EM 1110-2-1913
30 April 2000

US Army Corps
of Engineers
ENGINEERING AND DESIGN

Design and Construction of Levees

ENGINEER MANUAL

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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-2-1913

CECW-EG

Manual
No. 1110-2-1913

30 April 2000

Engineering and Design
DESIGN AND CONSTRUCTION OF LEVEES

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Chapter 5
Seepage Control

Section I
Foundation Underseepage

5-1. General

Without control, underseepage in pervious foundations beneath levees may result in (a) excessive hydrostatic pressures beneath an impervious top stratum on the landside, (b) sand boils, and (c) piping beneath the levee itself. Underseepage problems are most acute where a pervious substratum underlies a levee and extends both landward and riverward of the levee and where a relatively thin top stratum exists on the landside of the levee. Principal seepage control measures for foundation underseepage are (a) cutoff trenches, (b) riverside impervious blankets, (c) landside seepage berms, (d) pervious toe trenches, and (e) pressure relief wells. These methods will be discussed generally in the following paragraphs. Detailed design guidance is given in Appendixes B and C. Turnbull and Mansur (1959) have proposed control measures for underseepage also. Additional information on seepage control in earth foundations including cutoffs, impervious blankets, seepage berms, relief wells and trench drains is given in EM 1110-2-1901 and EM 1110-2-1914.

5-2. Cutoffs

A cutoff beneath a levee to block seepage through pervious foundation strata is the most positive means of eliminating seepage problems. Positive cutoffs may consist of excavated trenches backfilled with compacted earth or slurry trenches usually located near the riverside toe. Since a cutoff must penetrate approximately 95 percent or more of the thickness of pervious strata to be effective, it is not economically feasible to construct cutoffs where pervious strata are of considerable thickness. For this reason cutoffs will rarely be economical where they must penetrate more than 12.2 m (40 ft). Steel sheet piling is not entirely watertight due to leakage at the interlocks but can significantly reduce the possibility of piping of sand strata in the foundation. Open trench excavations can be readily made above the water table, but if they must be made below the water table, well point systems will be required. Cutoffs made by the slurry trench method (reference Appendix A) can be made without a dewatering system, and the cost of this type of cutoff should be favorable in many cases in comparison with costs of compacted earth cutoffs.

5-3. Riverside Blankets

Levees are frequently situated on foundations having natural covers of relatively fine-grained impervious to semipervious soils overlying pervious sands and gravels. These surface strata constitute impervious or semipervious blankets when considered in connection with seepage control. If these blankets are continuous and extend riverward for a considerable distance, they can effectively reduce seepage flow and seepage pressures landside of the levee. Where underseepage is a problem, riverside borrow operations should be limited in depth to prevent breaching the impervious blanket. If there are limited areas where the blanket becomes thin or pinches out entirely, the blanket can be made effective by placing impervious materials in these areas. The effectiveness of the blanket depends on its thickness, length, distance to the levee riverside toe, and permeability and can be evaluated by flow-net or approximate mathematical solutions, as shown in Appendix B. Protection of the riverside blanket against erosion is important.

5-4. Landside Seepage Berms

a. General. If uplift pressures in pervious deposits underlying an impervious top stratum landward of a levee become greater than the effective weight of the top stratum, heaving and rupturing of the top stratum may occur, resulting in sand boils. The construction of landside berms (where space is available) can eliminate this hazard by providing (a) the additional weight needed to counteract these upward seepage forces and (b) the additional length required to reduce uplift pressures at the toe of the berm to tolerable values. Seepage berms may reinforce an existing impervious or semipervious top stratum, or, if none exists, be placed directly on pervious deposits. A berm also affords some protection against sloughing of the landside levee slope. Berms are relatively simple to construct and require very little maintenance. They frequently improve and reclaim land as areas requiring underseepage treatment are often low and wet. Berms can also serve as a source of borrow for emergency repairs to the levee. Because they require additional fill material and space, they are used primarily with agricultural levees. Subsurface profiles must be carefully studied in selecting berm widths. For example, where a levee is founded on a thin top stratum and thicker clay deposits lie a short distance landward, as shown in Figure 5-1, the berm should extend far enough landward to lap the thick clay deposit, regardless of the computed required length. Otherwise, a concentration of seepage and high exit gradients may occur between the berm toe and the landward edge of the thick clay deposit.

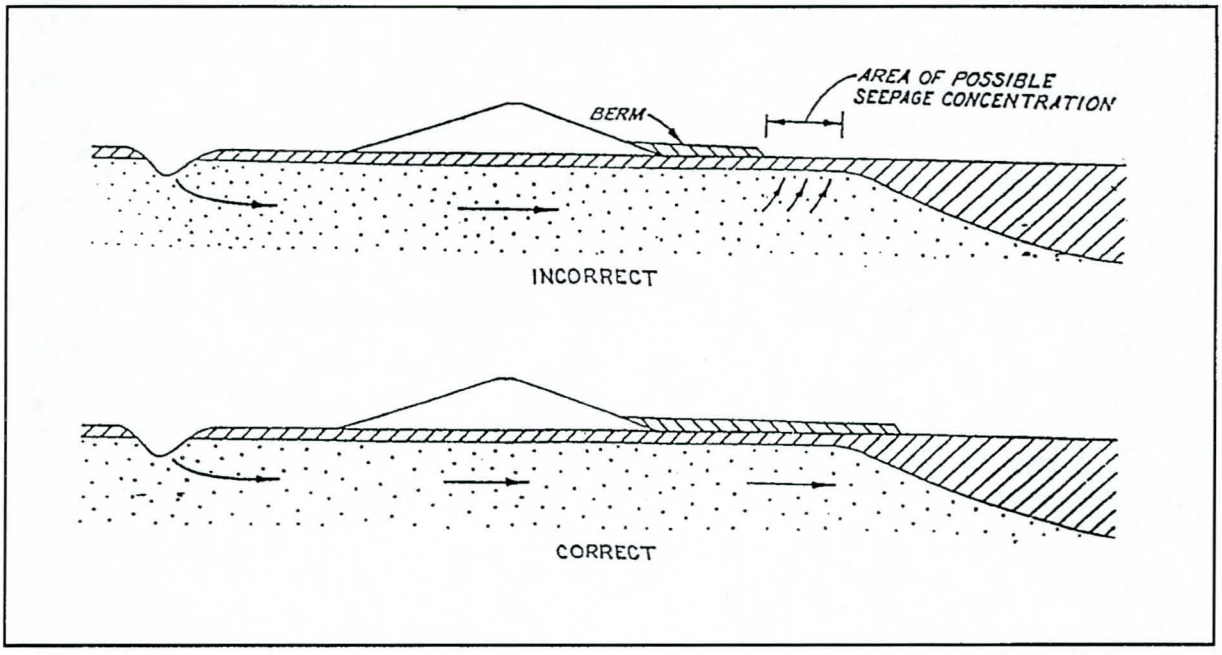


Figure 5-1. Example of incorrect and correct berm length according to existing foundation conditions

b. Types of seepage berms. Four types of seepage berms have been used, with selection based on available fill materials, space available landside of the levee proper, and relative costs.

(1) Impervious berms. A berm constructed of impervious soils restricts the pressure relief that would otherwise occur from seepage flow through the top stratum, and consequently increases uplift pressures

beneath the top stratum. However, the berm can be constructed to the thickness necessary to provide an adequate factor of safety against uplift.

(2) Semipervious berms. Semipervious material used in constructing this type of berm should have an in-place permeability equal to or greater than that of the top stratum. In this type of berm, some seepage will pass through the berm and emerge on its surface. However, since the presence of this berm creates additional resistance to flow, subsurface pressures at the levee toe will be increased.

(3) Sand berms. While a sand berm will offer less resistance to flow than a semipervious berm, it may also cause an increase in substratum pressures at the levee toe if it does not have the capacity to conduct seepage flow landward without excessive internal head losses. Material used in a sand berm should be as pervious as possible, with a minimum permeability of 100×10^{-4} cm per sec. Sand berms require less material and occupy less space than impervious or semipervious berms providing the same degree of protection.

(4) Free-draining berms. A free-draining berm is one composed of random fill overlying horizontal sand and gravel drainage layers (with a terminal perforated collector pipe system), designed by the same methods used for drainage layers in dams. Although the free-draining berm can afford protection against underseepage pressures with less length and thickness than the other types of seepage berms, its cost is generally much greater than the other types, and thus it is rarely specified.

c. Berm design. Design equations, criteria, and examples are presented in Appendix C for seepage berms.

d. Computer programs to use for seepage analysis.

(1) If the soil can be idealized with a top blanket of uniform thickness and seepage flow is assumed to be horizontal in the foundation and vertical in the blanket, then LEVSEEP (Brizendine, Taylor, and Gabr 1995) or LEVEEMSU (Wolff 1989; Gabr, Taylor, Brizendine, and Wolff 1995) could be used.

(2) If the soil profile is characterized by a top blanket and two foundation layers of uniform thickness, and seepage flow is assumed to be horizontal in the foundation, horizontal and vertical in the transition layer, and vertical in the blanket, then LEVEEMSU or the finite element method (CSEEP) could be used (Biedenharn and Tracy 1987; Knowles 1992; Tracy 1994; Gabr, Brizendine, and Taylor 1995). LEVEESMU would be simpler to use.

(3) If the idealized soil profile includes irregular geometry (slopes greater than 1 vertical to 100 horizontal), more than three layers and/or anisotropic permeability ($k_v \neq k_h$), then only the finite element method (CSEEP) is applicable. When using CSEEP it is recommended that FastSEEP, a graphical pre- and post-processor, be used for mesh generation, assigning boundary conditions and soil properties, and viewing the results (Engineering Computer Graphics Laboratory 1996).

5-5. Pervious Toe Trench

a. General. Where a levee is situated on deposits of pervious material overlain by little or no impervious material, a partially penetrating toe trench, as shown in Figure 5-2, can improve seepage conditions at or near the levee toe. Where the pervious stratum is thick, a drainage trench of any practicable depth would attract only a small portion of the seepage flow and detrimental underseepage would bypass the trench. Consequently, the main use of a pervious toe trench is to control shallow underseepage and protect the area in the vicinity of the levee toe. Pervious toe trenches may be used in conjunction with relief well systems;

the wells collect the deeper seepage and the trench collects the shallow seepage. Such a system is shown in Figure 5-3. The trench is frequently provided with a perforated pipe to collect the seepage. The use of a collector system is dependent on the volume of seepage and, to some degree, the general location of the levee. Collector systems are usually not required for agricultural levees but find wider use in connection with urban levees.

b. Location. As seen in Figures 5-2 and 5-3, pervious drainage trenches are generally located at the levee toe, but are sometimes constructed beneath the downstream levee slope as shown in Figure 5-4. Here the trench is located at the landward quarter point of the levee, and discharge is provided through a horizontal pervious drainage layer. Unless it is deep enough, it may allow excessive seepage pressures to act at the toe. There is some advantage to a location under the levee if the trench serves also as an inspection trench and because the horizontal pervious drainage layer can help to control embankment seepage.

c. Geometry. Trench geometry will depend on the volume of expected underseepage, desired reduction in uplift pressure, construction practicalities, and the stability of the material in which it is being excavated. Trench widths varying from 0.61 to 1.83 m (2 to 6 ft) have been used. Trench excavation can be expedited if a ditching machine can be used. However, narrow trench widths will require special compaction equipment. One such piece of equipment (Figure 5-5), which is a vibrating-plate type of compactor specially made to fit on the boom of a backhoe, has apparently performed satisfactorily.

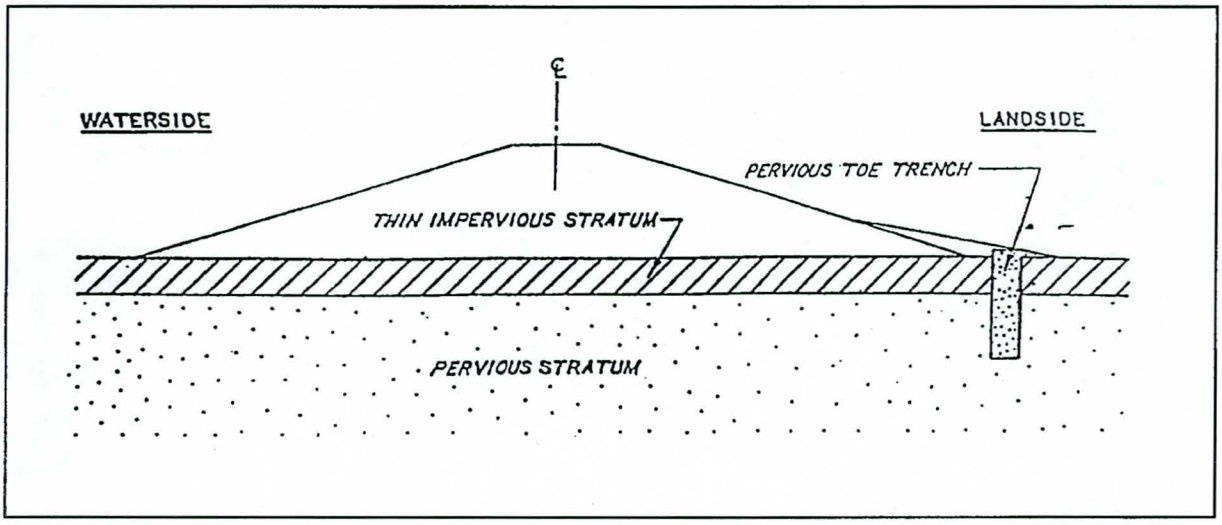


Figure 5-2. Typical partially penetrating pervious toe trench

d. Backfill. The sand backfill for trenches must be designed as a filter material in accordance with criteria given in Appendix D. If a collector pipe is used, the pipe should be surrounded by about a 305-mm (1-ft) thickness of gravel having a gradation designed to provide a stable transition between the sand backfill and the perforations or slots in the pipe. A typical section of a pervious drainage trench with collector pipe is shown in Figure 5-6. Placement of trench backfill must be done in such a manner as to minimize segregation. Compaction of the backfill should be limited to prevent breakdown of material or over compaction resulting in lowered permeabilities.

