1	28,700
30	73.6	32,950
36	106.0	47,500

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* To obtain discharge rates for other velocities, multiply the discharge rate shown in this tabulation by the desired velocity and divide by 15.

e. Laboratory Settling Test Data. The guidelines for sedimentation tests are given in Section 3-3. Depending on the results of the sedimentation tests, the dredged material slurry will settle by either zone processes (common for saltwater sediments) or flocculent processes (common for freshwater sediments). Regardless of the salinity, flocculent processes govern the concentration of solids in the effluent.

4-3. Sedimentation Basin Design.

a. Selection of Minimum Average Ponding Depth. Before a disposal site can be designed for effective settling or before the required disposal area geometry can be finalized, a ponding depth H_{pd} during disposal must be assumed. The design procedures in the following paragraphs call for an average ponding depth in estimating the residence time necessary for effective settling. A minimum average ponding depth of 2 feet should be used for the design. If the design objective is to minimize the surface area required, selection of a deeper ponding depth may be desirable. If conditions will allow for the greater ponding depth throughout the operation, the greater value can be used. For most cases, constant ponding depth can be maintained by raising the pond surface as settled material accumulates in the containment area by raising the elevation of the weir crest.

b. Calculation of Volume for Initial Storage.

(1) General. Containment areas must be designed to meet volume requirements for a particular disposal activity. The total volume required in a containment area includes volume for storage of dredged material, volume for sedimentation (ponding depths), and freeboard volume (volume above water surface). Volume required for storage of the coarse-grained (>No. 200 sieve) material must be determined separately since this material behaves independently of the fine-grained (<No. 200 sieve) material.

(2) Calculation of design concentration. The design concentration C_d is defined as the average concentration of the dredged material in the containment area at the end of the disposal activity and is estimated from the compression (15-day) settling test described in Chapter 3. This design parameter is required both for estimating initial storage requirements and for determining minimum required surface areas for effective zone settling. The following steps can be used to estimate C_d from the compression settling test.

(a) Estimate the time of dredging by dividing the dredge production rate into the volume of sediment to be dredged. Use Figure 4-2 for estimating the dredge production rate if no specific data are available from past dredging activities. (Note that curves in Figure 4-2 were developed for sand.) The total time required for dredging should allow for anticipated downtime.

(b) Enter the concentration versus time plot as shown in Figure 4-3 and determine the concentration at a time t equal to one-half the time required for the disposal activity determined in step (a).

(c) The value computed in step (b) is the design solids concentration C_d. Examples are shown in Appendix C.

(3) Volume estimation. The volume computed in the following steps is the volume occupied by dredged material in the containment area immediately after the completion of a particular disposal activity. This value is critical in determining the dike height requirements for the containment area. The volume is not an estimate of the long-term needs for multiple-disposal activities. Estimates for long-term storage capacity can be made using the procedures outlined in Chapter 5. The design for initial storage may be a controlling factor regardless of the settling behavior exhibited by the material. If the material initially exhibits compression settling at the expected inflow concentration, the design for initial storage is the only consideration (this is expected to be an exceptional case).

(a) Compute the average void ratio of the fine-grained dredged material in the containment area at the completion of the dredging operation using the

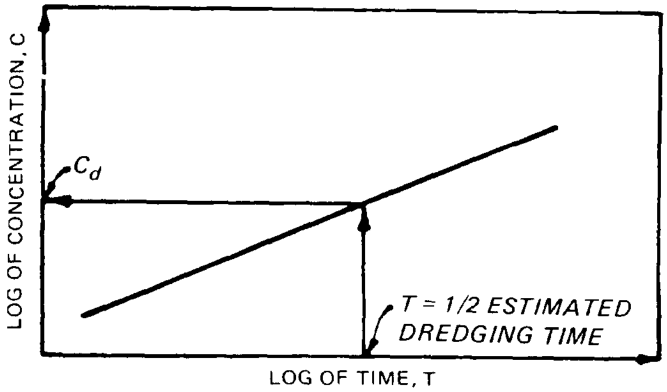

Figure 4-3. Conceptual time versus concentration plot

design concentration C_d determined in 4-3.b. Use the following equation to determine the void ratio:

$$e_o = \frac{G_s \gamma_w}{C_d} - 1 \qquad (4\text{-}2)$$

where

e_o = average void ratio of the dredged material in the containment area at the completion of the dredging operation
γ_w = density of water, grams per litre (normally 1,000 grams per litre)

(b) Compute the volume of the fine-grained channel sediments after disposal in the containment area:

$$V_f = V_i \left[\frac{e_o - e_i}{1 + e_i} \right] + 1 \qquad (4\text{-}3)$$

where

V_f = volume of the fine-grained dredged material after disposal in the containment area, cubic feet

V_i = volume of the fine-grained channel sediments, cubic feet

e_i = average void ratio of the in situ channel sediments

(c) Compute the volume required to store the dredged material in the containment area:

$$V = V_f + V_{sd} \qquad (4\text{-}4a)$$

where

V = total volume of the dredged material in the containment area at the end of the dredging operation, cubic feet

V_{sd} = volume of sand (use 1:1 ratio), cubic feet

(d) If there are limitations on the surface area available for disposal or if an existing disposal site is being evaluated, check whether the site conditions will allow for initial storage of the volume to be dredged. First determine the maximum height at which the material can be placed $H_{dm(max)}$ using the following equation:

$$H_{dm(max)} = H_{dk(max)} - H_{pd} - H_{fb} \tag{4-4b}$$

where

$H_{dk(max)}$ = maximum allowable dike height due to foundation conditions, feet

H_{pd} = ponding depth, feet

H_{fb} = freeboard (minimum of 2 feet can be assumed), feet

Compute the minimum surface area that could be used to store the material:

$$A_{ds} = \frac{V}{H_{dm(max)}\ (43,560)} \tag{4-4c}$$

where

A_{ds} = design surface area for storage, acres

If A_{ds} is less than the available surface area, then adequate volumetric storage is available at the site.

c. Calculation of Minimum Surface Area for Effective Zone Settling.

