Appendix A
References

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ER 500-1-1

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EM 1110-2-301

Guidelines for Landscape Planting at Floodwalls, Levees, and Embankment Dams

EM 1110-2-1601

Hydraulic Design of Flood Control Channels

EM 1110-2-1614

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Appendix B
Mathematical Analysis of Underseepage and Substratum Pressure

B-1. General

The design of seepage control measures for levees often requires an underseepage analysis without the use of piezometric data and seepage measurements. Contained within this appendix are equations by which an estimate of seepage flow and substratum pressures can be made, provided soil conditions at the site are reasonably well defined. The equations contained herein were developed during a study (reported in U.S. Army Engineer Waterways Experiment Station TM 3-424 (Appendix A) of piezometric data and seepage measurements along the Lower Mississippi River and confirmed by model studies. It should be emphasized that the accuracy obtained from the use of equations is dependent upon the applicability of the equation to the condition being analyzed, the uniformity of soil conditions, and evaluation of the various factors involved. As is normally the case, sound engineering judgment must be exercised in determining soil profiles and soil input parameters for these analyses.

B-2. Assumptions

It is necessary to make certain simplifying assumptions before making any theoretical seepage analysis. The following is a list of such assumptions and criteria necessary to the analysis set forth in this appendix.

a. Seepage may enter the pervious substratum at any point in the foreshore (usually at riverside borrow pits) and/or through the riverside top stratum.

b. Flow through the top stratum is vertical.

c. Flow through the pervious substratum is horizontal.

d. The levee (including impervious or thick berms) and the portion of the top stratum beneath it is impervious.

e. All seepage is laminar.

In addition to the above, it is also required that the foundation be generalized into a pervious sand or gravel stratum with a uniform thickness and permeability and a semipervious or impervious top stratum with a uniform thickness and permeability (although the thickness and permeability of the riverside and landside top stratum may be different).

B-3. Factors Involved in Seepage Analyses

The volume of seepage (Q_s) that will pass beneath a levee and the artesian pressure that can develop under and landward of a levee during a sustained high water are related to the basic factors given and defined in Table B-1 and shown graphically in Figure B-1. Other values used in the analyses are defined as they are discussed in subsequent paragraphs.

B-4. Determination of Factors Involved in Seepage Analyses

Table B-2 contains a brief summary of methods normally used to determine the factors necessary to perform a seepage analysis. The determination of these factors is discussed in more detail in the following paragraphs. Many of the methods given, such as exploration and testing, have previously been mentioned in the text; however, they will be discussed herein in more detail as they apply to each specific factor. The use of piezometric data, although rarely available on new projects, is mentioned primarily because it is not infrequent for seepage analyses to be performed as a part of remedial measures to existing levees in which case piezometric data often are available.

Table B-1
Factors Involved in Seepage Analyses

Factor	Definition
H	Net head on levee
M	Slope of hydraulic grade line (at middepth of pervious stratum) beneath levee
i_c	Critical gradient for landside top stratum
L_1	Distance from river to riverside levee toe
L_2	Base width of levee and berm
L_3	Length of foundation and top stratum beyond landside levee toe
L	Distance from effective seepage entry to effective seepage exit
s	Distance from effective seepage entry to landside toe of levee or berm
X_1	Distance from effective seepage entry to riverside levee toe
X_3	Distance from landside levee toe to effective seepage exit
d	Thickness of pervious substratum
z	Thickness of top stratum
z_b	Transformed thickness of top stratum
z_{bl}	Transformed thickness of landside top stratum
z_{br}	Transformed thickness of riverside top stratum
z_n	Thickness of individual layers comprising top stratum (n = layer number)
z_t	Transformed thickness of landside top stratum for uplift computation
k_b	Vertical permeability of top stratum
k_{bl}	Vertical permeability of landside top stratum
k_{br}	Vertical permeability of riverside top stratum
k_f	Horizontal permeability of pervious substratum
k_n	Vertical permeability of individual layers comprising top stratum (n = layer number)
Q_s	Total amount of seepage passing beneath the levee
h_o	Head beneath top stratum at landside levee toe
h_x	Head beneath top stratum at distance x from landside levee toe

Table B-2
Methods for Determination of Design Parameters

Factor	Method of Determination
H	From design flood stage or net levee grade
k_{bl}, k_{br}	From laboratory tests, estimations, and transformations
k_f	Field pump tests, correlations
z_b	Foundation exploration, knowledge of depth and locations of borrow pits, ditches, etc.
z_{bl} , z_{br}	From transformations
d	Foundation exploration
i_c	From equation B-9
M	From piezometers or from determining effective entrance and exit points of seepage
L_1	From maps
L_2	From preliminary or existing levee section
L_3	From foundation exploration and knowledge of location of levee
s	From piezometric data or estimated from equations
x_1	From knowing M or from equation B-7 or B-8
x_3	From knowing M or from equation B-3, B-3A, B-4, B-5, or B-6
Q_s	From equation B-11 or B-12
h_o	From piezometric data or estimated from equations
h_x	From piezometric data or estimated from equations

a. Net head, H. The net head on a levee is the height of water on the riverside above the tailwater or natural ground surface on the landside of the levee. H is usually based on the design or project flood stage but is sometimes based on the net levee grade.

b. Thickness, z and vertical permeability, k_b, of top stratum.

(1) Exploration. The thickness of the top stratum, both riverward and landward of the levee, is extremely important in a seepage analysis. Exploration to determine this thickness usually consists of auger borings with samples taken at 0.91- to 1.52-m (3- to 5-ft) intervals and at every change of material. Boring spacing will depend on the potential severity of the underseepage problem but should be laid out so as to sample the basic geologic features with intermediate borings for check purposes. Landside borings should be sufficient to delineate any significant geological features as far as 152.4 m (500 ft) away from the levee toe. The effect of ditches and borrow areas must be considered.

(2) Transformation. The top stratum in most areas is seldom composed of one uniform material but rather usually consists of several layers of different soils. If the in situ vertical permeability of each soil (k_n) is known, it is possible to transform an overall effective thickness and permeability. However, if good judgment is exercised in selection of these values, a reasonably accurate seepage analysis can be made by using a simplified procedure. Basically this procedure consists of assuming a uniform vertical permeability

for the generalized top stratum equal to the permeability of the most impervious strata and then using the transformation factor given in equation B-1 to determine a corresponding thickness for the entire top stratum.

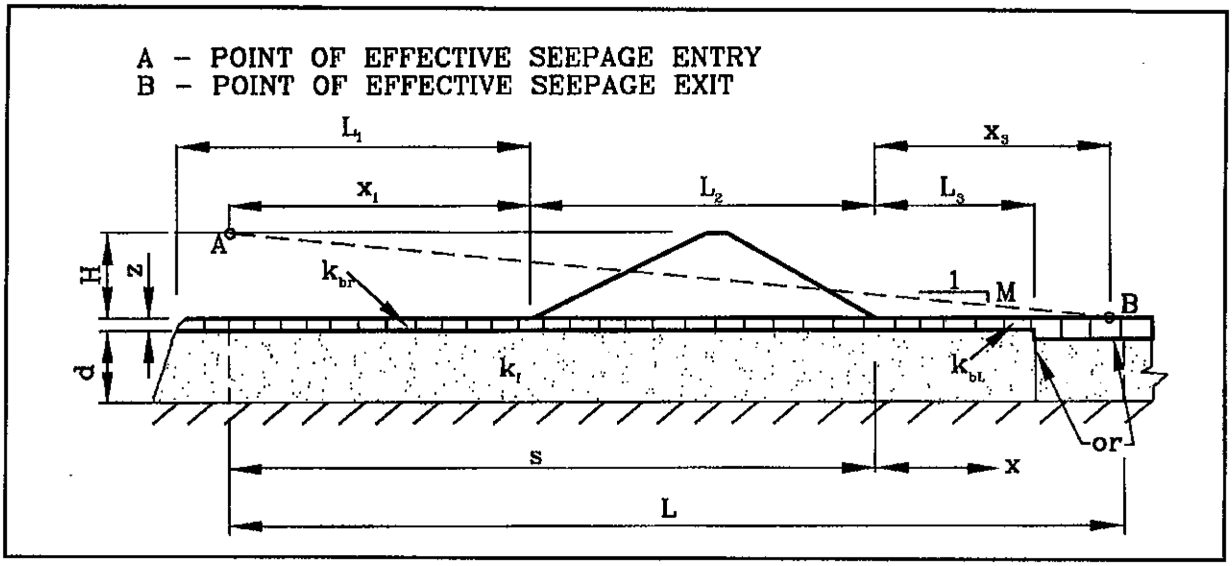


Figure B-1. Illustration of symbols used in Appendix B

$$F_t = \frac{k_b}{k_n} \qquad \text{(B-1)}$$

where F_t = transformation factor.

If the in situ thickness of each soil layer (z_n) is known, the value of corresponding transformed thickness (z_t) can be expressed as

$$z_t = z_n\, F_t = z_n\, \frac{k_b}{k_n} \qquad \text{(B-1a)}$$

The total in situ thickness (z) and total transformed can be expressed as

$$z = \sum_1^n z_n \qquad \text{(B-1b)}$$

$$z_b = \sum_1^n z_t \qquad \text{(B-1c)}$$

Some examples using this procedure are given in Table B-3 and in Figure B-2.

Table B-3
Examples of Transformation Procedure

Strata	Actual Thickness z_o, m (ft)	Actual Permeability cm/sec	$F_t = \frac{K_b}{K_n}$	Transformed Thickness, z_t, m (ft) ($k_b = 1 \times 10^{-4}$ cm/sec)
Clay	1.52 (5)	1×10^{-4}	1	1.52 (5.0)
Sandy silt	2.44 (8)	2×10^{-4}	1/2	1.22 (4.0)
Silty sand	1.52 (5)	10×10^{-4}	1/10	0.15 (0.5)
	z = 5.48 (18)			z_b = 2.90 (9.5)

A generalized top stratum having a uniform permeability of 1 x10^{-4} cm/sec and 2.9 m (9.5 ft) thick would then be used in the seepage analysis for computation of the length to the effective seepage exit. However, the thickness z_b may or may not be the effective thickness of the landside top stratum z_t that should be used in determining the allowable pressure beneath the top stratum. The transformed thickness of the top stratum for estimating allowable uplift z_t equals the in situ thicknesses of all strata above the base of the least pervious stratum plus the transformed thicknesses of the underlying more pervious top strata. This means that z_b will equal z_t only when the least pervious stratum is at the ground surface. Several examples of this transformation are given in Figure B-2. In making the final determination of the effective thicknesses and permeabilities of the top stratum, the characteristics of the top stratum at least 61 to 91.4 m (200 to 300 ft) landward of the levee must be considered. In addition, certain averaging assumptions are almost always required where soil conditions are reasonably similar. Thin or critical areas should be given considerable weight in arriving at such averages.

c. Thickness d and permeability k_f of pervious substratum. The thickness of the pervious substratum is defined as the thickness of the principal seepage-carrying stratum below the top stratum and above rock or other impervious base stratum. It is usually determined by means of deep borings although a combination of shallow borings and seismic or electrical resistivity surveys may also be employed. The thickness of any individual pervious strata within the principal seepage carrying stratum must be obtained by deep borings. The average horizontal permeability k_f of the pervious substratum can be determined by means of a field pump test on a fully penetrating well or by the use of correlations as shown in Figure 3-5(b) in the main text. For areas where such correlations exist their use will usually result in a more accurate permeability determination than that from laboratory permeability tests. In addition to the methods above, if the total amount of seepage per unit length passing beneath the levee (Q_s), the hydraulic grade line beneath the levee (M) and the thickness of pervious stratum (d) are known, k_f can be estimated from

$$k_f = \frac{Q_s}{M \cdot d} \qquad \text{(B-2)}$$

d. Distance from riverside levee toe to river, L_1. This distance can usually be estimated from topographic and stratigraphic maps.

e. Base width of levee and berm, L_2. L_2 can be determined from anticipated dimensions of new levees or by measurement in the case of existing levees.

f. Length of top stratum landward of levee toe, L_3. This distance can usually be determined from borings, topographic maps, and/or field reconnaissance. In determining this distance careful consideration

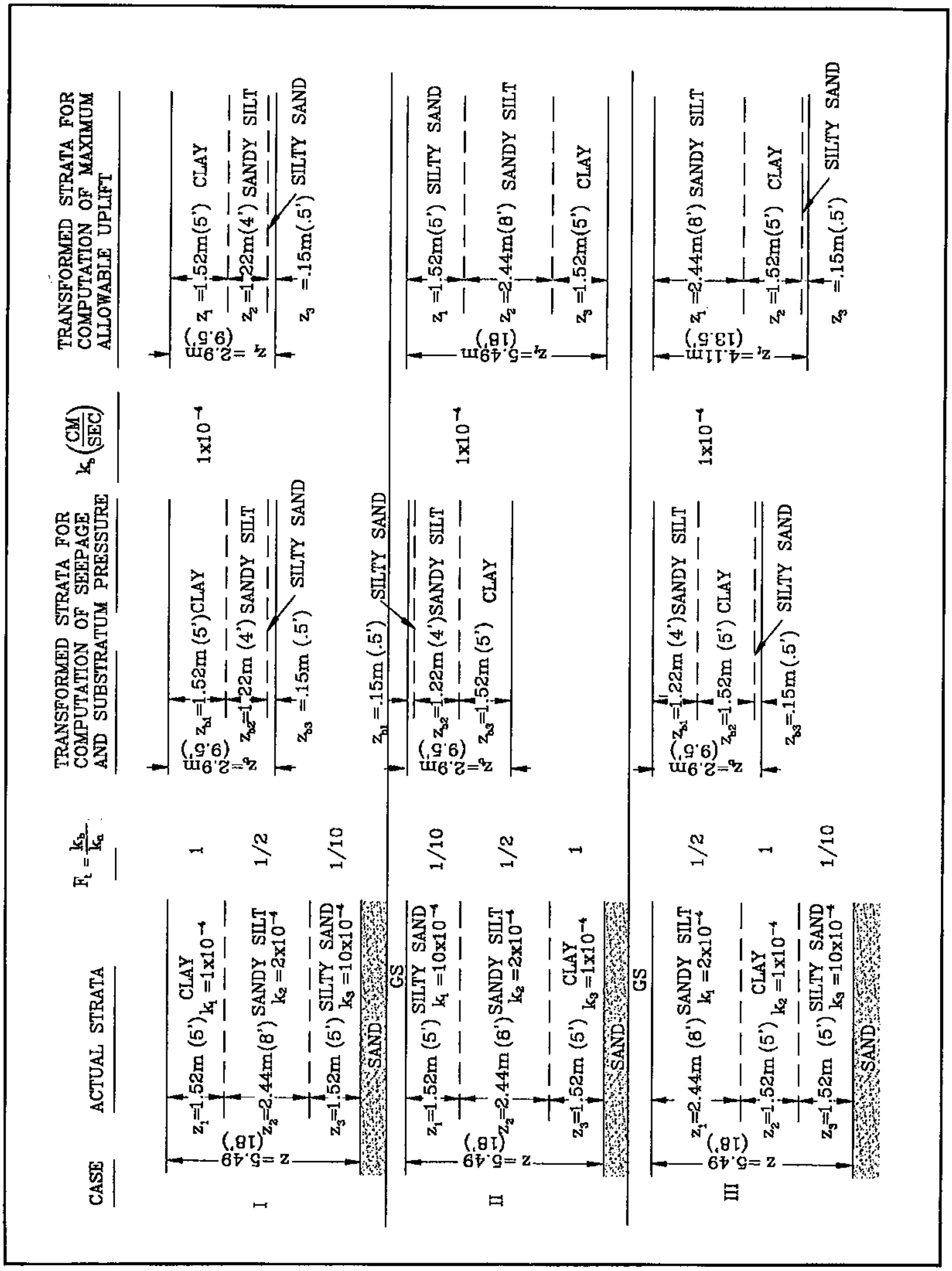


Figure B-2. Transformation of top strata must be given to any geological feature that may affect the seepage analysis. Of special importance are deposits of impervious materials such as clay plugs which can serve as seepage barriers and if located near the landside toe could force the emergence of seepage at their near edge, thus having a pronounced effect on the seepage analysis.

g. Distance from landside levee toe to effective seepage exit, x_3. The effective seepage exit (point B, Figure B-1) is defined as that point where a hypothetical open drainage face would result in the same hydrostatic pressure at the landside levee toe and would cause the same amount of seepage to pass beneath the levee as would occur for actual conditions. This point is also defined as the point where the hydraulic grade line beneath the levee projected landward with a slope M intersects the groundwater or tailwater. If the length of foundation and top stratum beyond the landside levee toe L_3 is known, x_3 can be estimated from the following equations:

(1) For $L_3 = \infty$

$$x_3 = \frac{1}{c} = \sqrt{\frac{k_f\, z_{bl}\, d}{k_{bl}}} \qquad \text{(B-3)}$$

where

$$c = \sqrt{\frac{k_{bl}}{k_f\, z_{bl}\, d}} \qquad \text{(B-3A)}$$

(2) For L_3 = finite distance to a seepage block

$$x_3 = \frac{1}{c\ \tanh\ (cL_3)} \qquad \text{(B-4)}$$

(3) For L_3 = finite distance to an open seepage exit

$$x_3 = \frac{\tanh\ (cL_3)}{c} \qquad \text{(B-5)}$$

(4) The relationship between z_{bl} and x_3 where L_3 is infinite in landward extent has been computed from equation B-3 and plotted in Figure B-3 for various values of k_f/k_{bl} and assuming d = 100 m or 100 ft. The x_3 value corresponding to values of d other than 100 m or 100 ft can be computed from equation B-6 below:

$$x_3 = (0.1\ \sqrt{d})\ x_3' \qquad \text{(B-6)}$$

where

x_3' is the value of x_3 for d = 100 m or 100 ft

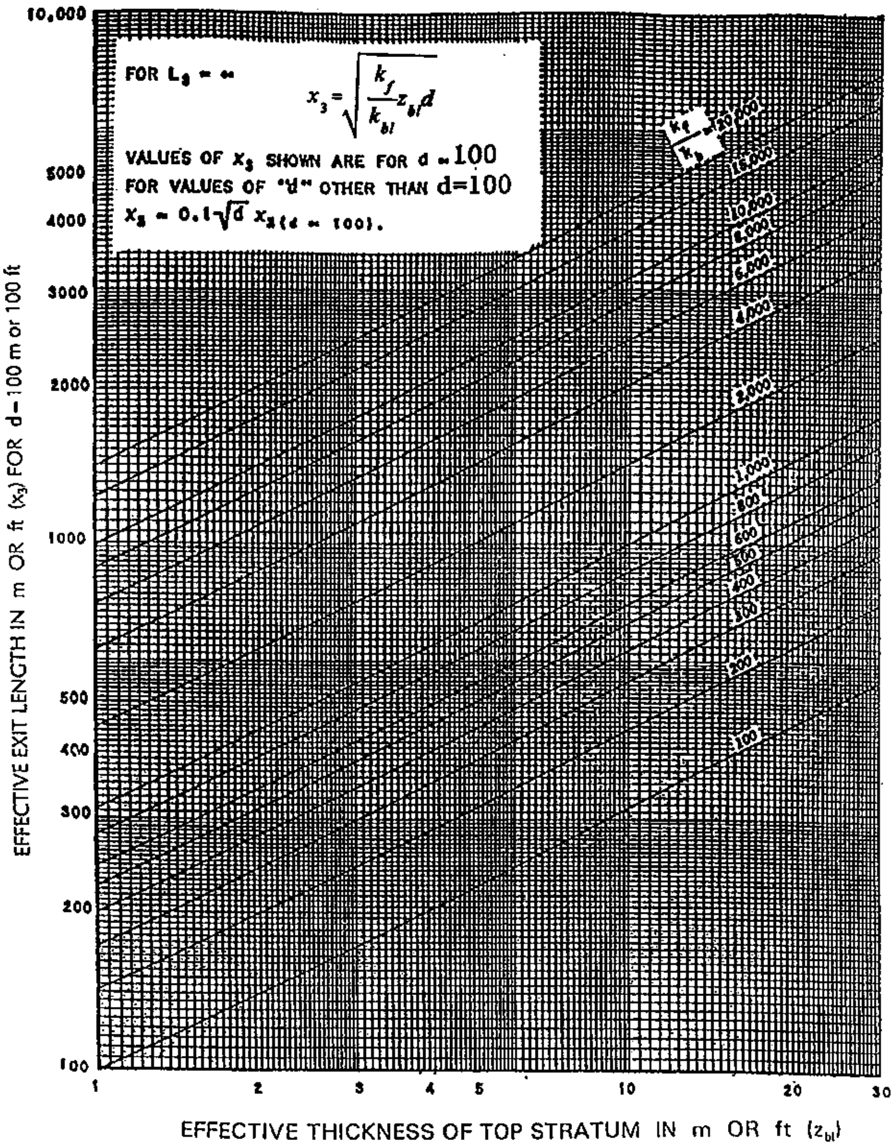


Figure B-3. Effective seepage exit length for $L_3 = \infty$ and d = 100 m or ft

Example: Using Figure B-3, find x_3 for soil with $\frac{k_f}{k_{bl}} = 200$, $z_{bl} = 3.05$ m (10 ft), and d = 45.7 m (150 ft)

Solution: 1 - From Figure B-3 the value x_3 for $\frac{k_f}{k_{bl}} = 200$, $z_{bl} = 3.05$ m (10 ft). Then for $z_{bL} = 3.05$ m and d = 100 m, $x_3' = 246$ m; or $z_{bL} = 10$ ft and d = 100 ft, $x_3' = 450$ ft

2 - Apply Equation B-6 to determine x_3 for d = 45.7 m (150 ft)

$$x_3 = 0.1\ \sqrt{45.7}\ x_3'$$

$$x_3 = (0.1)(6.76)(246) = 167\ m$$

or

$$x_3 = 0.1\ \sqrt{d}\ (450) = 0.1\sqrt{150}\ (450) = 551\ ft$$

(5) If L_3 is a finite distance either to a seepage block or an open seepage exit, the effective exit length x_3 can be computed by using equation B-4 or B-5 or by multiplying x_3 (for $L_3 = \infty$) by a factor obtained from Figure B-4.

h. Distance from effective source seepage entry to riverside levee toe, x_1. The effective source of seepage entry into the pervious substratum (point A in Figure B-1) is defined as that line riverward of the levee where a hypothetical open seepage entry face fully penetrating the pervious substratum and with an impervious top stratum between this line and the levee would produce the same flow and hydrostatic pressure beneath and landward of the levee as will occur for the actual conditions riverward of the levee. It is also defined as that line or point where the hydraulic grade line beneath the levee projected riverward with a slope M intersects the river stage.

(1) If the distance to the river from the riverside levee toe L_1 is known and no riverside borrow pits or seepage blocks exist, x_1 can be estimated from the following equation:

$$x_1 = \frac{\tanh\ cL_1}{c} \tag{B-7}$$

(2) If a seepage block (usually a wide, thick deposit of clay) exists between the riverside levee toe and the river so as to prevent any seepage entrance into the pervious foundation beyond that point, x_1 can be estimated from the following equation:

$$x_1 = \frac{1}{c\ \tanh\ cL_1} \tag{B-8}$$

where L_1 equals distance from riverside levee toe to seepage block and c is from equation B-3A.

i. Critical gradient for landside top stratum, i_c. The critical gradient is defined as the gradient required to cause boils or heaving (flotation) of the landside top stratum and is taken as the ratio of the submerged or buoyant unit weight of soil ã′ comprising the top stratum and the unit weight of water $ã_w$ or

$$i_c = \frac{ã'}{ã_w} = \frac{G_S - 1}{1 + e} \tag{B-9}$$

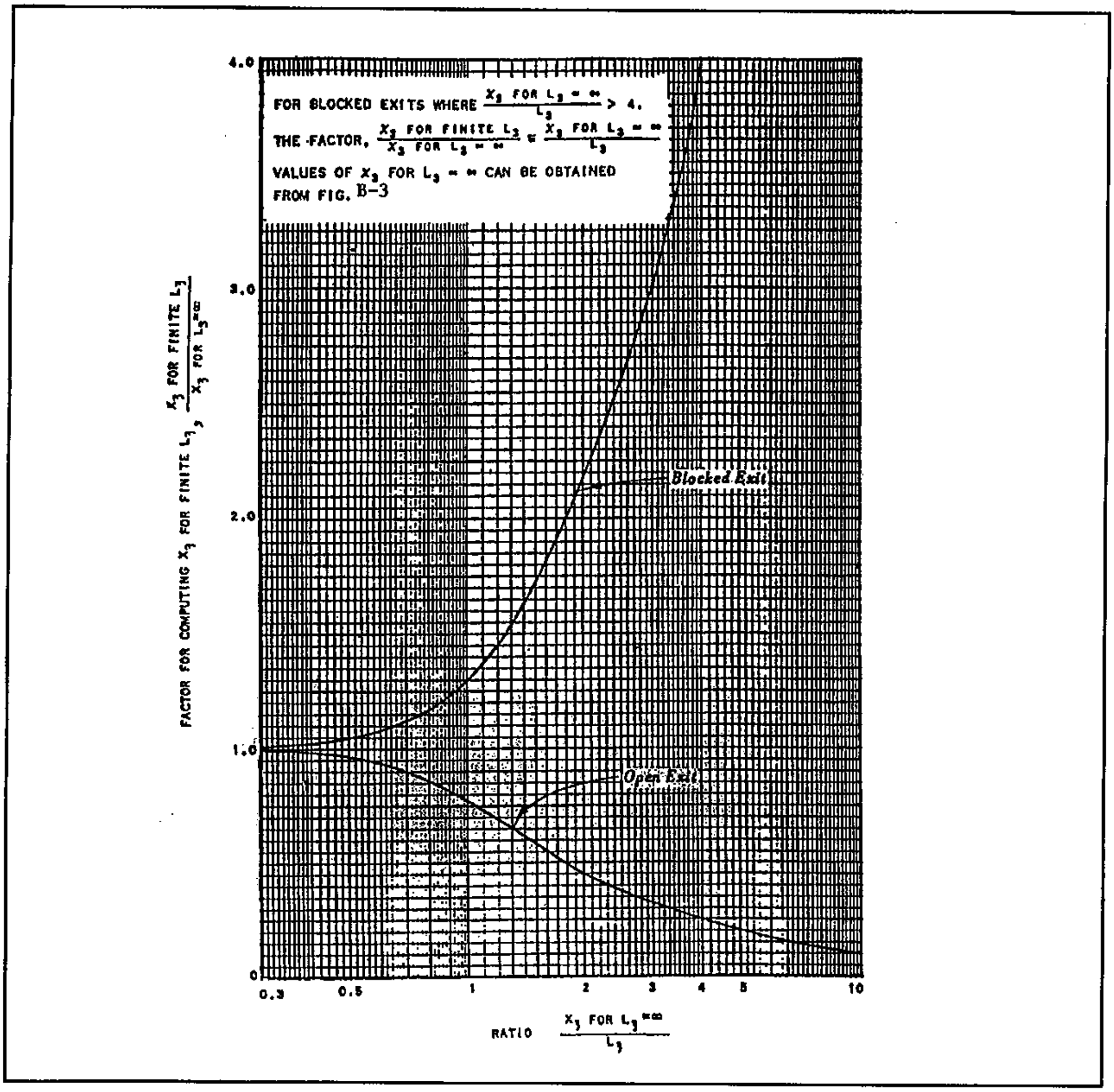


Figure B-4. Ratio between x_3 for blocked or open exits and x_3 for $L_3 = \infty$

where

G_S = specific gravity of soil solids
e = void ratio

j. Slope of hydraulic grade line beneath levee, M. The slope of the hydraulic grade line in the pervious substratum beneath a levee can best be determined from readings of piezometers located beneath the levee where the seepage flow lines are essentially horizontal and the equipotential lines vertical. If such readings during high water are available, M can be determined from the following relation:

$$M = \frac{Äh}{}$$ (B-10a)

where

Äh = the difference in piezometer readings
= the horizontal distance between piezometers

This relationship is not valid, however, until artesian flow conditions have developed beneath the levee. If no piezometer readings are available, as in the case for new levee design, M must be determined by exit points and first establishing the effective seepage entrance and then connecting these points with a straight line, the slope of which is M. For new levees M is expressed as

$$M = \frac{H}{x_1 + L_2 + x_3}$$ (B-10b)

B-5. Computation of Seepage Flow and Substratum Hydrostatic Pressures

a. General

(1) Seepage. For a levee underlain by a pervious foundation, the natural seepage per unit length of levee, Q_S, can be expressed by the general equation B-11.

$$Q_s = S k_f H$$ (B-11)

where

S = shape factor

This equation is valid provided the assumptions upon which Darcy's law is based are met. The mathematical expressions for the shape factor S (subsequently given in this appendix) depend upon the dimensions of the generalized cross section of the levee and foundation, the characteristics of the top stratum both riverward and landward of the levee, and the pervious substratum. Where the hydraulic grade line M is known from piezometer readings, the quantity of underseepage per unit length of the levee can be determined from equation B-12 as

$$Q_s = M k_f d$$ (B-12)

(2) Excess hydrostatic head beneath the landside top stratum.

(a) The excess hydrostatic head h_o beneath the top stratum at the landside levee toe is related to the net head on the levee, the dimensions of the levee and foundation, permeability of the foundation, and the character of the top stratum both riverward and landward of the levee. The head h_o can be expressed as a function of the net head H and the geometry of the piezometric line as subsequently shown.

(b) The head h_x beneath the top stratum at a distance x landward from the landside levee toe can be expressed as a function of the net head H and the distance x although it is more conveniently related to the head h_o at the levee toe. When h_x is expressed in terms of h_o it depends only upon the type and thickness of the top stratum and pervious foundation landward of the levee; the ratio h_x/h_o is thus independent of riverward conditions.

(c) Expressions for $\mathcal{S}$, h_o, and h_x are discussed in the following paragraphs.

b. Various underseepage flow and top substratum conditions.

Case 1 - No Top Stratum. Where a levee is founded directly on pervious materials and no top stratum exists either riverward or landward of the levee (Figure B-5a), the seepage Q_S can be obtained from equation B-11 in which

$$\mathcal{S} = \frac{d}{L_2 + 0.86d} \tag{B-13}$$

The excess hydrostatic head landward of the levee is zero and $h_o = h_x = 0$. The severity of such a condition in nature is governed by the exit gradient and seepage velocity that develop at the landside levee toe which can be estimated from a flow net compatible with the value of $\mathcal{S}$ computed from Equation B-13. The maximum allowable exit gradient should be 0.5.