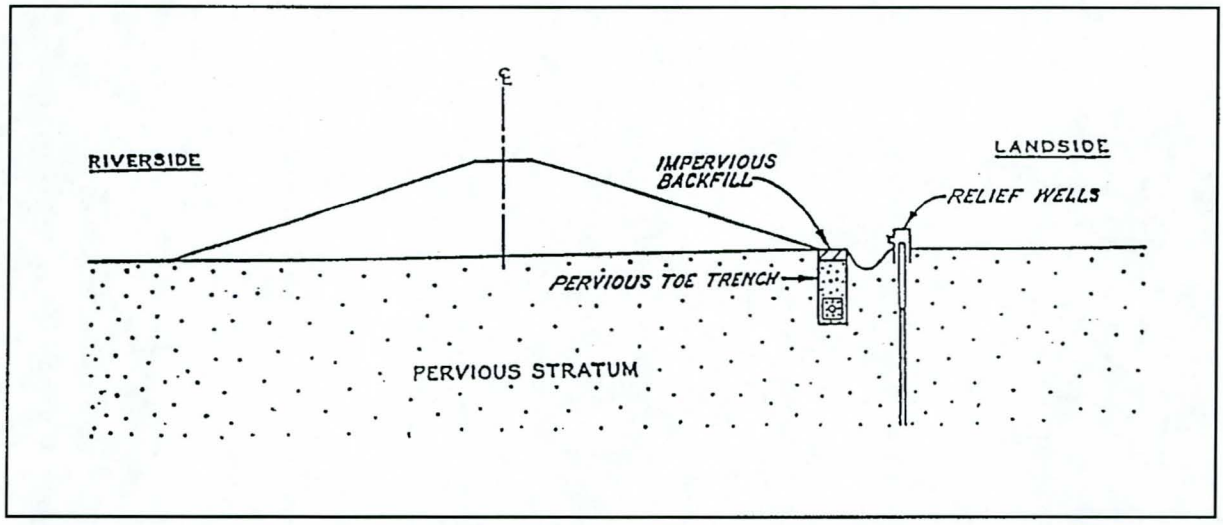


Figure 5-3. Typical pervious toe trench with collector pipe (Figure 5-6 shows trench details)

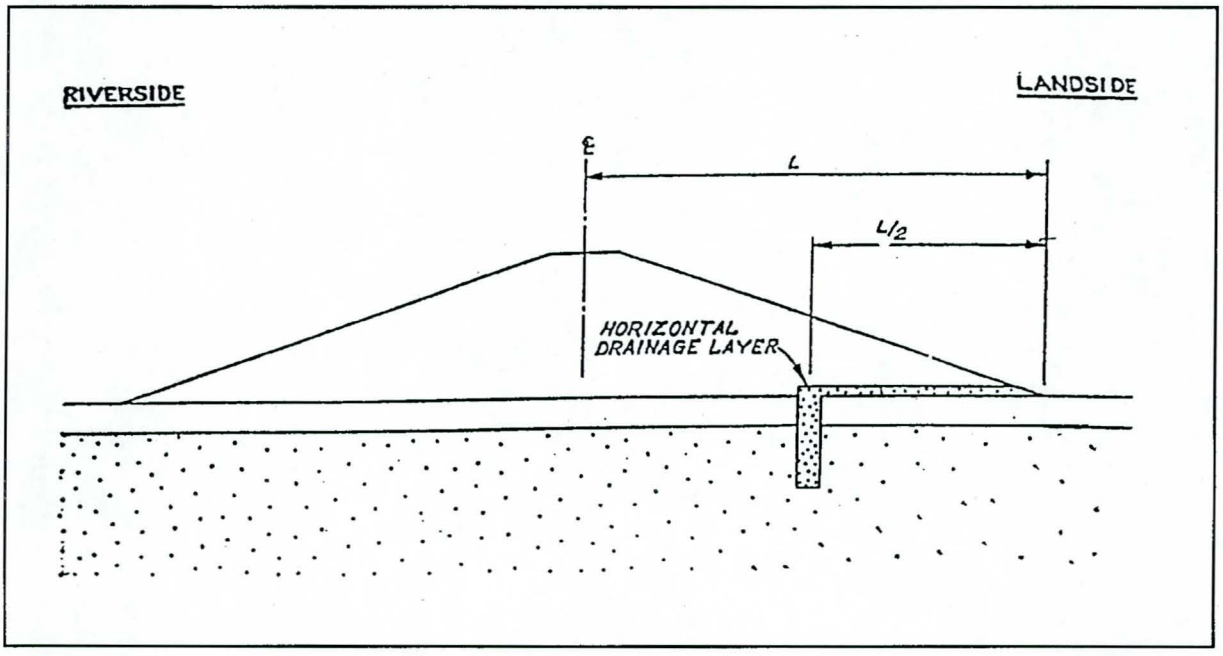


Figure 5-4. Pervious toe trench located beneath landward slope

5-6. Pressure Relief Wells

a. General. Pressure relief wells may be installed along the landside toe of levees to reduce uplift pressure which may otherwise cause sand boils and piping of foundation materials. Wells accomplish this by intercepting and providing controlled outlets for seepage that would otherwise emerge uncontrolled landward of the levee. Pressure relief well systems are used where pervious strata underlying a levee are

Figure 5-5. Special equipment for compacting sand in pervious toe trenches

too deep or too thick to be penetrated by cutoffs or toe drains or where space for landside berms is limited. Relief wells should adequately penetrate pervious strata and be spaced sufficiently close to intercept enough seepage to reduce to safe values the hydrostatic pressures acting beyond and between the wells. The wells must offer little resistance to the discharge of water while at the same time prevent loss of any soil. They must also be capable of resisting corrosion and bacterial clogging. Relief well systems can be easily expanded if the initial installation does not provide the control needed. Also, the discharge of existing wells can be increased by pumping if the need arises. A relief well system requires a minimum of additional real estate as compared with the other seepage control measures such as berms. However, wells require periodic maintenance and frequently suffer loss in efficiency with time, probably due to clogging of well screens by muddy surface waters, bacteria growth, or carbonate incrustation. They increase seepage discharge, and means for collecting and disposing of their discharge must be provided.

b. Design of well systems. The design of a pressure relief well system involves determination of well spacing, size, and penetration to reduce uplift between wells to allowable values. Factors to be considered are (a) depth, stratification, and permeability of foundation soils, (b) distance to the effective source of seepage, (c) characteristics of the landside top stratum, if any, and (d) degree of pressure relief desired. Guidance on the method used to determine well spacing, size, and penetration is contained in EM 1110-2-1914 and U.S. Army Engineer Waterways Experiment Station TM No. 3-424. Where no control measures are present, relief wells for agricultural and urban levees should be designed so that i_{max} midway between the wells or landward from the well line should not exceed 0.50 (equivalent to FS = 1.7 for an average soil saturated unit weight of 1840 kg/m^3 (115 pcf)). Many combinations of well spacing and penetration will produce the desired pressure relief; hence, the final selected spacing and penetration must be based on cost comparisons of alternative combinations. After the general well spacing for a given reach of levee has been determined, the actual location of each well should be established to ensure that the wells will be located at critical seepage points and will fit natural topographic features.

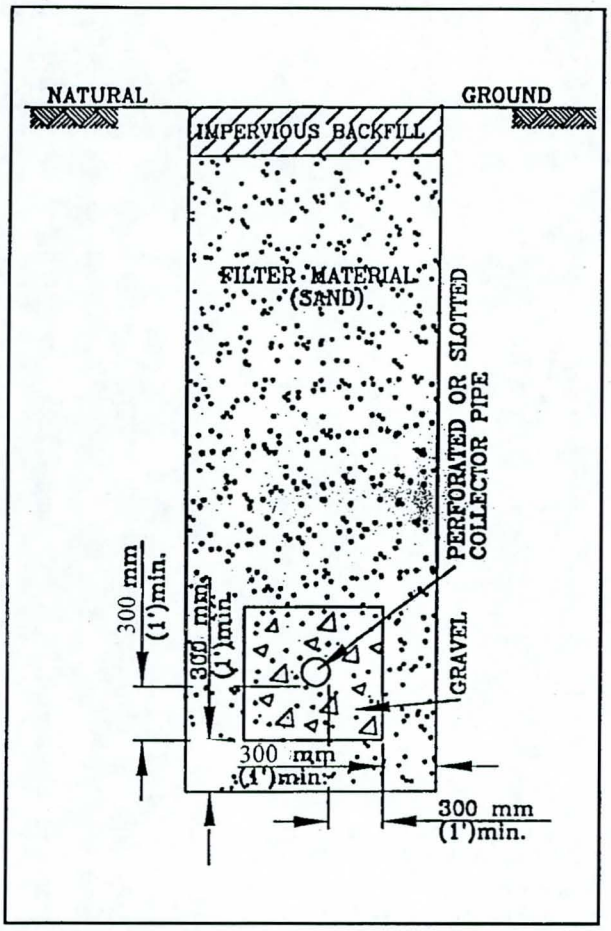


Figure 5-6. Pervious toe trench with collector pipe

c. Design of individual wells. The design of the well involves the selection of type and length of riser pipe and screen, design of the gravel pack, and design of well appurtenances. A widely used well design that has given good service in the past is shown in Figure 5-7.

(1) Riser pipe and screen. The well screen normally extends from just below the top of the pervious stratum to the bottom of the well, with solid riser pipe installed from the top of the pervious strata to the surface. In zones of very fine sand or silt, the screen is replaced by unperforated (blank) pipe. The type of material for the riser and screen should be selected only after a careful study of the corrosive properties of the water to be carried by the well. Many types of metals, alloys, fiberglass, plastics, and wood have been used in the past. At the present time, stainless steel and plastic are the most widely used, primarily because of their corrosion-resistant properties. Plastic risers should be considered with caution, being susceptible to damages during mechanical treatment or chemical treatment which develop excessive heat or cold.

(2) Filter. The filter that surrounds the screen must be designed in accordance with criteria given in Appendix D using the slot size of the screen and the gradation of surrounding pervious deposit as a basis of design. No matter what size screen is used, a minimum of 152.4 mm (6 in.) of filter material should surround the screen and the filter should extend a minimum of 610.8 mm (2 ft) above the top and 1.2 m (4 ft) below the bottom of the well screen. Above the filter to the bottom of the concrete or impervious backfill, sand backfill may be used.

(3) Well appurtenances. In selecting well appurtenances, consideration must be given to ease of maintenance, protection against contamination from back flooding, damage by debris, and vandalism. To prevent wells from becoming backflooded with muddy surface water, which greatly impairs their efficiency when they are not flowing, an aluminum check valve, rubber gasket, and plastic standpipe, as shown in Figure 5-7, can be installed on each well. To safeguard against vandalism, accidental damage, and the entrance of debris, the tops of the wells should be provided with a metal screen or flap-type gate. The elevation of the top of any protective standpipes must be used in design as the well discharge elevation.

d. Well installation. Proper methods of drilling, backfilling, and developing a relief well must be employed or the well will be of little or no use. These procedures are described in detail in EM 1110-2-1914.

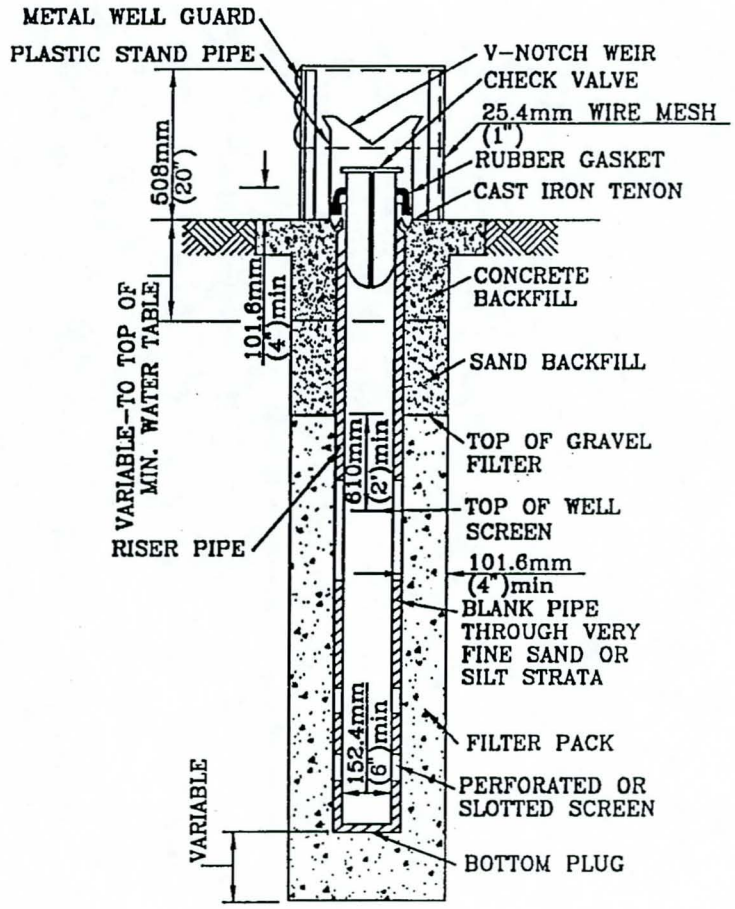


Figure 5-7. Typical relief well

Section II
Seepage Through Embankments

5-7. General

Should through seepage in an embankment emerge on the landside slope (Figure 5-8a), it can soften fine-grained fill in the vicinity of the landside toe, cause sloughing of the slope, or even lead to piping (internal erosion) of fine sand or silt materials. Seepage exiting on the landside slope would also result in high seepage forces, decreasing the stability of the slope. In many cases, high water stages do not act against the levee long enough for this to happen, but the possibility of a combination of high water and a period of heavy precipitation may bring this about. If landside stability berms or berms to control underseepage are required because of foundation conditions, they may be all that is necessary to prevent seepage emergence on the