(1) General. If the sediment slurry exhibited zone settling behavior at the expected inflow concentration, the zone settling test results are used to calculate a minimum required ponded surface area in the containment for effective zone settling to occur. The method is generally applicable to dredged material from a saltwater environment, but the method can also be used for freshwater dredged material if the laboratory settling tests indicate that zone settling describes the initial settling process. Additional calculations using flocculent settling data for the solids remaining in the ponded supernatant water are required for designing the containment area to meet a specific effluent quality standard for suspended solids.

(2) Compute area required for zone settling. The minimum surface area determined according to the following steps should provide removal of fine-grained sediments so that suspended solids levels in the effluent do not exceed several hundred milligrams per litre. The area is required for the zone settling process to remove suspended solids from the surface layers at

the rate sufficient to form and maintain a clarified supernatant that can be discharged.

(a) Determine the zone settling velocity v_s at the influent suspended solids concentration C_i as described in paragraph 3-3.d.

(b) Compute area requirements as

$$A_z = \frac{Q_i (3600)}{V_s} \tag{4-5}$$

where

A_z = containment surface area requirement for zone settling, square feet
Q_i = influent flowrate in ft^3/sec
3600 = conversion factor hours to seconds
V_s = zone settling velocity at influent solids concentration C_i, feet per hour

(c) Multiply the area by a hydraulic efficiency correction factor HECF to compensate for containment area inefficiencies:

$$A_{dz} = \frac{(HECF)A_z}{43{,}560} \tag{4-6}$$

where

A_{dz} = design basin surface area for effective zone settling, acres
HECF = hydraulic efficiency correction factor (determined as described in 4-3.g.)
A_Z = area determined from Equation 4-5, square feet

d. Calculation of Required Retention for Flocculent Settling.

(1) Sediments dredged from a freshwater environment normally exhibit flocculent settling properties. However, in some cases, the concentration of dredged material slurry is sufficiently high that zone settling will occur. The method of settling can be determined from the laboratory tests.

(2) Sediments in a dredged material containment area are composed of a broad range of particle floc sizes and surface characteristics. In the containment area, larger particle flocs settle at faster rates, thus overtaking finer flocs in their descent. This contact increases the floc sizes and enhances settling rates. The greater the ponding depth in the containment area, the greater is the opportunity for contact among sediments and flocs. Therefore, flocculent settling of dredged sediments is dependent on the ponding depth as well as the properties of the particles. For this reason, it is important that settling tests be performed with column heights corresponding to ponding depths expected under field conditions.

(3) The concentration of suspended solids in the effluent will depend on the total depth at which fluid is withdrawn at the weir, which is related to the hydraulic characteristics of the weir structure. The depth of withdrawal is equivalent to the depth of ponded water for weir configuration and flow rates that are normally encountered in containment areas. For this reason, the term "ponding depth" is used interchangeably with withdrawal zone in this manual in the context of effluent quality evaluations.

(4) Evaluation of the sedimentation characteristics of a sediment slurry exhibiting flocculent settling is accomplished as discussed in Chapter 3. The design steps to determine the required retention time for a desired effluent quality are as follows:

(a) Calculate the removal percentage at the selected minimum average ponding depth H_{pd} for various times using the concentration profile plot as shown in Figure 3-5. As an example, the removal percentage for H_{pd} = depth d_2 and time t_2 is computed as follows:

$$R = \frac{\text{Area right of profile}}{\text{Area total}} (100) = \frac{\text{Area } 0,\ 1,\ 2,\ 3,\ 0^*}{\text{Area } 0,\ 1,\ 2,\ 4,\ 0} (100) \qquad (4\text{-}5)$$

where R is the removal percentage. Determine these areas by either planimetering the plot or by direct graphical measurements and calculations. This approach is used to calculate removal percentages for the selected ponding depth as a function of time.

(b) Plot the solids removal percentages versus time as shown in Figure 4-4.

(c) Mean detention times can be selected from Figure 4-4 for various solids removal percentages. Select the residence time T_d that gives the desired removal percentage.

(d) The required mean residence time T_d should be multiplied by an appropriate hydraulic efficiency correction factor HECF to compensate for the fact that containment areas, because of inefficiencies, have field mean detention times less than theoretical (volumetric) detention times. The HECF is determined as described in 4-3.g. The basin volumetric or theoretical residence time is estimated as follows:

$$T = \text{HECF } T_d \qquad (4\text{-}8)$$

where T is the volumetric or theoretical residence time and T_d is selected from Figure 4-4.

(e) Note that for the case of flocculent settling of the entire slurry mass, the solids will be removed by gravity sedimentation to a level of 1 to

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* These numbers correspond to the numbers used in Figure 3-5 to indicate the area boundaries for the total area down to depth d_2 and the area to the right of the line for t_2 .