Case 2 - Impervious Top Stratum Both Riverside and Landside. This case is found in nature where the levee is founded on thick (z_{bl} > 4.58 m (15 ft)) deposits of clay or silts with clay strata. For such a condition little or no seepage can occur through the landside top stratum.

a. If the pervious substratum is blocked landward of the levee, no seepage occurs beneath the levee and $Q_s = 0$. The head beneath the levee and the landside top stratum is equal to the net head at all points so that $H = h_o = h_x$.

b. If the top stratum is impervious between the levee and river and has a length L_1, and if an open seepage exit exists in the impervious top stratum at some distance L_3 from the landside toe (i.e., L_3 is not infinite as shown in Figure B-5b), the distance from the landside toe of the levee to the effective seepage entry (river, borrow pit, etc.) is $S = L_1 + L_2$ and

$$\mathcal{S} = \frac{d}{L_1 + L_2 + L_3} \tag{B-14}$$

The heads h_o and h_x can be computed from

$$h_o = H\left(\frac{L_3}{L_1 + L_2 + L_3}\right) \tag{B-15}$$

$$h_x = h_o\left(\frac{L_3 - x}{L_3}\right) \textit{ for } x \quad L_3 \tag{B-16}$$

$$h_x = 0 \textit{ for } x \quad L_3$$

Case 3 - Impervious Riverside Top Stratum and No Landside Top Stratum. This condition may occur naturally or where extensive landside borrowing has taken place resulting in removal of all impervious material landward of the levee for a considerable distance. Seepage can be computed utilizing Equation B-11 and the following shape factor

$$\mathcal{S} = \frac{d}{L_1 + L_2 + 0.43d} \tag{B-17}$$

The excess head at the top of the sand landward of the levee is zero and the danger from piping must be evaluated from the upward gradient obtained from a flow net. This case is shown in Figure B-5c.

Case 4 - Impervious Landside Top Stratum and No Riverside Top Stratum. This is a more common case than Case 3, occurring when extensive riverside borrowing has resulted in removal of the riverside impervious top stratum (Figure B-5d). For this condition the seepage is computed from Equation B-11 utilizing the shape factor given in Equation B-18 below; the heads h_o and h_x can be computed from Equations B-19 and B-20, respectively.

$$\mathcal{S} = \frac{d}{0.43d + L_2 + L_3} \tag{B-18}$$

$$h_o = H\left(\frac{L_3}{0.43d + L_2 + L_3}\right) \tag{B-19}$$

$$h_x = h_o\left(\frac{L_3 - x}{L_3}\right) \tag{B-20}$$

Case 5 - Semipervious Riverside Top Stratum and No Landside Top Stratum. The same equation for the shape factor as was used in Case 3 can be applied to this condition provided x_1 is substituted for L_1 as follows:

$$\mathcal{S} = \frac{d}{x_1 + L_2 + 0.43d} \tag{B-21}$$

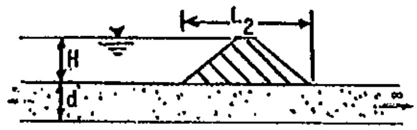
Since no landside top stratum exists, $h_o = h_x = 0$. This case is illustrated in Figure B-6a.

Case 6 - Semipervious Landside Top Stratum and No Riverside Top Stratum. The same equations for the shape factor and heads beneath the landside top stratum that are used for Case 4 are applicable to this case provided x_3 is substituted for L_3 (Figure B-6b). These equations are as follows:

$$\mathcal{S} = \frac{d}{0.43d + L_2 + x_3} \tag{B-22}$$

$$h_o = H\left(\frac{x_3}{0.43d + L_2 + x_3}\right) \tag{B-23}$$

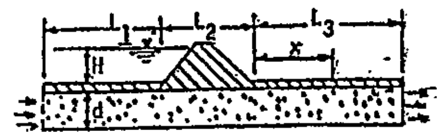
a. CASE 1 - No top stratum



$$\mathcal{S} = \frac{d}{L_2 + 0.86d}$$

$$h_o = h_x = 0$$

b. CASE 2 - Impervious topstratum both riverside and landside



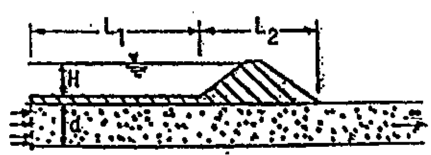
$$\mathcal{S} = \frac{d}{L_1 + L_2 + L_3}$$

$$h_o = H\left(\frac{L_3}{L_1 + L_2 + L_3}\right)$$

$$h_x = h_o\left(\frac{L_3 - x}{L_3}\right) \text{ for } x < L_3$$

$$h_x = 0 \text{ for } x > L_3$$

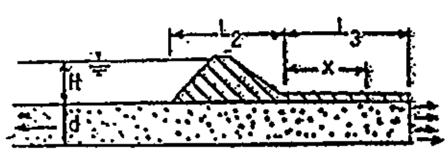
c. CASE 3 - Impervious riverside top stratum & no landside top stratum



$$\mathcal{S} = \frac{d}{L_1 + L_2 + 0.43d}$$

$$h_o = h_x = 0$$

d. CASE 4 - Impervious landside top stratum & no riverside top stratum



$$\mathcal{S} = \frac{d}{0.43d + L_2 + L_3}$$

$$h_o = H\left(\frac{L_3}{0.43d + L_2 + L_3}\right)$$

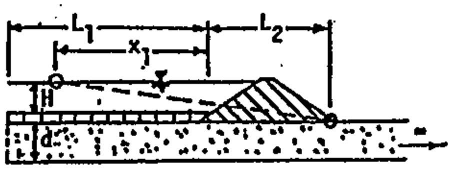
$$h_x = h_o\left(\frac{L_3 - x}{L_3}\right)$$

BASIC DEFINITIONS AND EQUATIONS

HEAD BENEATH TOP STRATUM AT LANDSIDE LEVEE TOE . . . . h_o
HEAD BENEATH TOP STRATUM AT DISTANCE x FROM LANDSIDE LEVEE TOE . . . . h_x
SHAPE FACTOR FOR USE IN SEEPAGE EQUATION . . . . $\mathcal{S}$
SEEPAGE PER UNIT LENGTH OF LEVEE . . . . $Q_s = \mathcal{S}\, k_f H$

Figure B-5. Equations for computation of underseepage flow and substratum pressures for cases 1 through 4

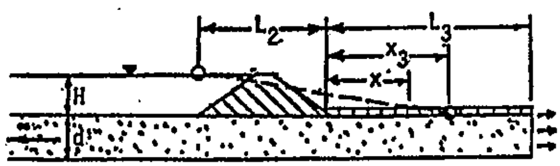
a. CASE 5 - Semipervious riverside top stratum and no landside top stratum



$$\mathcal{S} = \frac{d}{x_1 + L_2 + 0.43d}$$

$$h_o = h_x = 0$$

b. CASE 6 - Semipervious landside top stratum and no riverside top stratum



$$\mathcal{S} = \frac{d}{0.43d + L_2 + x_3}$$

$$h_o = H\left(\frac{x_3}{0.43d + L_2 + x_3}\right)$$

$$h_x = h_o\left(\frac{x_3 - x}{x_3}\right)$$

Figure B-6. Equations for computation of underseepage flow and substratum pressures for cases 5 and 6

$$h_x = h_o\left(\frac{x_3 - x}{x_3}\right) \tag{B-24}$$

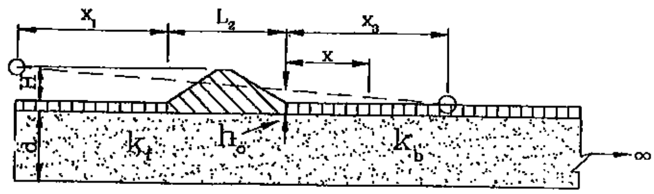
Case 7 - Semipervious Top Strata Both Riverside and Landside. Where both the riverside and landside top strata exist and are semipervious (Figure B-7), the quantity of underseepage can be computed from equation B-11 where $\mathcal{S}$ is defined in Equation B-25.

$$\mathcal{S} = \frac{d}{x_1 + L_2 + x_3} \tag{B-25}$$

The head beneath the top stratum at the landside toe of the levee is expressed by

$$h_o = H\left(\frac{x_3}{x_1 + L_2 + x_3}\right) \tag{B-26}$$

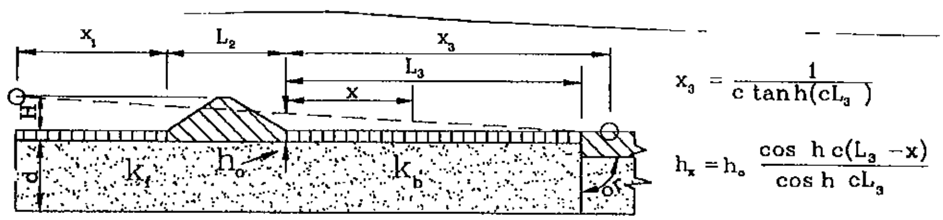
The equations above are valid for all conditions where the landside top stratum is semipervious. However, the head h_x beneath the semipervious top stratum depends not only on the head h_o but also on conditions landward of the levee. Expressions are given below for typical conditions encountered landward of levees.



$$x_3 = \frac{1}{c}$$

$$h_x = h_o \; e^{-cx}$$

a. $L_3 = \infty$

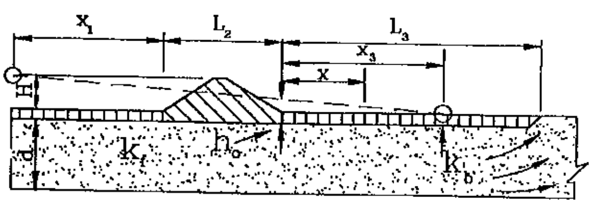


$$x_3 = \frac{1}{c \tanh(cL_3)}$$

$$h_x = h_o \frac{\cosh c(L_3 - x)}{\cosh cL_3}$$

$$h_x \text{ (for } x = L_3) = \frac{h_o}{\cosh cL_3}$$

b. L_3 is finite to a seepage block



$$x_3 = \frac{\tanh(cL_3)}{c}$$

$$h_x = h_o \frac{\sinh c(L_3 - x)}{\sinh cL_3}$$

$$h_x \text{ (for } x = L_3) = 0$$

c. L_3 is finite to an open seepage exit

BASIC DEFINITIONS AND EQUATIONS

SEEPAGE PER UNIT LENGTH OF LEVEE.. $Q_s = \$ k_f H = \dfrac{dk_f H}{x_1 + L_2 + x_3}$

HEAD BENEATH TOP STRATUM AT LANDSIDE LEVEE TOE............ $h_o = \dfrac{Hx_3}{x_1 + L_2 + x_3}$

THE FACTOR.. $c = \sqrt{\dfrac{kbL}{k_f z_b d}}$

Figure B-7. Equations for computation of underseepage and substratum pressures for Case 7

(1) For $L_3 = \infty$

$$h_x = h_o \, e^{-cx} \tag{B-27}$$

where

$e = 2.718$

(2) For L_3 = a finite distance to a seepage block

$$h_x = h_o \frac{\cosh c(L_3 - x)}{\cosh cL_3} \tag{B-28}$$

and

$$h_x \ (at \ x = L_3) = \frac{h_o}{\cosh cL_3} \tag{B-29}$$

(3) For L_3 = a finite distance to an open seepage exit

$$h_x = h_o \frac{\sinh c(L_3 - x)}{\sinh cL_3} \tag{B-30}$$

and

$$h_x \ (at \ x = L_3) = 0 \tag{B-31}$$

(4) Values of c and h_o in Equations B-27 through B-30 are as follows:

$$c = \sqrt{\frac{k_{bl}}{k_f \, z_{bl} \, d}} \ , \qquad h_o = H \frac{x_3}{x_1 + L_2 + x_3} \tag{B-32}$$

(5) In order to simplify the determination of h_x for various values of x, the relationship between h_x/h_o and x/x_3 is plotted in Figure B-8 for $L_3 = \infty$ and for various values of x_3/L_3 for both a seepage block and an open seepage exit. The procedure for determining h_x using Figure B-8 can be summarized as follows:

a. Determine x_1, L_2, x_3 and the head h_o at the landside toe of the levee.

b. For the given distance x where h_x needs to be determined find the ratios x/x_3 and x_3/L_3, then enter the appropriate graph in Figure B-8 to read the corresponding value of h_x/h_o.

c. Knowing the ratio h_x/h_o and the value of h_o compute h_x.

(6) Values of h_o and h_x resulting from the equations above are actually hydrostatic heads at the middle of the pervious substratum; where the ratio k_f/k_b is less than 100 to 500, values of h_o and h_x immediately

beneath the top stratum will be slightly less than those computed because of the head loss resulting from upward seepage through the sand stratum.

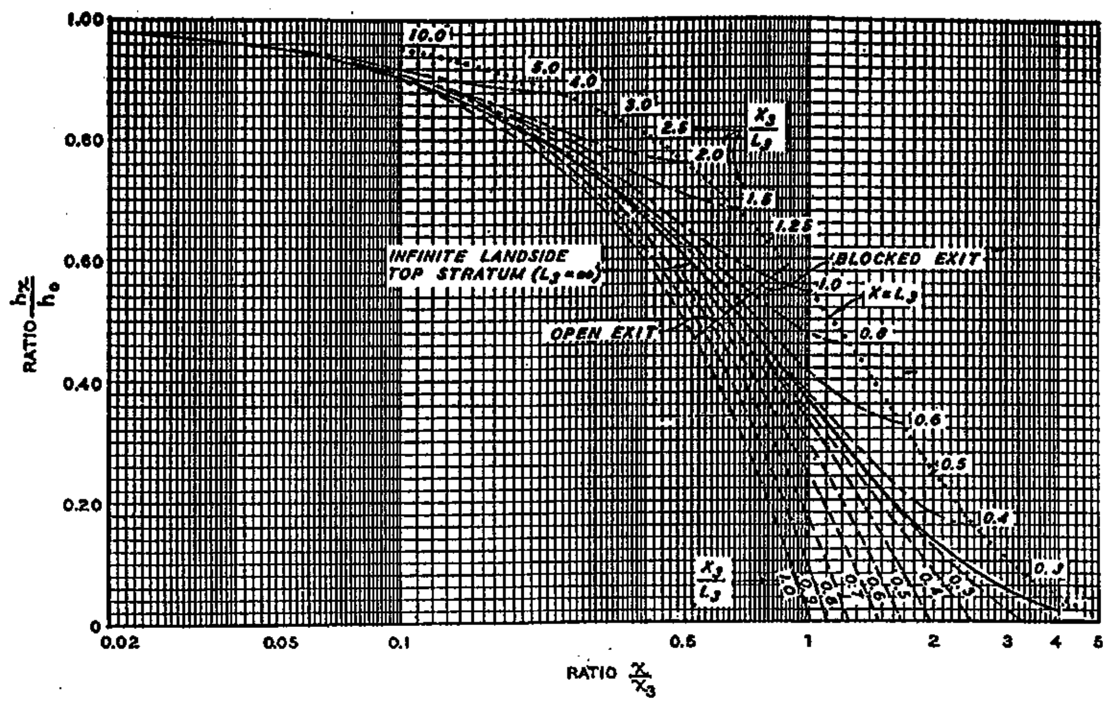


Figure B-8. Ratio between head landward of levee and head at landside toe of levee for levees founded on semipervious top stratum underlain by a pervious substratum

Appendix C
Design of Seepage Berms

C-1. General

This appendix presents design factors, equations, criteria, and examples of designing landside seepage berms. A discussion of the four major types of landside seepage berms is presented in the main text of this manual. The design equations presented are taken from U.S. Army Engineer Waterways Experiment Station TM 3-424 and EM 1110-2-1901 (Appendix A). Design procedures are taken from TM 3-424 and from procedures developed by the Lower Mississippi Valley Division (Appendix A).

C-2. Design Factors

a. Seepage records, if available, should be studied to determine the severity of the underseepage conditions during high water. A projection based upon these records of underseepage during high water to the design flood should be made based on experience and judgment. Aerial photographs and borings should be used to evaluate geologic and soil conditions. The location of drainage ditches and borrow pits should be noted and considered in design. Additional borings should be made where required to determine in situ soil and geological data needed for design.

b. The distance s from the landside toe of the levee to the point of effective seepage entry is equal to the base width of the levee L_2 plus the effective length of blanket x_1 on the riverside of the levee. The effective length of blanket x_1 can be determined by using blanket equations presented in Appendix B. The effect of riverside borrow pits or natural low areas such as oxbows, must be considered in determining x_1. The effective length of blanket x_1 should be the lesser of the distance based on the blanket thickness outside the riverside borrow pit and the distance based on the blanket thickness inside the riverside borrow pit plus the distance from the riverside toe of the levee to the borrow pit. The blanket equations assume an infinite blanket length. However, this assumption may not be valid if the river is close to the levee. If the computed value of x_1 is greater than L_1 (distance from riverside toe of levee to the river), then x_1 should equal L_1 . Distances to effective sources of seepage, effective lengths of riverside blankets, and vertical permeabilities of riverside blanket materials at different sites along the Mississippi River at the crest of the 1950 high water period are given in Table C-1. The values of x_1 are observed values adjusted to an assumed condition of a riverside blanket of infinite length with the same thickness as that of the borrow pit. The adjustment was made by use of blanket equations presented in Appendix B to partially eliminate the effect of different top strata riverward of the borrow pits and different distances between the levee and river at various sites.

c. The thickness d and permeability k_f of the pervious materials between the bottom of the blanket and the entrenched valley must be determined before designing a seepage berm. In Appendix B, paragraph B-4c, methods are described for determining d and k_f.

d. The permeability k_{bl} and effective thickness z_{bl} of the landside blanket must be determined before the seepage exit length x_3 can be computed. If the blanket is composed of more than one stratum and the vertical permeability of each stratum is known, the thickness of each stratum of the blanket can be transformed into an equivalent thickness of material having the same permeability as for one of the strata. A procedure and example for transforming the actual thickness of a stratified blanket into an effective thickness z_{bl} with a uniform vertical permeability is described in Appendix B, paragraph B-4b(2). The critical thickness of the landside top stratum z_t that should be used to determine if uplift pressure is within safe limits may or may not be equal to z_{bl} for stratified layers. The procedure and examples for computing z_t for different conditions of soil stratification are also presented in Appendix B, paragraph B-4b(2).

Table C-1a
Summary of Distances to Effective Source of Seepage, Effective Lengths of Riverside Blankets, and Vertical Permeability of Riverside Blanket Materials at the Crest of 1950 High Water (Metric Units)

Blanket in Riverside Borrow Pit		Number of Piezometer Lines from Which Data Were Obtained	S, m			x_1, m[a]			k_{br} x 10^{-4} cm/sec			Suggested Design Values			
Soil Type	Thickness in m		Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	$_{br}$ k		x_1, m	
Sand –	3	304.8		243.8	292.6	146.3	61	112.8	–	–	–	–	76.2		
Silty sand[b]	<1.52	3		243.8	170.7	204.2	97.5	70.1	85.3	14.2	1.6	7.0	7.0	91.4	
	1.52 to 3.05	1		170.7	170.7	170.7	85.3	85.3	85.3	1.8	1.8	1.8 5.7[c]	2.5	182.9	
Silt and sandy silt	<1.52	4		457.2	182.9	320	371.8	83.3	204.2	7.4	0.24	2.2	2.0	121.9	
	1.52 to 3.05	2		487.7	277.4	384	362.7	155.4	259.1	5.0	0.33	2.7	1.5	243.8	
	>3.05											2.4[c]	1.0	365.8	
Clay	<1.52	6		390.1	185.9	310.9	228.6	33.6	210.3	1.7	0.34	0.79	0.8	182.9	
	1.52 to 3.04	2		524.3	463.3	493.8	387.1	326.1	356.6	1.3[d]	0.86[d]	1.08[d]	0.5	396.2	
	3.05 to 4.58	2		–	–	–	–	–	–	–	–	–	0.2	762.0	
	>4.58	3		960.1	243.8	487.7	∞	∞	∞	0.0	0.00	0.00 0.4[c,f]	0.05	1219.2[e] or L_1	

[a] Values of x_1 computed from observed values of x_3 and adjusted to a condition where $L_1 = \infty$.
[b] Does not include Hole-in-the-Wall where values of S and x_3 may not be reliable because artesian flow conditions did not develop until near the crest of the 1950 high water.
[c] Averages of all values of k_{br} for a given soil type without regard to thickness.
[d] Values are considered to be too high as at these piezometer lines (Upper Francis) seepage could enter the pervious substratum through a silty blanket riverward of the borrow pit as well as through the clay in the borrow pit.
[e] Use the smaller of the two values.
[f] Average does not include k_{br} for blanket thickness between 1.52 and 3.05 m.

Table C-1b

Summary of Distances to Effective Source of Seepage, Effective Lengths of Riverside Blankets, and Vertical Permeability of Riverside Blanket Materials at the Crest of 1950 High Water (English Units)

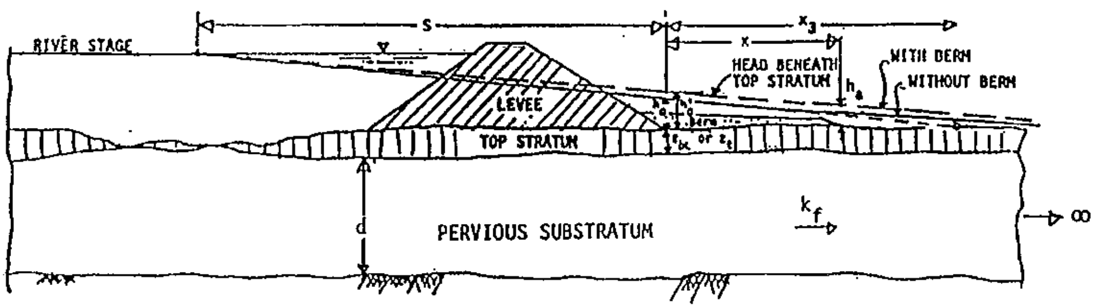
Blanket in Riverside Borrow Pit		Numer of Piezometer Lines from Which Data Were Obtained	S, ft			x_1, ft[a]			k_{br} x 10^{-4} cm/sec			Suggested Design Values		
Soil Type	Thickness in ft		Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	k_{br} k	x_1 (ft)	
Sand –	3	1080	800	960	480	200	370	–	–	–	–	250		
Silty sand[b]	<5	3	800	560	670	320	230	280	14.2	1.6	7.0	7.0	300	
	5 to 10	1	560	560	560	280	280	280	1.8	1.8	1.8	2.5	600	
											5.7[c]			
Silt and sandy silt	<5	4	1500	600	1050	1220	270	670	7.4	0.24	2.2	2.0	400	
	5 to 10	2	1600	910	1260	1190	510	850	5.0	0.33	2.7	1.5	800	
													1200	
	>10										2.4[c]	1.0		
Clay	<5	6	1280	610	1020	750	110	690	1.7	0.34	0.79	0.8	600	
	5 to 10	2	1720	1520	1620	1270	1070	1170	1.3[d]	0.86	1.08	0.5	1300	
	10 to 15	0	–	–	–	–	–	–	–	–	–	0.2	2500	
	>15	3	3150	800	1600	∞	∞	∞	0.0	0.00	0.00	0.05	4000 or L_1[e]	
											0.4[c,f]			

[a] Values of x_1 computed from observed values of x_3 and adjusted to a condition where $L_1 = \infty$.
[b] Does not include Hole-in-the-Wall where values of S and x_3 may not be reliable because artesian flow conditions did not develop until near the crest of the 1950 high water.
[c] Averages of all values of k_{br} for a given soil type without regard to thickness.
[d] Values are considered to be too high as at these piezometer lines (Upper Francis) seepage could enter the pervious substratum through a silty blanket riverward of the borrow pit as well as through the clay in the borrow pit.
[e] Use the smaller of the two values.
[f] Average does not include k_{br} for blanket thickness between 5 and 10 ft.

e. The seepage exit length x_3 can be calculated from equations presented in Appendix B, paragraph B-4g. These equations are applicable to conditions where the length of the landside blanket L_3 is either infinite or finite.

C-3. Design Equations and Criteria

a. Design equations. Equations for the design of landside seepage berms for the four major berm types are presented in Figure C-1. These equations are valid when a landside blanket of infinite length exists. A discussion of the four major landside seepage berms is presented in paragraph 5-4.



Notations

$c = \sqrt{\dfrac{k_{bl}}{k_f z_{bl} d}}$

γ'_z = submerged unit weight of top stratum

γ'_t = submerged unit weight of berm

γ_w = unit weight of water

H = total net head on levee

h_o = head at landside toe of levee without berm = $\dfrac{Hx_3}{s + x_3}$

h'_o = head at landside toe of levee with berm

i_o = allowable upward gradient at landside toe of levee

i_1 = allowable upward gradient at toe of berm

h_a = allowable head at toe of berm = $i_1 \cdot z_t$

X = required berm width

t = required thickness of berm at toe of levee

k_t = vertical permeability of berm

F = factor of safety against uplift at toe of levee

Q_b = flow into berm per foot of levee

s = distance from landside toe of levee to effective source of seepage entry

x_3 = distance from landside toe of levee to effective source of seepage exit

d = effective thickness of pervious substratum

Formulas

Impervious Berm $k_t = 0$

$$X_I = x_3\left(\frac{H}{h_a} - 1\right) - s$$

$$h'_o = H\left(\frac{x_3 + X}{s + x_3 + X}\right)$$

$$t = \frac{h'_o - z_t\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{F\gamma_w}}$$

USE $F \geq 2.8$

Semipervious Berm, $k_t = k_{bl}$

$$X_{sp} = \frac{-A + \sqrt{A^2 - 24(2 + r)\left(1 + sc - \dfrac{H}{h_a}\right)}}{2c(2 + r)}$$

where:

$A = 6 + 3sc(r + 1)$

$r = i_o/i_1$

$h'_o = h_a\left[1 + cX + \left(\frac{2 + r}{6}\right)(cX)^2\right]$

$t = \dfrac{h'_o - i_o z_t}{1 + i_o}$

Sand Berm

$$X_s = \frac{1}{3}(X_p + 2X_{sp})$$

$$h'_o = h_a\left[1 + cX + \left(\frac{2 + r}{6}\right)(cX)^2\right]$$

$$t = \frac{h'_o - z_t\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{F\gamma_w}}$$

Pervious Berm with Collector

$$X_p = x_3 \log_e\left(\frac{h'_o}{h_a}\right)$$

$$h'_o = h_o = \frac{Hx_3}{s + x_3}$$

$$t = \frac{h'_o - z_t\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{F\gamma_w}}$$

$$Q_b = \frac{k_f H d}{s + x_3}\left(1 - e^{-\frac{X}{x_3}}\right)$$

Figure C-1. Design of landside seepage berms on impervious top stratum

b. Design criteria

(1) Where a levee overlies a top stratum creating a landside blanket and the upward gradient through the blanket at the landside toe of the levee is greater than 0.8, a seepage berm should be designed with an allowable upward gradient of 0.3 through the blanket and berm at the landside toe of the levee. For a

saturated unit soil weight of 1840 kg/m^3 (115 pcf), this is equivalent to a factor of safety of 2.8. The factor of safety of 2.8 applies only to new construction, not to existing projects. A factor of safety lower than 2.8 may be used, based on sufficient soil data and past performance in the area. The berm width should be based on an allowable upward gradient of 0.8 through the top stratum at the landside toe of the berm, subject to the limitations in the paragraphs which follow. The thickness of the berm should be increased 25 percent to allow for shrinkage, foundation settlements and variations in design factors. Where field observations during lesser floods indicate severe seepage problems would occur at the design flood, the berm dimensions should be extended.

(2) All berms should have minimum thickness of 1.52 m (5 ft) at the levee toe, a minimum thickness of 0.61 m (2 ft) at the berm crown, and a minimum width of 45.7 m (150 ft).

(3) For conditions where the computed upward gradient at the landside toe of the levee is between 0.5 and 0.8 without a berm, a berm with minimum dimensions as specified in (2) above should be constructed. Also for conditions where the computed gradient is less than 0.5, but either severe seepage has been observed or seepage is expected to become severe and soften the landside portion of the levee, the minimum berm should be constructed.

(4) The width of the berm is usually limited to about 91.4 to 121.9 m (300 to 400 ft), although the design calculations may indicate that a greater berm width is required. When the selected width of the berm is less than the calculated width, using berm design equations of Figure C-1, the head h_o' and berm thickness t at the levee toe will be less than for the computed width. For the selected berm, h_o' should be recomputed assuming an i_1 of 0.8 at the toe of the new berm and a linear piezometric grade line between the toe of the new berm and the point of effective seepage entry. The design thickness of the selected berm at the toe of the levee and the estimated seepage flow under the levee will be based on the value of h_o' corresponding to the selected berm.

(5) For conditions where no landside blanket exists, the necessity for a landside seepage berm will be based on the exit gradient and seepage velocity as discussed in paragraph B-5b. The berm thickness at the landside toe should be of such magnitude that the upward gradient i_o does not exceed 0.3. The design thickness of the berm should be increased by 25 percent to allow for shrinkage, foundation settlements, and variations in design factors. The head h_o' beneath the berm at the landside toe of the levee can be determined from Equation C-1.

$$h_o' = \frac{H(X + 0.43\,\overline{D})}{x_1 + L_2 + X + 0.43\,\overline{D}} \qquad \text{(C-1)}$$

In the above equation $\overline{D}$ is the transformed thickness of the pervious stratum which is equal to $d\sqrt{k_h/k_v}$, L_2 is the base width of the levee, H is the total net head on levee, X is the berm width, and x_1, is the effective length of impervious blanket riverside of the levee. If no riverside blanket exists, the value of x_1 is assumed to be 0.43 $\overline{D}$. The rate of seepage Q_s below the levee per unit length of levee can be determined using Equation C-2.

$$Q_s = \frac{k_f H d}{x_1 + L_2 + X + 0.43\,\overline{D}} \qquad \text{(C-2)}$$

In the equation above, k_f is the permeability of the pervious substratum and d is the effective thickness of the pervious substratum. H , x_1 , L_2 , X , and $\overline{D}$ are as previously defined. If Q_s exceeds 757.1 ℓ/min (200 gal/min) per 30.5 m (100 ft) of levee, a riverside blanket should be designed to reduce the seepage. Riverside blankets are discussed in paragraph 5-3.

(6) The slope of berms should be generally 1V on 50H or steeper to ensure drainage. If the berm is constructed after the levee has caused the foundation to consolidate fully, a slope of 1V on 75H can be used. Where wide, thick berms are required, consideration may be given to using a berm with a broken surface slope to more closely simulate the theoretical thickness and consequently reduce the cost of the berm. Where this is done, the steeper riverward slope of the berm should be no flatter than 1V on 75H and the landward slope of the berm should be no flatter than 1V on 100H.