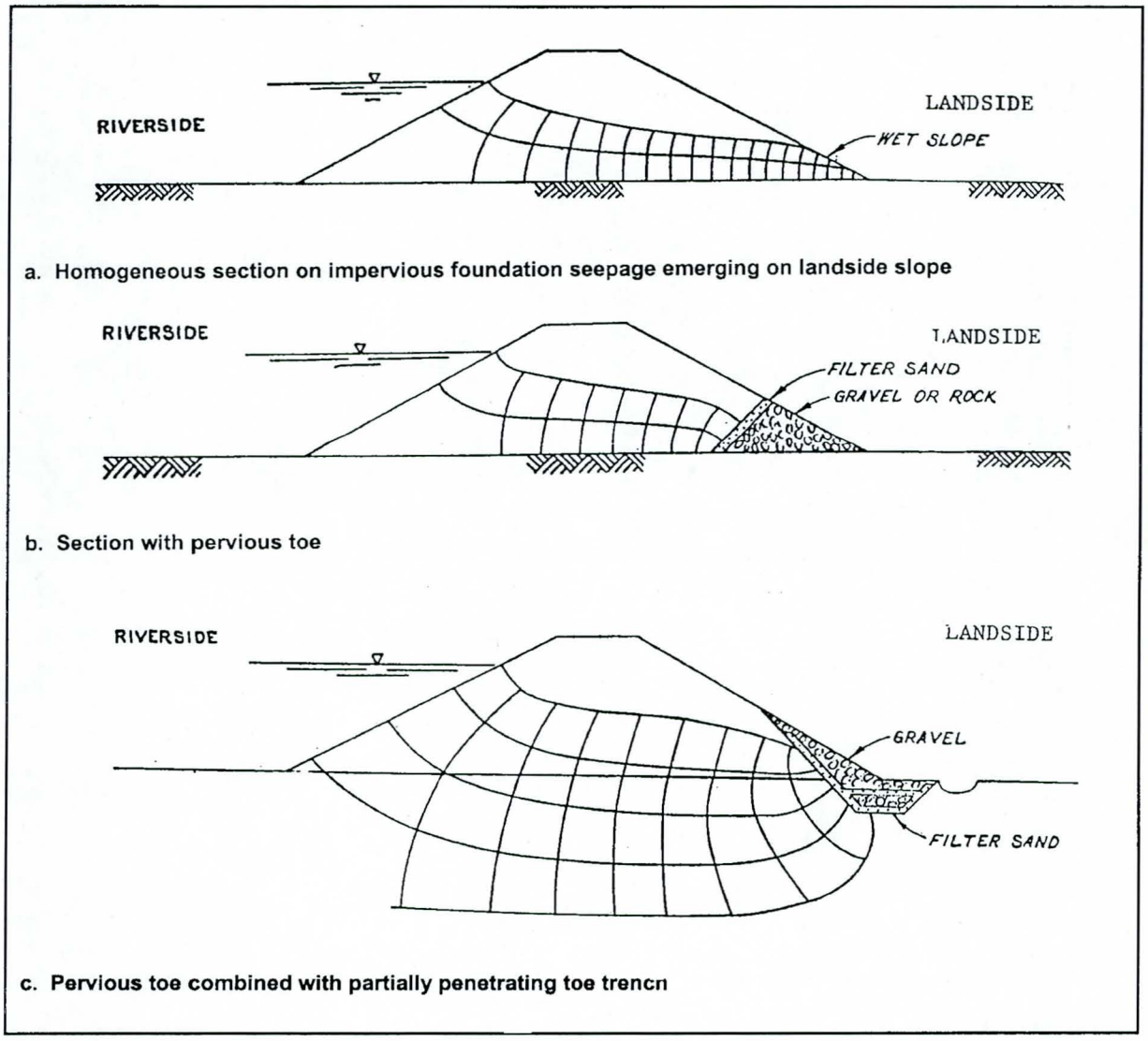


Figure 5-8. Embankment with through seepage

slope. On the other hand, if no berms are needed, landside slopes are steep, and floodstage durations and other pertinent considerations indicate a potential problem of seepage emergence on the slope, provisions should be incorporated in the levee section such as horizontal and/or inclined drainage layers or toe drains to prevent seepage from emerging on the landside slope. These require select pervious granular material and graded filter layers to ensure continued functioning, and therefore add an appreciable cost to the levee construction, unless suitable materials are available in the borrow areas with only minimal processing required. Where large quantities of pervious materials are available in the borrow areas, it may be more practicable to design a zoned embankment with a large landside pervious zone. This would provide an efficient means of through seepage control and good utilization of available materials. Additional information on seepage control in earth embankments including zoning embankments and vertical (or inclined) and horizontal drains is given in Chapter 8 of EM 1110-2-1901.

5-8. Pervious Toe Drain

A pervious toe (Figure 5-8b) will provide a ready exit for seepage through the embankment and can lower the phreatic surface sufficiently so that no seepage will emerge on the landside slope. A pervious toe can also be combined with partially penetrating toe trenches, which have previously been discussed, as a method for controlling shallow underseepage. Such a configuration is shown in Figure 5-8c.

5-9. Horizontal Drainage Layers

Horizontal drainage layers, as shown in Figure 5-9a, essentially serve the same purpose as a pervious toe but are advantageous in that they can extend further under the embankment requiring a relatively small amount of additional material. They can also serve to protect the base of the embankment against high uplift pressures where shallow foundation underseepage is occurring. Sometimes horizontal drainage layers serve also to carry off seepage from shallow foundation drainage trenches some distance under the embankment as shown previously in Figure 5-4.

5-10. Inclined Drainage Layers

An inclined drainage layer as shown in Figure 5-9b is one of the more positive means of controlling internal seepage and is used extensively in earth dams. It is rarely used in levee construction because of the added cost, but might be justified for short levee reaches in important locations where landside slopes must be steep and other control measures are not considered adequate and the levee will have high water against it for prolonged periods. The effect of an inclined drainage layer is to completely intercept embankment seepage regardless of the degree of stratification in the embankment or the material type riverward or landward of the drain. As a matter of fact, the use of this type of drain allows the landside portion of a levee to be built of any material of adequate strength regardless of permeability. When used between an impervious core and outer pervious shell (Figure 5-9c), it also serves as a filter to prevent migration of impervious fines into the outer shell. If the difference in gradation between the impervious and pervious material is great, the drain may have to be designed as a graded filter (Appendix D). Inclined drains must be tied into horizontal drainage layers to provide an exit for the collected seepage as shown in Figures 5-9b and 5-9c.

5-11. Design of Drainage Layers

The design of pervious toe drains and horizontal and inclined drainage layers must ensure that such drains have adequate thickness and permeability to transmit seepage without any appreciable head loss while at the same time preventing migration of finer soil particles. The design of drainage layers must satisfy the criteria outlined in Appendix D for filter design. Horizontal drainage layers should have a minimum thickness of 457.2 mm (18 in.) for construction purposes.

5-12. Compaction of Drainage Layers

Placement and compaction of drainage layers must ensure that adequate density is attained, but should not allow segregation and contamination to occur. Vibratory rollers are probably the best type of equipment for compaction of cohesionless material although crawler tractors and rubber-tired rollers have also been used successfully. Saturation or flooding of the material as the roller passes over it will aid in the compaction process and in some cases has been the only way specified densities could be attained. Care must always be taken to not overcompact to prevent breakdown of materials or lowering of expected permeabilities. Loading, dumping, and spreading operations should be observed to ensure that segregation does not occur. Gradation tests should be run both before and after compaction to ensure that the material meets specifications and does not contain too many fines.

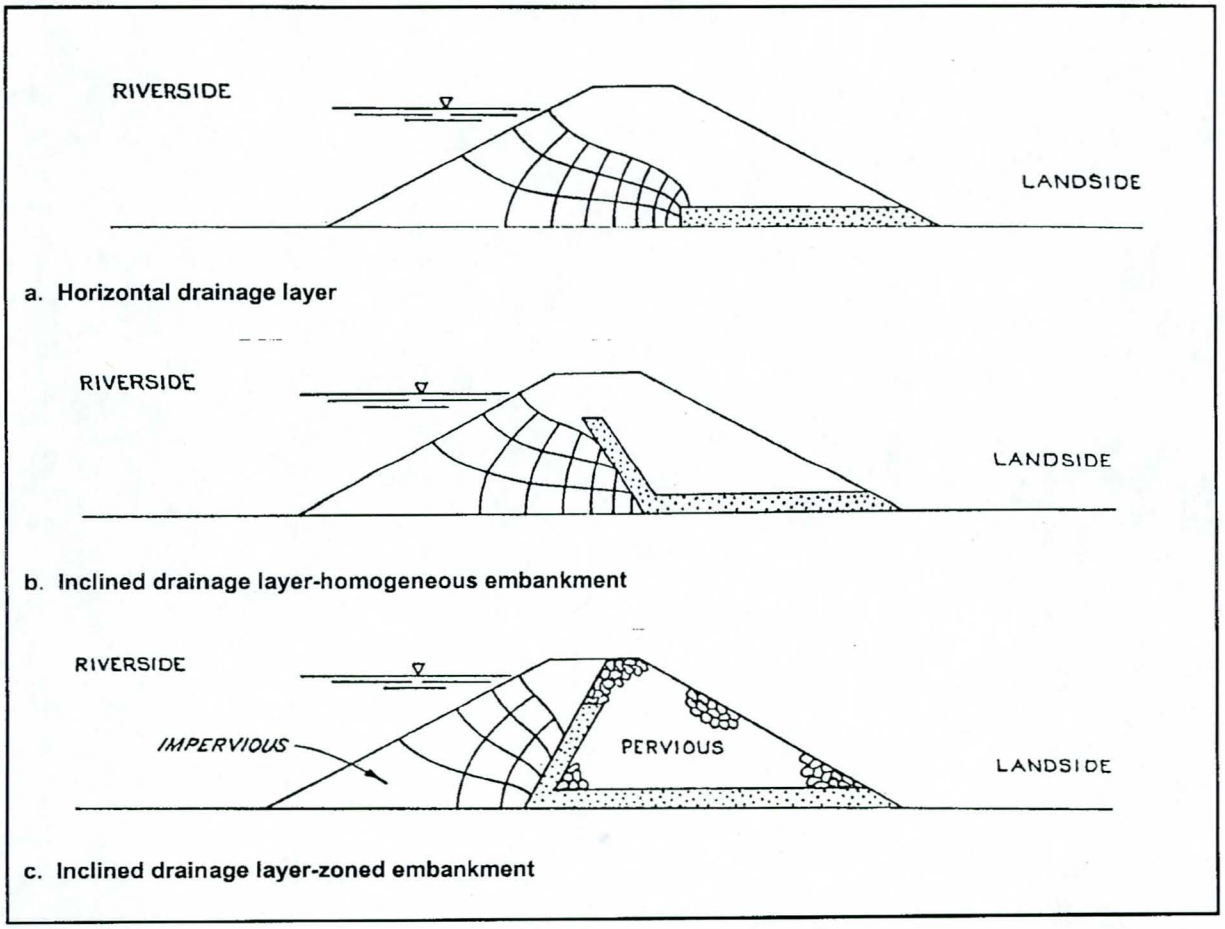


Figure 5-9. Use of horizontal and inclined drainage layers to control seepage through an embankment

Chapter 7
Levee Construction

Section I
Levee Construction Methods

7-1. Classification of Methods

a. Levee embankments classified according to construction methods used are listed in Table 7-1 for levees composed of impervious and semipervious materials (i.e., those materials whose compaction characteristics are such as to produce a well-defined maximum density at a specific optimum water content). While the central portion of the embankment may be Category I (compacted) or II (semicompacted), riverside and landside berms (for seepage or stability purposes) may be constructed by Category II or III (uncompacted) methods.

b. Pervious levee fill consisting of sands or sands and gravels may be placed either in the dry with normal earthmoving equipment or by hydraulic fill methods. Except in seismically active areas or other areas requiring a high degree of compaction, compaction by vibratory means other than that afforded by tracked bulldozers is not generally necessary. Where underwater placement is required, it can best be accomplished with pervious fill using end-dumping, dragline, or hydraulic means, although fine-grained fill can be so placed if due consideration is given to the low density and strength obtained using such materials.