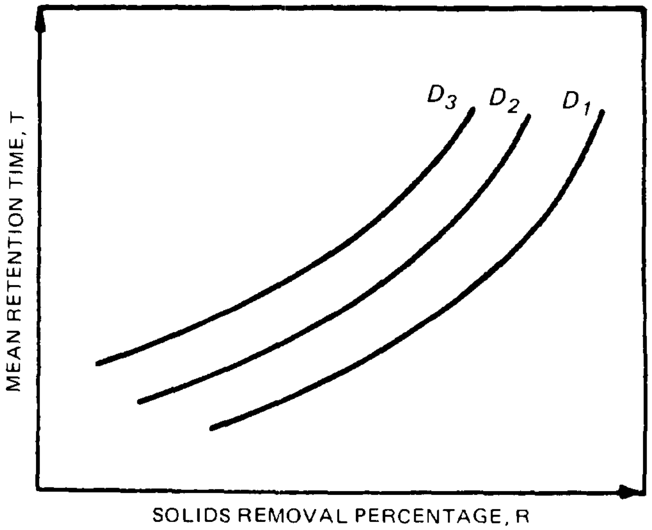


Figure 4-4. Conceptual plot of solids removal versus time for slurries exhibiting flocculent settling

2 grams per litre. For this case, the selection of a required residence time for a percentage removal is more convenient. For the case of flocculent settling in the supernatant water, where the slurry mass is undergoing zone settling, selection of a required residence time for an effluent suspended solids standard is more appropriate. Examples are shown in Appendix C.

e. Calculation of Required Retention Time for Flocculent Settling in Supernatant Water.

(1) Data analyses. For slurries exhibiting zone settling, flocculent settling behavior describes the process occurring in the supernatant water above the interface. Therefore, a flocculent data analysis procedure as outlined in the following paragraphs is required. The steps in the data analysis are as follows:

(a) Use the concentration profile diagram as shown in Figure 3-5 to graphically determine percentages removed, R , for the various time intervals and for the minimum ponding depth. This is done by graphically determining the areas to the right of each concentration profile and its ratio to the total area above the depth as described for the case of flocculent settling above.

(b) Compute the percentages remaining as follows:

$$P = 100 - R \qquad (4\text{-}9)$$

(c) Compute values for the average suspended solids concentration in the supernatant at each time of extraction as follows:

$$C_t = P_t C_o \qquad (4\text{-}10)$$

where

C_t = suspended solids concentration at time t , milligrams per litre

P_t = percentage remaining at time t

C_o = initial concentration in the supernatant, milligrams per litre

(d) Tabulate the data and plot a relationship for suspended solids concentration versus time using the value for each time of extraction as shown in Figure 4-5. An exponential curve fitted through the data points is recommended.

(2) Determination of retention time to meet an effluent suspended solids concentration. The relationship of supernatant suspended solids versus time developed from the column settling test is based on quiescent settling conditions found in the laboratory. The anticipated retention time in an existing disposal area under consideration can be used to determine a predicted suspended solids concentration from the relationship. This predicted value can be considered a minimum value able to be achieved in the field, assuming little or no resuspension of settled material. The relationship in Figure 4-5 can also be used to determine the required retention time to meet a standard for effluent suspended solids. For dredged material slurries exhibiting flocculent settling behavior, the concentration of particles in the ponded water is 1 gram per litre or higher. The resuspension resulting from normal wind conditions will not significantly increase this concentration; therefore, an adjustment for resuspension is not required for the flocculent settling case. However, an adjustment for anticipated resuspension is appropriate for dredged material exhibiting zone settling. The minimum expected value and the value adjusted for resuspension would provide a range of anticipated suspended solids concentrations in the effluent. The following procedure should be used:

(a) A standard for effluent suspended solids C_{eff} must be met considering anticipated resuspension under field conditions. A corresponding maximum concentration under quiescent laboratory conditions is calculated as:

$$C_{col} = \frac{C_{eff}}{RF} \tag{4-11}$$

where

C_{col} = Maximum suspended solids concentration of effluent as estimated from column settling tests, milligrams suspended solids per litre of water

C_{eff} = Suspended solids concentration of effluent considering anticipated resuspension, milligrams suspended solids per litre of water

RF = Resuspension factor selected from Table 4-1

Table 4-1 summarizes recommended resuspension factors based on comparisons of suspended solids concentrations as predicted from column settling tests and field data from a number of sites with varying site conditions.

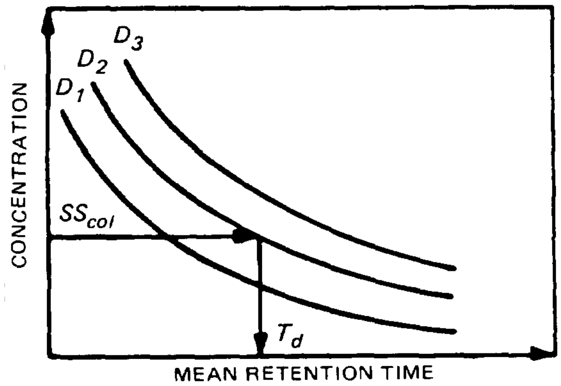


Figure 4-5. Conceptual plot of supernatant suspended solids concentration versus time from column settling test

Table 4-1

Recommended Resuspension Factors for the Zone Settling Case for Various Ponded Areas and Depths

Anticipated Ponded Area	Anticipated Average Ponded Depth less than 2 feet	 2 feet or greater
Less than 100 acres	2.0	1.5
Greater than 100 acres	2.5	2.0

(b) Using Figure 4-5, determine the required minimum mean residence time corresponding to C_{col} .