(7) In short reaches where computations indicate no berm is necessary, but berms are required in adjacent reaches, it may be advisable to continue the berm construction through such reaches due to concentration of seepage in these areas. Also, in areas where entrance conditions in adjacent reaches are highly variable, potential adverse effects of close entry in adjacent reaches should be taken in to consideration.

C-4. Design Example

An example design problem with solution is presented in Table C-2 illustrating the design of impervious, semipervious, sand, and free draining landside seepage berms overlaying a thin landside top stratum. Each berm is designed for the same conditions using the design equations and design criteria as presented in this appendix.

Table C-2a
Examples of Design of Seepage Berms (Metric Units)

Designs based on following conditions:

$H = 7.6$ m	$z_{bl} = z_t = 1.83$ m	$\gamma' = 840.5$ kg/m^3 for impervious berms
$k_f = 1{,}000 \times 10^{-4}$ cm/sec	$i_o = 0.30$	$\gamma' = 920.6$ kg/m^3 for sand berm or pervious berm with collector, $F = 1.6$
$d = 30.5$ m	$i_1 = 0.80$	$F = 1.6$ for impervious berm
$k_{bl} = 3 \times 10^{-4}$ cm/sec	$\gamma' = 840.5$ kg/m^3	$L_3 = \infty$
$s = 304.8$ m	$x_3 = 137.2$ m	

	Required Berm			Suggested Design Dimensions				Suggested Construction Dimensions			Approximate[d] Material Required
Type Berm	Width X, m	Thickness[a] t, m	h_o'[b] m	Thickness at Berm Crown m	Berm Width X, m	Berm Slope	Approximate Thickness Levee Toe m	Thickness[c] at Berm Crown m	Berm Width X, m	Thickness[c] at Levee Toe m	m^3 per 100 m of Levee
Impervious	268.2	2.22	4.33	0.61	243.8[e]	1 on 75	3.87	0.76	243.8	4.85	73,266
		1.49	3.23	0.61	121.9	1 on 75	2.22	0.76	121.9	2.77	23,312
Semipervious	85.34	1.16	2.62	0.61	83.82	1 on 75	1.74	0.76	83.82	2.16	13,321
Sand	79.20	1.0	2.53	0.61	76.20	1 on 75	1.61	0.76	76.20	2.01	11,528
Pervious with collector	65.53	0.88	2.35	0.61	60.96	1 on 75	1.43	0.76	60.96	1.80	8,454[f]

[a] At toe of levee.
[b] Head at toe of levee with berm, measured above surface of natural ground.
[c] Thickness increased 25 percent for shrinkage, foundation settlements, and variations in design factors.
[d] Calculations based on suggested construction dimensions.
[e] Berm width considered longer than necessary. If boils developed 121.9 m or farther landward of the toe of the levee, the levee probably would not be endangered. Therefore, an alternate design for an impervious berm with a width of 121.9 m is also given.
[f] Sand and gravel blankets and collector system are also required.

Table C-2b
Examples of Design of Seepage Berms (English Units)

Designs based on following conditions:

H = 25 ft		z_{bl} = z = 6.0 ft		ã′ = 52.5 pcf for impervious berms	
k_f = 1,000 × 10^{-4} cm/sec		i_o = 0.30		ã′ = 57.5 pcf for sand berm or pervious berm with collector, F = 1.6	
d = 100 ft		i_1 = 0.80		F = 1.6 for impervious berm	
k_{bl} = 3 × 10^{-4} cm/sec		ã′ = 52.5 pcf		L_3 = ∞	
s = 1,000 ft		x_3 = 450 ft			

Type Berm	Required Berm Width X, ft	Required Berm Thickness[a] t, ft	Required Berm h_o' ft	Suggested Design Dimensions Thickness at Berm Crown ft	Suggested Design Dimensions Berm Width X, ft	Suggested Design Dimensions Berm[b] Slope	Approximate Thickness Levee Toe ft	Suggested Construction Dimensions Thickness[c] at Berm Crown ft	Suggested Construction Dimensions Berm Width X, ft	Suggested Construction Dimensions Thickness at Levee Toe ft	Approximate[d] Material Required yd per 100 ft of Levee
Impervious	880	7.3	14.2	2.0	800[e]	1 on 75	12.7	2.5	800	15.9	28,600
		4.9	10.6	2.0	400	1 on 75	7.3	2.5	400	9.1	9,100
Semipervious	280	3.8	8.6	2.0	275	1 on 75	5.7	2.5	275	7.1	5,200
Sand	260	3.3	8.3	2.0	250	1 on 75	5.3	2.5	250	6.6	4,500
Pervious with collector	215	2.9	7.7	2.0	200	1 on 75	4.7	2.5	200	5.9	3,300[f]

[a] At toe of levee.
[b] Head at toe of levee with berm, measured above surface of natural ground.
[c] Thickness increased 25 percent for shrinkage, foundation settlements, and variations in design factors.
[d] Calculations based on suggested construction dimensions.
[e] Berm width considered longer than necessary. If boils developed 400 ft or farther landward of the toe of the levee, the levee probably would not be endangered. Therefore, an alternate design for an impervious berm with a width of 400 ft is also given.
[f] Sand and gravel blankets and collector system are also required.

Appendix D
Filter Design

D-1. General

The objective of filters and drains used as seepage control measures for embankments is to efficiently control the movement of water within and about the embankment. In order to meet this objective, filters and drains must, for the project life and with minimum maintenance, retain the protected materials, allow relatively free movement of water, and have sufficient discharge capacity. For design, these three necessities are termed, respectively, piping or stability requirement, permeability requirement, and discharge capacity. This appendix will explain how these requirements are met for cohesionless and cohesive materials, and provide general construction guidance for installation of filters and drains. The terms filters and drains are sometimes used interchangeably. Some definitions classify filters and drains by function. In this case, filters must retain the protected soils and have a permeability greater than the protected soil but do not need to have a particular flow or drainage capacity since flow will be perpendicular to the interface between the protected soil and filter. Drains, however, while meeting the requirements of filters, must have an adequate discharge capacity since drains collect seepage and conduct it to a discharge point or area. In practice, the critical element is not definition, but recognition, by the designer, when a drain must collect and conduct water. In this case the drain must be properly designed for the expected flows. Where it is not possible to meet the criteria of this appendix, the design must be cautiously done and based on carefully controlled laboratory filter tests.

D-2. Stability

Filters and drains[1] allow seepage to move out of a protected soil more quickly than the seepage moves within the protected soil. Thus, the filter material must be more open and have a larger grain size than the protected soil. Seepage from the finer soil to the filter can cause movement of the finer soil particles from the protected soil into and through the filter. This movement will endanger the embankment.[2] Destruction of the protected soil structure may occur due to the loss of material. Also, clogging of the filter may occur causing loss of the filter's ability to remove water from the protected soil. Criteria developed by many years of experience are used to design filters and drains which will prevent the movement of protected soil into the filter. This criterion, called piping or stability criterion, is based on the grain-size relationship between the protected soil and the filter. In the following, the small character "d" is used to represent the grain size for the protected (or base) material and the large character "D" the grain size for the filter material.

Determine filter gradation limits using the following steps:[3]

1. Determine the gradation curve (grain-size distribution) of the base soil material. Use enough samples to define the range of grain sizes for the base soil or soils and design the filter gradation based on the base soil that requires the smallest D_{15} size.

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[1] In paragraphs D-2 and D-3 the criteria apply to drains and filters; for brevity, only the word filter will be used.

[2] In practice, it is normal for a small amount of protected soil to move into the filter upon initiation of seepage. This action should quickly stop and may not be observed when seepage first occurs. This is one reason that initial operation of embankment seepage control measures should be closely observed by qualified personnel.

[3] Guide for Determining the Gradation of Sand and Gravel Filters, Soil Mechanics Note No. 1, U.S. Department of Agriculture Soil Conservation Services, Engineering Division, January 1986.

2. Proceed to step 4 if the base soils contains no gravel (materials larger than No. 4 sieve).

3. Prepare adjusted gradation curves for base soils with particles larger than the No. 4 (4.75 mm) sieve.

a. Obtain a correction factor dividing 100 by the percent passing the No. 4 (4.75 mm) sieve.

b. Multiply the percentage passing each sieve size of the base soil smaller than No. 4 (4.75 mm) by the correction factor from step 3a.

c. Plot these adjusted percentages to obtain a new gradation curve.

d. Use the adjusted curve to determine the percent passing the No. 200 (0.075 mm) sieve in step 4.

4. Place the base soil in a category based on the percent passing the No. 200 (0.075 mm) sieve in accordance with Table D-1.

Table D-1
Categories of Base Soil Materials

Category	Percent Finer Than the No. 200 (0.075 mm) Sieve
1	>85
2	40-85
3	15-39
4	<15

5. Determine the maximum D_{15} size for the filter in accordance with Table D-2. Note that the maximum D_{15} is not required to be smaller than 0.20 mm.

6. To ensure efficient permeability, set the minimum D_{15} greater than or equal to 3 to 5 × maximum d_{15} of the base soil before regrading but no less than 0.1 mm.

7. Set the maximum particle size at 75 mm (3 in.) and the maximum passing the No. 200 (0.075 mm) sieve at 5 percent. The portion of the filter material passing the No. 40 (0.425 mm) sieve must have a plasticity index (PI) of zero when tested in accordance with EM 1110-2-1906, "Laboratory Soils Testing."

8. Design the filter limits within the maximum and minimum values determined in steps 5, 6, and 7. Standard gradations may be used if desired. Plot the limit values, and connect all the minimum and maximum points with straight lines. To minimize segregation and related effects, filters should have relatively uniform grain-size distribution curves, without "gap-grading"—sharp breaks in curvature indicating absence of certain particle sizes. This may require setting limits that reduce the broadness of filters within the maximum and minimum values determined. Sand filters with D_{90} less than about 20 mm generally do not need limitations on filter broadness to prevent segregation. For coarser filters and gravel zones that serve both as filters and drains, the ratio D_{90}/D_{10} should decrease rapidly with increasing D_{10} size. The limits in Table D-3 are suggested for preventing segregation during construction of these coarser filters.

Table D-2
Criteria for Filters

Base Soil Category	Base Soil Description and Percent Finer Than the No. 200 (0.075 mm) Sieve (a)	
1	Fine silts and clays; more than 85 percent finer	(c) $D_{15} \le 9 \times d_{85}$
2	Sands, silts, clays and silty and clayey sands; 40 to 85 percent finer	$D_{15} \le 0.7$ mm
3	Silty and clayey sands and gravels; 15 to 39 percent finer	(d), (e) $D_{15} \le \left(\frac{40 - A}{40 - 15}\right)$ $[(4 \times d_{85}) - 0.7 \text{ mm}] + 0.7$ mm
4	Sands and gravels; less than 15 percent finer	(f) $D_{15} \le 4$ to $5 \times d_{85}$

(a) Category designation for soil containing particles larger than 4.75 mm is determined from a gradation curve of the base soil which has been adjusted to 100 percent passing the No. 4 (4.75 mm) sieve.

(b) Filters are to have a maximum particle size of 75 mm (3 in.) and a maximum of 5 percent passing the No. 200 (0.075 mm) sieve with the plasticity index (PI) of the fines equal to zero. PI is determined on the material passing the No. 40 (0.425 mm) sieve in accordance with EM 1110-2-1906, "Laboratory Soils Testing." To ensure sufficient permeability, filters are to have a D_{15} size equal to or greater than $4 \times d_{15}$ but no smaller than 0.1 mm.

(c) When $9 \times d_{85}$ is less than 0.2 mm, use 0.2 mm.

(d) A = percent passing the No. 200 (0.075 mm) sieve after any regrading.

(e) When $4 \times d_{85}$ is less than 0.7 mm, use 0.7 mm.

(f) In category 4, the $D_{15} \le 4 \times d_{85}$ criterion should be used in the case of filters beneath riprap subject to wave action and drains which may be subject to violent surging and/or vibration.

Table D-3
D_{10} and D_{90} Limits for Preventing Segregation

Minimum D_{10} (mm)	Maximum D_{90} (mm)
< 0.5	20
0.5 - 1.0	25
1.0 - 2.0	30
2.0 - 5.0	40
5.0 - 10	50
10 - 50	60

D-3. Permeability

The requirement that seepage move more quickly through the filter than through the protected soil (called the permeability criterion) is again met by a grain-size relationship criterion based on experience:

Permeability

$$\frac{15 \text{ } percent \text{ } size \text{ } of \text{ } filter \text{ } material}{15 \text{ } percent \text{ } of \text{ } the \text{ } protected \text{ } soil} \ge 3 \text{ } to \text{ } 5 \tag{D-1}$$

Permeability of a granular soil is roughly proportional to the square of the 10- to 15-percent size material. Thus, the permeability criterion ensures that filter materials have approximately 9 to 25 or more times the permeability of the protected soil. Generally, a permeability ratio of at least 5 is preferred; however, in the case of a wide band of uniform base material gradations, a permeability ratio as low as 3 may be used with respect to the maximum 15-percent size of the base material. There may be situations, particularly where the filter is not part of a drain, where the permeability of the filter is not important. In those situations, this criterion may be ignored.

D-4. Applicability

The previously given filter criteria in Table D-2 and Equation D-1 are applicable for all soils (cohesionless or cohesive soils) including dispersive soils.[1] However, laboratory filter tests for dispersive soils, very fine silt, and very fine cohesive soils with high plastic limits are recommended.

D-5. Perforated Pipe[2]

The following criteria are applicable for preventing infiltration of filter material into perforated pipe, screens, etc.:

$$\frac{\textit{Minimum 50 percent size of filter material}}{\textit{hole diameter or slot width}} \geq 1.0 \qquad \text{(D-2)}$$

In many instances a filter material meeting the criteria given by Table D-2 and Equation D-1 relative to the material being drained is too fine to meet the criteria given by Equation D-2. In these instances, multilayered or "graded" filters are required. In a graded filter each layer meets the requirements given by Table D-2 and Equation D-1 with respect to the pervious layer with the final layer in which a collector pipe is bedded also meeting the requirements given by Equation D-2. Graded filter systems may also be needed when transitioning from fine to coarse materials in a zoned embankment or where coarse material is required for improving the water-carrying capacity of the system.

D-6. Gap-Graded Base

The preceding criteria cannot, in most instances, be applied directly to protect severely gap- or skip-graded soils. In a gap-graded soil such as that shown in Figure D-1. The coarse material simply floats in the matrix of fines. Consequently, the scattered coarse particles will not deter the migration of fines as they do in a well-graded material. For such gap-graded soils, the filter should be designed to protect the fine matrix rather than the total range of particle sizes. This is illustrated in Figure D-1. The 85-percent size of the total sample is 5.2 mm. Considering only the matrix material, the 85-percent size would be 0.1 mm resulting in a much finer filter material being required. This procedure may also be followed in some instances where the material being drained has a very wide range of particle sizes (e.g., materials graded from coarse gravels to significant percentages of silt or clay). For major structures such a design should be checked with filter tests.

1 Sherard, J. L., Dunnigan, L. P., "Filters and Leakage Control in Embankment Dams," *Proceeding of the Symposium on Seepage and Leakage from Dams and Impoundments*, ASCE National Convention, Denver, Colorado, 1985.

2 EM 1110-2-2300 states, "Collector pipe should not be placed within the embankment, except at the downstream toe, because of the danger of either breakage or separation of joints, resulting from fill placement and compaction operations, or settlement, which might result in either clogging and/or piping."

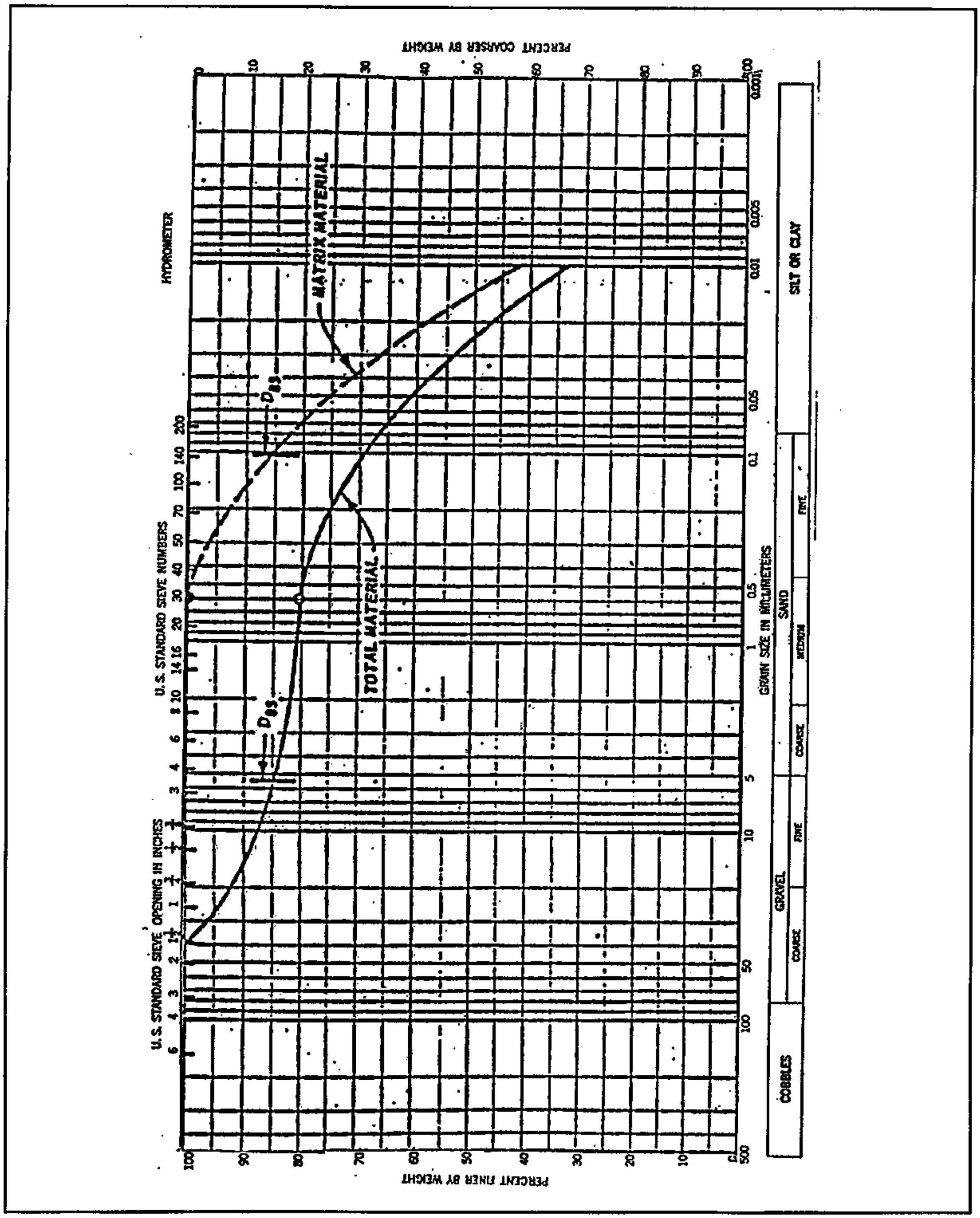

Figure D-1. Analysis of a gap-graded material

D-7. Gap-Graded Filter

A gap-graded filter material must never be specified or allowed since it will consist of either the coarse particles floating in the finer material or the fine material having no stability within the voids produced by the coarse material. In the former case the material may not be permeable enough to provide adequate drainage. The latter case is particularly dangerous since piping of the protected material can easily occur through the relatively large, loosely filled voids provided by the coarse material.

D-8. Broadly Graded Base

One of the more common soils used for embankment dams is a broadly graded material with particle sizes ranging from clay sizes to coarse gravels and including all intermediate sizes. These soils may be of glacial, alluvial-colluvial, or weathered rock origin. As noted by Sherard (1979)[1], since the 85-percent size of the soil is commonly on the order of 20 to 30 mm, a direct application of the stability criteria $D_{15}/d_{85} \leq 4$ to 5 would allow very coarse uniform gravel without sand sizes as a downstream filter, which would not be satisfactory. The typical broadly graded soils fall in Soil Category 2 in Table D-2 and require a sand or gravelly filter with $D_{15} \leq 0.7$ mm.

D-9. Example of Graded Filter Design for Drain

Seldom, if ever, is a single gradation curve representative of a given material. A material is generally represented by a gradation band which encompasses all the individual gradation curves. Likewise, the required gradation for the filter material is also given as a band. The design of a graded filter which shows the application of the filter criteria where the gradations are represented by bands is illustrated in Figure D-2. A typical two-layer filter for protecting an impervious core of a dam is illustrated. The impervious core is a fat clay (CH) with a trace of sand which falls in Category 1 soil in Table D-2. The criterion $D_{15} \leq 9 \times d_{85}$ is applied and a "point a" is established in Figure D-2. Filter material graded within a band such as that shown for Filter A in Figure D-2 is acceptable based on the stability criteria. The fine limit of the band was arbitrarily drawn, and in this example, is intended to represent the gradation of a readily available material. A check is then made to ensure that the 15-percent size of the fine limit of the filter material band (point b) is equal to or greater than 3 to 5 times the 15-percent size of the coarse limit of the drained material band (point c). Filter A has a minimum D_{10} size and a maximum D_{90} size such that, based on Table D-3, segregation during placement can be prevented. Filter A meets both the stability and permeability requirements and is a suitable filter material for protecting the impervious core material. The second filter, Filter B, usually is needed to transition from a fine filter (Filter A) to coarse materials in a zoned embankment dam. Filter B must meet the criteria given by Table D-2 with respect to Filter A. For stability, the 15-percent size of the <u>coarse</u> limit of the gradation band for the second filter (point d) cannot be greater than 4 to 5 times the 85-percent size of the <u>fine</u> limit of the gradation bank for Filter A (point e). For permeability, the 15-percent size of the <u>fine</u> limit (point f) must be at least 3 to 5 times greater than the 15-percent size of the <u>coarse</u> limit for Filter A (point a). With points d and f established, the fine and coarse limits for Filter B may be established by drawing curves through the points approximately parallel to the respective limits for Filter A. A check is then made to see that the ratio of maximum D_{90}/minimum D_{10} size Filter B is approximately in the range as indicated in Table D-3. A well-graded filter which usually would not meet the requirements in Table D-3 may be used if segregation can be controlled during placement. Figure D-2 is intended to show only the principles of filter design. The design of thickness of a filter for sufficient seepage discharge capacity is done by applying Darcy's Law, $Q = kia$.

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[1] Sherard, J. L., "Sinkholes in Dams of Coarse, Broadly-Graded Soils," <u>Transactions of the 13th International Congress on Large Dams</u>, New Delhi, India, Vol. II, 1979, pp 25-35.

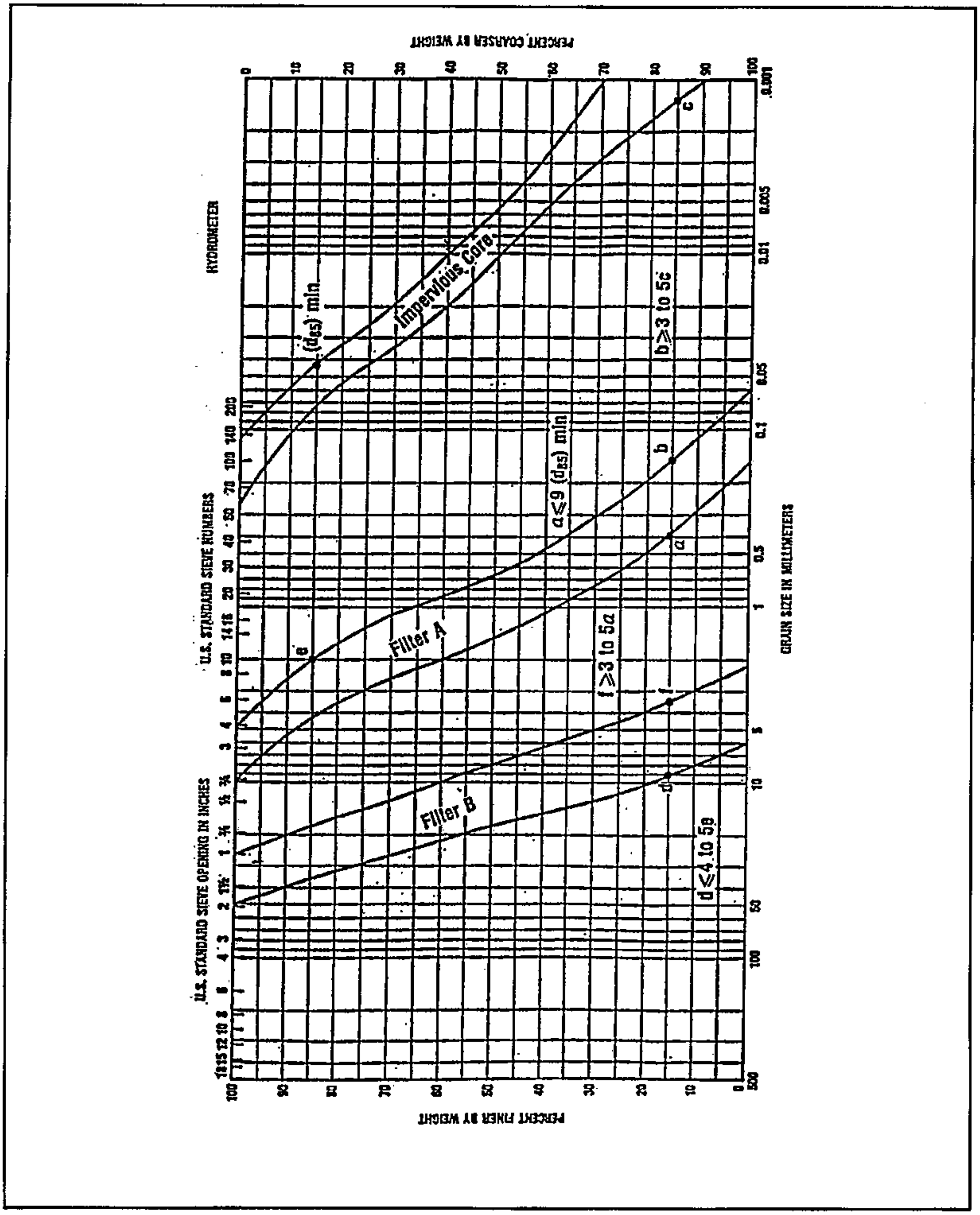


Figure D-2. Illustration of the design of a graded filter

D-10. Construction

EM 1110-2-1911 and EM 1110-2-2300 provide guidance for construction. Major concerns during construction include:

a. Prevention of contamination of drains and filters by runoff containing sediment, dust, construction traffic, and mixing with nearby fine-grained materials during placement and compaction. Drain and filter material may be kept at an elevation higher than the surrounding fine-grained materials during construction to prevent contamination by sediment-carrying runoff.

b. Prevention of segregation, particularly well-graded filters, during handling and placement.

c. Proper in-place density is usually required to be no less than 80-percent relative density. Granular materials containing little or no fines should be saturated during compaction to prevent "bulking" (low density) which can result in settlement when overburden materials are placed and the drain is subsequently saturated by seepage flows.

d. Gradation should be monitored closely so that designed filter criteria are met.

e. Thickness of layers should be monitored to ensure designed discharge capacity and continuity of the filter.

Thus, quality control/assurance is very important during filter construction because of the critical function of this relatively small part of the embankment.

D-11. Monitoring

Monitoring of seepage quantity and quality (see Chapter 13 of EM 1110-2-1901 for method of monitoring seepage) once the filter is functioning is very important to the safety of the embankment. An increase in seepage flow may be due to a higher reservoir level or may be caused by cracking or piping. The source of the additional seepage should be determined and action taken as required (see Chapters 12, 13, and 14 of EM 1110-2-1901). Decreases in seepage flows may also signal dangers such as clogging of the drain(s) with piped material, iron oxide, calcareous material, and effects of remedial grouting. Again, the cause should be determined and appropriate remedial measures taken. Drain outlets should be kept free of sediment and vegetation. In cold climates, design or maintenance measures should be taken to prevent clogging of drain outlets by ice.

Appendix E
Drainage Trench

E-1. General

This appendix presents the design and analysis of drainage trenches. The design criteria and the example presented are taken from U.S. Army Engineer Waterways Experiment Station TM 3-424 (Appendix A).

E-2. Applicability

A drainage trench can be used to control underseepage where the top stratum is thin and the pervious foundation is relatively shallow so that the trench substantially penetrates the aquifer. Where the pervious foundation is deep, a drainage trench of moderate depth would attract only a small portion of the underseepage. The drainage trench method is known to be effective where the ratio of the thickness of the landside blanket, z_{bl}, to the depth of the pervious foundation, d, is greater than 25 percent. While only substantial penetration by the drainage trench provides significant landside relief, a trench with limited penetration may be used in conjunction with a landside blanket to contain seepage pressures.