Section II
Foundations

7-2. Foundation Preparation and Treatment

a. General. Minimum foundation preparation for levees consists of clearing and grubbing, and most levees will also require some degree of stripping. Clearing, grubbing, stripping, the disposal of products therefrom, and final preparation are discussed in the following paragraphs.

b. Clearing. Clearing consists of complete removal of all objectional and/or obstructional matter above the ground surface. This includes all trees, fallen timber, brush, vegetation, loose stone, abandoned structures, fencing, and similar debris. The entire foundation area under the levee and berms should be cleared well ahead of any following construction operations.

c. Grubbing. Grubbing consists of the removal, within the levee foundation area, of all stumps, roots, buried logs, old piling, old paving, drains, and other objectional matter. Grubbing is usually not necessary beneath stability berms. Roots or other intrusions over 38.1 mm (1-1/2 in.) in diameter within the levee foundation area should be removed to a depth of 0.91 m (3 ft) below natural ground surface. Shallow tile drains sometimes found in agricultural areas should be removed from the levee foundation area. The sides of all holes and depressions caused by grubbing operations should be flattened before backfilling. Backfill, consisting or material similar to adjoining soils, should be placed in layers up to the final foundation grade and compacted to a density equal to the adjoining undisturbed material. This will avoid "soft spots" under the levee and maintain the continuity of the natural blanket.

d. Stripping. After foundation clearing and grubbing operations are complete, stripping is commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil. The depth of stripping

Table 7-1
Classification According to Construction Method of Levees Composed of Impervious and Semipervious Materials

Category	Construction Method	Use
I. Compacted	Specification of:	Provides embankment section occupying minimum space. Provides strong embankments of low compressibility needed adjacent to concrete structures or forming parts of highway systems.
	a. Water content range with respect to standard effort optimum water content	
	b. Loose lift thickness (152.4 mm to 228.6 mm (6-9 in.))	
	c. Compaction equipment (sheepsfoot or rubber-tired rollers)	Requires strong foundation of low compressibility and availability of borrow materials with natural water contents reasonably close to specified ranges.
	d. Number of passes to attain a given percent compaction based on standard maximum density	
	e. Minimum required density	Used where field inspection is not constant throughout the project.
II. Semicompacted	Compaction of fill materials at their natural water content (i.e., no water content control). Borrow materials known to be too wet would require some drying before placement. Placed in thicker lifts than Category I (about 304.8 mm (12 in.)) and compacted either by controlled movement of hauling and spreading equip ment or by fewer passes of sheepsfoot or rubber-tired rollers. Compaction evaluated relative to 15-blow compaction test.	The most common type of levee construction used in reaches where:
		a. There are no severe space limitations and steep-sloped Category I embankments are not required.
		b. Relatively weak foundations could not support steep-sloped Category I embankments.
		c. Underseepage conditions are such as to required wider embankment base than is provided by Category I construction.
		d. Water content of borrow materials or amount of rainfall during construction season is such as not to justify Category I compaction.
III. Uncompacted	*a.* Fill cast or dumped in place in thick layers with little or no spreading or compaction.	Levees infrequently constructed today using method except for temporary emergency. Both methods are used for construction of stability berms, pit fills and seepage berms.
	b. Hydraulic fill by dredge, often from channel excavation.	

is determined by local conditions and normally varies from 152.4 to 304.8 mm (6 to 12 in.) Stripping is usually limited to the foundation of the levee embankment proper, not being required under berms. All stripped material suitable for use as topsoil should be stockpiled for later use on the slopes of the embankment and berms. Unsuitable material must be disposed of by methods described in the next paragraph.

e. Disposal of debris. Debris from clearing, grubbing, and stripping operations can be disposed of by burning in areas where this is permitted. When burning is prohibited by local regulations, it needs to be disposed of in an environmentally approved manner.

f. Exploration trench. An exploration trench (often termed "inspection trench") should be excavated under all levees unless special conditions as discussed later warrant its omission. The purpose of this trench is to expose or intercept any undesirable underground features such as old drain tile, water or sewer lines, animal burrows, buried logs, pockets of unsuitable material, or other debris. The trench should be located at or near the centerline of hauled fill levees or at or near the riverside toe of sand levees so as to connect with waterside impervious facings. Dimensions of the trench will vary with soil conditions and embankment configurations. Backfill should be placed only after a careful inspection of the excavated trench to ensure that seepage channels or undesirable material are not present; if they are, they should be dug out with a base of sufficient width to allow backfill compaction with regular compaction equipment. To backfill narrower trenches properly, special compaction procedures and/or equipment will be required. Trenches should have a minimum depth of 1.83 m (6 ft) except for embankment heights less than 1.83 m (6 ft), in which case the minimum depth should equal the embankment height. Exploration trenches can be omitted where landside toe drains beneath the levee proper constructed to comparable depths are employed (toe drains are discussed in more detail later in this chapter).

g. Dewatering. Dewatering levee foundations for the purpose of excavation and back filling in the dry is expensive if more than simple ditches and sumps are required, and is usually avoided if at all possible. The cost factor may be an overriding consideration in choosing seepage control measures other than a compacted cutoff trench, such as berms, blankets, or relief wells. Where a compacted cutoff trench involving excavation below the water table must be provided, dewatering is essential. TM 5-818-5 provides guidance in dewatering system design.

h. Final foundation preparation. Soft or organic spots in the levee foundation should be removed and replaced with compacted material. Except in special cases where foundation surfaces are adversely affected by remolding (soft foundations for instance), the foundation surface upon or against which fill is to be placed should be thoroughly broken up to a depth of at least 152.4 mm (6 in.) prior to the placement of the first lift of fill. This helps to ensure good bond between the foundation and fill and to eliminate a plane of weakness at the interface. The foundation surface should be kept drained and not scarified until just prior to fill placement in order to avoid saturation from rainfall.

7-3. Methods of Improving Stability

a. General. Levees located on foundation soils that cannot support the levee embankment because of inadequate shear strength require some type of foundation treatment if the levee is to be built. Foundation deposits that are prone to cause problems are broadly classified as follows: (1) very soft clays, (2) sensitive clays, (3) loose sands, (4) natural organic deposits, and (5) debris deposited by man. Very soft clays are susceptible to shear failure, failure by spreading, and excessive settlement. Sometimes soft clay deposits have a zone of stronger clay at the surface, caused by dessication, which if strong enough may eliminate the need for expensive treatment. Sensitive clays are brittle and even though possessing considerable strength in the undisturbed state, are subject to partial or complete loss of strength upon disturbance. Fortunately,

extremely sensitive clays are rare. Loose sands are also sensitive to disturbance and can liquefy and flow when subjected to shock or even shear strains caused by erosion at the toe of slopes. Most organic soils are very compressible and exhibit low shear strength. The physical characteristics and behavior of organic deposits such as peat can sometimes be predicted with some degree of accuracy. Highly fibrous organic soils with water contents of 500 percent or more generally consolidate and gain strength rapidly. The behavior of debris deposited by man, such as industrial and urban refuse, is so varied in character that its physical behavior is difficult, if not impossible, to predict. The following paragraphs discuss methods of dealing with foundations that are inadequate for construction of proposed levees.

b. Excavation and replacement. The most positive method of dealing with excessively compressible and/or weak foundation soils is to remove them and backfill the excavation with suitable compacted material. This procedure is feasible only where deposits of unsuitable material are not excessively deep. Excavation and replacement should be used wherever economically feasible.

c. Displacement by end dumping.

(1) Frequently low levees must be constructed across sloughs and stream channels whose bottoms consist of very soft fine-grained soils (often having high organic content). Although the depths of such deposits may not be large, the cost of removing them may not be justified, as a levee of adequate stability can be obtained by end-dumping fill from one side of the slough or channel, pushing the fill over onto the soft materials, and continually building up the fill until its weight displaces the foundation soils to the sides and front. By continuing this operation, the levee can finally be brought to grade. The fill should be advanced with a V-shaped leading edge so that the center of the fill is most advanced, thereby displacing the soft material to both sides. A wave of displaced foundation material will develop (usually visible) along the sides of the fill and should not be removed. A disadvantage of this method is that all soft material may not be displaced which could result in slides as the embankment is brought up and/or differential settlement after construction. Since this type of construction produces essentially uncompacted fill, the design of the levee section should take this into account.

(2) When this method of foundation treatment is being considered for a long reach of levee over unstable areas such as swamps, the possibility of facilitating displacement by blasting methods should be evaluated. Blasters' Handbook (1966) (Appendix A-2) presents general information on methods of blasting used to displace soft materials.

(3) The end-dumping method is also used to provide a working platform on soft foundation soils upon which construction equipment can operate to construct a low levee. In this case, only enough fill material is hauled in and dozed onto the foundation to build a working platform or pad upon which the levee proper can be built by conventional equipment and methods. Material forming the working platform should not be stockpiled on the platform or a shear failure may result. Only small dozers should be used to spread and work the material. Where the foundation is extremely weak, it may be necessary to use a small clamshell to spread the material by casting it over the area.

d. Stage construction.

(1) General. Stage construction refers to the building of an embankment in stages or intervals of time. This method is used where the strength of the foundation material is inadequate to support the entire weight of the embankment, if built continuously at a pace faster than the foundation material can drain. Using this method, the embankment is built to intermediate grades and allowed to rest for a time before-placing more fill. Such rest periods permit dissipation of pore water pressures which results in a gain in strength so that higher embankment loadings may be supported. Obviously this method is appropriate when pore water

pressure dissipation is reasonably rapid because of foundation stratification resulting in shorter drainage paths. This procedure works well for clay deposits interspersed with highly pervious silt or sand seams. However, such seams must have exits for the escaping water otherwise they themselves will become seats of high pore water pressure and low strengths (pressure relief wells can be installed on the landside to increase the efficiency of pervious layers in foundation clays). Initial estimates of the time required for the needed strength gain can be made from results of consolidation tests and study of boring data. Piezometers should be installed during construction to monitor the rate of pore water dissipation, and the resumption and rate of fill placement should be based on these observations, together with direct observations of fill and foundation behavior. Disadvantages of this method are the delays in construction operation, and uncertainty as to its scheduling and efficiency.

(2) Prefabricated vertical (wick) drains. If the expected rate of consolidation under stage construction is unacceptably slow, it may be increased by the use of prefabricated vertical (wick) drains. Such drains are geotextile wrapped plastic cores that provide open flowage areas in the compressible stratum. Their purpose is to reduce the length of drainage paths, thus speeding up primary consolidation. The wick drains are very thin and about 101.6 mm (4 in.) wide. They can be pushed into place through soft soils over 30.5 m (100 ft) deep. Before the drains are installed, a sand drainage blanket is placed on the foundation which serves not only to tie the drains together and provide an exit for escaping pore water, but as a working platform as well. This drainage blanket should not continue across the entire base width of the embankment, but should be interrupted beneath the center.

e. Densification of loose sands. The possibility of liquefaction of loose sand deposits in levee foundations may have to be considered. Since methods for densifying sands, such as vibroflotation, are costly, they are generally not considered except in locations of important structures in a levee system. Therefore, defensive design features in the levee section should be provided, such as wider levee crest, and flatter slopes.

Section III
Embankments

7-4. Embankment Construction Control

a. Construction control of levees may present somewhat different problems from that of dams because:

(1) Construction operations may be carried on concurrently along many miles of levee, whereas the majority of dams are less than about 0.8 km (0.5 mile) in length and only in a few cases are dams longer than 4.8 km (3 miles). This means that more time is needed to cover the operations on many levee jobs.

(2) While inspection staff and testing facilities are located at the damsite, levee inspection personnel generally operate out of an area office which may be a considerable distance from the levee project.

(3) There are frequently fiscal restraints which prevent assigning an optimum number of inspectors on levee work or even one full-time inspector on small projects. Under these conditions, the inspectors used must be well-trained to observe construction operations, minimizing the number of field density tests in favor of devoting more time to visual observations, simple measurements, and expedient techniques of classifying soils, evaluating the suitability of their water content, observing behavior of construction equipment on the fill, and indirectly assessing compacted field densities.

b. Although it has previously been stated that only limited foundation exploration and embankment design studies are generally needed in areas where levee heights are low and foundation conditions adequate

(i.e., no question of levee stability), the need for careful construction control by competent inspection exists as well as at those reaches where comprehensive investigations and analyses have been made. Some of the things that can happen during construction that can cause failure or distress of even low embankments on good foundations are given in Table 7-2.

Table 7-2
Embankment Construction Deficiencies

Deficiency	Possible Consequences
Organic material not stripped from foundation	Differential settlements; shear failure; internal erosion caused by through seepage
Highly organic or excessively wet or dry fill	Excessive settlements; inadequate strength
Placement of pervious layers extending completely through the embankment	Allows unimpeded through seepage which may lead to internal erosion and failure
Inadequate compaction of embankment (lifts too thick, haphazard coverage by compacting equipment, etc.)	Excessive settlements; inadequate strength; through seepage
Inadequate compaction of backfill around structures in embankment	Excessive settlements; inadequate strength; provides seepage path between structure and material which may lead to internal erosion and failure by piping

7-5. Embankment Zoning

As a general rule levee embankments are constructed as homogeneous sections because zoning is usually neither necessary nor practicable. However, where materials of varying permeabilities are encountered in borrow areas, the more impervious materials should be placed toward the riverside of the embankment and the more pervious material toward the landside slope. Where required to improve underseepage conditions, landside berms should be constructed of the most pervious material available and riverside berms of the more impervious materials. Where impervious materials are scarce, and the major portion of the embankment must be built of pervious material, a central impervious core can be specified or, as is more often done, the riverside slope of the embankment can be covered with a thick layer of impervious material. The latter is generally more economical than a central impervious core and, in most cases, is entirely adequate.

7-6. Protection of Riverside Slopes

a. The protection needed on a riverside slope to withstand the erosional forces of waves and stream currents will vary, depending on a number of factors:

(1) The length of time that floodwaters are expected to act against a levee. If this period is brief, with water levels against the levee continually changing, grass protection may be adequate, but better protection may be required if currents or waves act against the levee over a longer period.

(2) The relative susceptibility of the embankment materials to erosion. Fine-grained soils of low plasticity (or silts) are most erodible, while fat clays are the least erodible.

(3) The riverside slope may be shielded from severe wave attack and currents by timber stands and wide space between the riverbank and the levee.

(4) Structures riverside of the levee. Bridge abutments and piers, gate structures, ramps, and drainage outlets may constrict flow and cause turbulence with resultant scour.

(5) Turbulence and susceptibility to scour may result if levee alignment includes short-radius bends or if smooth transitions are not provided where levees meet high ground or structures.