(c) As in the case for flocculent settling of the entire slurry mass, the mean residence time should be increased by an appropriate hydraulic efficiency factor HECF using Equation 4-8. The resulting minimum volumetric or theoretical residence time T can be used to determine the required disposal area geometry.

f. Computation of Design Surface Area for Flocculent Settling. The design surface area for flocculent settling can be calculated as follows:

$$A_{df} = \frac{T\ Q_i}{H_{pd}\ (12.1)} \qquad (4\text{-}12)$$

where

A_{df} = design surface area for flocculent settling, acres

T = minimum mean residence time, hours

Q_i = average inflow rate, cubic feet per second

H_{pd} = average ponding depth, feet

12.1 = conversion factor acre-feet per cubic feet per second to hours

g. Estimation of Hydraulic Efficiency Correction Factor.

(1) Estimates of the field mean retention time for expected operational conditions are required for prediction of suspended solids concentrations in the effluent. Estimates of the retention time must consider the hydraulic efficiency of the disposal area, defined as the ratio of mean retention time to theoretical retention time, Field mean retention time T_d for given flow rate and ponding conditions and the theoretical residence time T are related by a hydraulic efficiency correction factor as follows:

$$T_d = \frac{T}{(HECF)} \tag{4-13}$$

where

T_d = mean residence time, hours

T = theoretical residence time, hours

HECF = hydraulic efficiency correction factor (HECF > 1.0) defined as the inverse of the hydraulic efficiency, T_d/T .

(2) The hydraulic efficiency correction factor HECF can be estimated by several methods. The most accurate estimate is that made from dye tracer studies to determine T_d at the actual site under operational conditions at a previous time, with the conditions similar to those for the operation under consideration (see Appendix J). This approach can be used only for existing sites.

(3) Alternatively, the ratio $T_d/T = 1/HECF$ can be estimated from the equation:

$$\frac{T_d}{T} = 0.9 \left[1 - \exp\left(-0.3 \frac{L}{W}\right)\right] \tag{4-14}$$

where L/W is the length-to-width ratio of the proposed basin. The L/W ratio can be increased greatly by the use of internal spur dikes, resulting in a higher hydraulic efficiency and a lower required total area.

h. Determination of Disposal Area Geometry. Previous calculations have provided minimum required surface area for storage A_{ds} , a minimum required surface area for zone settling (if applicable) A_{dz} , and a minimum required surface area for flocculent settling A_{df} . A ponding depth H_{pd} was also assumed. These values are then used, as described in the following paragraphs, to determine the required disposal area geometry. Throughout the design process, the existing topography of the containment area site must be considered since it can have a significant effect on the resulting geometry of the containment area. Any limitations on dike height should also be determined based on an appropriate geotechnical evaluation of dike stability (See Chapter 6).

(1) Select the design surface area. Select the design surface area A_d as the largest of A_{ds} , A_{dz} , and/or A_{df} . If A_d exceeds the real estate available for disposal, consider a smaller flow rate (dredge size), deeper average ponding depth, chemical clarification, or an alternate site, and repeat the design. If the surface area for an existing site exceeds A_d , the existing surface area may be used for A_d .

(2) Compute height of the dredged material and dikes. The following procedure should be used:

(a) Estimate the thickness of the dredged material at the end of the disposal operation:

$$H_{dm} = \frac{V}{A_d} \tag{4-15}$$

where

H_{dm} = thickness of the dredged material layer at the end of the dredging operation, feet

V = volume of dredged material in the basin, cubic feet (from Equation 4-4)

A_d = design surface area, square feet (as determined above)

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

$$H_{dk} = H_{dm} + H_{pd} + H_{fb} \tag{4-16}$$

where

H_{dk} = dike height, feet

H_{pd} = average ponding depth, feet (a minimum of 2 feet is recommended)

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

4-4. Weir Design and Operation. The purpose of the weir structure is to regulate the release of ponded water from the containment area. Proper weir design and operation can control resuspension and withdrawal of settled solids.

a. Guidelines for Weir Design.

(1) Weir design and containment sizing. Weir design is based on providing the capability for selective withdrawal of the clarified upper layer of ponded water. The weir design guidelines as developed in the following paragraphs are based on the assumptions that the design of the containment area has provided sufficient area and volume for sedimentation and that short-circuiting is not excessive.

(2) Effective weir length and ponding depth.

(a) Ponding depth and effective weir length are the two most important parameters in weir design. The weir design guidelines presented in this section allow evaluation of the trade-off involved between these parameters.

(b) In order to maintain acceptable effluent quality, the upper layers containing low levels of suspended solids should be ponded at depths greater than or equal to the minimum depth of the withdrawal zone, which will prevent scouring settled material. The withdrawal zone is the area through which fluid is removed for discharge over the weir as shown in Figure 4-6. The size of the withdrawal zone affects the approach velocity of flow toward the weir and is generally equal to the depth of ponding.

(c) The weir shape or configuration affects the dimensions of the withdrawal zone and consequently the approach velocity. Since weirs do not extend across an entire side of the containment area, flow concentrations of varying degree occur near the weir, resulting in higher local velocities and possible resuspension of solids. Longer effective weir lengths result in less concentration of flow. The minimum width through which the flow must pass may be termed the effective weir length L_e .

(d) The relationship between effective weir length and ponding depth necessary to discharge a given flow without significantly entraining settled material is illustrated by the nomograph in Figure 4-7.

(3) Design procedure. To design a new weir to meet a given effluent suspended solids level, the following procedure should be used:

(a) Select the appropriate operating line in the lower portion of the nomograph based on the governing settling behavior of the dredged material slurry (zone or flocculent).