E-3. Design Criteria

The design criteria and graphs are applicable only for homogeneous, isotropic pervious foundations having an impervious top stratum landward of the drainage trench. The distance from the source of seepage to the landside toe of the levee, S, to be used in the design may be estimated by a procedure outlined in Appendix B. Seepage into a drainage trench, Q_{st}, and the maximum head landside of the levee, h_∞, where the blanket landside of the levee consists of impervious or relatively impervious soil, can be computed using the graphs presented in Figure E-1. The analysis and design procedure is as follows:

a. Where $k_h > k_v$ transform the in situ pervious stratum into a homogeneous, isotropic formation using k'_f and d' for k_f and d, respectively, as follows:

$$k'_f = \sqrt{k_h\, k_v} \tag{E-1}$$

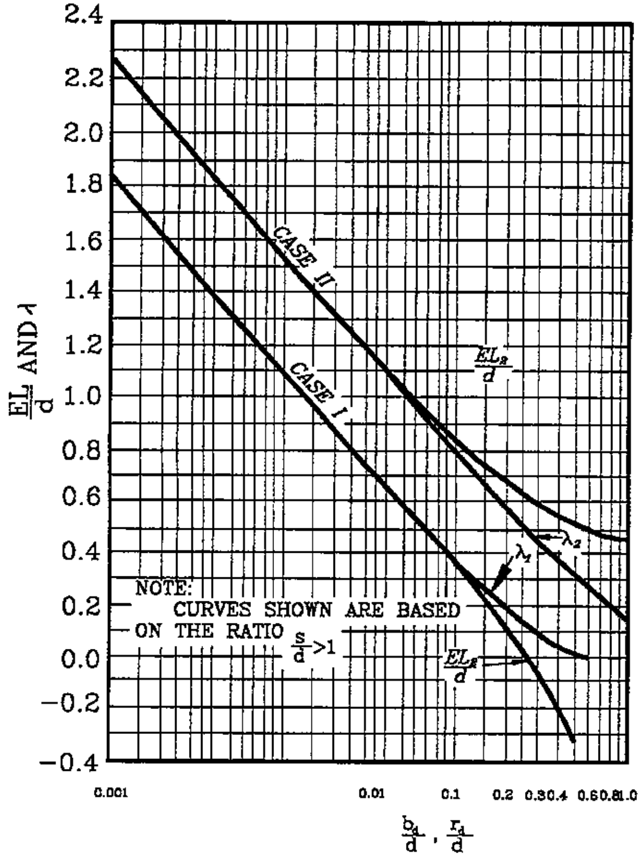
$$d' = d\sqrt{k_h/k_v} \tag{E-2}$$

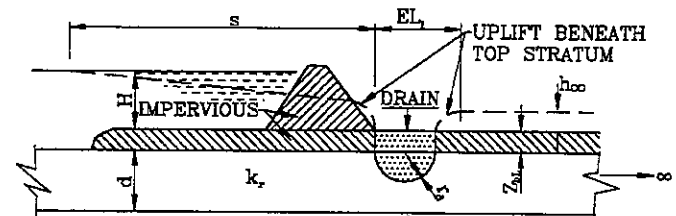
where

d = thickness of the pervious foundation
k_h = coefficient of horizontal permeability
k_v = coefficient of vertical permeability
k'_f = transformed coefficient of permeability of the foundation
d' = transformed depth of the pervious foundation

b. From the geometry of the drainage trench, Figure E-1, find the ratio of r_d/d, (Case I) or b_d/d (Case II) where:

r_b = radius of the circular sector of the trench for Case I
b_d = width of the rectangular trench section for Case II

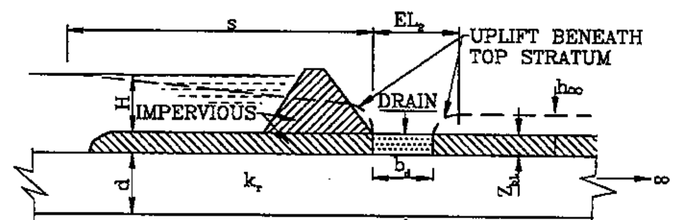




SHAPE FACTOR $\$ = \dfrac{1}{\frac{S+r_d}{d} + \frac{EL_1}{d}}$

MAXIMUM NET UPLIFT RATIO UNDER TOP IMPERVIOUS BLANKET LANDSIDE OF DRAIN IN $\dfrac{h_\infty}{H} = \$_1\ \lambda_1$

CASE I



SHAPE FACTOR $\$ = \dfrac{1}{\frac{S}{d} + \frac{EL_2}{d}}$

MAXIMUM NET UPLIFT RATIO UNDER TOP IMPERVIOUS BLANKET LANDSIDE OF DRAIN IN $\dfrac{h_\infty}{H} = \$_2\ \lambda_2$

CASE II

SEEPAGE PER UNIT LENGTH OF STRUCTURE IS A $Q = k_f\ H\$$

NOTE:
WHERE $K_H > K_V$, TRANSFORM PERVIUS SUBSTRATUM TO A HOMOGENEOUS, ISOTROPIC FOUNDATION AND USE $\bar{k}_f$ FOR k_f AND $\bar{d}$ FOR d IN ABOVE FORMAULAS.

$\bar{k}_f = \sqrt{k_h\ k_v}$ AND $\bar{d} = d\sqrt{\frac{k_H}{k_v}}$

Figure E-1. Formulas and design curves for drainage trenches (ref. EM 1110-2-1601) *c.* Use the computed ratio of r_d/d or b_d/d to enter the appropriate graph of Figure E-1 to determine the corresponding value of EL/d and ë. The factor EL is the extra length of pervious substratums corresponding to the increased resistance to flow into a drainage trench as compared to a fully penetrated open trench, and ë is an uplift factor. The values of EL_1/d and ë, are related to Case I, while EL_2/d and $ë_2$ are related to Case II.

d. Once the magnitude of EL is determined, the value of the shape factor \$ which is equivalent to the ratio in the flow net of the number of flow channels to the number of equipotential drops, can be determined as:

Case I:

$$\$_1 = \frac{d}{S + r_d + EL_1} \tag{E-3}$$

Case II:

$$\$_2 = \frac{d}{S + EL_2} \tag{E-4}$$

e. Calculate the quantity of discharge per unit length of the levee, Q_{st}, and the maximum head landside of the trench, h_∞ , using the following expressions:

$$Q_{st} = \$ \, k'_f \, H \tag{E-5}$$

$$h_\infty = H \, \$ \, ë \tag{E-6}$$

where

H = total head acting on the levee, or the height of flood stage above the average low-ground surface or tail water

Where there is no top stratum landside of the levee, seepage flow into the drainage trench and beyond can be estimated from the flow net analysis.

E-4. Limitations of the Method

The method of controlling underseepage using the trench drain method has several limitations:

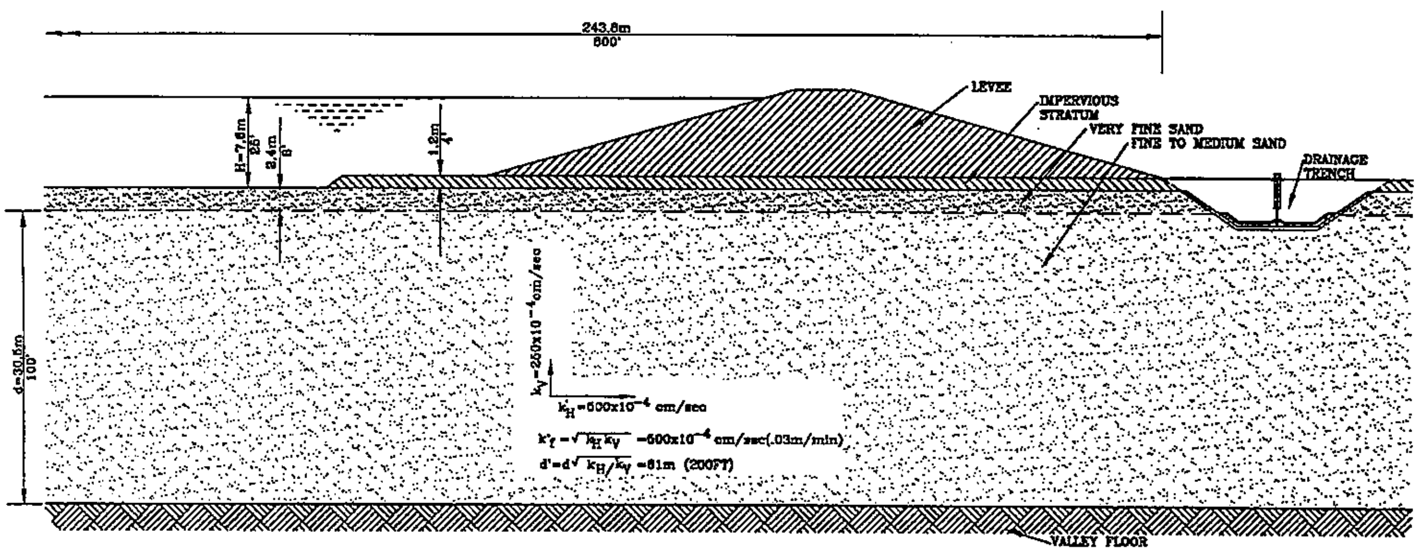
a. If the top stratum landside of the drainage trench has a certain degree of perviousness, seepage into the trench and the maximum head landward of the levee will be somewhat less than those computed from Figure E-1. Therefore, design based on Figure E-1 will be slightly on the conservative side if the top stratum landside of the drain trench is semipervious.

b. If the pervious foundation is highly stratified, seepage may bypass the drain and emerge landward of the drain thereby defeating its purpose. For such cases, other methods of seepage control are more effective.

c. If the trench is underlain by either impervious or semipervious strata of either clay, silt, or fine sand, the formulas presented in Part E-3 are no longer applicable.

E-5. Design Example

Figure E-2 illustrates the design of a drainage trench in a thin impervious blanket overlying a shallow pervious stratum. The trench drain (Case II) is designed using equations presented in Part E-3 of this appendix.



CROSS SECTION OF LEVEE, FOUNDATION AND DRAINAGE TRENCH

b_d/d =7.6m(25')/61m(200') =0.125 ; EL_2/d' =0.80 and λ_2=0.75

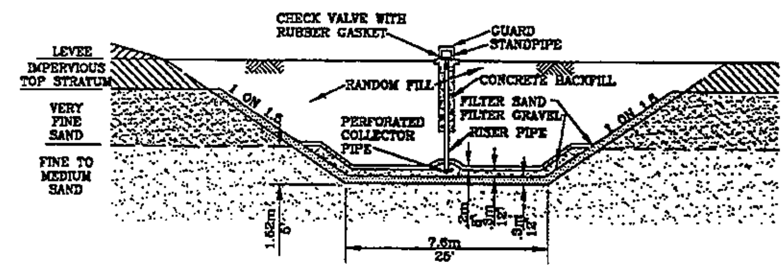
$$\oint = \frac{1}{S/d'+EL_2/d'} = \frac{1}{243.8m(800')/61m(200')+0.8} = \frac{1}{4.8} = .208$$

SEEPAGE FLOW PER UNIT LENGTH OF LEVEE, Q_s:

$Q_s = \oint$ k H = 0.208x.03m(.1')/min x 7.6m(25') =.0474m^3/mpm OF LEVEE
or 1.445m^{3}pm/30.5m(100') OF LEVEE

MAXIMUM HEAD BENEATH LANDSIDE BLANKET, h_∞ :

$h_\infty/H = \oint$ .208x.75=.156 OR 15.6%
h_∞=1.56xH=.156x7.6m(25')= 1.17m(3.9')



DETAILS OF DRAINAGE TRENCH

Figure E-2. Example of design of a drainage trench

Appendix F
Emergency Flood Protection

F-1. Introduction

a. Flood fighting. Flood fighting can be defined as those emergency operations that are taken in advance of and during a flood to prevent or minimize damages to public and private property. As defined herein, flood fighting includes the hasty construction of emergency levees; the overbuilding of existing levees or natural river banks; ring and U-shaped levees constructed around facilities or areas of high property value; preservation of vital facilities including water treatment plants and wells; power and communication facilities; protection of sanitary and storm sewer systems; and provisions for interior drainage treatment during flood stages. Flood fighting plans should acknowledge that it may not be feasible to protect entire communities based on economic or time and equipment considerations; therefore, evacuation of certain areas may be a necessary facet of an emergency operation.

b. Recommended local organization. Each community with a flood history should establish an organization and written plans for the purpose of conducting flood fighting operations. These plans should include identification of flood-prone areas and previous high water marks; flood fighting or evacuation plans; delegation of responsibilities; lists of important suppliers of materials and special equipment; lists of local contractors; and establishment of earth borrow sites, etc. The plan should further provide for the establishment of an emergency operation center and list various assistance programs available, either through the State or Federal government. Further assistance in developing these plans can be provided by the State or local Civil Defense Director in the area.

F-2. Flood Barrier Construction

a. Introduction. The two basic features of an emergency levee system include the flood barrier, generally constructed of earth fill, and the related interior drainage treatment. It is desirable that individuals assigned to a flood-fight situation have prior knowledge of flood barrier construction, interior drainage, and related flood-fight problems which they may encounter. They should also be acquainted with the past flood emergency efforts, historical flood stages, and forecasted stages for the community. The following information will provide personnel with guidelines based on actual experience. However, it cannot be over emphasized that individual resourcefulness is a key element in a successful flood fight.

b. Preliminary work.

(1) Alignment. A complete alignment for the barrier should be established promptly and, if possible, in cooperation with State or local floodplain management officials. The alignment should be the shortest practical route, provide the maximum practical protection, and take advantage of any high ground where practicable. The flood barrier should be kept as far landward of the river as possible to prevent encroachment on the floodway and to provide maximum space for overbank flows. This is especially important for smaller floodways where encroachment would directly impact the water surface profile. Sharp bends should be avoided. Topographic, plat, or city street maps may be useful in selecting alignment. In choosing the alignment, consideration should be given as to whether sufficient time remains to complete construction before the flood crest arrival. Potentially unstable river banks should be avoided. Keep as many trees and brush between the levee and river as possible to help deflect current, ice, and debris. However, in constricted areas of the river, 1.52 m (5 ft), and preferably 3.05 m (10 ft), should be allowed between the levee toe and vertical obstructions such as trees. In urban areas, many communities within a flood prone

area already have some levees in-place. These communities typically fight the flood along this primary line of defense. Moving the alignment farther landward creates problems in determining methods to stop floodwater backup through storm and sanitary sewer lines. It could also leave storm and sanitary lift stations on the riverside of the flood barrier. Leaving some homes outside the line of protection also exposes the watermains to floodwater infiltration. Right-of-way considerations may also influence the final alignment. Generally, the city or county engineer will assist in laying out the line and grade for the barrier, or a professional surveyor may be available. However, if help is not available, a hand-level along with a known elevation can be used to lay out rough grade. As soon as the alignment is firm, quantities of earthwork should be estimated for establishing equipment and borrow requirements.

(2) Drainage. In laying out a flood barrier, the problem of interior drainage from snowmelt, rain, or sewer backup should be considered. A certain amount of ponding, if valuable property will not be damaged, is not detrimental and may be allowed. The excess interior water can be pumped out over the levee if pumps are available.

(3) Borrow area and haul road. The two prime requisites for a borrow area are that adequate material be available and that the site be accessible at all times. The quantity estimate plus an additional 50 percent should provide the basis for the area requirement. The area must be located so that it will not become isolated from the project by high water. The borrow area should also be located where the present water table, if known, and the water table levels caused by high water will not hinder or stop its use. If possible, a borrow area should be selected which will provide suitable materials for levee construction as covered below. Local contractors and local officials are the best source of information on available borrow areas. If undeveloped, the area should be cleared of brush, trees, and debris, with topsoil and surface humus being stripped. In cold regions in early spring, it will probably be necessary to rip the area to remove frozen material. An effort should be made to borrow from the area in such a manner that the area will be relatively smooth and free-draining when the operation is complete. The haul road may be an existing road or street, or it may have to be constructed. To mitigate damages it is highly desirable to use unpaved trails and roads, or to construct a road if the haul distance is short. In any case, the road should be maintained to avoid unnecessary traffic delays. The use of flagmen and warning signs is mandatory at major crossings such as highways, near schools, and at major pedestrian crossings. A borrow area, or source of sand for sandbags, should also be located promptly. This can become a critical item of supply in some areas due to long haul, project isolation, etc. It may become necessary to stockpile material near anticipated trouble areas.

(4) Equipment. One of the important considerations in earthwork construction is the selection of proper equipment to do the work. Under emergency conditions, obtaining normally specified earthwork equipment will be difficult and the work will generally be done with locally available equipment. It may be wise to call for technical assistance in the early contract stage to insure that proper and efficient equipment use is proposed. If possible, compaction equipment should be used in flood-barrier construction. This may involve sheepsfoot, rubber-tired, or vibratory rollers. Scrapers should be used for hauling when possible because of speed (on short haul) and large capacity. Truck haul, however, has been the most widely used. A ripper will be necessary for opening borrow areas in the early spring if the ground is frozen. A bulldozer of some size is mandatory on the job to help spread dumped fill and provide minimum compaction.

(5) Construction contract. The initiation of a construction contract under emergency conditions is very unique in that sole judgment as to the competence and capabilities of the contractor lies with field personnel. Field personnel, therefore, must be somewhat knowledgeable in construction operations. The initial contract is very important in that it delineates what equipment must be accounted for on the project and what is available for construction. During construction, if it becomes obvious that the equipment provided by the initial contract is inadequate to provide reasonably good construction or timely completion, a new or supplemental contract may be required. Procedures are the same as in the initial contract. Flexibility may be

built into the original contract if it can be foreseen that additional pieces of equipment will ultimately be used.

(6) Supplies. Early anticipation of floodfight problems will aid greatly in providing necessary and sufficient supplies on hand. These include sandbags, polyethylene, pumps, etc. The importance of initiative, resourcefulness, and foresight of the individual on the project cannot be over emphasized. Technical assistance may be invaluable in locating potential problem areas which, with proper materials at hand, could be alleviated early.

c. Earth fill levees.

(1) Foundation preparation. Prior to embankment construction, the foundation area along the levee alignment should be prepared. This is particularly important if the levee is to be left in place. Since spring flooding is the only condition providing much advance warning, the first item of work in cold regions probably will be snow removal. The snow should be pushed riverward so as to decrease ponding when the snow melts. Trees should be cut and the stumps removed. All obstructions above the ground surface should be removed, if possible. This will include brush, structures, snags, and similar debris. The foundation should then be stripped of topsoils and surface humus. (Clearing and grubbing, structure removal, and stripping should be performed only if time permits.) Stripping may be impossible if the ground is frozen. In this case, the foundation should be ripped or scarified, if possible, to provide a rough surface for bond with the embankment. Every effort should be made to remove all ice or soil containing many ice lenses. Frost or frozen ground can give a false sense of security in the early stages of a flood fight. It can act as a rigid boundary and support the levee; but on thawing, soil strength may be reduced sufficiently for cracks or slides to develop. It also forms an impervious barrier to prevent seepage. This may result in a considerable buildup in pressure under the soils landward of the levee, and upon thawing pressure may be sufficient to cause sudden blowouts. If this condition exists it must be monitored, and one must be prepared to act quickly if sliding or sand boils develop. If stripping is possible, the material should be pushed landward and riverward of the toe of levee and windrowed. After the flood, this material may be spread on the slopes to provide topsoil for vegetation.

(2) Materials. Earth fill materials for emergency levees will usually come from local borrow areas. An attempt should be made to utilize materials which are compatible with the foundation materials. Due to time limitations, however, any local materials may be used if reasonable construction procedures are followed. The material should be relatively clean (free of debris) and should not contain large frozen pieces of earth.

(a) Clay. Clay is preferred because the section can be made smaller (steeper side slopes). Clay is also relatively impervious (will not readily permit passing of water) and has relatively high resistance to erosion in a compacted state. A disadvantage in using clay is that adequate compaction is difficult to obtain without proper equipment and when the material is wet. Another disadvantage is if the clay is wet and sub-freezing temperatures occur, this may cause the material to freeze in the borrow pit and hauling equipment. Weather could cause delays and should definitely be considered in the overall construction effort.

(b) Sand. If sand is used, the section should comply as closely as possible with recommendations in paragraph F-2.C.(3)(b) below. Steep slopes without poly coverage on the riverside slope will result in seepage through the levee that exits on the landward slope causing slumping of the slope and potential overall failure if it occurs over an extended period of time.

(c) Silt. Material which is primarily silt should be avoided. If used, poly facing must be applied to the river slope. In addition to being very erodible, silt, upon wetting, tends to collapse if not properly compacted.

(3) Levee section.

(a) General. In standard levee design the configuration of the levee is generally dictated by the foundation soils and the materials available for construction. Therefore, even under emergency conditions, an attempt should be made to make the embankment compatible with the foundation. Information on foundation soils may be available from local officials or engineers, and it should be utilized. The two levee sections cited below are classical and idealized, and usual field conditions depart from them to various degrees. However, if they are used as a guide, possible serious flood-fight problems could be lessened during high water. In determining the top width of any type of section, consideration should be given as to whether a revised forecast will require additional fill to be placed. A top width adequate for construction equipment will facilitate raising the levee. Finally, actual dike construction will, in most cases, depend on time, materials, and right-of-way available.

(b) Sand section. Use 1 V (Vertical) on 3 H (Horizontal) toward a river, 1 V on 5 H landward slope, and 3.05-m (10-ft) top width.

(c) Clay section. Generally 1 V on 2 1/2 H slopes are used but for low height levees 1 V on 2 H slopes have been used successfully. It is important to always use a 3.05 m (10 ft) top width. When clay levees are constructed on pervious foundations, the bottom width may not be adequate to reduce the potential for foundation piping. This can be accomplished by using berms either landward or riverward of the levee. Berm thickness will be site specific. Berms reduce the potential for foundation piping, but do not reduce foundation seepage.

(4) Placement and compaction. As stated above, obtaining proper compaction equipment for a given soil type will be difficult. It is expected in most cases that the only compaction will be from that due to the hauling and spreading equipment; i.e., construction traffic routed over the fill. Levee height should provide 0.61 m (2 ft) of freeboard above forecast flood crest. In urban areas, the upstream end of the project should use a larger freeboard than the downstream end.

(5) Slope protection.

(a) General. Methods of protecting levee slopes from current scour, wave wash, seepage, and debris damage are numerous and varied. However, during a flood emergency, time, availability of materials, cost and construction capability preclude the use of all accepted methods of permanent slope protection. Field personnel must decide the type and extent of slope protection the emergency levee will need. Several methods of protection have been established which prove highly effective in an emergency. Again, resourcefulness on the part of the field personnel may be necessary for success.

(b) Polyethylene and sandbags. Experience has shown that a combination of polyethylene (poly) and sandbags is one of the most expedient, effective, and economical methods of combating slope attack in a flood situation. Poly and sandbags can be used in a variety of combinations, and time becomes the factor that may determine which combination to use. Ideally, poly and sandbag protection should be placed in the dry. However, many cases of unexpected slope attack will occur during high water, and a method for placement in the wet is covered below. See Figures F-1 to F-4 for suggested methods of laying poly and sandbags. Since each flood fight project is generally unique (river, personnel available, materials, etc.), specific details of placement and materials handling will not be covered. Personnel must be aware of resources available when using poly and sandbags.

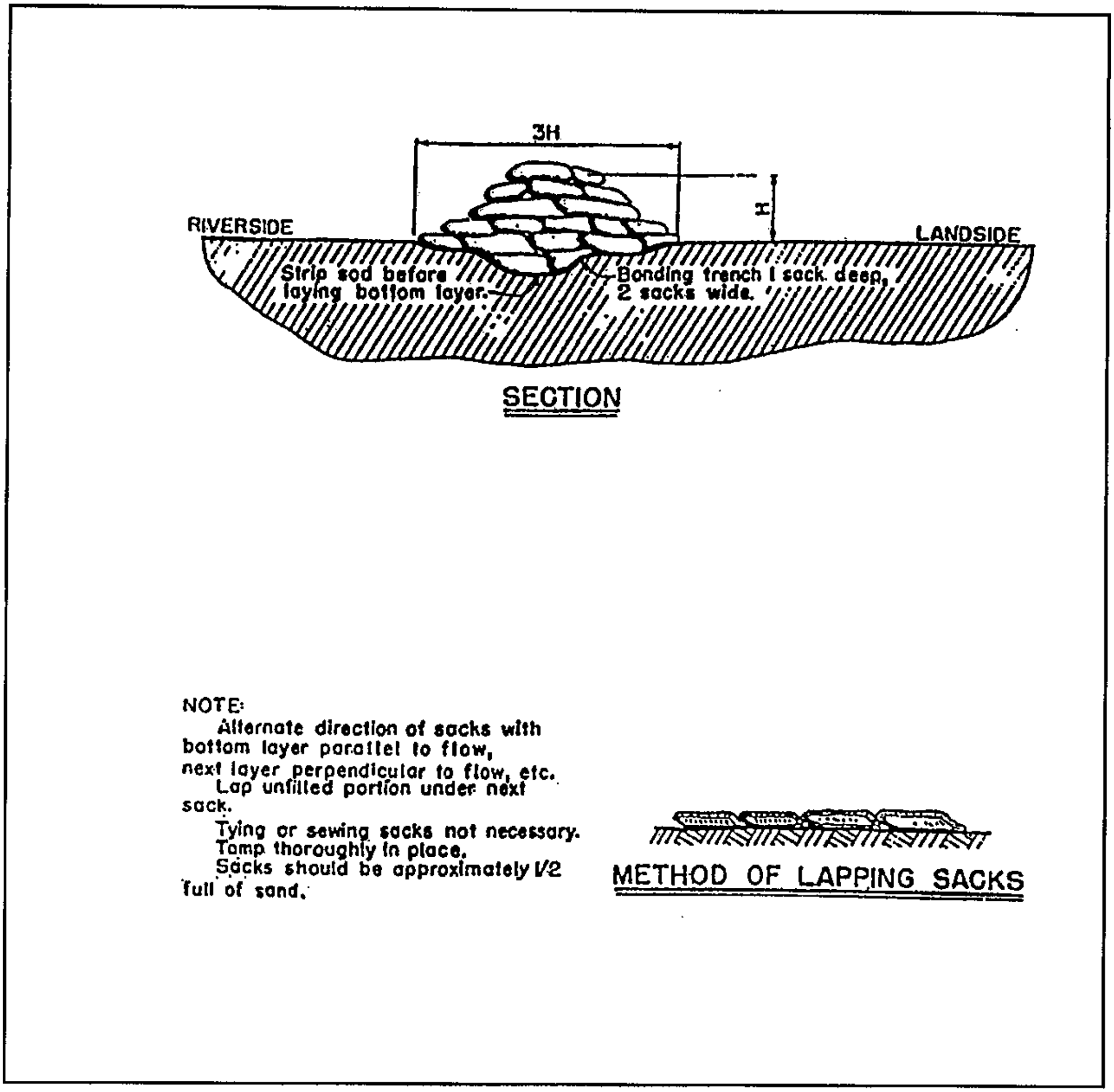


Figure F-1. Sandbag barrier

(c) Toe anchorage and poly placement. Anchoring the poly along the riverward toe is important for a successful job. It may be done in three different ways: (1) After completion of the levee, a trench excavated along the toe, poly placed in the trench, and the trench backfilled; (2) Poly placed flat-out away from the toe, and earth pushed over the flap; and (3) Poly placed flat-out from the toe and one or more rows of sandbags placed over the flap. The poly should then be unrolled up the slope and over the top enough to allow for anchoring with sandbags. Poly should be placed from downstream to upstream along the slopes and overlapped at least 0.61 m (2 ft). The poly is now ready for the "hold-down" sandbags.

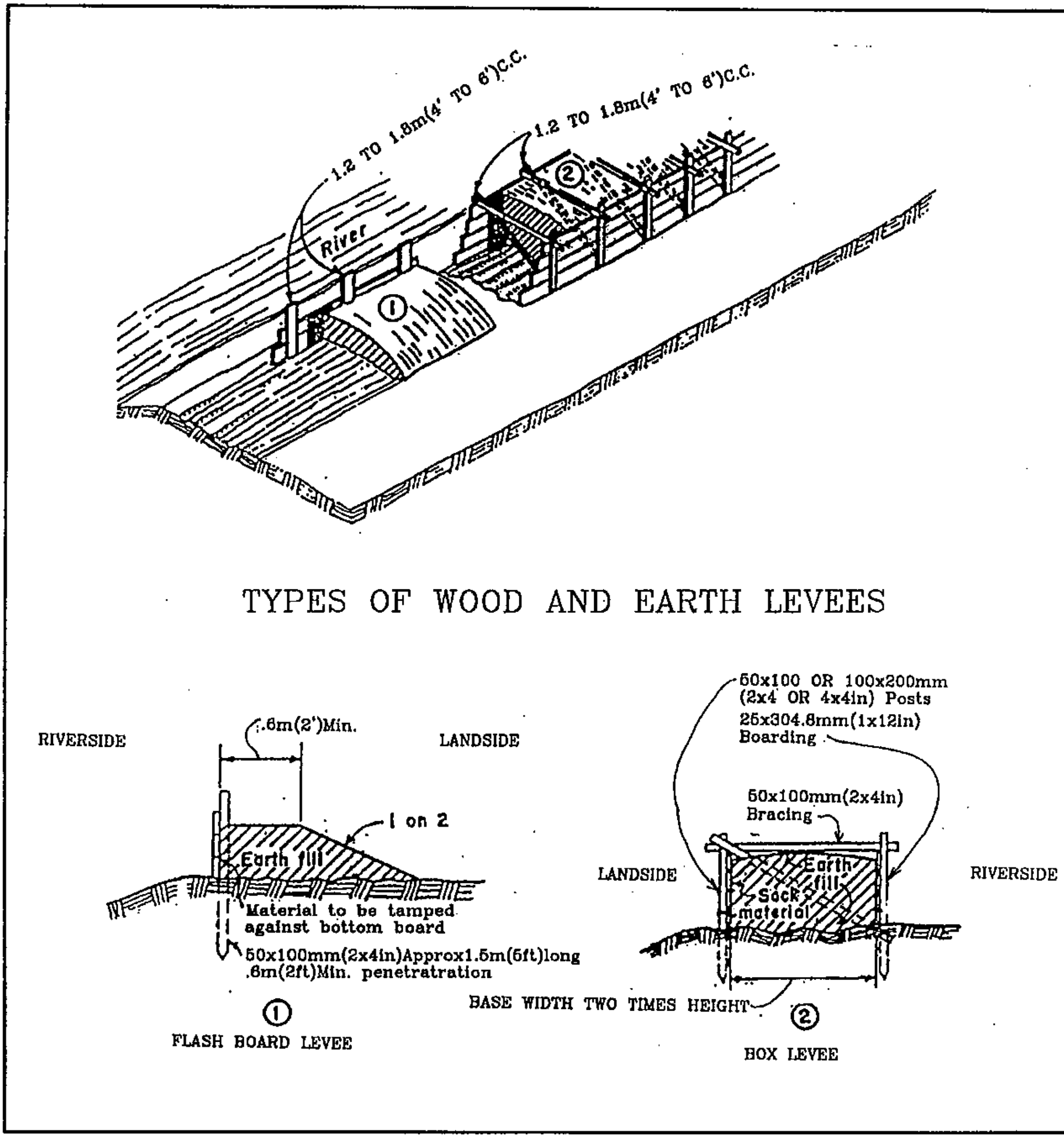


Figure F-2. Flash board and box levee

(d) Slope anchorage. It is mandatory that poly placed on levee slopes be held down. An effective method of anchoring poly is a grid system of sandbags, unless extremely high velocities, heavy debris, or a large amount of ice is anticipated. Then a solid blanket of bags over the poly should be used. A grid system can be constructed faster and requires fewer bags and much less labor than a total covering. Various grid systems include vertical rows of lapped bags, two-by-four lumber held down by attached bags, and rows