(6) Requirements for slope protection are reduced when riverside levee slopes are very flat as may be the case for levees on soft foundations. Several types of slope protection have been used including grass cover, gravel, sand-asphalt paving, concrete paving, articulated concrete mat, and riprap, the choice depending upon the degree of protection needed and relative costs of the types providing adequate protection.

b. Performance data on existing slopes under expected conditions as discussed above are invaluable in providing guidance for the selection of the type of slope protection to be used.

c. Sometimes it may be concluded that low cost protection, such as grass cover, will be adequate in general for a levee reach, but with a realization that there may be limited areas where the need for greater protection may develop under infrequent circumstances. If the chances of serious damage to the levee in such areas are remote, good engineering practice would be to provide such increased protection only if and when actual problems develop. Of course, it must be possible to accomplish this expeditiously so that the situation will not get out of hand. In any event, high-class slope protection, such as riprap, articulated mat, or paving should be provided on riverside slopes at the following locations:

(1) Beneath bridges, since adequate turf cannot be generally established because of inadequate sunlight.

(2) Adjacent to structures passing through levee embankments.

d. Riprap is more commonly used than other types of revetments when greater protection than that afforded by grass cover is required because of the relative ease of handling, stockpiling, placement, and maintenance. Guidance on the design of riprap revetment to protect slopes against currents is presented in EM 1110-2-1601. Where slopes are composed of erodible granular soils or fine-grained soils of low plasticity, a bedding layer of sand and gravel or spalls, or plastic filter cloth should be provided beneath the riprap.

e. When suitable rock is not available within economical haul distances, soil cement may provide the most economical slope protection (see Appendix G).

Chapter 8
Special Features

Section I
Pipelines and Other Utility Lines Crossing Levees

8-1. General Considerations

a. Serious damage to levees can be caused by inadequately designed or constructed pipelines, utility conduits, or culverts (all hereafter referred to as "pipes") beneath or within levees. Each pipe crossing should be evaluated for its potential damage which would negatively impact the integrity of the flood protection system and could ultimately lead to catastrophic failure. During high water, seepage tends to concentrate along the outer surface of pipes resulting in piping of fill or foundation material. High water also results in uplift pressures that may cause buoyancy of some structures. Seepage may also occur because of leakage from the pipe. In the case of pipes crossing over levees, leakage can cause erosion in the slopes. In addition, loss of fill or foundation material into the pipe can occur if joints are open. The methods of pipe installation should be understood by the designer to anticipate problems that may occur. Some of the principal inadequacies that are to be avoided or corrected are as follows:

(1) Pipes having inadequate strength to withstand loads of overlying fill or stresses applied by traffic.

(2) Pipe joints unable to accommodate movements resulting from foundation or fill settlement.

(3) Unsuitable backfill materials or inadequately compacted backfill.

(4) High pressures from directional drilling that could result in hydro-fracturing the surrounding materials.

b. Some state and local laws prohibit pipes from passing through or under certain categories of levees. As a general rule, this should not be done anyway, particularly in the case of pressure lines. However, since each installation is unique, pipes in some instances may be allowed within the levee or foundation. Major factors to be considered in deciding if an existing pipe can remain in place under a new levee or must be rerouted over the levee, or if a new pipe should be laid through or over the levee are as follows:

(1) The height of the levee.

(2) The duration and frequency of high water stages against the levee.

(3) The susceptibility to piping and settlement of levee and foundation soils.

(4) The type of pipeline (low or high pressure line, or gravity drainage line).

(5) The structural adequacy of existing pipe and pipe joints, and the adequacy of the backfill compaction.

(6) The feasibility of providing closure in event of ruptured pressure lines, or in the event of failure of flap valves in gravity lines during high water.

(7) The ease and frequency of required maintenance.

(8) The cost of acceptable alternative systems.

(9) Possible consequences of piping or failure of the pipe.

(10) Previous experience with the owner in constructing and maintaining pipelines.

General criteria for pipes crossing levees are given in Table 8-1.

Table 8-1
Criteria for Pipelines Crossing Levees

Pipelines	Leaving Existing Pipeline in Foundations of Proposed Levees	New Pipeline Installation	
		Pipes Through Levees	Pipes Over Levees
Must be known to be in good condition	X		
Must have adequate strength to withstand levee loading	X	X	
Must have adequate cover as needed to prevent damage by vehicular traffic or heavy equipment			X
Must have adequate cover for frost protection			X
Must have sufficient flexibility in joints to adjust under expected settlement and stretching of pipe	X	X	X
Pressure lines must have provisions for rapid closure in event of leakage or rupture	X	X	X
Gravity discharge pipes must have provisions for emergency closure in event of inoperative flap valves on riverside end	X	X	
Must have pervious backfill under landside third of levee where:			
a. Foundation materials are susceptible to piping	X		
b. Levee materials are susceptible to piping		X	

8-2. General Considerations for Pipelines Crossing Through or Under Levees

a. General. As has been noted previously, it is preferable for all pipes to cross over a levee rather than penetrate the embankment or foundation materials. This is particularly true for pipes carrying gas or fluid under pressure. Before consideration is given to allowing a pressure pipe (and possibly other types of pipe) to extend through or beneath the levee, the pipe owner should provide an engineering study to support his request for such installation. The owner, regardless of the type of pipe, should show adequate capability to properly construct and/or maintain the pipe. Future maintenance of pipe by the owner must be carefully

evaluated. It may be necessary to form an agreement to the effect that should repairs to a pipe in the levee become necessary, the pipe will be abandoned, sealed, and relocated over the levee.

b. Existing pipes

(1) All existing pipelines must be located prior to initiation of embankment construction. As previously noted, inspection trenches may reveal abandoned pipes not on record. It is preferable that all abandoned pipes be removed during grubbing operations and the voids backfilled. Any existing pipe should meet or be made to meet the criteria given in Table 8-1. If this is not feasible and removal is not practical, they should be sealed, preferably by completely filling them with concrete. Sealed pipes must also meet the criteria given in Table 8-1 relating to prevention of seepage problems.

(2) In general, existing pressure pipes should be relocated over the proposed new levee. Rupture or leakage from such pipes beneath a levee produces extremely high gradients that can have devastating effects on the integrity of the foundation. Therefore, as indicated by the criteria in Table 8-1, it is imperative that pressure pipes be fitted with rapid closure valves or devices to prevent escaping gas or fluid from damaging the foundation.

(3) Although gravity drainage lines may be allowed or even required after the levee is completed, it is likely that existing pipes will not have sufficient strength to support the additional load induced by the embankment. Therefore, existing pipes must be carefully evaluated to determine their supporting capacity before allowing their use in conjunction with the new levee.

c. New Pipelines. Generally, the only new pipelines allowed to penetrate the foundation or embankment of the levee are gravity drainage lines. The number of gravity drainage structures should be kept to an absolute minimum. The number and size of drainage pipes can be reduced by using such techniques as ponding to reduce the required pipe capacity.

8-3. General Considerations for Pipelines Crossing Over Levees

In the past the term and concept of freeboard was used to account for hydraulic, geotechnical, construction, operation and maintenance uncertainties. Pipelines crossing over the levee were encouraged to be within the freeboard zone to reduce or eliminate many of the dangers that are inherent with pipelines crossing through the embankment or foundation. The term and concept of freeboard to account for these uncertainties is no longer used in the design of levee projects. Therefore, since freeboard no longer exists, pipes must cross over the completed levee cross section. Problems do exist, however, with pipelines crossing over the levee. These pipes must be properly designed and constructed to prevent (a) flotation if submerged, (b) scouring or erosion of the embankment slopes from leakage or currents, and (c) damage from debris carried by currents, etc. In some areas climatic conditions will require special design features. Guidance on design methods and construction practices will be given later in this chapter.

8-4. Pipe Selection

a. EM 1110-2-2902 contains a discussion of the advantages and disadvantages of various types of pipe (i.e., corrugated metal, concrete, cast iron, steel, clay, etc.). The selection of a type of pipe is largely dependent upon the substance it is to carry, its performance under the given loading, including expected deflections or settlement, and economy. Although economy must certainly be considered, the overriding factor must be safety, particularly where urban levees are concerned.

b. The earth load acting on a pipe should be determined as outlined in EM 1110-2-2902. Consideration must also be given to live loads imposed from equipment during construction and the loads from traffic and maintenance equipment after the levee is completed. The respective pipe manufacturers organizations have recommended procedures for accounting for such live loads. These recommended procedures should be followed unless the pipe or roadway owners have more stringent requirements.

c. Required strengths for standard commercially available pipe should be determined by the methods recommended by the respective pipe manufacturers organizations. Where cast-in-place pipes are used, design procedures outlined in EM 1110-2-2902 should be followed. Abrasion and corrosion of corrugated steel pipe should be accounted for in design using the method given in Federal Specification WW-P-405B(1) (Appendix A) for the desired design life. The design life of a pipe is the length of time it will be in service without requiring repairs. The term does not imply the pipe will fail at the end of that time. Normally, a design life of 50 years can be economically justified. Corrugated pipe should always be galvanized and protected by a bituminous or other acceptable coating as outlined in EM 1110-2-2902. Protective coatings may be considered in determining the design life of a pipe.

d. Leakage from or infiltration into any pipe crossing over, through, or beneath a levee must be prevented. Therefore, the pipe joints as well as the pipe itself must be watertight. For pipes located within or beneath the embankment, the expected settlement and outward movement of the soil mass must be considered. Where considerable settlement is likely to occur the pipe should be cambered (para 8-7). Generally, flexible corrugated metal pipes are preferable for gravity lines where considerable settlement is expected. Corrugated metal pipe sections should be joined by exterior coupling bands with a gasket to assure watertightness. Where a concrete pipe is required and considerable settlement is anticipated, a pressure-type joint with concrete alignment collars should be used. The collars must be designed either to resist or accommodate differential movement without losing watertight integrity. Where settlement is not significant, pressure-type joints capable of accommodating minor differential movement are sufficient. Design details for concrete collars are shown in EM 1110-2-2902. Cast iron and steel pipes should be fitted with flexible bolted joints. Steel pipe sections may be welded together to form a continuous conduit. All pressure pipes should be pressure tested at the maximum anticipated pressure before they are covered and put into use.

e. During the design, the potential for electrochemical or chemical reactions between the substratum materials or groundwater and construction materials should be determined. If it is determined that there will be a reaction, then the pipe and/or pipe couplings should be protected. The protective measures to be taken may include the use of cathodic protection, coating of the pipe, or use of a corrosion-resistant pipe material.

8-5. Antiseepage Devices

a. Antiseepage devices have been employed in the past to prevent piping or erosion along the outside wall of the pipe. The term "antiseepage devices" usually referred to metal diaphragms (seepage fins) or concrete collars that extended from the pipe into the backfill material. The diaphragms and collars were often referred to as "seepage rings." However, many piping failures have occurred in the past where seepage rings were used. Assessment of these failures indicated that the presence of seepage rings often results in poorly compacted backfill at its contact with the structure.

b. Where pipes or conduits are to be constructed through new or existing levees:

(1) Seepage rings or collars should not be provided for the purpose of increasing seepage resistance. Except as provided herein, such features should only be included as necessary for coupling of pipe sections or to accommodate differential movement on yielding foundations. When needed for these purposes, collars with a minimum projection from the pipe surface should be used.

(2) A 0.45-m (18-in.) annular thickness of drainage fill should be provided around the landside third of the pipe, regardless of the size and type of pipe to be used, where landside levee zoning does not provide for such drainage fill. For pipe installations within the levee foundation, the 0.45-m (18-in.) annular thickness of drainage fill shall also be provided, to include a landside outlet through a blind drain to ground surface at the levee toe, connection with pervious underseepage features, or through an annular drainage fill outlet to ground surface around a manhole structure. Figure 8-1 shows typical sections of drainage structures through levees. Figure 8-2 shows typical precast conduits through the levee.