(b) Construct horizontal lines at the design inflow rate Q_i and the ponding depth expected at the weir as shown in the key in Figure 4-7. This ponding depth may be larger than the average ponding depth for large containment areas as the result of a slope taken by the settling material. The ponding depth at the weir may be estimated by using the following equation:

$$H_{pd(weir)} = H_{pd} + 1/2\ Lps\ (0.001) \tag{4-17}$$

where

$H_{pd(weir)}$ = estimated ponding depth at the weir, feet

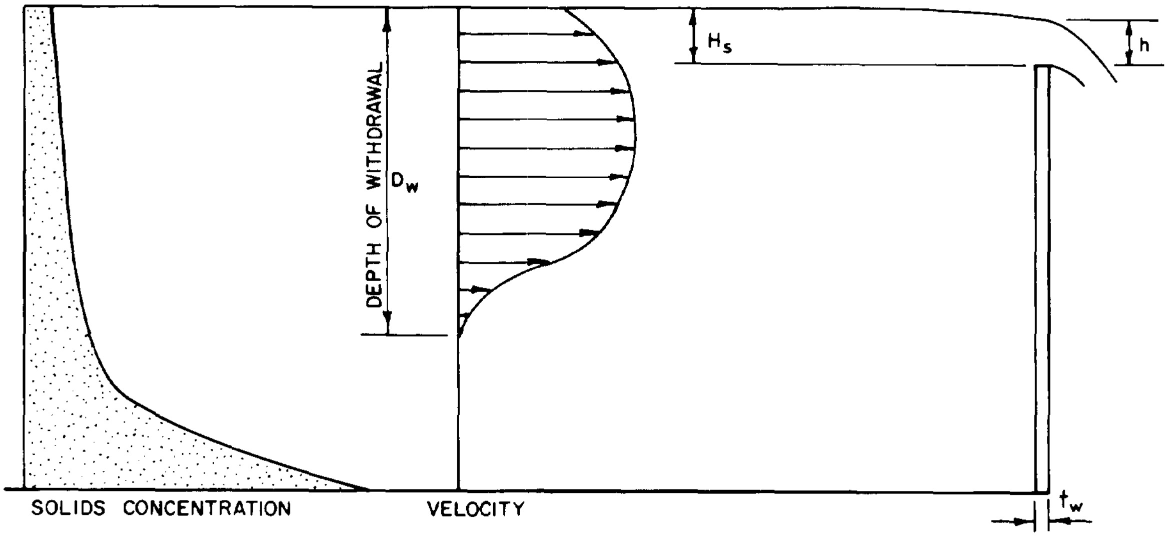


Figure 4-6. Conceptual illustration of withdrawal depth and velocity profile

H_{pd} = average ponding depth, feet

L_{ps} = length of ponded surface between inflow point and weir, feet

(c) Construct a vertical line from the point of intersection of the horizontal ponding depth line and the selected operating line of the nomogram. The required effective weir length is found at the intersection of the vertical line and the horizontal design flow line. An example is shown in the key in Figure 4-7.

(d) Determine the number of weir structures, the physical dimensions of each, and the locations, based on the weir type to be used and the configuration of the containment area. If a satisfactory balance between effective weir length and ponding depth cannot be achieved, intermittent operation or use of a smaller dredge may be required to prevent resuspension at the weir as the containment area is filled. An illustrative problem is given in Appendix C.

(4) Effect of weir type.

(a) Rectangular weirs. Rectangular weirs are the commonly used weir type and may consist of a rectangular wood- or metal-framed inlet(s) or half-cylindrical corrugated metal pipe riser(s). The effective weir length is equal to the actual weir crest length for rectangular weirs as illustrated in Figure 4-8a.

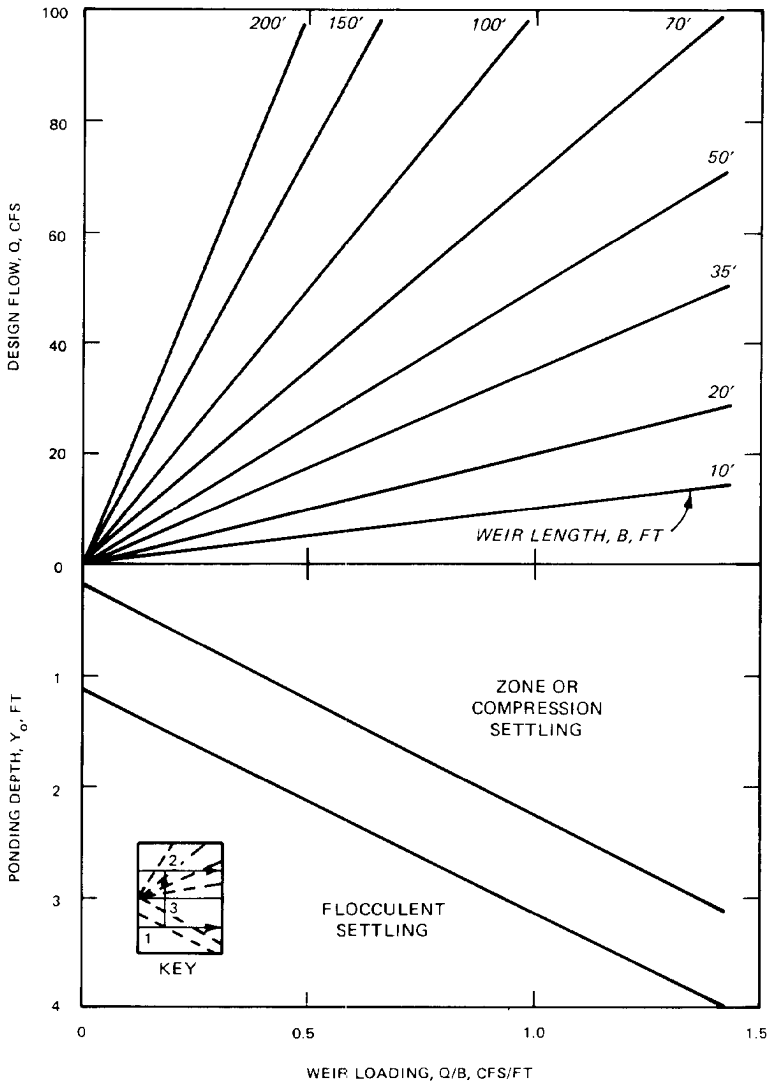
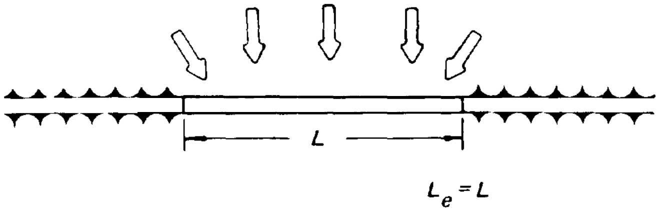