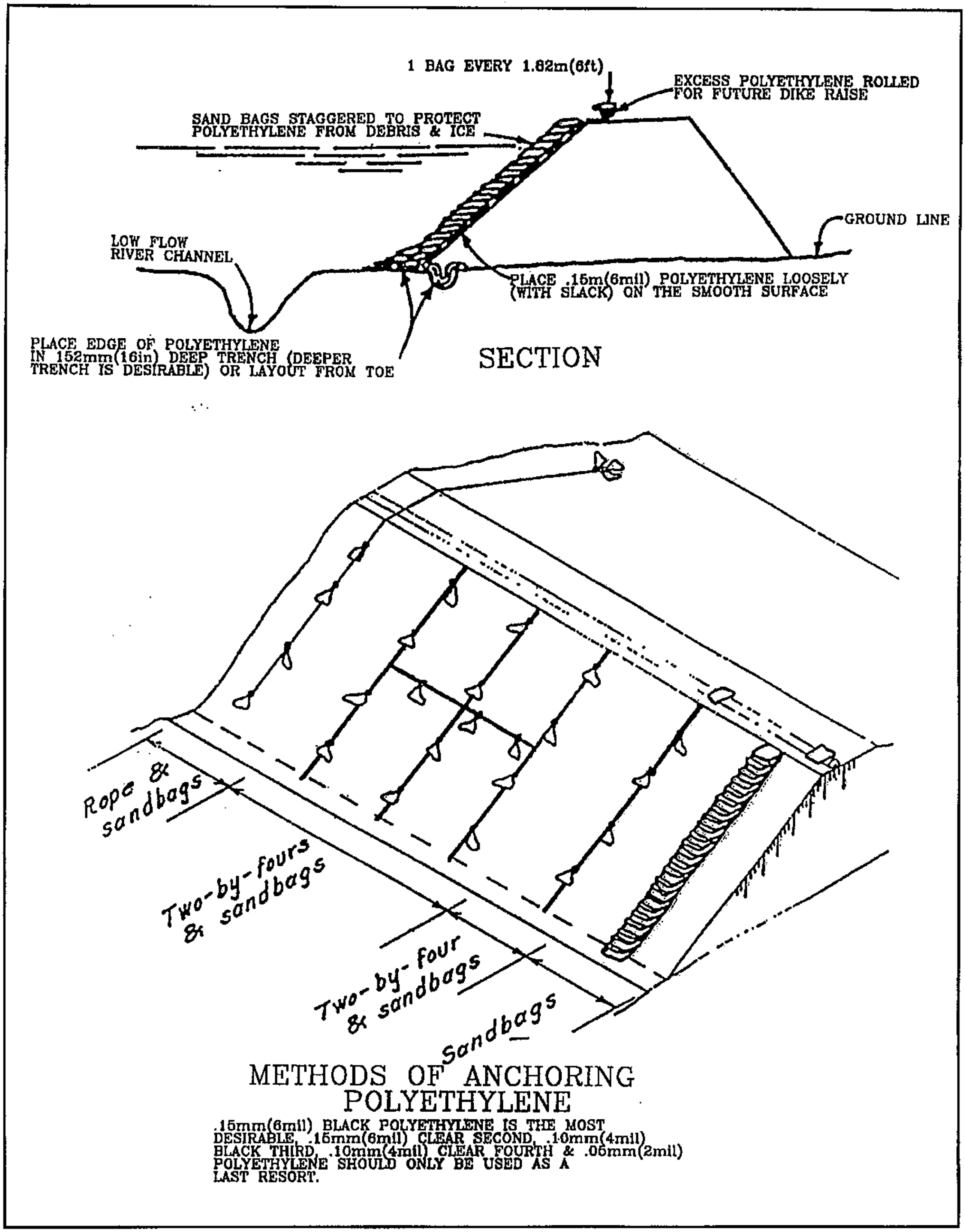


Figure F-3. Placement of polyethylene sheeting on temporary levee

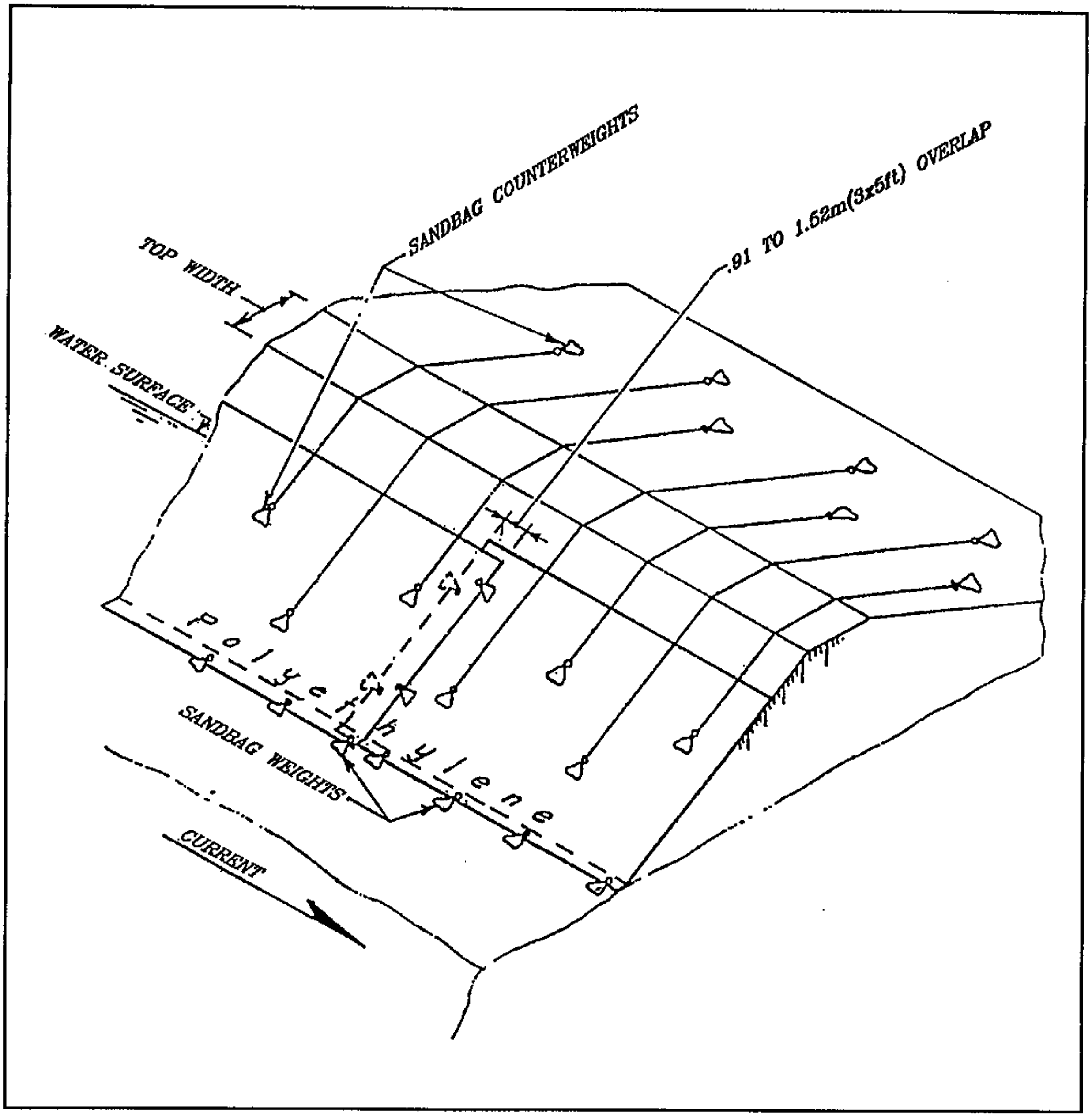


Figure F-4. Placement of polyethylene sheeting in the wet

of bags held by a continuous rope tied to each bag. Poly can also be held down by a system using two bags tied with rope and the rope saddled over the levee crown with a bag on each slope.

(e) Placement in the wet. In many situations during high water, poly and sandbags placed in the wet must provide the emergency protection. Wet placement may also be required to replace or maintain damaged poly or poly displaced by current action. Figure F-4 shows a typical section of levee covered in the wet. Sandbag anchors are formed at the bottom edge and ends of the poly by bunching the poly around a fistful of hand or rock and tying the sandbags to this fist-sized ball. Counterweights consisting of two or more sandbags connected by a length of 6.35-mm (1/4-in.) rope are used to hold the center portion of the poly down. The number of counterweights will depend on the uniformity of the levee slope and current velocity. Placement of the poly consists of first casting out the poly sheet with the bottom weights and then adding counterweights to slowly sink the poly sheet into place. The poly, in most cases, will continue to move down slope until the bottom edge reaches the toe of the slope. Sufficient counterweights should be added to insure that no air voids exist between the poly and the levee face and to keep the poly from flapping or being carried away in the current. For this reason, it is important to have enough counterweights prepared prior to the placement of the sheet.

(f) Overuse of poly. In past floods there has been a tendency to overuse and in some cases misuse poly on slopes. For example, on well compacted clay embankments, in areas of relatively low velocities, use of poly would be unnecessary. Also, placement of poly on landward slopes to prevent seepage must not be done. It will only force seepage to another exit and may prove detrimental. Poly has been used on the landside slope of levees to prevent rainwater from entering a crack where slope movement has occurred, particularly in fat clay soils. Keeping water out of the cracks resulting from slope movements is desirable to prevent lubrication and additional hydrostatic pressure on the slip surface.

(g) Riprap. Riprap is a positive means of providing slope protection and has been used in a few cases where erosive forces were too large to effectively control by other means. Objections to using riprap when flood fighting are: (1) rather costly; (2) large amount necessary to protect a given area; (3) availability; and (4) little control over its placement, particularly in the wet.

(h) Groins. In the past, small groins, extending 3.05 m (10 ft) or more into the channel were effective in deflecting current away from the levees. Groins can be constructed by using sandbags, snow fence, rock, compacted earth, or any other substantial materials that are available. Preferably groins should be placed in the dry and at locations where severe scour may be anticipated. Consideration of the hydraulic aspects of placing groins should be given, because haphazard placement may be detrimental. Hydraulic technical assistance should be sought if doubts arise in the use of groins. Construction of groins during high water will be very difficult and results will generally be minimal. If something other than compacted fill is used, some form of anchorage or bonding should be provided. (For example, snow fence anchored to a tree beyond the toe of the levee.)

(i) Log booms. Log booms have been used to protect levee slopes from debris or ice attack. Logs are cabled together and anchored with a dead man in the levee. The boom will float out in the current and, depending on log size, will deflect floating objects.

(j) Miscellaneous measures. Several other methods of slope protection have been used. Straw bales pegged into the slope may be successful against wave action, as is straw spread on the slope and overlain with snow fence.

(6) Sandbag dikes. The sandbag dike should not be considered as a primary-flood barrier. The main objections to their use are that the materials (bags and sand) are quite costly; they require a tremendous amount of manpower; and are time consuming to construct. They are also very difficult to raise if the flood forecasts are revised. Sandbag dikes should be used where a very low and relatively short barrier is required and earth fill would not be practicable, such as in the freeboard range along an arterial street. They are very useful in constricted areas such as around or very close to buildings, where rights-of-way would preclude using earth fill. They are also useful where temporary closure is required, such as roads and railroad tracks. A polyethylene seepage barrier should be incorporated into the sandbag structure. The poly must be on the riverward slope and brought up immediately behind the outermost layer of bags. The poly should be

keyed-in to a trench at the toe and anchored, or, at best, lapped under the sandbags for anchorage. See Figure F-1 for recommended practices in sandbag dike construction. A few points to be aware of in sandbag construction are: (1) sand, or predominantly sandy or gravelly material should be used; (2) extremely fine, clean sand, such as washed mortar sand, should be avoided; (3) bags should be 1/2 full; (4) bags should be lapped when placing; (5) bags should be tamped tightly in place; and (6) the base width should be wide enough to resist the head at high water. Sandbagging is also practical for raising a narrow levee, or when construction equipment cannot be used. Sandbag raises should be limited to 0.91 m (3 ft), if possible.

(7) Miscellaneous flood barriers. In addition to earth fill and sandbag levees, two other types of flood barriers should be mentioned. They are the flashboard and the box levees, both of which are constructed using lumber and earth fill (see Figure F-2). They may be used for capping a levee or as a barrier in highly constricted areas. Two disadvantages in using these barriers are the long construction time involved and very high cost. Therefore, these barriers are not recommended, unless a very unusual situation warrants their use.

F-3. Emergency Interior Drainage Treatment

a. General. High river stages often disrupt the normal drainage of sanitary and storm sewer systems, render sewage treatment plants inoperative, and cause backup in sewers and the discharge of untreated sewage directly into the river. When the river recedes, some of the sewage may be trapped in low lying pockets to remain as a possible source of contamination. Hastily constructed dikes intended to keep out river waters may also seal off normal outlet channels for local runoff, creating large ponds on the landward side of the dikes, making the levees vulnerable from both sides. If the ponding is excessive, it may nullify the protection afforded by the dikes even if they are not overtopped. Sewers may also back up because of this ponding.

b. Preliminary work. In order to arrive at a reasonable plan for interior drainage treatment, several items of information must be obtained by field personnel. These are:

(1) Size of drainage area.

(2) Pumping capacity and/or ponding required.

(3) Basic plan for treatment.

(4) Storm and sanitary sewer and water line maps, if available.

(5) Location of sewer outfalls (abandoned or in use).

(6) Inventory of available local pumping facilities.

(7) Probable location of pumping equipment.

(8) Whether additional ditching is necessary to drain surface runoff to ponding and/or pump locations.

(9) Location of septic tanks and drain fields (abandoned or in use).

c. Pumps, types, sizes, and capacities.

(1) Storm sewer pumps. Table F-1 indicates the size of pump needed to handle the full flow discharge from sewer pipes up to 610 mm (24 in.) in diameter. Table F-2 shows sizes and capacities of agricultural--type pumps which may be useful in ponding areas.

Table F-1
Matching Pipe Size to Pump Size

Sewer Pipe Size, mm (in.)	Probable Required Pump Size, mm (in.)
152.4 (6)	50.8 (2)
203.2 (8)	50.8 to 76.2 (2 to 3)
254.0 (10)	76.2 to 101.6 (3 to 4)
304.8 (12)	101.6 to 152.4 (4 to 6)
381.0 (15)	152.4 to 203.2 (6 to 8)
457.2 (18)	152.4 to 254 (6 to 10)
533.4 (21)	203.2 to 254 (8 to 10)
609.6 (24)	254 to 304.8 (10 to 12)

(2) Fire engine pumps. The ordinary fire pumper has a 101.6 mm (4-in.) suction connection and a pumping capacity of about 2838.75 ℓ/min (750 gpm). Use only if absolutely necessary.

(3) Pump discharge piping. The Crisafulli pumps are generally supplied with 15.24-m (50-ft) lengths of butyl rubber hose. Care must be taken to prevent damage to the hose. Irrigation pipe or small diameter culverts will also serve as discharge piping. Care should be taken to extend pump discharge lines riverward far enough to not cause erosion of the levee. On 304.8 mm (12-in.) or larger lines, substantial anchorage is required. These pumps must not be operated on slopes greater than 20 degrees from horizontal.

(4) Sanitary sewage pumping. During high water, increased infiltration into sanitary sewers may necessitate increased pumping at the sewage treatment plant or at manholes at various locations to keep the system functioning. To estimate the quantity of sewage, allow 0.378 m^3 (100 gal) per capita per day for sanitary sewage and an infiltration allowance of 35.28 m^3 per km-day (15,000 g/mile-day) of sewer per day. In some cases, it will be necessary to pump the entire amount of sewage, and in other cases only the added infiltration will have to be pumped to keep a system in operation.

Example: Estimate pumping capacity required at an emergency pumping station to be set up at the first manhole above the sewage treatment plant for a city of 5,000 population and approximately 48.24 km (30 miles) of sewer (estimated from map of city). In this case, it is assumed that the treatment plant will not operate at all.

Required capacity = (infiltration) + (sewage)

Sewage demand: $$\frac{5000 \text{ per} \times 0.378 \text{ m}^3\text{/person/day}}{24 \text{ hr} \times 60 \text{ min}} = 1.314\ m^3\text{/min}$$

$$\frac{5000 \text{ persons} \times 100 \text{ gal/person/day}}{24 \text{ hr} \times 60 \text{ min/hr}} = 347\ gpm$$

Table F-2
Crisafulli Pumps - Model CP 2 in. to 24 in.

Size mm (in.)	m³/min (gal/min)	Head[a] m (ft)	Elec. kW (hp)	Gas or Diesel kW (hp)
50.8 (2)	0.56 (150)		0.745 (1)	
101.6 (4)	1.88 (500)		5.59 (7.5)	11.18(15)
152.4 (6)	3.76 (1000)		7.45 (10)	14.9 (20)
203.2 (8)	11.27 (3000)	3.04 (10)	11.18 (15)	18.62(25)
304.8 (12)	18.79 (5000)		18.62 (25)	29.8 (40)
406.4 (16)	35.70 (9500)		29.8 (40)	48.4 (65)
609.6 (24)	93.95 (25000)		55.88 (75)	104.3(140)
50.8 (2)	0.49 (130)		0.745 (1)	
101.6 (4)	1.84 (490)		7.45 (10)	14.9 (20)
152.4 (6)	3.19 (850)		11.18 (15)	18.62(25)
203.2 (8)	9.21 (2450)	6.1 (20)	14.9 (20)	26.08(35)
304.8 (12)	14.09 (3750)		22.35 (30)	37.2 (50)
406.4 (16)	30.06 (8000)		33.52 (45)	63.3 (85)
609.6 (24)	71.4 (19000)		74.5 (100)	141.6(190)
50.8 (2)	0.45 (120)		0.745 (1)	
101.6 (4)	1.79 (475)		8.94 (12)	18.62(25)
152.4 (6)	2.99 (795)		14.9 (20)	26.08(35)
203.2 (8)	8.08 (2150)	9.84 (30)	18.62 (25)	33.52(45)
304.8 (12)	12.96 (3450)		26.08 (35)	52.15(70)
406.4 (16)	26.68 (7100)		44.70 (60)	93.12(125)
609.6 (24)	62.38 (16600)		93.12 (125)	186.24(250)

[a] Use high head pumps for heads over 6.1 m or 59.71 KPa (20 ft).

$$\text{Infiltration: } \frac{35.31\ \text{m}^3}{\text{km}} \times 48.24\ \text{km}\ \frac{1}{24\ \text{hr} \times 60\ \text{min}} = 1.18\ \text{m}^3/\text{min}$$

$$\frac{15000\ \text{gal/mile/day} \times 30\ \text{miles}}{24\ \text{hr} \times 60\ \text{min/hr}} = 312\ gpm$$

Required pumping capacity: 2.49 m³/min (659 gpm). From Table F-3, use one 101.6 mm (4-in.) pump or its equivalent.

Table F-3
Marlow Self Priming Centrifugal Pumps

Size mm (in.)	AGC Rating[a]	Capacity[b] m³/min (gal/min)	Horsepower kW (hp)
38.1 (1.5)	4M	0.25 (67)	1.34 (1.8)
50.8 (2)	7-10M	0.44-0.63 (117-167)	1.71-3.66(2.3-4.9)
76.2 (3)	20-30M	1.26-1.89 (334-500)	3.66-8.36 (4.9-11.2)
101.6 (4)	30-40M	1.89-2.51 (500-665)	14.92-28.94 (20-38.8)
152.4 (6)	90M	5.67 (1500)	32.46 (43.5)
203.2 (8)	125M	7.87 (2080)	46.25 (62)
254.0 (10)		12.6 (3330)	46.25 (62)

[a] Gallons per hour, thousands.
[b] At 75 kPa (7.67-m, 25-ft) head.

d. Metal culverts.

(1) Pumping of ponded water is usually preferable to draining the water through a culvert since the tailwater (drainage end of culvert) could increase in elevation to a point higher than the inlet, and water could back up into the area being protected. Installation of a flapgate at the outlet end may be desirable to minimize backup.

(2) Table F-4 shows the capacity of corrugated pipe culverts on a flat slope, with H factor (head) representing the difference between the headwater level and tailwater level, assuming the outlet is submerged. If the outlet is not submerged the head equals the difference in elevation between the headwater level and 0.6 of the diameter of the pipe measured from the bottom of the pipe upward. The capacity would change for smooth pipe, pipe laid on a slope, or if headwalls or wingwalls are used.

(3) If a culvert is desired to pass water from a creek through a levee, a computation of the drainage basin by an engineer is required to determine pipe size.

e. Preventing backflow in sewer lines.

(1) Watertight sluice gates or flap gates are one answer. Emergency stoppers may be constructed of lumber, sandbags, or other materials, using poly as a seal, preferably placed on the discharge end of the outfall pipe.

(2) Figures F-5 and F-6 contain manufacturer's literature on prefabricated rubber pipe stoppers which can be placed in the outlet opening of a manhole.

(3) Figures F-7 to F-11 illustrate methods of sealing off the outlet openings of a manhole with standard materials which are normally available so that the manhole may be used as an emergency pumping station.

F-4. Flood Fight Problems

a. General. Problem situations which arise during a flood fight are varied and innumerable. The problems covered below and in "Emergency Interior Drainage Treatment" are those which are considered most critical to the integrity of the flood barrier system. It would be impossible to enumerate all of the problems, such as supplies, personnel, communication, etc., which field personnel must handle. The most valuable asset of field personnel under emergency conditions is their common sense. Many problems can be solved instantly and with less effort through the application of good common sense and human relations. Problems, such as those below, can be identified early only if a well organized levee patrol system with a good communication system exists. The problems are presented with the assumption that high water is on the levee slopes.

b. Overtopping. Overtopping of a levee is the flowing of water over the levee crown. Since most emergency levees are of an urban nature, overtopping should be prevented at any cost. Overtopping will generally be caused by: (1) unusual hydrologic phenomena, including unexpected rainfall, faster than expected rainfall, faster than expected snowmelt, and ice and debris blockages, which cause a much higher stage than anticipated; (2) insufficient time in which to complete the flood barrier; or (3) unexpected settlement of the barrier. Generally, the flood barriers are constructed 0.61 m (2 ft) above the crest prediction. If the crest prediction is raised during construction, additional height must be added to the barrier. Capping should be done with earth fill or sandbags, using normal construction procedures. For levee construction, the 3.05 m (10 ft) top width allows the barrier to be raised relatively quickly with regular

Table F-4a
Capacity of Corrugated Metal Pipe Culverts
Without Headwalls and With Outlet Submerged (outlet control-full flow) (Circular) (metric Units)

Dia. In mm		CUBIC METERS PER SECOND Pipe Pressure in kPa 0.003	0.006	0.008	0.011	0.014	0.017	0.023	0.028	0.034	0.040	0.045	0.050	0.056	0.071	0.085	0.1	0.113
304.2	Length = 61 m	2.98	4.17	5.07	5.96	6.6	7.15	8.34	9.23	10.1	11	11.9	12.5	13.1	14.9	16.1	17.3	18.5
381		5.07	7.15	8.64	10.13	11.3	12.2	14.3	15.8	17.3	18.8	20.3	21.2	22.4	25	27.4	29.5	32.8
457.2		7.75	10.73	13.11	15.5	17	18.5	21.5	23.8	26.2	28.3	29.8	32.8	32.8	38.7	41.7	44.7	47.7
533.4		10.7	15.2	18.5	21.4	23.8	26.2	29.8	32.8	35.8	38.7	41.7	44.7	47.7	53.6	56.6	62.6	65.6
609.6		14.6	20.3	25	28.6	32.8	35.8	41.7	44.7	50.7	53.6	56.6	59.6	62.6	71.5	77.5	83.4	89.4
685.8		18.5	26.2	32.8	35.8	41.7	44.7	53.6	60	62.6	68.5	74.5	77.5	83.1	92.3	101	107	116
762.0		23.2	32.8	41.7	47.7	50.7	56.6	65.6	74.5	80.5	86.4	95.4	98.3	98	116	125	137	146
914.4		35.8	47.7	59.6	68.5	77.5	83.4	98.3	110	119.2	128.1	137	146	155	170	188	203	215
1066.8		47.7	68.5	83.4	95.4	107.2	116.2	134.1	152	164	178.8	191	203	212	235	256	277	298
1238.4		65.6	89.4	110	128	143	155	179	203	221	238.4	253	268	280	316	349	373	399
1371.6		83.4	116.2	143	164	181.8	200	232	260	289	304	325	346	361	405	444	477	510
1524		101	143	175	203	226.5	247	286	319	352	375	400	423	447	498	542	587	626

Dia. In mm		CUBIC METERS PER SECOND Pipe Pressure in kPa 0.003	0.006	0.008	0.011	0.014	0.017	0.023	0.028	0.034	0.040	0.045	0.050	0.056	0.071	0.085	0.100	0.113
304.2	Length = 122 m	2.38	3.27	4.17	4.8	5.4	6	6.9	7.5	8.3	8.9	9.5	10.1	10.7	11.9	13.1	14.3	15.2
381		4.17	5.66	7.15	8.05	9.24	10.1	11.6	12.8	14.3	15.5	16.4	17.6	18.5	20.6	22.6	24.4	26.2
457.2		6.26	8.94	11	12.8	14.3	15.5	17.9	20.3	22	23.8	25.6	26.8	28.6	32.8	35.7	38.7	41.7
533.4		8.94	12.8	15.8	18.2	20.3	22.1	25.6	28.6	32.8	35.8	35.8	38.7	42	44.7	50.7	536	56.6
609.6		12.5	17.6	21.5	25	28	29.8	35.8	38.7	44.7	47.7	51	53.6	57	62.6	68.5	74.5	80.5
685.8		16.4	23.2	28.6	32.8	35.8	41.7	47.7	51	56.6	62.6	66	68.6	74.5	83	89.4	98.3	104
762.0		20.9	29.2	35.8	41.7	47.7	50.7	60	66	71.5	77.5	83	89.4	92	104	113	125	131
914.4		29.8	44.7	53.6	62.6	71.5	77.5	89.4	98	107	116	125	134	140	158	173	185	197
1066.8		44.7	62.6	77.5	89.4	98.3	107	125	140	152	164	175	185	197	221	238	262	277
1238.4		59.6	83.4	104.3	119.2	134	146	167	188	206	220	238	250	265	295	25	352	378
1371.6		77.5	107.3	134.1	152	170	188	215	241	265	286	307	325	343	381	417	453	486
1524		95	134.1	164	191	215	232	268	298	328	358	381	405	426	477	522	566	602

Dia. In mm		CUBIC METERS PER SECOND Pipe Pressure in kPa 0.003	0.006	0.008	0.011	0.014	0.017	0.023	0.028	0.034	0.040	0.045	0.050	0.056	0.071	0.085	0.100	0.113
304.2	Length = 197 m	2.09	2.98	3.6	4.17	4.77	5.07	5.96	6.56	7.15	7.75	8.12	8.64	9.24	10.4	11.3	12.2	13.11
381		3.58	5.07	6.3	7.15	8.05	8.94	10.1	11.3	12.5	13.4	14.3	15.2	16.1	17.9	19.7	21.2	22.6
457.2		5.66	8.05	9.8	11.3	12.5	13.7	15.8	17.6	19.4	20.9	22.3	23.8	25	27.7	29.8	32.8	35.8
533.4		8.05	11.6	14.3	16.4	18.2	19.7	22.9	25.9	28.3	29.8	32.8	35.8	35.8	41.7	44.7	47.7	50.7
609.6		11.3	16.1	19.7	22.6	25	27.4	32.8	35.8	38.7	41.7	44.7	47.7	50.7	56.6	65.6	65.6	71.5
685.8		14.9	21.1	25.9	29.8	33	35.8	41.7	47.7	50.7	56.6	59.6	62.6	65.6	74.5	80.5	86.4	92.4
762.0		19.1	26.8	32.8	38.7	42	47.7	53.6	59.6	66	68.5	74.5	80.5	83.4	95.4	104	110	119
914.4		28.9	41.7	50.7	56.6	65.6	71.5	80.5	92.4	98.3	107	113	122	128	143	155	167	179
1066.8		41.7	56.6	72	83.4	92.3	101	116	131	143	152	164	173	182	203	221	238	256
1238.4		56.6	80.5	95	113	125	137	158	176	191	206	221	232	244	274	298	322	346
1371.6		71.5	101	125	143	161	175	203	229	247	268	286	304	322	358	390	423	453
1524		92.3	128	158	182	203	221	256	289	313	337	358	381	402	447	495	530	566

Table F-4b
Capacity of Corrugated Metal Pipe Culverts
Without Headwalls and With Outlet Submerged (outlet control-full flow) (Circular) (Metric Units)

Dia. In m		m^3 PER SECOND — Head on Pipe in m																
		0.003	0.006	0.008	0.011	0.014	0.017	0.023	0.028	0.034	0.040	0.045	0.051	0.057	0.071	0.085	0.099	0.113
0.305	Length = 6.10 m	0.3	0.43	0.53	0.61	0.67	0.73	0.85	0.94	1.04	1.13	1.22	1.28	1.34	1.52	1.64	1.77	1.88
0.381		0.52	0.73	0.88	1.04	1.16	1.25	1.46	1.61	1.77	1.92	2.07	2.16	2.29	2.56	2.8	3.02	3.35
0.457		0.79	1.1	1.34	1.58	1.74	1.89	2.19	2.44	2.68	2.9	3.05	3.35	3.35	3.96	4.27	4.57	4.88
0.533		1.10	1.55	1.89	2.19	2.44	2.68	3.05	3.35	3.66	3.96	4.27	4.57	4.88	5.49	5.79	6.4	6.71
0.610		1.49	2.07	2.56	2.93	3.35	3.66	4.27	4.57	5.18	5.49	5.79	6.1	6.4	7.32	7.92	8.53	9.14
0.686		1.89	2.68	3.35	3.66	4.27	4.57	5.49	6.1	6.4	7.01	7.62	7.92	8.53	9.45	10.36	10.79	11.89
0.762		2.38	3.35	4.27	4.88	5.18	5.79	6.7	7.62	8.23	8.84	9.75	10.06	10.7	11.89	12.8	14.02	14.94
0.914		3.66	4.88	6.1	7.01	7.92	8.53	10.06	11.28	12.19	13.11	14.02	14.94	15.85	17.37	19.2	20.72	21.95
1.067		4.88	7.01	8.53	9.77	10.79	11.89	13.72	15.54	16.76	18.29	19.5	20.73	21.64	24.07	26.21	28.35	30.48
1.219		6.71	9.14	11.28	13.11	14.63	15.84	18.29	20.73	22.56	24.38	25.9	27.42	28.65	32.31	35.66	38.1	40.84
1.372		8.53	11.89	14.63	16.76	18.59	20.42	23.72	26.52	29.52	31.09	33.22	35.36	36.88	41.45	45.41	48.77	52.12
1.524		10.36	14.63	17.98	20.73	23.16	25.30	24.26	32.61	35.97	38.4	40.89	3.28	45.72	50.9	55.47	60.04	64.0