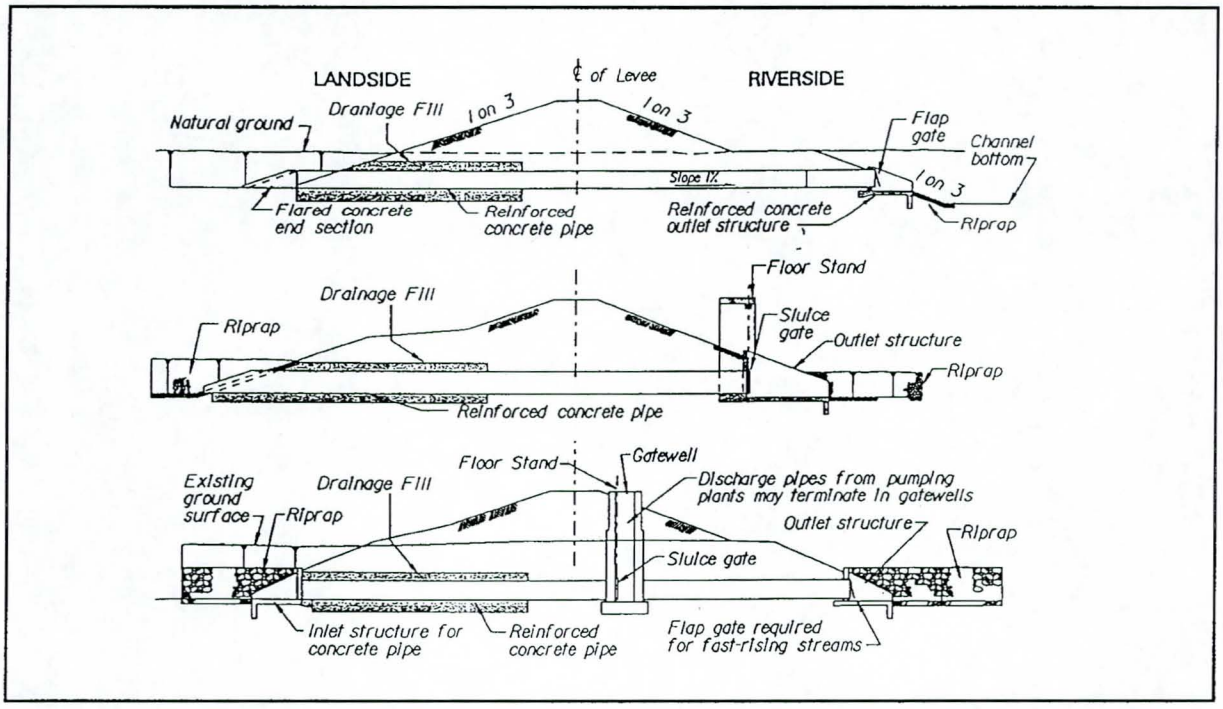


Figure 8-1. Typical sections, drainage structures through levees

8-6. Closure Devices

a. All pipes allowed to penetrate the embankment or foundation of a levee must be provided with devices to assure positive closure. Gravity lines should be provided with flap-type or slide-type service gates on the riverside of the levee. Automatic flap-type gates are usually used where the water is likely to rise to the "Gate Closing Stage" rather suddenly and where the water stage is likely to fluctuate within a few feet above and below the "Gate Closing Stage" for prolonged periods of time during flood season. Automatic gates are also required on slower rising streams or bodies of water where frequent visits from operating personnel are not practical.

b. Slide-type gates are usually preferred as service gates where the rate of rise of the water during major floods is slow, enough (minimum of 12-hr flood prediction time) to give ample time for safe operation. The principal advantages of the slide gate in comparison with automatic flap gates are greater reliability of operation and the ease with which emergency closure can be made in event obstructions prevent closure of the gate. Usually emergency closure can be made by filling the manhole with sandbags. The obvious

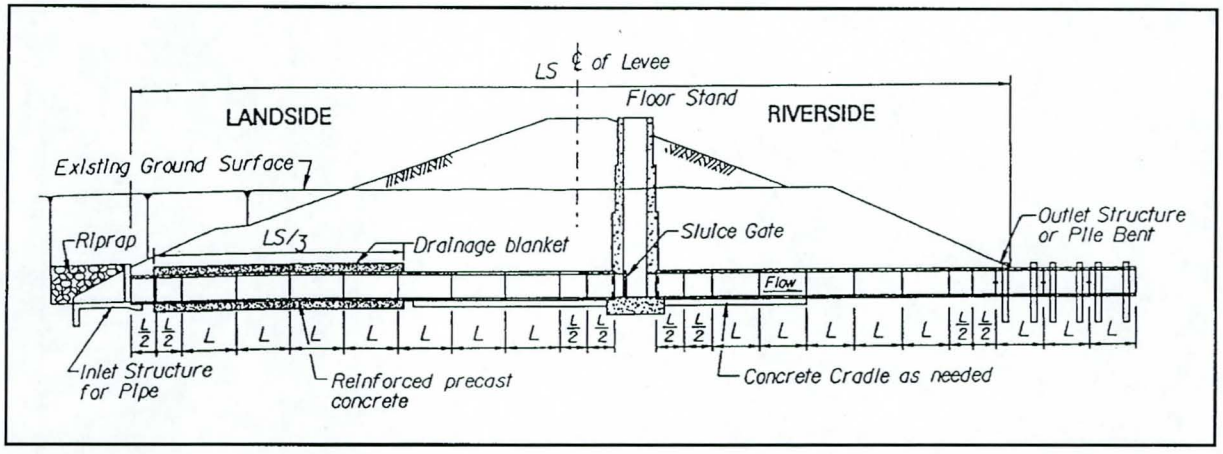


Figure 8-2. Typical precast conduit (levees)

disadvantage of slide type gates is that personnel must be on hand for their operation. Also their initial cost is generally greater than that for a flap-type gate.

c. A slide-type gate with a flap-type gate attachment is often used and affords the advantages of automatic flap gate operation with the added safety of the slide-type gate. Such installations usually eliminate the need for a supplemental emergency gate as described below.

d. Experience has shown that service gates occasionally fail to close completely during critical flood periods because of clogging by debris, mechanical malfunctions, or other causes. This, of course, can cause flooding of, the protected areas. Supplemental emergency gates are intended to minimize these risks insofar as necessary and economically practical. For an emergency gate to be effective it must be located so that its controls are accessible during flood stage. Provisions required for emergency protection of other areas should be consistent with the risks and cost involved.

e. Pressure pipes should be fitted with valves at various stations that can be closed rapidly to prevent gas or fluid from escaping within or beneath a levee should the pipe rupture within these areas. Provisions for closure of pressure pipes on the water side must also be provided to prevent backflow of floodwater into the protected area should the pipe rupture. These requirements should generally be followed in other areas, but may be relaxed to be consistent with the risks and costs involved.

8-7. Camber

The alignment of a gravity structure must be such as to provide for a continuous slope toward the outlet. Settlement of the embankment and foundation can significantly alter the initial grade line of a pipe. Therefore, the expected settlement of the levee must be considered in establishing the initial grade line. If the settlement will result in an upward gradient in the direction of flow or not allow the desired gradient to be maintained, the pipe should be cambered. The amount of camber required can usually be taken as the mirror image of the settlement curve along a line established by the final required grade. The camber should then be laid out, preferably as a vertical curve, on a grade such that all parts of the pipe will slope toward the outlet when installed. If the gradient of the pipe is limited and the camber will initially result in a slope away from the outlet, the portion of the pipe from the inlet up to the point of greatest load may be installed level. The remaining portion of the pipe is then installed on a vertical curve tangent to the first portion of the pipe.

Regardless of the type of pipe selected, movements at the joints must be considered as discussed in paragraph 8-4d.

8-8. Installation Requirements

a. General. The installation of pipes or other structures within the levee or foundation probably requires the greatest care and the closest supervision and inspection of any aspect of levee construction. Most failures of levee systems have initiated at the soil-structure interface and therefore every effort must be made to ensure that these areas are not susceptible to piping. Of overriding importance is good compaction of the backfill material along the structure. Pipes installed by open trench excavation should be installed in the dry and a dewatering system should be used where necessary. Pipes installed by directional drilling, microtunneling, or other trenchless methods require special consideration.

b. Pipes crossing through or beneath levees

(1) The preferred method of installing pipes within the embankment or foundation of a levee has historically been by the open cut method. Preferably, new levees should be brought to a grade about 610.8 mm (2 ft) above the crown of the pipe. This allows the soil to be preconsolidated before excavating the trench. The trench should be excavated to a depth of about 610.8 mm (2 ft) below the bottom of the pipe and at least 1.2 m (4 ft) wider than the pipe. The excavated material should be selectively stockpiled so that it can be replaced in a manner that will not alter the embankment zoning if there is some or will result in the more impervious soils on the riverside of the levee.

(2) After the trench has been excavated, it should be backfilled to the pipe invert elevation. In impervious zones, the backfill material should be compacted with mechanical compactors to 95 percent standard density at about optimum water content.

(3) First-class bedding should be used for concrete pipe and other rigid pipe, as shown in EM 1110-2-2902 except no granular bedding should be used in impervious zones. For flexible pipe, the trench bottom should be flat to permit thorough tamping of backfill under the haunches of the pipe. Backfill should be compacted to 95 percent standard density at about optimum water content. The backfill should be brought up evenly on both sides of the pipe to avoid unequal side loads that could fail or move the pipe. Special care must be taken in the vicinity of any protrusions such as joint collars to ensure proper compaction. Where granular filter material is required, it should be compacted to a minimum of 80 percent relative density. In areas where backfill compaction is difficult to achieve, flowable, low strength concrete fill has been used to encapsulate pipes in narrow trenches.

(4) In existing levees, the excavation slopes should be stable, meet OSHA criteria, but in no case be steeper than 1V on 1H. The excavated material should be selectively stockpiled as was described for new levees. The pipe is installed as described in the previous paragraphs. Impervious material within 0.61 m (2 ft) of the pipe walls should be compacted to 95 percent standard density at optimum water content, with the remainder of the backfill placed at the density and water content of the existing embankment.

(5) Installation of pipes in existing levees by directional drilling, microtunneling, tunneling or jacking may be considered. It is recognized, that in some instances, installation by the open cut method is not feasible or cannot be economically justified. Where trenchless methods are allowed, special considerations are required.

(6) Pipes under levees.

(a) General. Pipes crossing beneath levees also require special considerations. Such crossings should be designed by qualified geotechnical engineers. Pipes constructed with open excavation methods should proceed in accordance with the requirements stated in the above paragraph, Pipes Crossing Through or Beneath Levees. If directional drilling or other trenchless methods are used, seepage conditions may be aggravated by the collapse of levee foundation material into the annular void between the bore and pipe. Penetration through the top stratum of fine-grained materials may concentrate seepage at those locations. Pipes constructed with trenchless methods should proceed only after a comprehensive evaluation of the following: comprehensive understanding of the subsurface soil and groundwater conditions to a minimum depth of 6.1 m (20 ft) below the lowest pipe elevation, locations of the pipe penetration entry and exit, construction procedure, allowable uplift pressures, on-site quality control and quality assurance monitoring during construction operation, grouting of the pipe annulus, backfilling of any excavated areas, and repair and reinstatement of the construction-staging areas. Guidance for construction of pipelines beneath levees using directional drilling is provided in Appendix A of WES CPAR-GL-98-1 (Staheli, et al. 1998). Guidance for construction of pipelines using microtunneling methods is provided in WES CPAR-GL-95-2 (Bennett, et al. 1995).

(b) Pipes installed by directional drilling. The pipe entry or exit location, when located on the protected (land) side, should be set back sufficiently from the land side levee toe to ensure that the pipe penetrates some depth of a pervious sand stratum but is no less than 91.5 m (300 ft) from the centerline of the levee crest. The pipe entry or exit location, when located on the unprotected (river) side, should be located at least 6.1 m (20 ft) riverward of the levee stability control line. This is the distance between the river side levee toe and an eroding bank line which will maintain the minimum design criteria for slope stability.

If directional drilling is to be used, the depth of the pipe under the levee should be at a level to maintain an adequate factor of safety against uplift from the pressurized drilling fluid during the drilling operation. A positive means of maintaining an open vent to the surface should be required whether through bored holes or downhole means while installing the drill pipe.

The drilling fluid should consist of a noncolloidal lubricating admixture to ensure suspension and removal of drilling cuttings. The pilot hole should be advanced at a rate to maintain a continuous return flow. The annular space should be sufficient to ensure that no blockage occurs with the drilling cuttings. The prereamer boring diameter should be of sufficient size to ensure that the production pipe can be advanced without delay and undue stress to the surrounding soils. The prereamer boring operation should be continuous for the down-slope and up-slope cutting segments. Excessive drilling fluid pressures can hydraulically fracture the levee foundation and levee embankment and should be avoided.

Where economically feasible, the pipeline should be bored through rock where the pipeline crosses the levee centerline.

The maximum allowable mud pressure acting against the borehole wall should be evaluated using the Delft equation presented in the Appendix A of WES CPAR-GL-98-1 (Staehli, et al., 1998). During construction, the actual mud pressure existing in the borehole must be measured by a pressure measuring device located on the outside of the drill string no more than 5 ft from the drill bit. The drilling operator should be required to monitor these pressures and adjust the drilling mud pressure so as not to exceed the maximum pressure determined by Delft equation.

Where the casing pipe is carrying multiple fibre optic cables and each cable is installed within its own HDPE inner duct, the detail shown in Figure 8-3a should be used to prevent preferred seepage path (both external and internal). The casing pipe must end in the encasements.

The directional drilling contract should be required to show proof that all of his pressure sensors and readout devices have been calibrated by a national standard within the last 6 months.

A full time inspector, not on directional drilling contractor's payroll, should be required to observe the construction.

The drilling fluid should be processed through an active drilling mud conditioning unit to remove the cuttings from the drill fluid and maintain its viscosity.

c. Pipes crossing over levees. Pipe crossings on the surface of the levee should be designed to counteract uplift of the empty pipe at the design high water stage. This may be accomplished by soil cover, anchors, headwalls, etc. All pipes on the riverside of the levee should have a minimum of 305 mm (1 ft) of soil cover for protection from debris during high water. It is desirable for pipe on the landward side to also be covered with soil. Pipes crossing beneath the levee crown should be provided with sufficient cover to withstand vehicular traffic as outlined in paragraph 8-4b. Depth of cover should also be at least the depth of local frost protection. Where mounding of soil over the pipe is required, the slope should be gentle to allow mowing equipment or other maintenance equipment to operate safely on the slopes. The approach ramps on the levee crown should not exceed 1V on 10H in order to allow traffic to move safely on the crown. The trenching details for pipelines cross-up and over-levees are shown in Figure 8-3b and Figure 8-3c.