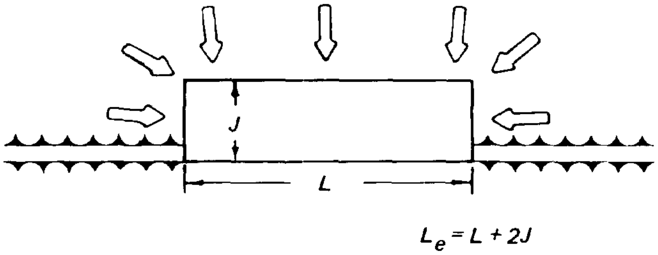
Figure 4-7. Weir design nomograph

(b) Jutting weirs. A modified form of the rectangular weir is the jutting weir (see Figure 4-8b). It is possible to achieve a greater effective weir length using a jutting weir since the effective length L_e equals L + 2J as shown in Figure 4-8b.

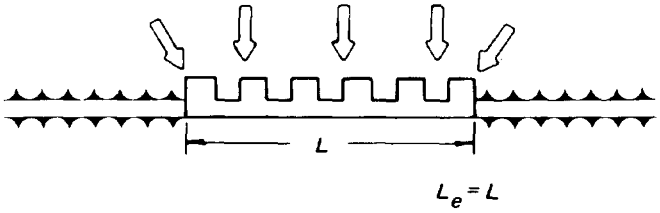
(c) Polygonal (labyrinth) weirs. Polygonal (labyrinth) weirs have been used to reduce the depth of flow over the weir. However, use of such weirs has little impact on effluent suspended solids concentrations since the controlling factor for the depth of withdrawal is usually not the flow over the weir but the approach velocity. Therefore, the approach velocity and the withdrawal depth for the rectangular weir in Figure 4-8a would be the same as that for the polygonal weir in Figure 4-8c since both weirs have the same effective length L_e , even though the total weir crest length for the polygonal weir is considerably greater. Use of polygonal weirs is not recommended



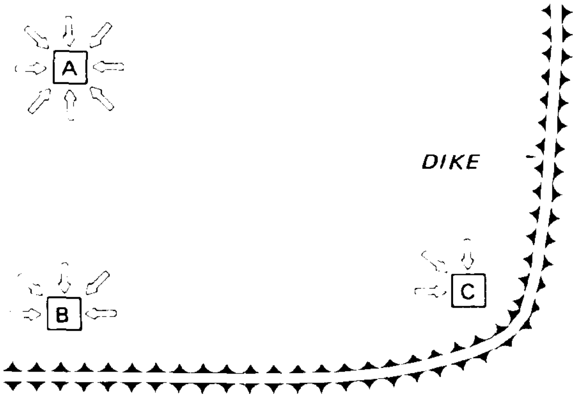
a. RECTANGULAR WEIR



b. JUTTING RECTANGULAR WEIR



c. POLYGONAL WEIR



d. SHAFT WEIRS

Figure 4-8. Effective lengths of various weir types because of the greater cost and the marginal improvement of effluent quality realized when using such a weir.

(d) Shaft-type weirs. In some cases, the outflow structure is a four-sided drop inlet or shaft located within the containment area as shown in Figure 4-8d. In evaluating the effective weir length for shaft-type weirs, the approach velocity is a 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 4-8d, location A is the most desirable since flow can approach from all sides (four effective sides). Location B is less desirable since flow can approach from only three directions (three effective sides). Location C is the least desirable since it has only two effective sides. Since effluent pipes must run from the shaft weir under the dike to the receiving stream, a location such as A in Figure 4-8 may not be optimal since it is far from the dike and will require a longer pipe than Location B.

(e) Converting weir length. To convert the weir length determined from the design nomographs to length L_s of a side of the square shaft weir, use the following formula:

$$L_s = \frac{L_e}{n} \tag{4-18}$$

where n is the number of effective sides of a shaft-type weir. A side is considered effective if it is at least 1.5 L_s feet away from the nearest dike, mounded area, or other dead zone. This distance is generally accepted as being sufficient to prevent the flow restriction caused by the flow contraction and bending due to the walls.