Dia. In m		m^3 PER SECOND — Head on Pipe in m																
		0.003	0.006	0.008	0.011	0.014	0.017	0.023	0.028	0.034	0.040	0.045	0.051	0.057	0.071	0.085	0.099	0.113
0.305	Length = 12.19 m	0.26	0.34	0.43	0.49	0.55	0.61	0.70	0.76	0.85	0.91	0.97	1.04	1.1	1.22	1.34	1.46	1.55
0.381		0.43	0.58	0.73	0.82	0.94	1.04	1.19	1.31	1.46	1.58	1.68	1.8	1.89	2.10	2.32	2.5	2.68
0.457		0.64	0.91	1.13	1.31	1.46	1.58	1.83	2.07	2.26	2.44	2.62	2.74	2.93	3.35	3.66	3.96	4.27
0.533		0.91	1.31	.61	1.86	2.07	2.26	2.62	2.93	3.35	3.66	3.66	3.96	4.27	4.57	5.18	5.49	5.79
0.610		1.28	1.8	2.19	2.56	2.86	3.05	3.66	3.96	4.57	4.88	5.18	5.49	5.79	6.4	7.01	7.62	8.23
0.686		1.68	2.38	2.93	3.35	3.66	4.27	4.88	5.18	5.79	6.4	6.71	7.01	7.62	8.53	9.14	10.1	10.67
0.762		2.13	2.99	3.66	4.27	4.88	5.18	6.1	6.71	7.32	7.92	8.53	9.14	9.45	10.7	11.58	12.8	13.41
0.914		3.05	4.57	5.49	6.4	7.32	7.92	9.14	10.1	10.79	11.89	12.8	13.72	14.32	16.15	17.68	18.9	20.11
1.067		4.57	6.4	7.92	9.14	10.1	10.97	12.8	14.32	15.54	16.76	17.98	18.9	20.11	22.56	24.36	26.82	28.35
1.219		6.1	8.53	10.69	12.19	13.72	14.94	17.07	18.20	14.69	22.56	24.38	25.6	27.13	30.17	33.22	36.0	38.71
1.372		7.92	10.87	13.72	15.54	17.37	19.20	21.95	24.69	27.13	29.26	31.39	33.22	35.06	39.01	42.67	46.32	49.7
1.524		9.75	13.72	16.76	19.51	21.95	23.77	27.43	30.48	33.53	36.58	39.01	41.45	43.59	48.77	53.34	57.91	61.6

Dia. In m		m^3 PER SECOND — Head on Pipe in m																
		0.003	0.006	0.008	0.011	0.014	0.017	0.023	0.028	0.034	0.040	0.045	0.051	0.057	0.071	0.085	0.099	0.113
0.305	Length = 18.29 m	0.21	0.3	0.36	0.43	0.49	0.52	0.61	0.67	0.73	0.79	0.85	0.88	0.94	1.07	1.16	1.25	1.34
0.381		0.37	0.52	0.64	0.73	0.82	0.91	1.04	1.16	1.28	1.37	1.46	1.55	1.64	1.83	2.0	2.16	2.32
0.457		0.58	0.82	1.0	1.16	1.28	1.4	1.61	1.8	1.98	2.13	2.29	2.44	2.56	2.83	3.05	3.35	3.66
0.533		0.82	1.19	1.46	1.68	1.86	2.01	2.35	2.65	2.9	3.05	3.35	3.66	3.66	4.27	4.57	4.88	5.18
0.610		1.16	1.65	2.01	2.32	2.56	2.8	3.35	3.66	3.96	4.27	4.57	4.88	5.18	5.79	6.4	6.71	7.32
0.686		1.52	2.16	2.66	3.05	3.35	3.66	4.27	4.88	5.18	5.74	6.1	6.4	6.7	7.62	8.23	8.84	9.45
0.762		1.95	2.74	3.35	3.96	4.26	4.88	5.49	6.1	6.71	7.01	7.62	8.23	8.53	9.75	10.67	11.28	12.14
0.914		2.96	4.27	5.18	5.79	6.7	7.32	8.23	9.45	10.1	10.70	11.58	12.5	13.11	14.63	15.85	17.07	18.29
1.067		4.26	5.79	7.32	8.53	9.45	10.36	11.89	13.41	14.6	15.54	16.76	17.68	18.59	20.73	22.56	24.38	26.21
1.219		5.79	8.23	9.75	11.58	12.8	14.02	16.15	17.98	19.51	14.63	22.56	23.77	25.0	28.04	30.48	32.92	35.36
1.372		7.32	10.36	12.8	14.63	16.45	17.98	20.73	23.47	25.3	27.43	29.26	31.09	32.92	36.58	39.93	43.28	46.33
1.524		9.44	13.11	16.15	18.59	20.73	22.56	26.21	29.57	32.0	34.44	36.58	39.01	41.15	45.72	50.6	54.25	57.91

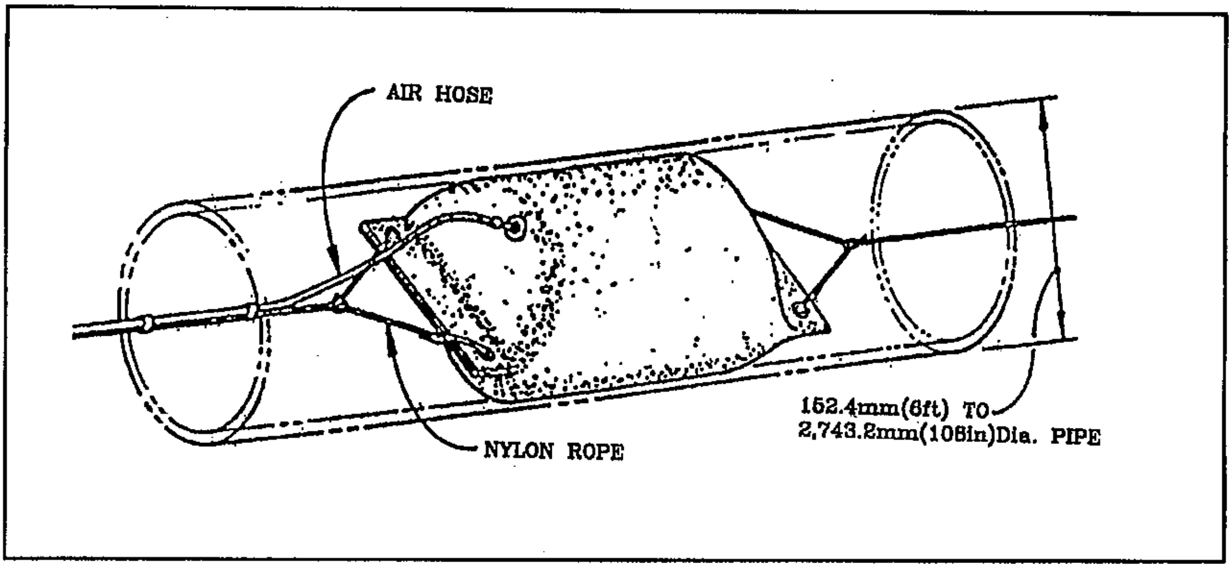


Figure F-5. Prefabricated rubber pipe stoppers for outlet opening of a manhole

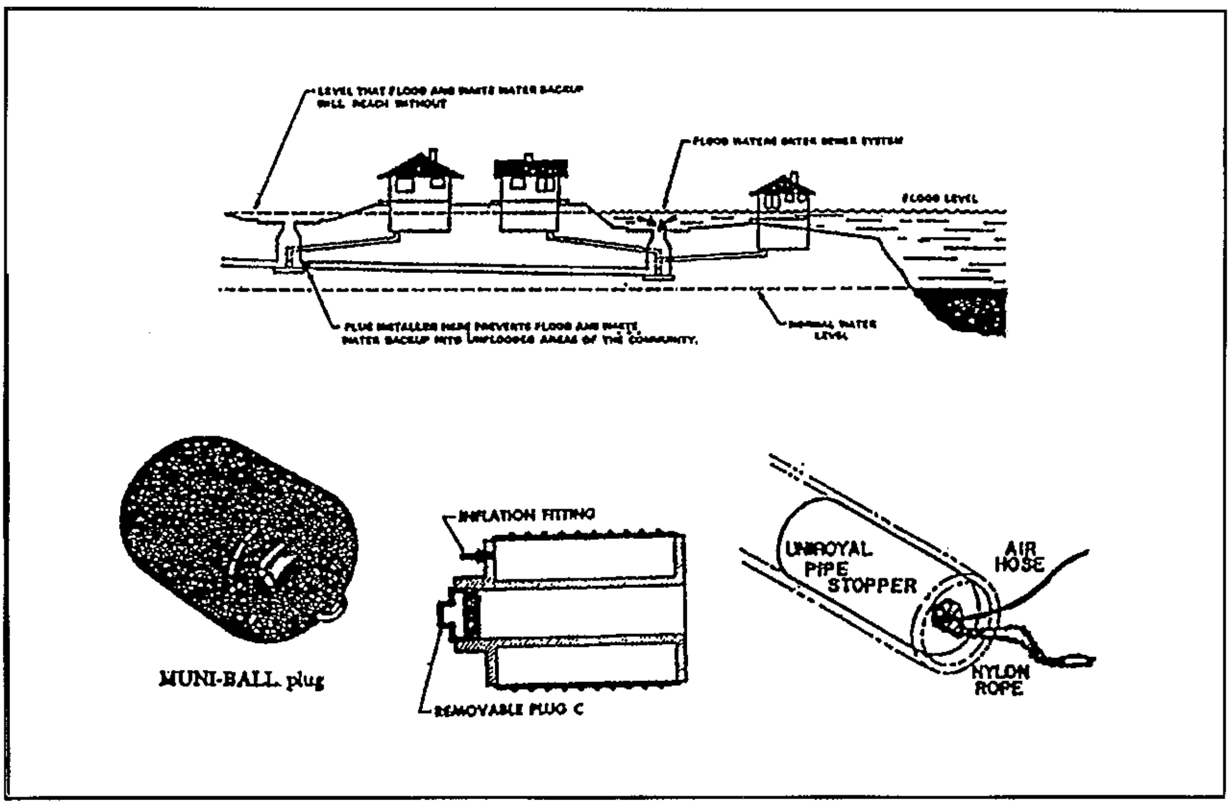


Figure F-6. Prefabricated rubber pipe stoppers for outlet opening of a manhole

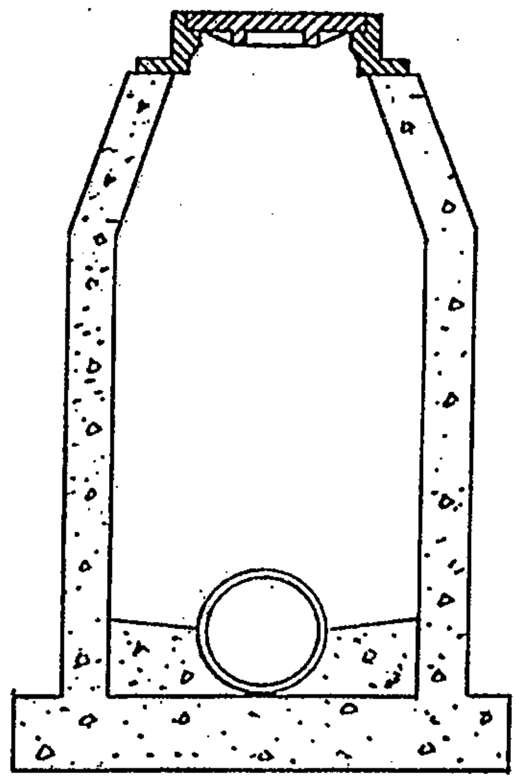


Figure F-7. Typical manhole

construction equipment. However, if the flood barrier consists of poly and sandbags having a minimum top width and limited base width, raising the barrier is very time consuming and labor intensive. Experience has shown that sandbag barriers over 0.91 m (3 ft) in height do not perform well for prolonged floods; underseepage becomes a real problem and failures have occurred as the water approaches the top of protection.

c. Seepage. Seepage is percolation of water through or under a levee, generally appearing first at the landside toe. Seepage through the levee is applicable only to a relatively pervious section. Seepage, as such, is generally not a problem unless (1) the landward levee slope becomes saturated over a large area; (2) seepage water is carrying material from the levee; or (3) pumping capacity is exceeded. Seepage which causes severe sand boils and piping is covered below. Seepage is difficult to eliminate, and attempts to do so may create a much more severe condition. Pumping of seepage should be held to a minimum, based on the maximum ponding elevation without damages. Seepage should be permitted if no apparent ill-effects are observed, and if adequate pumping capacity is available. If seepage causes sloughing of the landward slope, it should be flattened to 1V on 4H or flatter. Material for flattening should be at least as pervious as the embankment material.

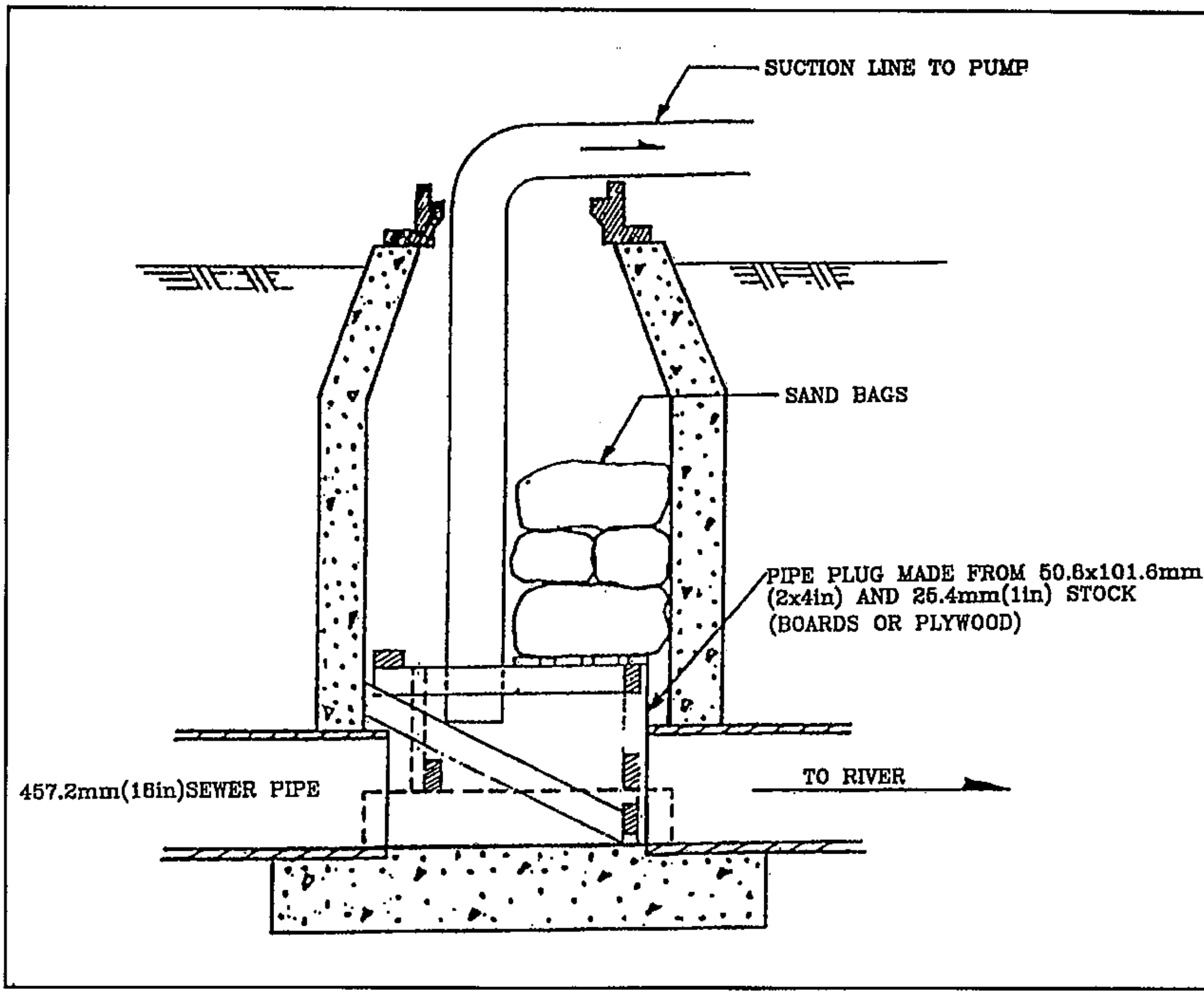


Figure F-8. Adapting manhole for use as emergency pumping station

d. Sand boils.

(1) Description. A sand boil is the rupture of the top foundation stratum landward of a levee caused by excess hydrostatic head in the substratum. Even when a levee is properly constructed and of such mass to resist the destructive action of floodwater, water may seep through a sand or gravel stratum under the levee and break through the ground surface on the landside in the form of bubbling springs. When such eruptions occur, a stream of water bursts through the ground surface, carrying with it a volume of sand or silt which is distributed around the hole. A sand boil may eventually discharge relatively clear water, or the discharge may contain quantities of sand and silt, depending upon the magnitude of pressure and the size of the boil. They usually occur within 3.05 m to 91.4 m (10 to 300 ft) from the landside toe of the levee, and in some instances have occurred up to 304.8 m (1,000 ft) away. .

(2) Destructive action. Sand boils can produce three distinctly different effects on a levee, depending upon the condition of flow under the levee.

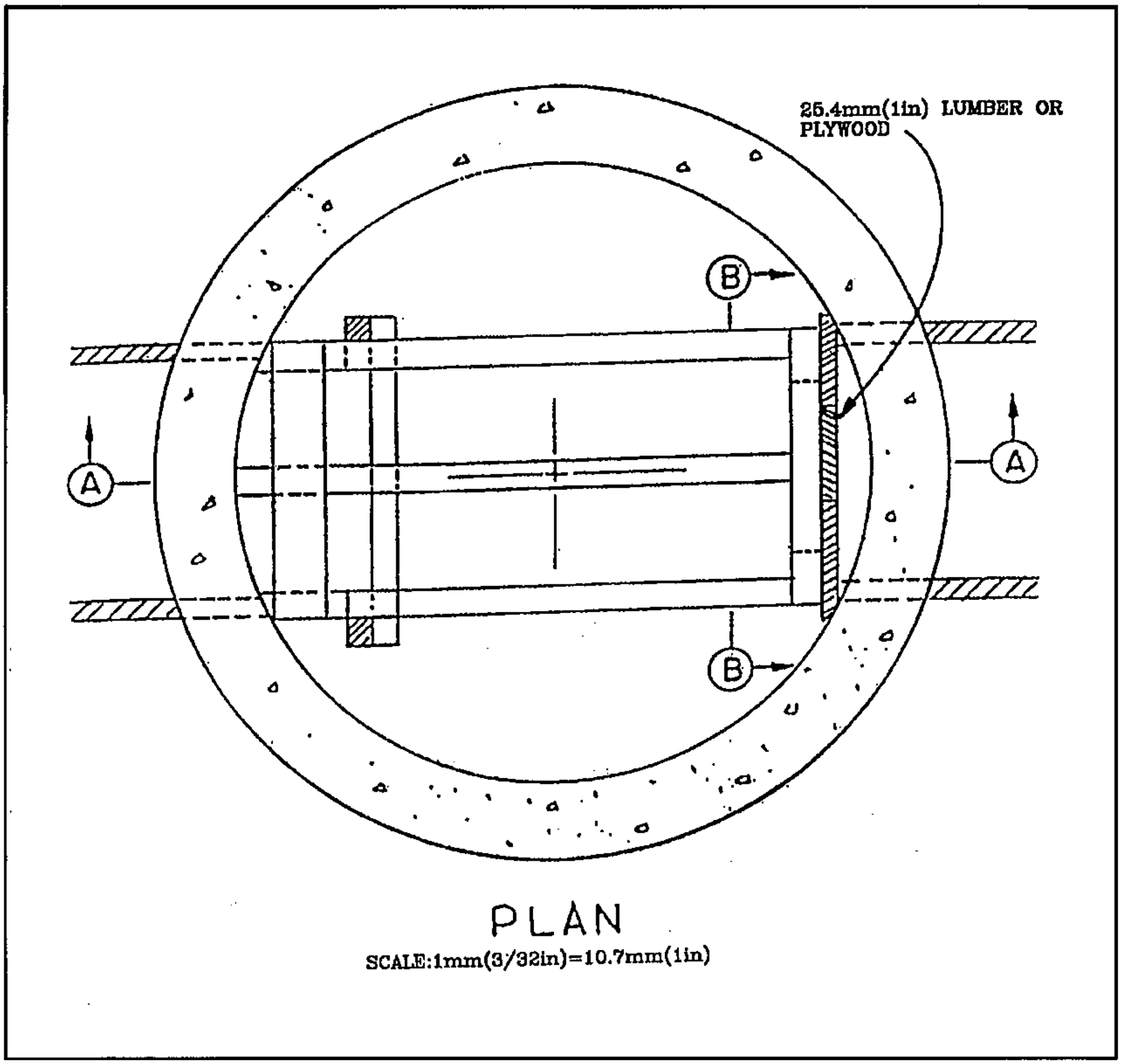


Figure F-9. Sealing top of manhole with wood

(a) Piping flow. Piping is the active erosion of subsurface material as a result of substratum pressure and concentration of seepage in the localized channels. The flow breaks out at the landside toe in the form of one or more large sand boils. Unless checked, this flow causes the development of a cavern under the levee, resulting in the subsidence of the levee and possible overtopping. This case can be easily recognized by the slumping of the levee crown.

(b) Non-piping flow. In this case, the water flows under pressure beneath the levee without following a defined path, as in the case above. This flow results in one or more boils outcropping at or near the landside toe. The flow from these boils tends to undercut and ravel the landside toe, resulting in sloughing

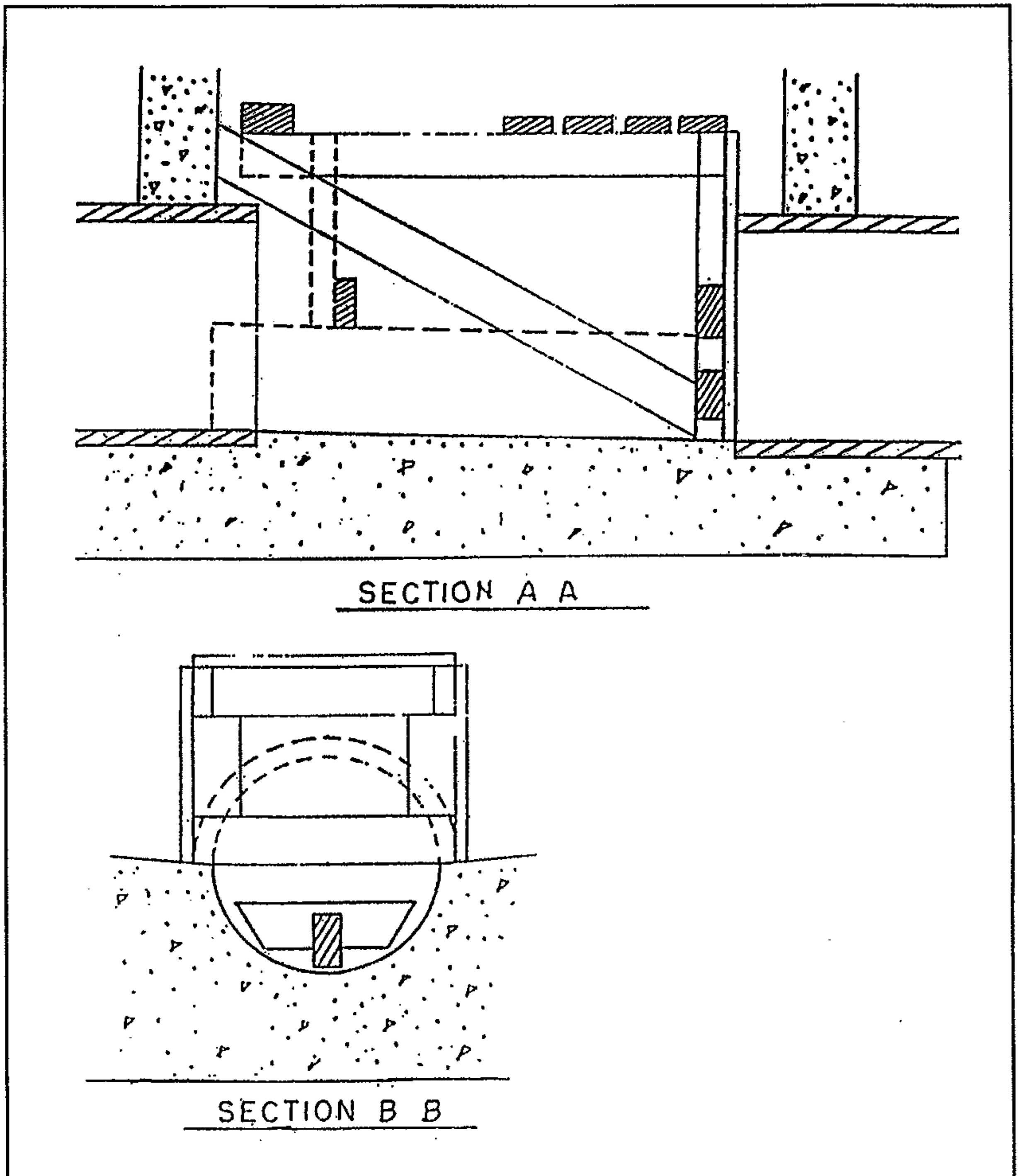


Figure F-10. Treatment of bottom of manhole

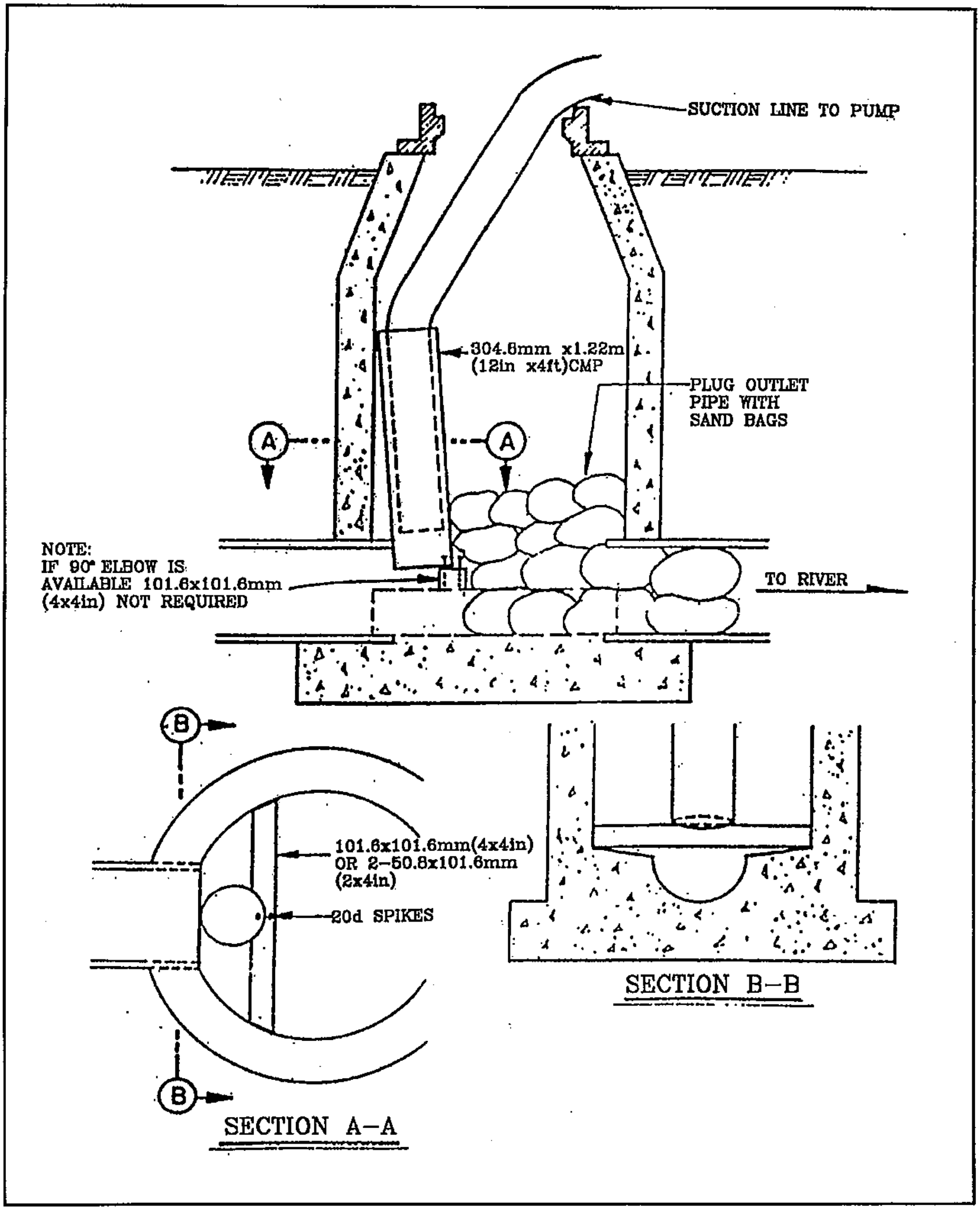


Figure F-11. Suction line to pump from manhole of the landward slope. Evidence of this type of failure is found in undercutting and ravelling at the landside toe.

(c) Saturating flow. In this case, numerous small boils, many of which are scarcely noticeable, outcrop at or near the landside toe. While no boil may appear to be dangerous in itself, the consequence of the group of boils may cause flotation ("quickness") of the soil, thereby reducing the shearing strength of the material at the toe, where maximum shearing stress occurs, to such an extent that failure of the slope through sliding may result.

(3) Combating sand boils. All sand boils should be watched closely, especially those within 30.5 m (100 ft) of the toe of the levee. All boils should be conspicuously marked with flagging so that patrols can locate them without difficulty and observe changes in their condition. A sand boil which discharges clear water in a steady flow is usually not dangerous to the safety of the levee. However, if the flow of water increases and the sand boil begins to discharge material, corrective action should be undertaken immediately. The accepted method of treating sand boils is to construct a ring of sandbags around the boil, building up a head of water within the ring sufficient to check the velocity of flow, thereby preventing further movement of sand and silt. See Figure F-12 for technique in ringing a boil. Actual conditions at each sand boil will determine the exact dimensions of the ring. The diameter and height of the ring depend on the size of the boil and the flow of water from it. In general, the following considerations should control: (1) the base width of the sandbag section should be no less than 1 1/2 times the contemplated height; (2) encompass weak soils near the boil within the ring of sandbags, thereby preventing a potential failure later; and (3) the ring should be of sufficient size to permit sacking operations to keep ahead of the flow of water. The height of the ring should only be that necessary to stop movement of soil, and not as high as to completely eliminate seepage. The practice of carrying the ring to the river elevation is not necessary and may be dangerous in high stages. If seepage flow is completely stopped, a new boil will likely develop beyond the ring; this boil could then suddenly erupt and cause considerable damage. Where many boils are found to exist in a given area, a ring levee of sandbags should be constructed around the entire area and, if necessary, water should be pumped into the area to provide sufficient weight to counterbalance the upward pressure.

e. Erosion. Erosion of the riverside slope is one of the most severe problems which will be encountered during a flood fight. Emergency operations to control erosion have been presented earlier under "Slope Protection."

f. Storm and sanitary sewers.