Section II
Access Roads and Ramps

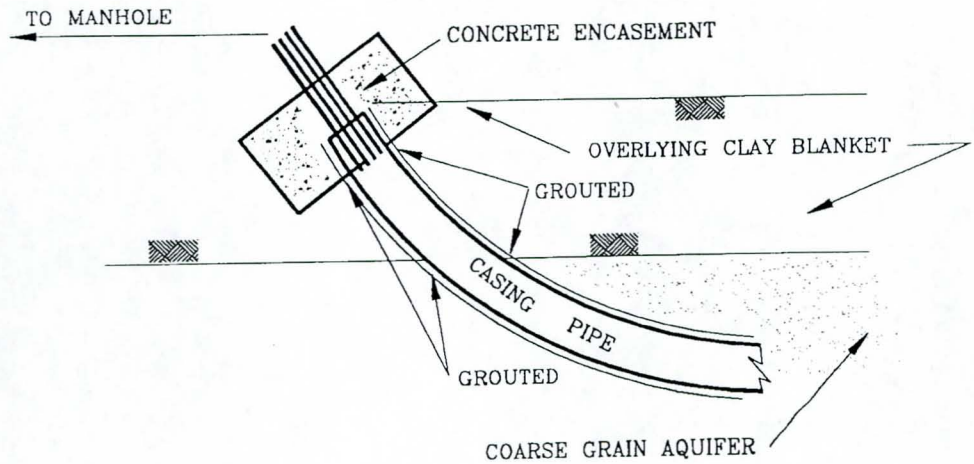
8-9. Access Roads

a. Access road to levee. Access roads should be provided to levees at reasonably close intervals in cooperation with state and local authorities. These roads should be all-weather roads that will allow access for the purpose of inspection, maintenance, and flood-fighting operations.

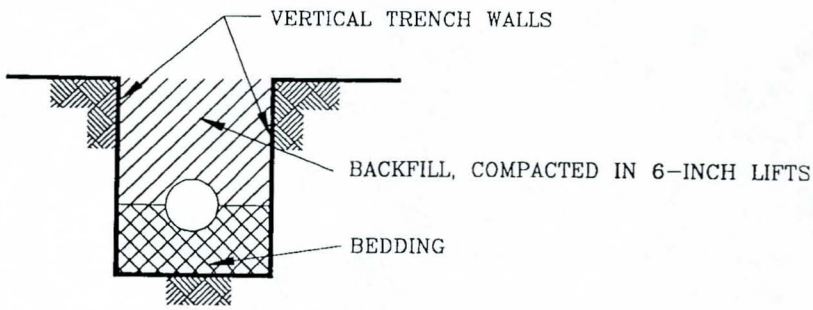
b. Access road on levee. Access roads, sometimes referred to as patrol roads, should be provided also on top of the levees for the general purpose of inspection, maintenance, and flood-fighting operations. This type of road should be surfaced with a suitable gravel or crushed stone base course that will permit vehicle access during wet weather without causing detrimental effects to the levee or presenting safety hazards to the levee inspection and maintenance personnel. The width of the road surfacing will depend upon the crown width of the levee, where roadway additions to the crown are not being used, and upon the function of the roadway in accommodating either one- or two-way traffic. On levees where county or state highways will occupy the crown, the type of surfacing and surfacing width should be in accordance with applicable county or state standards. The decision as to whether the access road is to be opened to public use is to be made by the local levee agency which owns and maintains the levee.

(1) Turnouts. Turnouts should be used to provide a means for the passing of two motor vehicles on a one-lane access road on the levee. Turnouts should be provided at intervals of approximately 762 m (2500 ft), provided there are no ramps within the reach. The exact locations of the turnouts will be dependent upon various factors such as sight distance, property lines, levee alignment, and desires of local interests. An example turnout for a levee with a 3.65 m (12-ft) levee crown is shown in Figure 8-4.

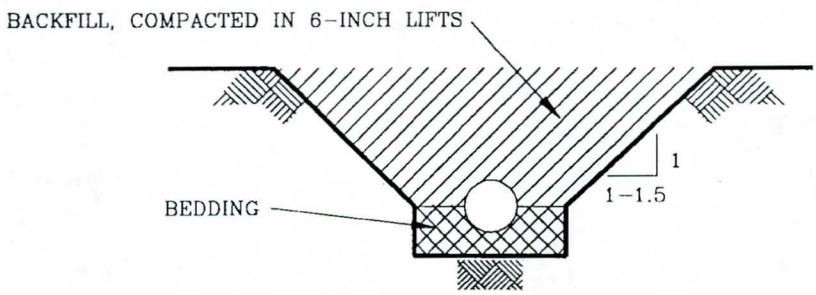
(2) Turnarounds. Turnarounds should be provided to allow vehicles to reverse their direction on all levees where the levee deadends, and no ramp exists in the vicinity of the deadend. An example turnaround for a levee with a 3.65-m (12-ft) crown is shown in Figure 8-5.



a. Detail of end casing (for pipe lines beneath the levees)



b. On riverside, within 50 ft of levee (for pipe lines cross-up and over-levees)



c. On landside (for pipelines cross-up and over-levees)

Figure 8-3. Details of pipeline levee crossing

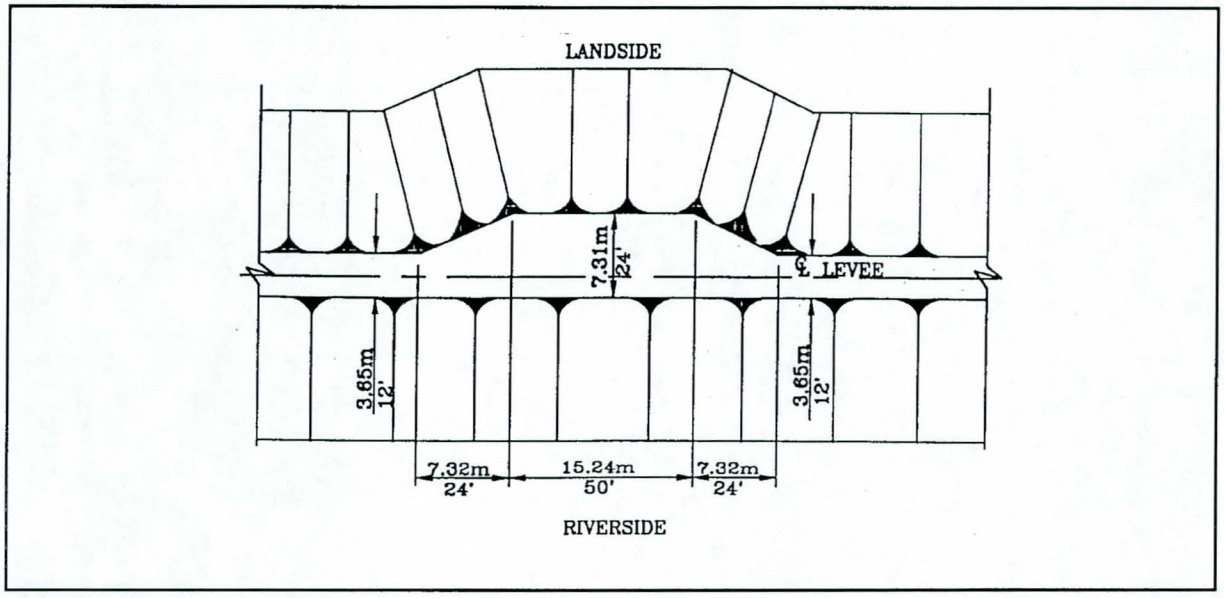


Figure 8-4. Example of levee turnout

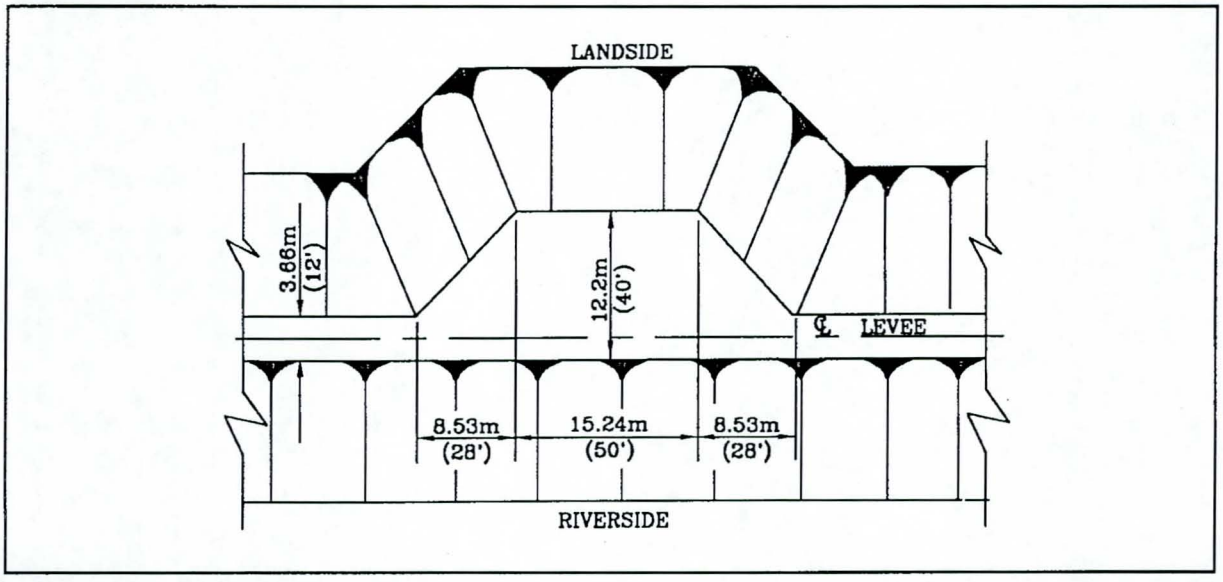


Figure 8-5. Example of levee turnaround

8-10. Ramps

a. Ramps should be provided at sufficient locations to permit vehicular traffic to access onto and from the levee. Ramps may be located on both the landside and the riverside of the levee. Ramps on the landside of the levee are provided to connect access roads leading to a levee with access roads on top of a levee and

at other convenient locations to serve landowners who have property bordering the levee. Ramps are also provided on some occasions on the riverside of the levee to connect the access road on top of the levee with existing levee traverses where necessary. The actual locations of the ramps should have the approval of the local levee agency which owns and maintains the levee. When used on the riverside of the levee, they should be oriented to minimize turbulence during high water.

b. Ramps are classified as public or private in accordance with their function. Public ramps are designed to satisfy the requirements of the levee owner: state, county, township, or road district. Private ramps are usually designed with less stringent requirements and maximum economy in mind. Side-approach ramps should be used instead of right angle road ramps because of significant savings in embankment. The width of the ramp will depend upon the intended function. Some widening of the crown of the levee at its juncture with the ramp may be required to provide adequate turning radius. The grade of the ramp should be no steeper than 10 percent. Side slopes on the ramp should not be less than 1V on 3H to allow grass-cutting equipment to operate. The ramp should be surfaced with a suitable gravel or crushed stone. Consideration should be given to extending the gravel or crushed stone surfacing to the levee embankment to minimize erosion in the gutter. In general, private ramps should not be constructed unless they are essential and there is assurance that the ramps will be used. Unused ramps lead to maintenance neglect.

c. Both public and private ramps should be constructed only by adding material to the levee crown and slopes. The levee section should never be reduced to accommodate a ramp.

Section III
Levee Enlargements

8-11. General

The term levee enlargement pertains to that addition to an existing levee which raises the grade. A higher levee grade may be required for several reasons after a levee has been constructed. Additional statistical information gathered from recent floodings or recent hurricanes may establish a higher project flood elevation on a river system or a higher elevation for protection from incoming tidal waves produced by hurricane forces in low-lying coastal areas. The most economical and practical plan that will provide additional protection is normally a levee enlargement. Levee enlargements are constructed either by adding additional earth fill or by constructing a flood-wall, "I"-type or "inverted T"-type, on the crown.

8-12. Earth-Levee Enlargement

a. The earth-levee enlargement is normally preferred when possible, since it is usually more economical. This type of enlargement is used on both agricultural and urban levees where borrow sites exist nearby and sufficient right-of-way is available to accommodate a wider levee section.

b. An earth-levee enlargement is accomplished by one of three different methods: riverside, straddle, or landside enlargement. A riverside enlargement is accomplished by increasing the levee section generally at the crown and on the riverside of the levee as shown in Figure 8-6a. A straddle enlargement is accomplished by increasing the levee section on the riverside, at the crown, and on the landside of the levee-as shown in Figure 8-6b. A landside enlargement is accomplished by increasing the levee section, generally at the crown and on the landside of the levee as shown in Figure 8-6c. There are advantages and disadvantages to each enlargement method that will have to be looked at for each project. The riverside enlargement would be more costly if the riverside slope has riprap protection and it could also be an encroachment for narrow floodways that would impact top of levee designs. Landside enlargements would require additional right-of-way and larger fill quantities for levees with flatter landside slopes. The strattle

enlargement would require the whole levee system to be stripped with work being done on both sides of the levee.