(5) Structural design. Weirs should be structurally designed to withstand anticipated loadings at maximum ponding elevations. Considerations should be given to uplift forces, potential settlement, access, corrosion protection, and potential piping beneath or around the weir. Additional information regarding structural design of weirs is found in WES TR D-77-9 (item 16). Outlet pipes for the weir structure must be designed to carry flows in excess of the flow rate for the largest dredge size expected. Larger flow capacity of the outlet pipes may be needed if an emergency release of ponded water is required.

b. Weir Operation.

(1) Weir boarding.

(a) Adequate ponding depth during the dredging operation is maintained by controlling the weir crest elevation. Weir crest elevations are usually controlled by placing boards within the weir structure. The board heights should range in size from 2 to 10 inches, and thickness should be sufficient to avoid excessive bending as the result of the pressure of the ponded water.

(b) Weir boarding should be determined based on the desired ponding elevation as the dredging operation progresses. Small boards (e.g., 2 inches) should be placed at the top of the weir in order to provide more flexibility in controlling ponding depth. Use of larger boards in this most critical area may result in increased effluent suspended solids concentrations as weir boards are manipulated during the operation. Figure 4-9 shows the recommended weir boarding used for a minimum ponding depth of 2 feet.

(2) Operational guidelines for weirs. Some basic guidelines for weir operation are given below:

(a) If the weir and the disposal site are properly designed, intermittent dredging operation should not become necessary unless the required ponding depth cannot be maintained.

(b) While the weir is in operation, floating debris should be periodically removed from the front of the weir to prevent larger withdrawal flows at greater depths.

(c) If multiple weirs or a weir with several sections is used in a basin, the crests of all weirs or weir sections should be maintained at equal elevations, in order to prevent local high velocities and resuspension in front of the weir with lower elevation.

(d) If the effluent suspended solids concentration increases above acceptable limits, the ponding depth should be increased by raising the elevation of the weir crest. However, if the weir crest is at the maximum ponding elevation and the effluent quality is still unacceptable, the flow into the basin should be decreased by operating intermittently.

(e) The weir may be controlled in the field by using the head over the weir as an operational parameter since the actual volumetric flow over the weir cannot easily be measured.

(3) Operating head. The static head with the related depth of flow over the weir is the best criterion now available for controlling weir operation in the field. Weirs utilized in containment areas can usually be considered sharp crested where the weir crest thickness t_w is less than two-thirds the depth of flow over the weir h as seen in Figure 4-6. The ratio of depth of flow over the weir to the static head h/H_s equals 0.85 for rectangular sharp-crested weirs. Other values for the ratio of depth of flow to static head for various weir configurations may be found in the Handbook of Applied Hydrology (item 7). The weir crest length L , static head H_s , and depth of flow over the weir h are related by the following equations for rectangular sharp-crested weirs:

$$H_s = \left[0.3 \frac{Q}{L}\right]^{2/3} \qquad (4\text{-}19)$$

and

$$h = 0.85 H_s \qquad (4\text{-}20)$$

where

H_S = static head above the weir crest, feet

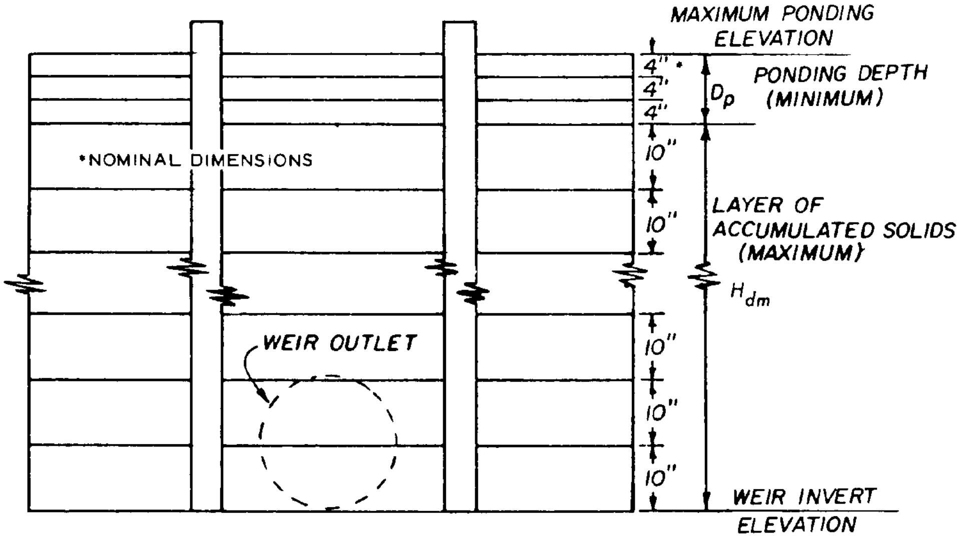


Figure 4-9. Recommended boarding configuration

Q = flow rate, cubic feet per second ($Q = Q_i = Q_e$ for continuous operation)

Q_e = clarified effluent rate, cubic feet per second

L = weir crest length, feet

h = depth of flow over the weir crest, feet

These relationships are shown graphically in Figure 4-10. If a given flow rate is to be maintained, Figure 4-10 can be used to determine the corresponding head and depth of flow. If the head in the basin exceeds this value, additional weir boards can be added, or the dredge can be operated intermittently until sufficient water is discharged to lower the head to an acceptable level. Since the depth of flow over the weir is directly proportional to the static head, it may be used as an operating parameter. The operator need not be concerned with head over the weir if effluent suspended solids concentrations are acceptable.