(1) Problems. Existing sewers in the protected area may cause problems because of seepage into the lines, leakage through blocked outlets to the river, manhole pumps not spread throughout the sewer system, and old or abandoned sewer locations which were not found during preflood preparations. Any of these conditions can cause high pressures in parts of the sewer system and lead to the collapse of lines at weak points and blowing off of manhole covers.

(2) Solutions. During the flood fight, continued surveillance of possible sewer problems is necessary. If the water level in a manhole approaches the top, additional pumps in manholes may alleviate the problem. In sanitary sewers, additional pumping may be required at various locations in the system to provide continued service to the homes in the protected area. When pumps are not available, manholes may have to be ringed with sandbags or by some other method which allows the water to head up above the top of the manhole. To eliminate the problem of disposing of this leakage from manholes the ring dike would have to be raised above the river water surface elevation. This creates high pressures on the sewer and should not be done. As with sand boils, it is best to ring the manhole part way to reduce the head and dispose of what leakage occurs. Directly weighing down manhole covers with sandbags or other-items is not recommended

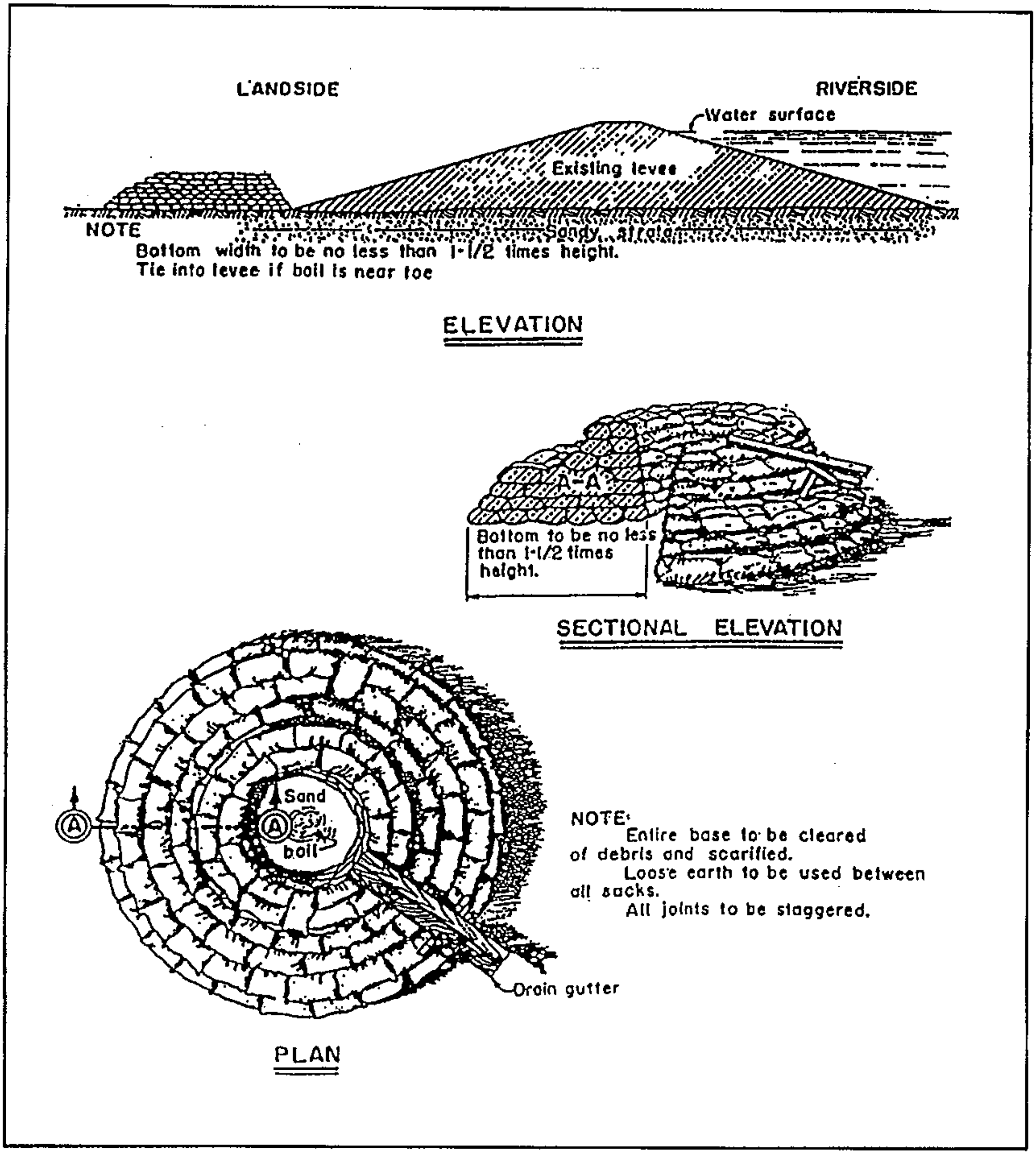


Figure F-12. Ringing sand boils

where high heads are possible. A 30-kPa (10-ft) head on a manhole cover 0.61 m (2 ft) in diameter would exert a force of 9.16 kN (2,060 lb-force). Thus, a counterweight of more than a ton would have to be placed directly on the cover.

g. Slope stability on weak foundations. In areas that have very weak foundation soils it may not be possible to construct full height flood barriers in preferred locations because of inadequate slope stability. However, if flood waters are slow to rise and fall, it is possible to use the rising floodwater as a restraining load on the riverside slope to meet stability criteria. This is usually used for closure structures or for staged construction where the flood barrier is only constructed after the river reaches an established level. This procedure would also require that the flood barrier be removed before the river went down below the established level.

h. Causes of levee failures. In addition to the problems covered above, the following conditions could contribute to failure:

(1) Joining of a levee to a solid wall, such as concrete or piling. Flood barriers consisting of sandbags greater than 0.91 m (3 ft) in height and joining a solid wall have performed poorly in the past due to excessive underseepage and instability of the sandbag prism.

(2) Structures projecting from the riverside of levee.

(3) A utility line crossing or a drain pipe through the fill.

(4) Tops of stoplogs on roads or railroad tracks at a lower elevation than the levee.

(5) Joining a sandbag barrier to a levee. Seepage problems at the juncture with the levee fill have caused very poor performance.

Appendix G
Use of Soil Cement for Levee Protection

G-1. Purpose

The purpose of this appendix is to provide guidance on the design and construction of soil cement slope protection for levees and embankments. This includes soil cement, materials, mixture proportioning, design of slope protection, construction, quality control, inspection, and testing.

G-2. General Considerations

a. Soil Cement. The American Concrete Institute defines soil cement as a mixture of soil and measured amounts of portland cement and water compacted to a high density. Soil cement can be further defined as a material produced by blending, compacting, and curing a mixture of soil/aggregate, portland cement, possibly admixtures including pozzolans, and water to form a hardened material with specific engineering properties.

b. Application. Although riprap has historically been used for slope protection for levees, dams, channels, etc.,there are situations when suitable rock is not available within economical haul distances and soil cement slope protection may be the most economical and appropriate selection.

c. History. The use of soil cement for slope protection has increased considerably over the past 30 years. The main focus of this effort has come from the U.S. Bureau of Reclamation (USBR) in the construction of dams. The first experimental use of soil-cement for slope protection was a test section constructed by USBR at Bonny reservoir in eastern Colorado in 1951. Observation of the performance of this test section for the first 10-year period of service indicated excellent performance of the soil cement which was subject to harsh wave action and repeated cycles of freezing and thawing. This lead to the conclusion that use of soil cement for slope protection was feasible based on both economical and service life considerations.

d. Economics. The decision to use soil cement instead of riprap is primarily an economic one. However, not every soil is suitable for producing soil cement for this application. Therefore, the designer must compare the availability of suitable soil for soil cement versus the availability of suitable rock for riprap. The designer must prepare a cost analysis in arriving at a decision. Factors that must be considered for soil cement include cost of cement, location of suitable soil, special processing requirements if needed, haul distance, dimensions and configuration of the slope protection and mixing and placement methods. For riprap, considerations include cost and availability of rock, size and availability of rock, haul distance, special processing requirements, configuration of placement and placement effort. Cost estimates of the alternative methods provide the basis for the economic analysis.

G-3. Materials

a. Soils. In general most soils of medium to low plasticity (Plasticity Index (PI) equal to or less than 12) can be used for soil cement. However for levee protection, better quality granular materials are recommended since the soil cement may be subjected to repeated cycles of wetting-drying, freezing-thawing and wave action. It is recommended that the soil should not contain any material retained on a 2-in. (50.8 mm) sieve, nor more than 45 percent retained on a No. 4 (4.75-mm) sieve, nor more than 35 percent or less than 5 percent passing the No. 200 (0.075-mm) sieve. The PI should be equal to or less than 12 and the organics content should be less than 2 percent. It should be noted that clay balls (nodules of clay and silt mixed with sand materials) can form when the PI is as low as 8. Clay balls can be detrimental when soil cement is exposed to weathering and the clay tends to wash out leaving voids in the soil cement structure. Clay balls greater than 25.4 mm (1-in.) should be removed and the minus 25.4-mm (1-in.) clay ball content should be limited to 10 percent. For economic reasons, the soil should be obtained from a borrow area close to the construction site. Samples from borrow sources must be evaluated for gradation and PI. If in-situ soils are not suitable it may be necessary to blend materials from several borrow sources.

b. Cement. Portland cements meeting specifications of ASTM C 150 are suitable. Generally, Type I is used for soil cement. However, soil cement can be subject to sulfate attack and it is the lime in the cement that is involved in the reaction. Therefore, sulfate bearing soils or water should be avoided. There is no definitive test to determine the threshold sulfate content at which a soil is deemed to be potentially reactive however experience has shown that soils with a sulfate content as low as low as 0.3 percent have developed reactions. If exposure to sulfates is not avoidable, Type II cement is recommended. Use of fly ash as a replacement for portland cement is not recommended in that experience has indicated that fly ash reduces early age compressive strength and durability when used in soil cement.

c. Water. Most water is acceptable for soil-cement. The primary requirement is that water should be free from substances deleterious to hardening of the soil cement. Specifically, water should be free from objectionable quantities of organic matter, alkali, salts, and other impurities. Presence of soluble sulfates should be of concern. Seawater has been used satisfactorily. The presence of chlorides in seawater may increase early strength. The quality of water for soil cement should be similar to that used for mixing concrete. Guidance on water quality may be found in Corps of Engineers CRD-C 400.

G-4. Proportioning Soil Cement Mixtures

a. General. One of the key factors that accounts for the successful use of soil cement is careful predetermination of engineering control factors in the laboratory and their application during construction. The composition of soils varies considerably and these variations affect the manner in which the soils react when combined with portland cement and water. The way a given soil reacts with cement is determined by simple laboratory tests conducted on mixtures of cement, soil, and water. These tests determine three fundamental requirements for soil cement: the minimum cement content needed to harden the soil adequately; the proper moisture content; and the density to which the soil cement must be compacted. Generally, the procedure to determine the mixture cement content consists of the following steps: soil classification test to determine an appropriate soil type; moisture density tests at a selected initial cement content to determine target density and water content values; durability tests at a range of cement content values including the initial cement content; unconfined compressive strength tests; and selection of final cement content based on test results.

b. Selection of soils. The design of a soil cement mixture begins with selection of a suitable soil type, The objective is to select a soil that can be stabilized with the minimum cement content and that will be suitably durable for the range of service conditions to which it will be subjected. Guidance on specifications for grading and plasticity of soils were given previously. Generally, soil cement made with granular materials requires less cement than soil cement made with sands and fine grained soils. The latter materials are also less durable. If the soils available in the immediate area of construction do not meet desired specifications it may be necessary to blend several soil types to obtain the desired characteristics. However, before blending is specified, the increased costs of processing and monitoring should be compared to the increased cost of additional cement required for the natural material. Occasionally the designer may encounter soils that are unreactive or are marginally reactive requiring apparently excessive amounts of cement. Often such soils contain acidic organic materials that affect the reaction.

c. Cement content general. A series of laboratory tests must be conducted to determine cement content. Inherent in these tests is also the determination of design soil density and water content. If the project is large and more than one candidate soil is available, it may be appropriate to conduct the entire series of tests on each soil to determine the most economical mixture for the project. Also, if several borrow areas having significantly different soils are involved it may be necessary to conduct laboratory tests on soil from each borrow area to determine the appropriate mixture for each soil. The tests involved in this process include: moisture density tests (ASTM D 558) to determine initial design density and moisture content based on a selected initial cement content and durability tests (ASTM D 559 and D560) to determine resistance to repeated cycles of wetting and drying and freezing and thawing which might be expected under natural climatic changes. Compressive strength tests (ASTM D 1632 and D 1633) should be conducted on laboratory prepared specimens. Tests are conducted at several cement content values and the final cement content is that which produces the required durability and strength at the lowest practical cement content. Strength and rate of strength gain are important factors in performance of the soil cement. Adequate strength is required to resist forces of wave action and uplift pressures.

d. Moisture density tests. Moisture density tests are conducted to determine values of density and water content for molding soil cement durability samples and for field control of compaction during construction. The cement content for moisture density tests is selected based on soil classification. Soils should be classified following procedures indicated in ASTM D 2487, Standard Test Method of Classification of Soils for Engineering Purposes. Initial cement contents for different soil classifications are indicated in Table G-1. The appropriate value of cement content for moisture-density tests may be selected from this table. Only coarse grained soil symbols are shown as these are the soil types preferred for soil cement for slope protection. Representative soil samples should be collected and moisture density tests conducted following procedures indicated in ASTM D 558, Standard Test Methods for Moisture Density Relations of Soil Cement Mixtures. Results of the tests are plotted as shown in Figure G-1 from which values of dry density and moisture content are selected for molding durability specimens. The dry density may be the maximum or a percentage of the maximum density indicated on the plot. Past experience has indicated that a minimum density of 98 percent of the maximum ASTM D 558 density is adequate. The water content is the value associated with the selected density. The water content at maximum dry density is termed the "Optimum Water Content" (OWC).

Table G-1
Initial Cement Content for Moisture Density Tests

Soil Classification (ASTM D 2487)	Initial Cement Content (percent dry weight of soil)
GW, GP SW, SP	7
GM, SM	8
GC, SC	9
SP	11

e. Durability tests. Two types of durability tests are conducted: ASTM D 559, Standard Test Methods for Wetting and Drying of Compacted Soil Cement Mixtures and ASTM D 560, Standard Test Methods of Freezing and Thawing of Compacted Soil Cement Mixtures. These tests were designed to reproduce in the laboratory the moisture and temperature changes expected under field conditions. These tests measure the effect of internal volume changes produced by changes in moisture and temperature. From these tests the minimum cement content required to produce a structural material that will resist volume changes produced by changes in moisture and temperature can be determined. Wet dry tests should be conducted in all geographic areas. Freeze-thaw tests should be conducted in all areas that experience at least one cycle of freezing and thawing per year since levee protection is expected to be subjected to this condition over a long

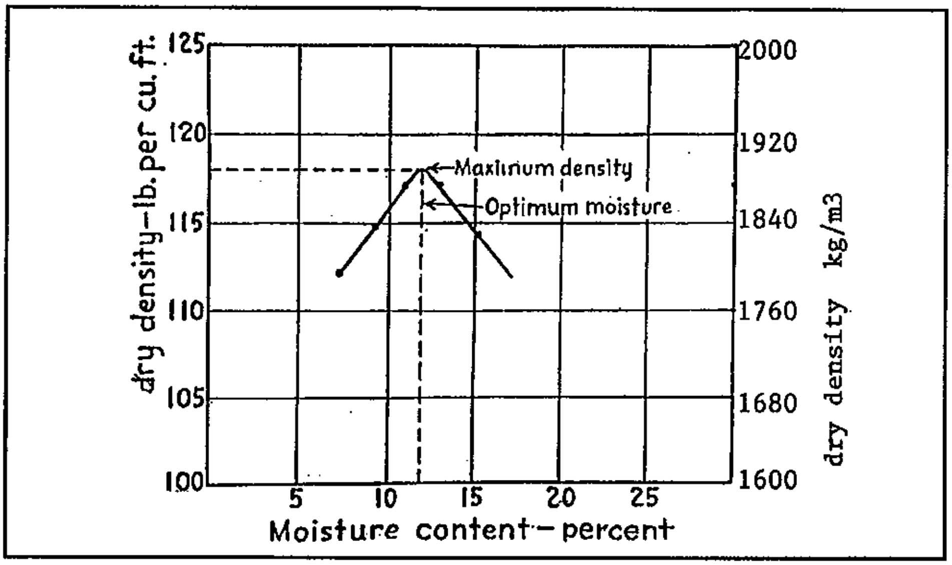


Figure G-1. Typical moisture-density curve

period of time. If there is absolutely no expectancy of freeze thaw cycles in the geographic area this test may be omitted. Each type of test consists of twelve two-day cycles of wetting/drying or freezing/thawing as appropriate and thus requires 24 days to complete.

For each type of test, duplicate specimens of soil cement should be prepared at cement contents equal to the cement content used for the moisture density test and at cement contents 2 percent above and 2 percent below that used for the moisture density test. For example, if the cement content for moisture density tests is 7 percent, samples for durability tests should be molded at 5, 7, and 9 percent cement. Ideally, a moisture-density test should be conducted for each cement content to determine maximum density and optimum moisture water content for that particular design mixture since these values vary with cement content. If this is not possible the density and moisture content determined from the initial tests may be used.

After each cycle (of either the wet-dry or freeze-thaw) the specimen is scrubbed with a wire brush to remove soil cement that becomes loosened or unbonded as a result of exposure to the test environment. After the twelve cycles are completed, the total weight loss is calculated and this value is compared to established criteria. The weight loss criteria are shown in Table G-2. Assuming both tests are conducted, specimens must meet both criteria. If specimens do not meet both criteria, adjustments must be made in the soil gradation and/or cement content based on engineering judgment and at least one set of tests should be rerun. Adjustments may include blending of aggregate to the soil and/or increasing the cement content.

Table G-2
Durability Test Weight Loss Criteria

Type of Durability Test	Maximum Weight Loss After 12 Cycles (percent)
Wet Dry (ASTM D 558)	6
Freeze Thaw (ASTM D 559)	8

f. Unconfined compressive strength tests. The next step is to conduct unconfined compressive strength tests (ASTM D 1632 Making and Curing Soil Cement Compression and Flexure Test Specimens in the Laboratory, and ASTM D 1633 Compressive Strength of Molded Soil Cement Cylinders). Strength of the soil cement is important in slope protection to provide resistance to wave action and uplift pressures. In fact, strength may be the determining factor in arriving at the final design cement content. Experience has shown that often the cement content of specimens meeting compressive strength criteria is higher than that necessary to meet durability requirements. The cement content for specimens for initial compressive strength tests will be the minimum cement content of the specimens that met durability criteria. The water content and dry density will be that used to mold durability specimens. Duplicate specimens should be prepared and tested as indicated according to the ASTM procedures previously indicated. Minimum compressive strength criteria are indicated in Table G-3. If strengths of specimens tested at the initial cement content do not meet minimum criteria, then the cement content should be increased in two percentage point increments and compressive strength tests rerun until criteria are met or it is determined that another mix design approach must be undertaken. If time constraints do not permit conduct of unconfined compressive strength tests until the durability tests have been completed, it may be necessary to conduct these tests simultaneously. If this is necessary, the unconfined compressive strength tests should be conducted on specimens prepared at all of the cement contents used in the durability tests. This approach obviously requires that many more specimens be prepared and tested however the savings in time may be more economical than conducting the tests in sequence.

Table G-3
Unconfined Compressive Strength Criteria (ASTM D 1633)

Cure Time (days)	Minimum Compressive Strength, kPa (psi)
7	4138 (600)
28	6034 (875)

g. Final cement content. The final cement content is the minimum cement content used in specimens that met or exceeded both the durability and compressive strength criteria. Some designers have added one or two percentage points to this cement content to account for variability in the field cement content where the proposed method of construction is mixed in place. Where central plant mix procedures are used control of cement content is generally accurate.

G-5. Design of Slope Protection

a. General considerations. Design of slope protection with soil cement is somewhat similar to design with riprap in that protection must be provided against erosional forces from wave action and stream currents. Soil cement slope protection can be provided in two configurations: stair step or plating. In stair step slope protection the soil cement is usually placed in successive horizontal layers adjacent to the slope. This method is preferred for slopes exposed to moderate to severe wave action or debris carrying, rapidly flowing water. The plating method consists of placing one or more layers of soil cement parallel to, i.e., directly on, the slope. This method is used where less severe exposure is expected.

b. Stair step method. The stair step method consists of constructing successive horizontal lifts of compacted soil cement up the slope to the desired height of protection (Figure G-2). Each successive lift is set back by an amount equal to the compacted lift thickness times the cotangent of the slope which results in a stair step pattern approximately parallel to the embankment slope. Layer thickness can be from 152.4 to 304.8 mm (6 to 12 in.) depending on the type of compaction equipment used. Historically, stair step construction has been accomplished with 152.4 mm (6 in.) compacted lifts. However, thicker lifts require less

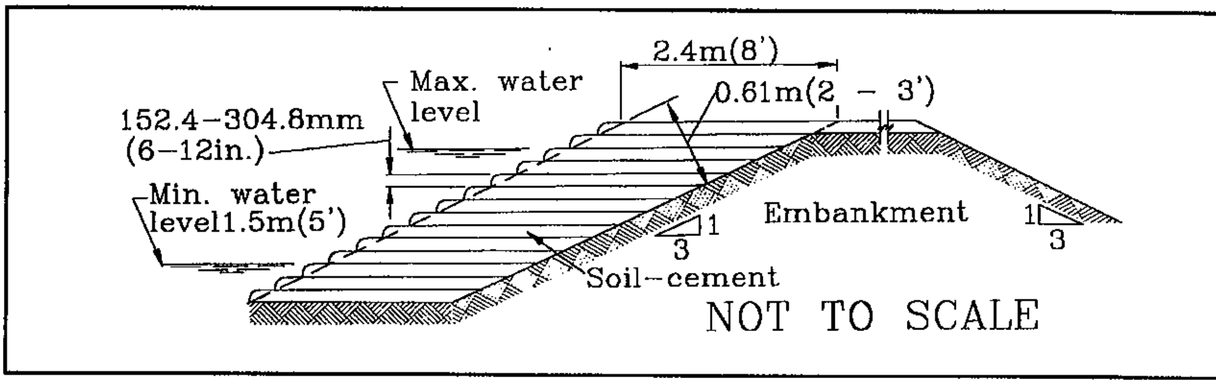


Figure G-2. Stair-step method of slope protection

construction effort and result in fewer bond surfaces. The disadvantage of thicker lifts is more loss of soil cement at the exposed edge during construction and additional effort is required to obtain desired density throughout the lift. The width of the layer also is a function of type and size of construction equipment. Experience has shown that a layer width of about 2.4 m (8 ft) is generally most convenient. Since stair step protection is indicated for more severe environmental conditions, a thicker covering over the slope is generally specified. Experience has indicated that the total thickness of soil cement measured perpendicular to the slope should be 0.61 to 0.92 m (2 to 3 ft). The relationships between slope, facing thickness, layer thickness and horizontal layer width are shown in Figure G-3.

c. Plating method. The plating method consists of lifts placed parallel to, i.e., directly on, the slope and is used in areas where a thinner facing is required. Generally two 152.4 mm (6-in.) lifts or one 203.2-mm (8-in.) lift are used for plating. One of the primary considerations in plating protection is providing resistance to high flow especially with debris. To date there are no definitive design criteria to determine lift thickness based on abrasion, however, since the plating method is applicable for areas subjected to less harsh environments, experience has shown 304.8 mm (12 in.) of protection is adequate. In the plating method, lifts can be constructed so that the resulting construction joints are either parallel or perpendicular to the flow of water. If placement and compaction of the soil cement are up and down the slope, the construction joint will be perpendicular to the water flow. If placement and compaction are along the slope, the construction joints will be parallel to the flow of water. For the plating method of construction, the slope should be 3H:IV or flatter in order to properly spread and compact the soil cement. Construction on steeper slopes may be accomplished if special compaction equipment is used.

d. Freeboard and wave runup. Freeboard is the vertical distance from the top of the levee to the water surface. The freeboard should be sufficient to prevent waves from overtopping the levee or damaging the crest. Slope protection should be provided in the freeboard area to prevent erosion. When a wave contacts the face of the levee it will run up the slope. Wave run up is the vertical height above the still-water level to which the uprush from a wave will rise on a structure. It is not the distance measured along the inclined surface. To calculate the wave run up for soil cement slope protection, the wave run up value based on riprap protection is first calculated and this value is multiplied by a factor based on the type and condition of the soil cement slope protection. For calculation of wave run up for riprap, designers should consult the following references: EM 1110-2-1614, Design of Coastal Revetments, Seawall, and Bulkheads, dated 30 June 1995; and the Automated Coastal Engineering System (ACES) computer program. For stair step construction with vertical faces on the layers the run up factor 1.2. Where the faces have become rounded due to weathering and erosion the run up factor is 1.3. For plating slope protection the run up factor is 1.4.

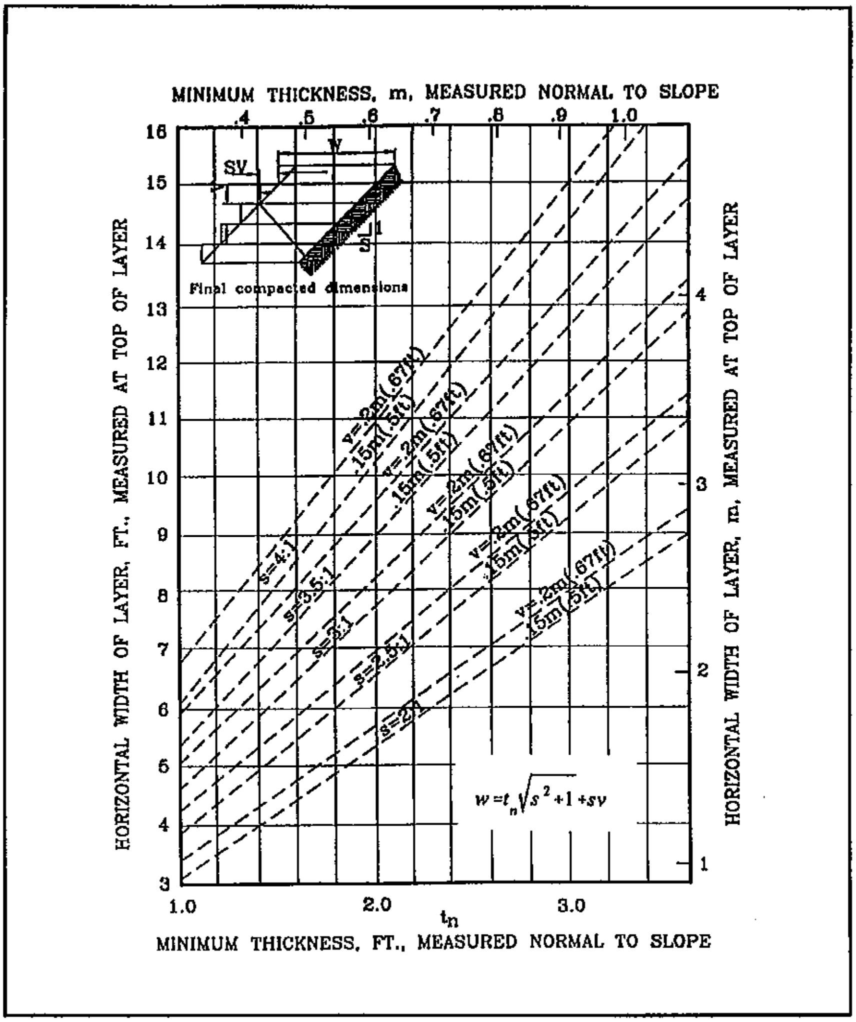


Figure G-3. Relationship of slope, facing thickness, layer thickness, and horizontal layer width

e. Transitions. Transitions between soil cement and earth or other structures should be addressed. Tie-backs similar to riprap emplacements can be designed to avoid flanking of the structure. An alternative is to use a riprap section at either end of the soil cement structure. Where soil cement joins other structures and compaction is difficult it may be appropriate to use lean concrete.

f. Drainage and seepage. Although no distress to soil cement slope protection due to rapid drawdown has been reported and the current thinking is that drainage is not required unless severe drawn down is anticipated, the designer should be aware of the preventative measures can be used. Three concepts are presented. One is design of the levee so that the least permeable zone is adjacent to the soil cement. This will provide protection against build up of excess pore water pressure. A second method is to determine that the weight of the facing is sufficient to resist uplift pressures. Here, there may be some pore pressure relief through shrinkage cracks in the soil cement. Obviously, some estimate must be made of the gross hydraulic conductivity of the soil cement. A third measure is to provide deliberate drainage conduits through the soil cement. This approach was used by the Bureau of Reclamation at Merrit Dam. Three rows of 76.2- to

127-mm- (3- to 5-in.-) diameter weep holes were drilled into the facing after construction and included 118 holes on 3.05 m (10 ft) centers. In such arrangements, a filter is placed in the area of weep hole before soil cement construction.

G-6. Construction

a. General. There are two general methods in common use for constructing soil-cement: mixed-in-place and central mix plant. Regardless of the equipment and methods used the goal is to obtain thoroughly mixed and adequately compacted and cured soil-cement. The central mix method involves mixing of a borrow material with cement and water, at a centrally located plant. The mixture is then transported to the site. The mixed-in-place method involves mixing of cement and water with the in-place soil at the site, and is infrequently used for embankment soil cement applications.

The most common method of soil-cement construction for bank protection is central mix plant. For soil--cement used as bank protection, particularly where banks experience higher flow velocity forces, adequate strength and durability, and consistent quality, are primary requirements. It is harder to achieve these objectives using mixed-in-place construction than central mix plant.

Two methods are used for placement and compaction of soil cement for embankments: stair step or plating. Design for these methods was discussed earlier in this document. The stair step method is the predominant method used, although construction using both methods is discussed in the subsequent sections on spreading and compaction.

Soil cement should not be mixed or placed when the soil or subgrade is frozen or when the air temperature is below 9°C (45°F). Specifications may allow soil cement construction to proceed if the air temperature is at least 4°C (40°F) and rising. Hot weather poses a few problems for soil cement construction, requiring sometimes additional moisture application to the materials, faster placement and compaction operations, and additional curing effort.

b. Central mix plant construction. There are two basic types of central mix plants: pugmill mixers either continuous or batch type, and rotary drum mixers (also a batch type of mixer). The uniformity of soil cement produced by these plant types is generally roughly equivalent, provided they have been properly calibrated. Continuous mix pugmill plants have higher production rates, while batch plants are often easier to calibrate, and require less frequent calibration. Batch-type pugmill plants have been used, but infrequently. Production rates between 76.4 and 152.9 m^3 (100 and 200 cu yd/hr) are common for stair-step soil cement construction. The basic steps of central mix plant construction of soil cement are: subgrade preparation, borrow materials, mixing, transporting, spreading, compacting, bonding lifts, finishing, construction joints, and curing and protection.

(1) *Subgrade preparation.* A firm subgrade is necessary to compact the overlying layers of soil cement to the required density. The subgrade is prepared by removing and replacing, or stabilizing, soft or wet areas, removing deleterious materials, and grading and compaction to construction plans and specifications. Most overly wet subgrade areas can be corrected by aerating and recompacting, or some type of chemical stabilization. Dry subgrades are surface moistened immediately prior to soil-cement placement.

(2) *Borrow materials.* Soil borrow sources are usually near the construction site and may consist partially or wholly of excavated bed and/or bank material. Native borrow materials are naturally variable in composition. Excavation, blending and stockpiling methods for borrow material should be selected to minimize this variation, and produce as consistent a material as possible. Horizontally stratified soil layers can be blended by deep excavation using full face cuts, insuring all layers are cut with each equipment pass.

If materials vary laterally across the borrow areas, loads from different locations should be blended in a systematic fashion. Further blending can also be done as materials are brought to the plant stockpile area. Alternating the loads from different parts of the plant stockpiles, or even using a front-end loader to take a vertical cut of the stockpiles, also helps blend materials as they are fed to the mixing plant.

Screening the borrow material through a 25-mm (1-in.) to 38.1-mm (1-1/2-in.) mesh at the pit or at the plant can help remove oversize clay balls and other oversize materials. Selective excavation may be necessary to avoid excessive clay balls or clay content in the borrow area.

Stockpiles should be separated from each other and all plant equipment by at least 15.2 m (50 ft). Where the soil contains coarse aggregate, stockpiling is done in layers to minimize segregation.

(3) *Mixing.* Central mixing plants with rated capacities of 227 to 907 metric tons (250 to 1,000 tons) per hour (about 95.56 to 382.3 m^3 (125 to 500 cu yd)) are used commonly. Special blending requirements may require several stockpiles and separate storage feeder bins. Prior to mixing and placing, it is necessary to measure the quantities and proportions of material supplied by the plant. The plant should be accurately calibrated.

(a) *Pugmill mixers.* The most common continuous mixing plants contain a twin shaft pugmill. Figure G-4 shows a diagram of a typical pugmill central mix plant. USBR recommends a twin-shaft pugmill with a rated capacity of at least 152.9 m^3 (200 cu yd)/hr. A pugmill mixing chamber contains twin shafts rotating in opposite directions, with paddles (see Figure G-5) that force mix the soil cement and move it through the chamber by the pitch of the paddles. Material feeds (by adjusting gate openings and belt speed) and pugmill features (such as pugmill tilt and paddle pitch) may be adjusted to optimize the mixing actions and production. Thoroughness of blending is partly determined by the length of mixing time. A mixing time of 30 sec is commonly specified, although shorter times have also been shown to be adequate, depending on the mixer efficiency.

Batch type pugmill mixers, where the materials are delivered to a pugmill mixer in a discrete batch rather than as a continuous ribbon of material, can provide effective mixing of soil cement, but are seldom used, largely due to lower production capacity and lack of availability.

(b) Rotary drum mixers. Although rotary drum (also called tilt drum) mixers are sometimes used, they are generally lower in production capacity than pugmill mixers. These plants are typically converted central mix concrete plants, and function in the same manner. Mixing times for these plants are typically about 60 sec.

(4) *Transporting.* Haul trucks can be of the end or bottom dump variety, although many types are used. Where conditions are extremely hot and/or windy or where sudden showers are a possibility, soil cement should be protected by using canvas covers on haul vehicles. Equipment should be clean. The elapsed time between mixing and compacting should be kept to a minimum. Sixty minutes is usually the maximum. Therefore, most specifications require haul times to be kept below a maximum of thirty minutes.

In stair step construction, temporary ramps are constructed at intervals along the bank to enable trucks to reach the layer to be placed. These temporary ramps should have a minimum 0.457 m (18-in.) thickness of material to protect the edge of the previous lift from truck traffic. There is also a requirement, where streambeds are dry, for ramps to be spaced to allow egress from the channel in case of a flood. These are constructed at 45° angles, with a minimum of 0.61 m (2 ft) of cover over the soil cement, and spaced about 91.4 to 121.9 m (300 to 400 ft) apart.

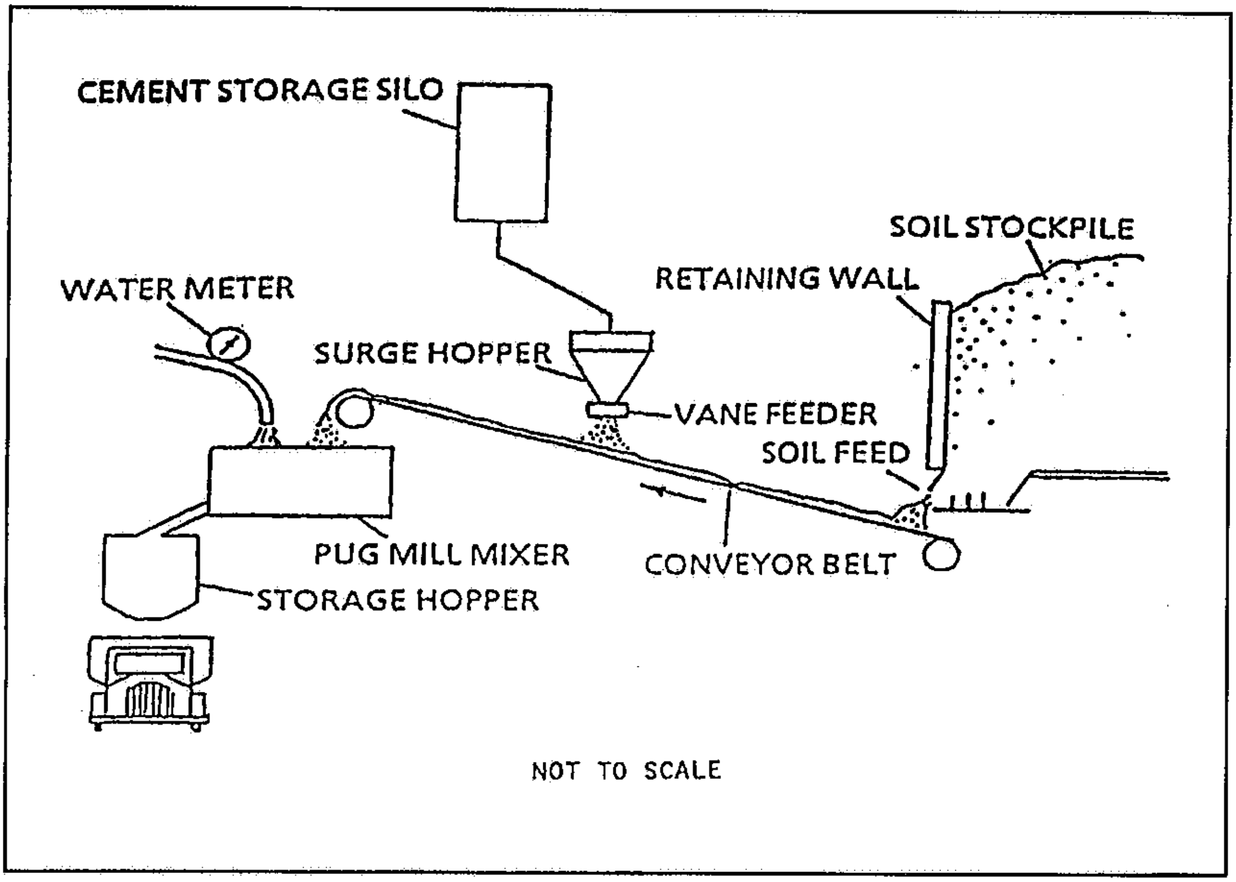


Figure G-4. Typical pug mill central plant

Figure G-5. Mixing paddles of a twin-shaft, continuous-flow central mixing plant

Figure G-6 shows a typical step-construction sequence. Frequently time and cost savings have been realized by using conveyor systems to deliver the soil cement to the spreader. This removes the necessity for ramp construction and truck maneuvering and provides a cleaner end product. Narrower layers and plating applications can also be placed using a conveyor system. The soil cement can be delivered from above or below directly to a spreader box.

(5) *Spreading*. Soil cement must be spread in a manner that will provide a compacted layer of uniform thickness and density, conforming to the design grade and cross section.

(a) *Stair step method.* There are a wide variety of spreading devices and methods for stair step construction. One of the most common is the spreader box attached to a dozer or grader. An

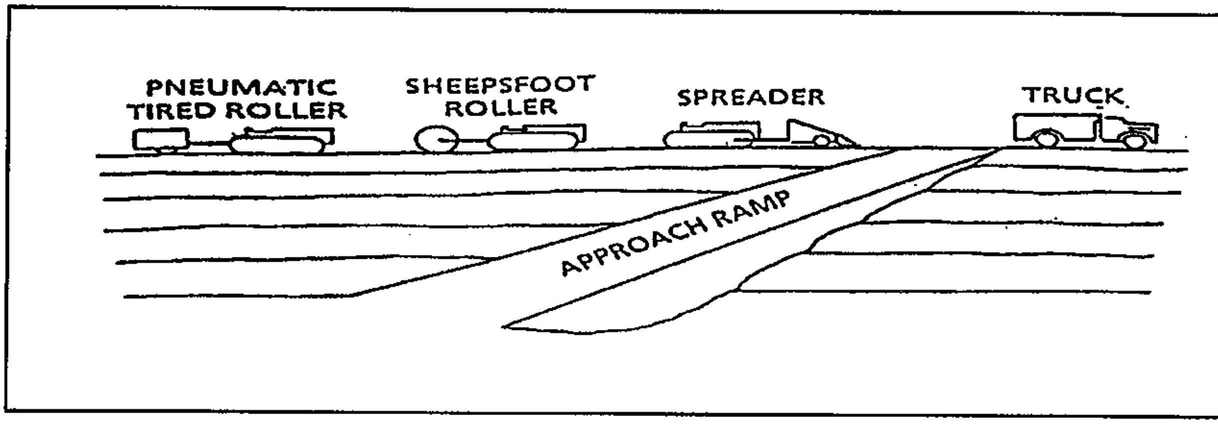


Figure G-6. Typical construction sequence

alternate method is to place material in windrows to be spread by a grader. Care must be taken with the windrow operation not to over manipulate the material which may cause separation and premature drying. Layers are spread 15 to 30 percent greater than the required compacted thickness. Experimentation may be necessary to determine the appropriate spread thickness since different combinations of equipment and soil type may produce different amounts of precompaction. Spreading may also be done with asphalt-type or RCC pavers. Some of these pavers are equipped with one or more tamping bars which provide some initial compaction.

Placement of stair-step sections may need to be limited to a maximum of 1.22 m (4 ft) height in a single shift to avoid instability producing bulging in the outer face from the surcharge weight of material and equipment above.

(b) *Plating method.* A variety of methods may be used for spreading of soil cement for plating applications. On relatively level surfaces, the methods are the same as for stair step placement. Plating construction on steeper slopes requires different procedures than stair step construction. Dozers are commonly used to spread soil cement on steeper slopes. USBR has reported best results in terms of producing uniform thickness and minimum waste when soil cement was spread from the top to the bottom, rather than from bottom to top. Whatever method is used, careful attention needs to be paid to achieving uniform thickness.

(6) *Compaction.* Minimum compaction to be achieved in the field is normally specified as a percentage of maximum density determined by ASTM D 558 or ASTM D 1557, typically requiring 98 percent of maximum density. Moisture content of the soil cement mixture must be controlled within tight limits to ensure consistent optimum conditions for compaction. USBR practice has been to place soil cement at water contents at or slightly dry of optimum. This can help avoid excessively wet mixes that may cause traffic and compaction difficulties, as well as lift distortion and increased cracking due to shrinkage. Compaction should begin as soon as possible and be completed within about one hour after initial mixing. No section of soil cement should be left unworked for longer than 30 min. Climatic conditions at some sites, such as very cool, humid weather, may allow relaxation of this guidance. Moisture loss by evaporation during hot weather compaction should be replaced by light applications of water. Compaction is done by various types of rollers. For fine grained soils, a sheepsfoot roller is generally used for initial compaction, followed by a pneumatic-tire roller for final compaction. USBR practice has often been to compact the lower portion of the lift with a towed sheepsfoot roller, using the vibratory steel-wheeled roller for the upper portion of the lift. Some problems have been encountered with vibratory roller compactors when used for finer grained materials. Vibratory rollers may create fine transverse cracks in the soil cement surface, requiring a

rubber-tired roller for final compaction to close most of the cracks. Compacting soil cement at or above optimum moisture can produce rutting from pneumatic tire rolling. For coarse grained soils, vibratory steel-wheeled or heavy pneumatic rollers are generally used. Compacted layer thickness is typically from 152.4 to 228.6 mm (6 in. to 9 in.), although greater thicknesses of coarse grained soils can be compacted with heavy equipment designed for thicker lifts. The specified minimum density must be achieved throughout the lift thickness, regardless of the lift thickness and compaction equipment used. Compactor weight, and vibration amplitude and frequency must be adjusted during construction to obtain the best compaction. Test sections are a valuable aid in determining the optimum compaction equipment characteristics and procedures.

(a) *Stair step method.* Compaction of the outer edge of the layer is usually not necessary from the standpoint of structural integrity. However, uniform edges provide a better appearance and allow for easier emergency egress from streambeds. Sharp edges reduce wave runup but increase roughness. Edge compaction can be accomplished by hand tampers or through the use of some type of edge support during compaction.

(b) *Plating method.* Compaction is done with various roller types. Construction on near horizontal surfaces is similar to layered construction. Compaction on steeper side-slopes requires different procedures. A rolling deadman (Figure G-7) has been used to winch the roller up and down slope. Adequate compaction has been achieved using bulldozers, although their use is not recommended. Multiple overlapping passes are usually required. Surface tearing can be minimized by using cut grousers or street pads. Compaction from bottom to top has been most successful.

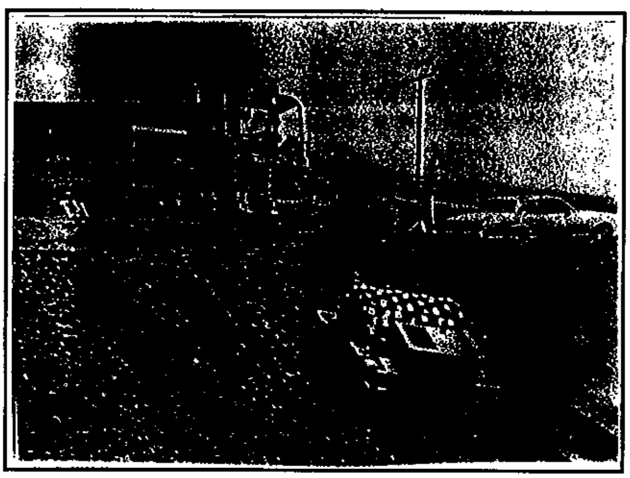


Figure G-7. "Deadman" pulling vibratory sheepsfoot roller up the slope

(7) *Bonding lifts.* The bond between soil cement layers is generally weak. No definite criteria is available on the most effective methods of bonding between layers; however, bonding may be considered if layer separation is anticipated. Layer separation may be a concern from strong wave action, or at the upper lift of some sections, where there is little weight above the lift to mobilize shear resistance. The most significant factor in bond strength is time delay between lifts. The shorter the time between lifts the better the bond. Long placements may be broken up into shorter segments, enabling subsequent lifts to be placed more rapidly. Moist curing increases the bond strength but excess water tends to decrease it. Most specifications require temporarily exposed surfaces to be kept moist and clean. Care must be taken to avoid tracking clay or other materials onto the layer which would reduce bond.

Power brooms should be used for lift surface cleaning to remove loose and unbonded material. USBR studies have suggested that roughening the lift surface with steel power brooming does not significantly contribute to increased bond strength. Brooming is not permitted prior to 1 hr after compaction to allow adequate set of the soil cement.

Both dry cement and cement slurry lift bonding have been used and evaluated in USBR test sections, with encouraging results. A slurry mix should have a water/cement ratio of about 0.70 to 0.80 and an application the latter rate. Dry cement applications have a disadvantage of being susceptible to wind, while cement slurry is susceptible to rapid drying. Whichever method may be used, the material should be applied immediately before placement of the next lift.

(8) *Finishing.* As compaction nears completion the entire layer should be shaped to specified lines, grades, and cross sections. Edge shaping can be done with a modified blade or a curved attachment on the roller. The lift may require scarification to take out imprints left by equipment or to remove thin surface compaction planes. Scarification can be done with a variety of spring tooth or spike toothed harrows, or similar equipment. Soils containing gravel may not require scarification. Final surface compaction following scarification is performed with a steel-wheeled roller in nonvibratory mode, or a rubber-tired roller. A smooth "table top" finish is not required and may be detrimental to lift joint shear strength. Wheel marks are acceptable, although they may make lift joint cleanup more difficult

The edges on stair-stepped soil cement applications have been finished by cutting back the uncompacted edges, by using special rounded attachments on compaction equipment, and by leaving sacrificial uncompacted edge material in place to be eroded later.

(9) *Construction joints.* Construction joints are required at the completion of each day's work or when work must be stopped for time periods longer than allowed for placement and compaction of fresh soil cement. They are made by cutting back into the finished work to proper crown and grade. The joint must be vertical, full depth, and transverse to the layer direction and is usually done with the toe of a grader blade or bulldozer blade. Care must be taken that no debris is present on the joint edge, and that new material placed against the joint adheres to the previous work. Joints should be staggered to inhibit cracking throughout the structure

(10) *Curing and protection.* Proper curing is essential, because strength gain and durability is dependent upon time, temperature and the presence of moisture. All permanently exposed surfaces should be moist cured for a period of seven days. Traffic should be kept off the soil cement during the curing period. Light traffic is sometimes allowed on the completed soil cement, provided the curing is not disrupted.

Soil cement must be protected from freezing during the curing period. Insulation blankets, straw, or a soil cover are commonly used. Light rainfall should not interrupt construction. However, a heavy rain prior to compaction can be detrimental. For mixed-in-place operations, if rain falls during the cement spreading operation, the cement already spread must be quickly mixed with the soil, and compaction must proceed immediately. After soil cement has been compacted, rain will seldom have detrimental effects.

(a) *Moist curing.* Water curing may be done with fog spraying, or with weighted and secured plastic sheeting if wind is not a problem. Wet burlap can also be used if a moist condition can be maintained. A minimum of 152.4 mm (6 in.) of moist earth can be specified as an alternative. The earth cover may also inhibit freezing should colder temperatures be expected.

(b) *Bituminous membrane curing.* Membrane curing using some types of bituminous material (generally an emulsified asphalt) can be used as an option to water curing where no succeeding layers will come in contact with the membrane. However, the black color may be objectionable to owners. Bituminous membrane curing should not be used for levees, ponds or reservoirs which will have water frequently in contact with the membrane, without evaluation of environmental effects of the bituminous membrane. An application rate of 0.68 to 1.4 ℓ/m^2 (0.15 to 0.30 gal/sq yd) is required. The soil cement should be moistened just prior to the membrane application. Sand can be spread over the bituminous membrane curing if light traffic is necessary, to prevent tracking of the bituminous material.

c. *Mixed-in-place construction.* In-place mixing is generally not used nor recommended for multi-layer construction. Plating type embankment applications are possible with the mixed-in-place method of soil cement, although again are not recommended. The basic steps in mixed-in-place construction are: soil preparation, cement addition, pulverization and mixing, compaction, finishing, curing, and protection. Following mixing, the construction techniques are essentially identical to central plant soil cement and are not further discussed under the mixed-in-place method. Although windrow type mobile pugmill mixers are used for pavement mixed-in-place construction, they are seldom used for embankment applications. Mix-in-place operations are generally performed using transverse single or multiple-shaft rotary mixers (see Figure G-8). In-place strength of the soil cement using mixed-in-place construction may be only 60 to 80 percent of the laboratory values, due partly to less efficient mixing compared to central mixing. Adding one to two percent cement is common practice to compensate for the higher variation in strength using mixed-in-place construction.

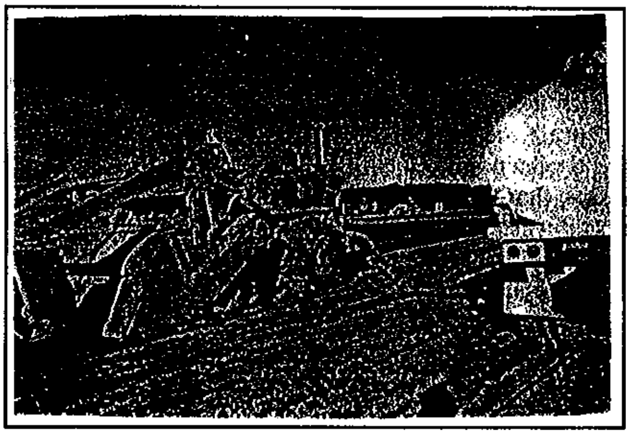


Figure G-8. Transverse single-shaft rotary mixer

(1) *Soil preparation and pulverization.* The soil is prepared by removing and replacing, or stabilizing, soft or wet areas, removing deleterious materials such as stumps, large roots, organic soils, and aggregate greater than 76.2 mm (3 in.) in size, and grading to the approximate final design profile. Most overly wet areas can be corrected by aerating and recompacting, or some type of chemical stabilization. Proper moisture content is essential for unimpeded construction traffic and for satisfactory pulverization and mixing. Dry soils may be disced and wetted by spray trucks until moisture content is near optimum for the soil cement. A moisture content near optimum may be necessary for pulverizing fine grained soils. Pulverization of soil prior to cementitious materials spreading is generally necessary to insure uniform cement mixing. Pulverization of soils with higher fines content or higher plasticity may be difficult without proper moisture control and proper equipment.

(2) *Cementitious materials application.* Cementitious materials are distributed on the soil surface using a bulk mechanical spreader (see Figure G-9), or for smaller projects, by hand placing cement bags. Mechanical spreaders must be operated at uniform speed with a relatively constant level of cement in the hopper to produce a uniform spread of cement. Mechanical spreaders also require sufficient traction for proper distribution, sometimes requiring wetting and rolling the soil prior to spreading. Some spreaders are directly attached behind a bulk cement truck, where cement is pneumatically moved into the spreader hopper for distribution. PCA (1995) has convenient tables to convert the required cement content as a percentage by weight of oven-dry soil into a cement spread quantity in terms of weight of cement per square foot of soil surface. Cement spreading can be performed only when wind is absent and may require environmental permits. Although cement slurry spray applicators, including admixture capability, are available, they have not been widely used as yet.

(3) *Pulverization and mixing.* Most soils must be pulverized prior to mixing operations, using the rotary mixers. For mixing, single-shaft mixers require at least two passes; one to mix the soil and cement, and the second to add water. Multiple-shaft mixers handle these functions in one pass. Agricultural equipment does not generally give adequate results. In-place mixing efficiency is generally poorer than central mixed soil cement.

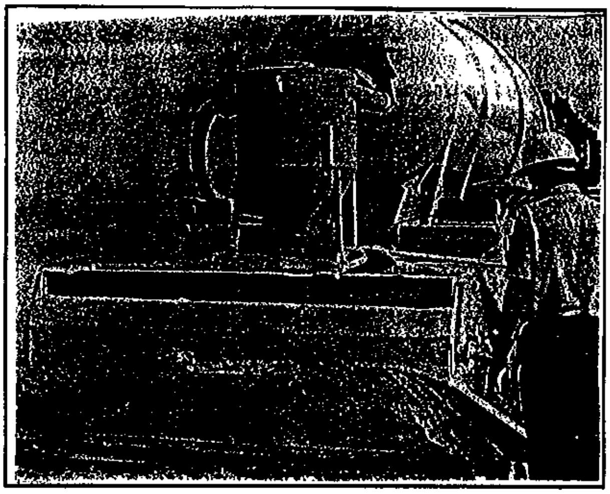


Figure G-9. Bulk mechanical spreader

(4) *Compaction, finishing, curing, and protection.* These construction techniques for mixed-in-place construction are essentially identical to those for central plant soil cement.

G-7. Quality Control, Inspection, and Testing

Adequate quality control and inspection procedures are important factors in successful soil-cement construction. Construction control procedures for soil-cement are fairly standardized. The quality of the two basic operations (soil-cement mixing and actual construction) are insured through control of four basic factors: cement content, moisture content, compaction, and curing. These factors can be controlled easily by organizing the inspection steps into a routine that fits in with the sequence of construction steps. These steps are slightly different for central-plant construction and mixed-in-place construction.

a. Central-plant construction. The inspector checks on the following items.

(1) *Construction site and equipment.* Equipment must be clean, appropriate for the soil type, adjusted properly, and designed to preclude contamination introduction. Hauling vehicles must have protective covers where appropriate. The site should be set up to meet production and timing requirements and provide efficient traffic flow and proper separation distances for material stockpiles.

(2) *Soil.* Soil must match identification data given in the laboratory report. The inspector should check for uniformity of color, texture, and moisture. The soil should be monitored as it is stockpiled. Upon completion of the stockpile it is sampled and tested for acceptance. Gradation, specific gravity, and Atterberg limits should be tested regularly.

(3) *Cement application.* The amount of cement is specified either as a percentage of cement by weight of oven-dry soil material, or in pounds of cement per cubic foot of compact soil-cement. Pre-construction plant calibration and daily calibration checks insure an accurate mix. Different types of calibration procedures are applicable depending on the type of mixing plant used. In addition to plant calibration and daily checks of mix proportions, freshly mixed soil-cement cement content can be tested using a titration test and hardened soil-cement cement content can be tested using ASTM D 806.

(4) *Water Application.* Water is added at the central mixing plant in quantities sufficient to bring the mixture to the optimum moisture content as determined by a laboratory moisture-density test. Generally the moisture content should not be more than two percentage points below or above the specified optimum moisture. To estimate mixing water requirements stockpile moisture content is determined and additional water requirements calculated. Experienced inspectors can determine, in a qualitative way, the moisture requirements just prior to compaction by squeezing the mixture in the palm of the hand. A mixture near optimum moisture content is just moist enough to dampen the hands when packed tightly and can be broken in two with little or no crumbling. During compaction the surface of the material may dry out (indicated by a graying of the surface). Moisture is brought back to optimum by fog spraying.

(5) *Mix uniformity.* Uniformity is checked visually by noting color uniformity either at the plant or by digging a hole in the loosely placed material in the layer. If, due to lightness of soil material color, it is difficult to determine mixing, a 2 percent solution of phenolphthalein can be sprayed on a cut face of the material to determine if any cement is present. The cement in the mixture will turn treated material pinkish-red while untreated soil will retain its natural color.

(6) *Transporting and spreading.* Specified timing requirements for transporting and spreading should be monitored. Traffic patterns and possible material contamination (especially near layer edges and ramps) should be checked. Layer offset distances and layer thickness and uniformity should also be checked. The spreader should not be allowed to empty, but should be stopped while there is still mix left in the hopper. This insures uniform spreader operation.

(7) *Compaction.* Samples of the soil-cement are taken from the batch and prepared for laboratory moisture-density testing at the same time compaction is taking place. This accounts for timing parameters. In-place density testing is conducted as soon as possible after compaction in a spot where the laboratory material has been taken. Field and laboratory densities are then compared.

(8) *Curing.* Curing specifications and placement procedures should be closely monitored by the inspector. If water curing is used, the equipment must be capable of fog, rather than pressure, spraying. The surface must be kept continuously moist. Exposed surfaces should be cured for seven days. Curing times must be satisfied as well as provisions made in the case of freezing temperatures. Membrane cures must be of sufficient thickness to hold in moisture.

G-8. References

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Appendix H
Notation

The symbols that follow are used throughout this manual and correspond wherever possible to those recommended by the American Society of Civil Engineers.

Symbol	Term
c	Cohesion per unit area; a constant for natural top stratum where $c = \sqrt{\dfrac{k_{bl}}{k_f\, z_{bl}\, d}}$
c'	Effective cohesion in terms of effective stress
c_v	Coefficient of consolidation
C_c	Compression index
C_α	Coefficient of secondary compression
d	Effective thickness of pervious substratum
e	Void ratio
F_t	Transformation factor for permeability
h_o	Excess hydrostatic head
h'_o	Hydrostatic head beneath landside toe of levee
h_x	Hydrostatic head beneath top stratum
H	Net head
i_c	Critical gradient for landside top stratum
i_1	Upward gradient at landside toe of berm
i_o	Upward gradient at landside toe of levee
k	Coefficient of permeability
k_b	Coefficient of permeability (top stratum)
k_f	Average horizontal coefficient of permeability
k_n	Coefficient of permeability (vertical)
k_{bl}	Permeability of landside stratum
k_{br}	Permeability of riverside stratum
L_1	Distance from riverside levee toe to river
L_2	Base width of levee and berm
L_3	Length of top stratum landward of levee toe
M_d	Slope of hydraulic grade line

(Continued)

Symbol	Term
Q	Shear test for specimen tested at constant water content (unconsolidated-undrained)
Q_s	Total amount of seepage passing beneath levee
R	Shear test for specimen consolidated and then sheared at constant water content (consolidated-undrained)
s	(a) distance from the landside toe of the levee to the point of effective seepage entry
	(b) shear test for specimen consolidated and sheared without restriction of change in water content (consolidated-drained)
x_1	Effective length of riverside blanket
x_3	Distance from landside levee toe to effective seepage exit
z_b	Effective thickness of stratum
z_t	Transformed thickness of top stratum
z_{bl}	Effective thickness of landside top stratum
z_{br}	Effective thickness of riverside top stratum
z_{bt}	Effective thickness of top stratum
$ã_t$	Wet unit weight of soil
$ã_w$	Unit weight of water
ã′	Submerged or buoyant unit weight of soil
ö′	Angle of internal friction based on effective stresses
S	Shape factor to generalized cross section of the levee and foundation