(4) Weir operation for undersized basins. If the basin is undersized and/or inefficient settling is occurring in the basin, added residence time and reduced approach velocities are needed to achieve efficient settling and to avoid resuspension, respectively. Added residence time can be obtained by raising the weir crest to its highest elevation to maximize the ponding depth or by operating the dredge intermittently. The residence time with intermittent dredging can be controlled by maintaining a maximum allowable static head or depth of flow over the weir based on the effluent quality achieved at various weir crest elevations.

(5) Weir operation for decanting, Once the dredging operation is completed, the ponded water must be removed to promote drying and consolidation of dredged material. Weir boards should be removed one row at a time to slowly decant the ponded water. Preferably, 2- by 4-inch boards should be located as described in previous paragraphs in order to minimize the withdrawal of settled solids. A row of boards should not be removed until the

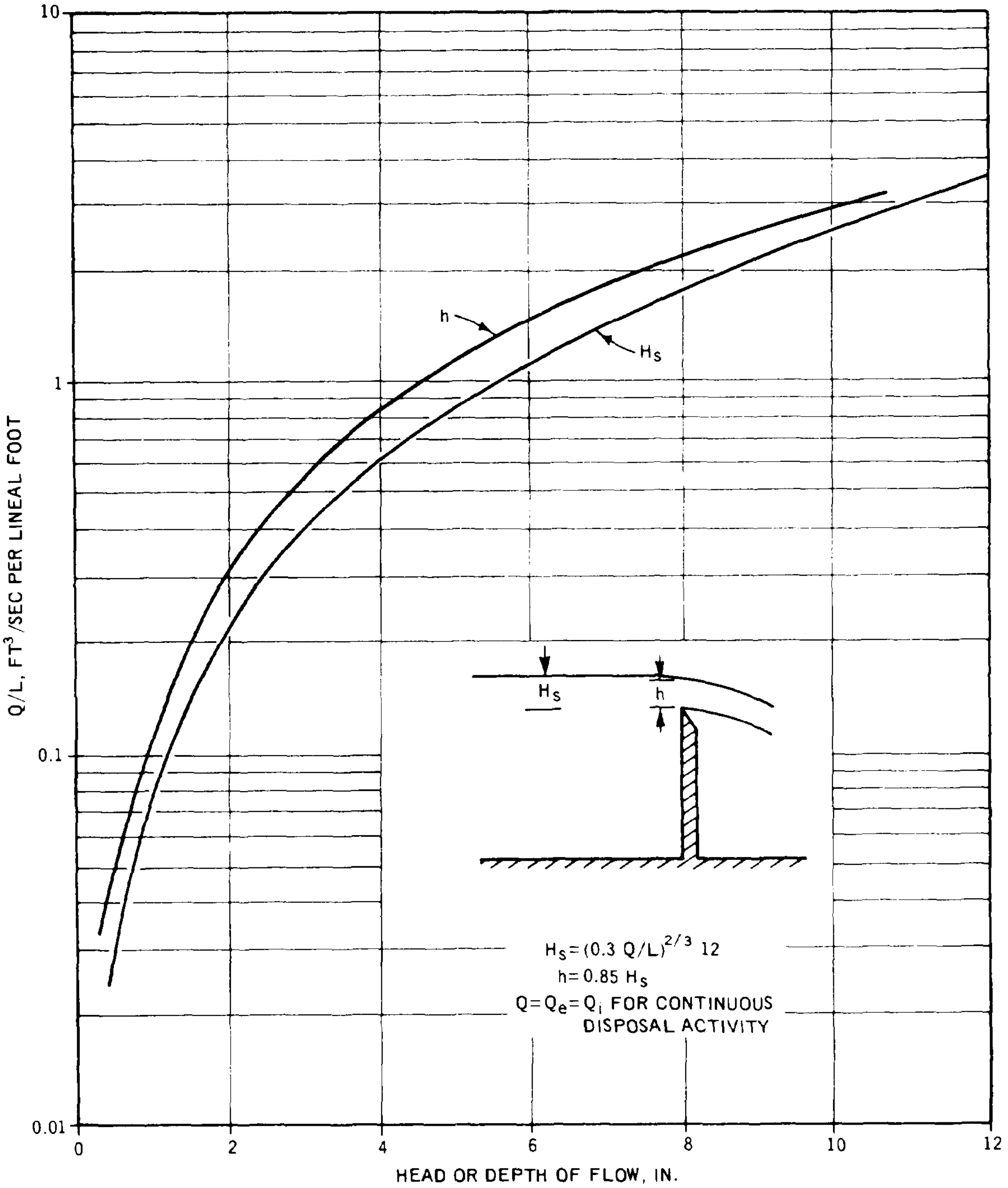


Figure 4-10. Relationship of flow rate, weir length, and head

water level is drawn down close to the weir crest and the outflow is low. This process should be continued until the decanting is completed. It is desirable to eventually remove the boards below the dredged material surface so that rainwater can drain from the area. These boards can be removed only after the material has consolidated sufficiently so that it will not flow from the basin. If it begins to do so, the boards should be replaced. In the final stages of decanting ponded water, notched boards may be placed in the weir, allowing low flow for slow removal of surface water.

4-5. Design of Chemical Clarification Systems. Pipeline injections of chemicals for clarification into the dredge inflow pipeline have shown only limited effectiveness and require much higher dosages of chemicals, This section therefore presents only the design procedures for chemical clarification of primary containment area effluents. The design is composed of three subsystems: the polymer feed system including storage, dilution, and injection; the weir and discharge culvert for mixing; and the secondary basin for settling and storage. The treatment system should be designed to minimize equipment needs and to simplify operation. Detailed procedures and examples are presented in Appendix G.