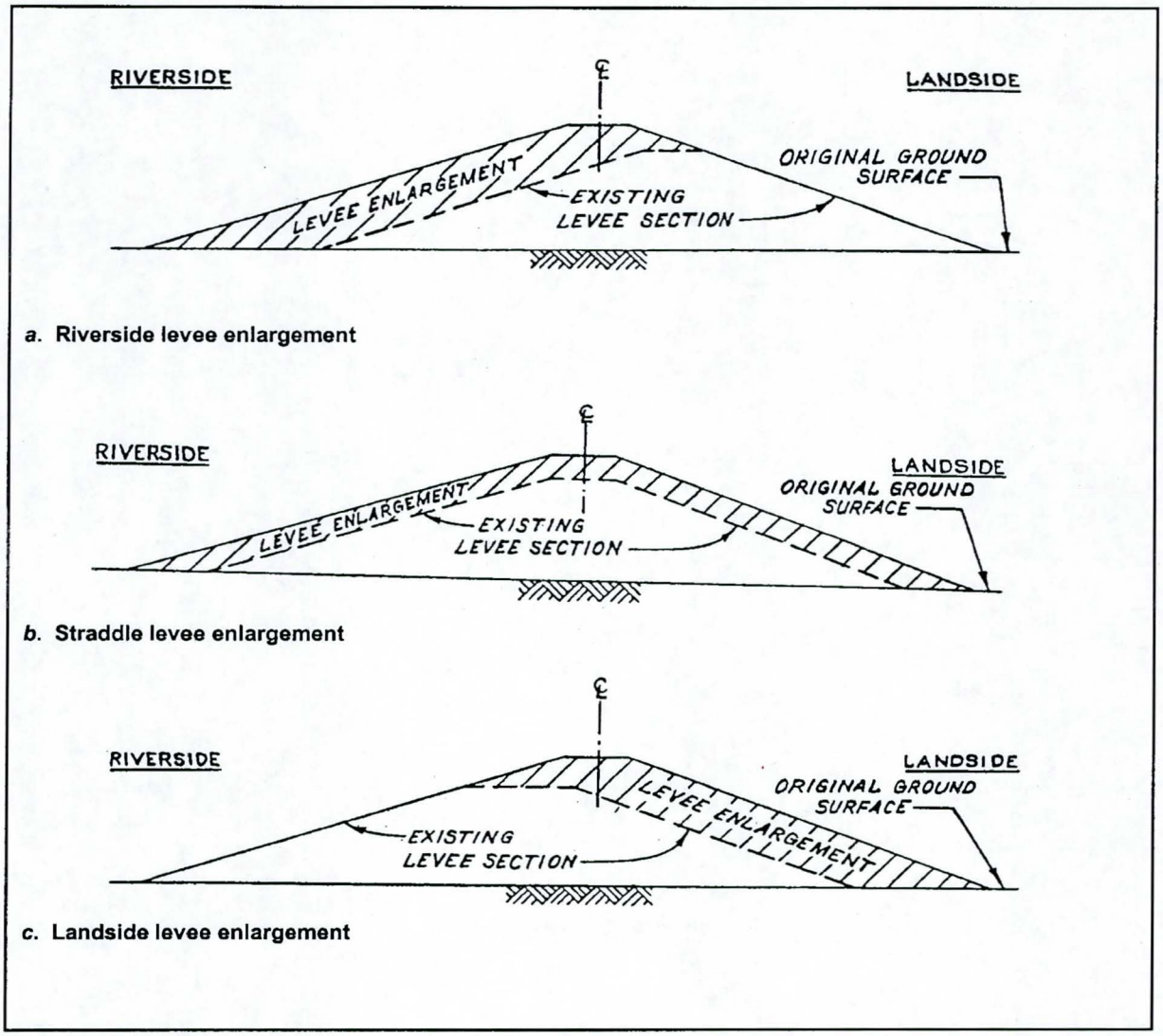


Figure 8-6. Enlargements

c. The modified levee section should be checked for through seepage and underseepage as discussed in Chapter 5 and for foundation and embankment stability as discussed in Chapter 6. Sufficient soil borings should be taken to determine the in situ soil properties of the existing levee embankment for design purposes.

d. An earth-levee enlargement should be made integral with the existing levee. Every effort should be made such that the enlargement has at least the same degree of compaction as the existing levee on which it is constructed. Preparation of the interface along the existing levee surface and upon the foundation should be made to ensure good bond between the enlargement and the surfaces on which it rests. The foundation surface should be cleared, grubbed, and stripped as described in Chapter 6. The existing levee surface upon which the levee enlargement is placed should also be stripped of all low-growing vegetation and organic

topsoil. The topsoil that is removed should be stockpiled for reuse as topsoil for the enlargement. Prior to constructing the enlargement, the stripped surfaces of the foundation and existing levee should be scarified before the first lifts of the enlargements are placed.

8-13. Floodwall-Levee Enlargement

a. A floodwall-levee enlargement is used, when additional right-of way is not available or is too expensive or if the foundation conditions will not permit an increase in the levee section. Economic justification of floodwall-levee enlargement cannot usually be attained except in urban areas. Two common types of floodwalls that are used to raise levee grades are the I wall and the inverted T wall.[1]

b. The I floodwall is a vertical wall partially embedded in the levee crown. The stability of such walls depends upon the development of passive resistance from the soil. For stability reasons, I floodwalls rarely exceed 2.13 m (7 ft) above the ground surface. One common method of constructing an I floodwall is by combining sheet pile with a concrete cap as shown in Figure 8-7. The lower part of the wall consists of a row of steel sheet pile that is driven into the levee embankment, and the upper part is a reinforced concrete section capping the steel piling.

c. An inverted T floodwall is a reinforced concrete wall whose members act as wide cantilever beams in resisting hydrostatic pressures acting against the wall. A typical wall of this type is shown in Figure 8-8. The inverted T floodwall is used to make floodwall levee enlargements when walls higher than 2.13 m (7 ft) are required.

d. The floodwall should possess adequate stability to resist all forces which may act upon it. An I floodwall is considered stable if sufficient passive earth resistance can be developed for a given penetration of the wall into the levee to yield an ample factor of safety against overturning. The depth of penetration of the I wall should be such that adequate seepage control is provided. Normally the penetration depth of the I wall required for stability is sufficient to satisfy the seepage requirements. For the inverted T floodwall, the wall should have overall dimensions to satisfy the stability criteria and seepage control as presented in EM 1110-2-2502.

e. The existing levee section should be checked for through seepage and underseepage as discussed in Chapter 5 and for embankment and foundation stability as discussed in Chapter 6 under the additional hydrostatic forces expected. If unsafe seepage forces or inadequate embankment stability result from the higher heads, seepage control methods as described in Chapter 5 and methods of improving embankment stability as described in Chapter 6 may be used. However, some of these methods of controlling seepage and improving embankment stability may require additional right-of-way for construction which could eliminate the economic advantages of the floodwall in comparison with an earth levee enlargement. As in earth levee enlargements, a sufficient number of soil borings should be taken to determine the in situ soil properties of the existing levee embankment for design purposes.

[1] Structural design of crest walls is given in ETL 1110-2-341.

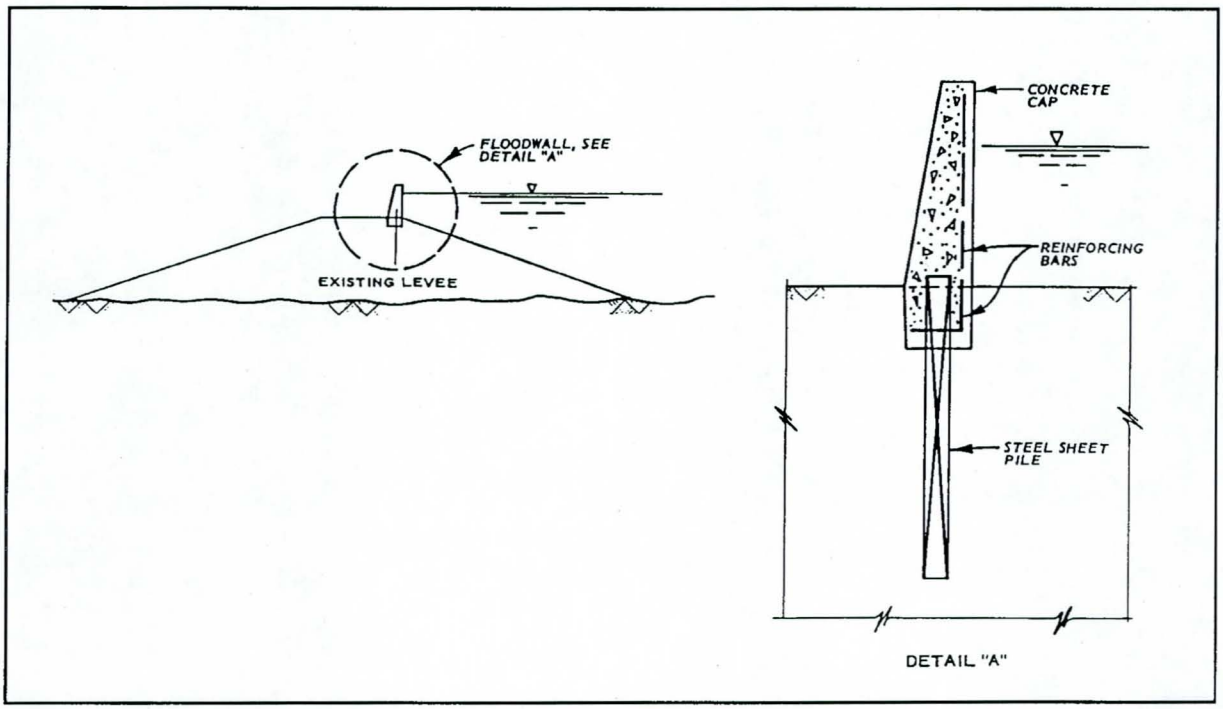


Figure 8-7. I-type floodwall-levee enlargement

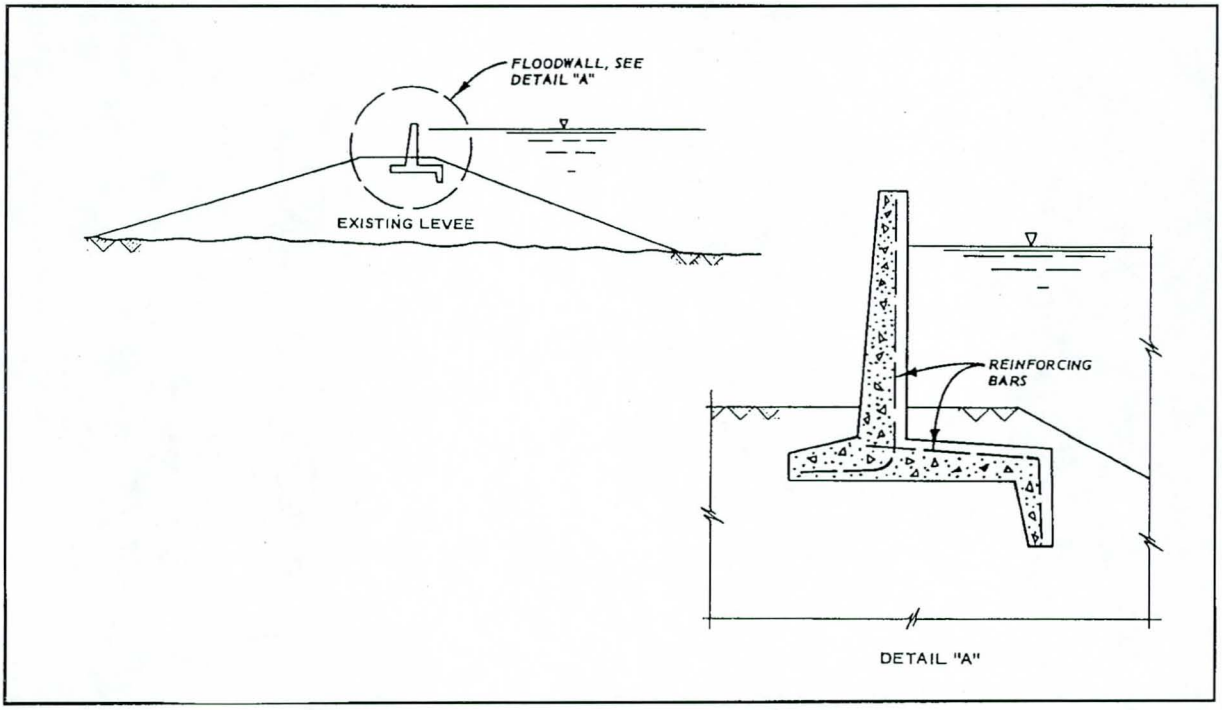


Figure 8-8. Inverted T-type floodwall-levee enlargement

Section IV
Junction with Concrete Closure Structures

8-14. General

In some areas, a flood protection system may be composed of levees, floodwalls, and drainage control structures (gated structures, pumping plants, etc.). In such a system, a closure must be made between the levee and the concrete structure to complete the flood protection. One closure situation occurs when the levee ties into a concrete floodwall or a cutoff wall. In this closure situation the wall itself is usually embedded in the levee embankment. In EM 1110-2-2502 a method of making a junction between a concrete floodwall and levee is discussed and illustrated. Another closure situation occurs when the levee ties into a drainage control structure by abutting directly against the structure as shown in Figure 8-9. In this situation the abutting end walls of the concrete structure should be battered 10V on 1H to ensure a firm contact with the fill.

8-15. Design Considerations

When joining a levee embankment with a concrete structure, items that should be considered in the design of the junction are differential settlement, compaction, and embankment slope protection.

a. Differential settlement. Differential settlement caused by unequal consolidation of the foundation soil at the junction between a relatively heavy levee embankment and a relatively light concrete closure structure can be serious if foundation conditions are poor and the juncture is improperly designed. Preloading has been used successfully to minimize differential settlements at these locations. In EM 1110-2-2502 a transitioning procedure for a junction between a levee embankment and a floodwall is presented that minimizes the effect of differential settlement.

b. Compaction. Thorough compaction of the levee embankment at the junction of the concrete structure and levee is essential. Good compaction decreases the permeability of the embankment material and ensures a firm contact with the structure. Heavy compaction equipment such as pneumatic or sheepsfoot rollers should be used where possible. In confined areas such as those immediately adjacent to concrete walls, compaction should be by hand tampers in thin loose lifts as described in EM 1110-2-1911.

c. Seepage. Seepage needs to be analyzed to determine the embedment length of the structure-levee junction. Zoning of the embankment materials needs to be maintained through the junction unless analysis indicates different zoning is required.

d. Slope protection. Slope protection should be considered for the levee embankment at all junctions of levees with concrete closure structures. Turbulence may result at the junction due to changes in the geometry between the levee and the structure. This turbulence will cause scouring of the levee embankment if slope protection is not provided. Slope protection for areas where scouring is anticipated is discussed in paragraph 7-6.

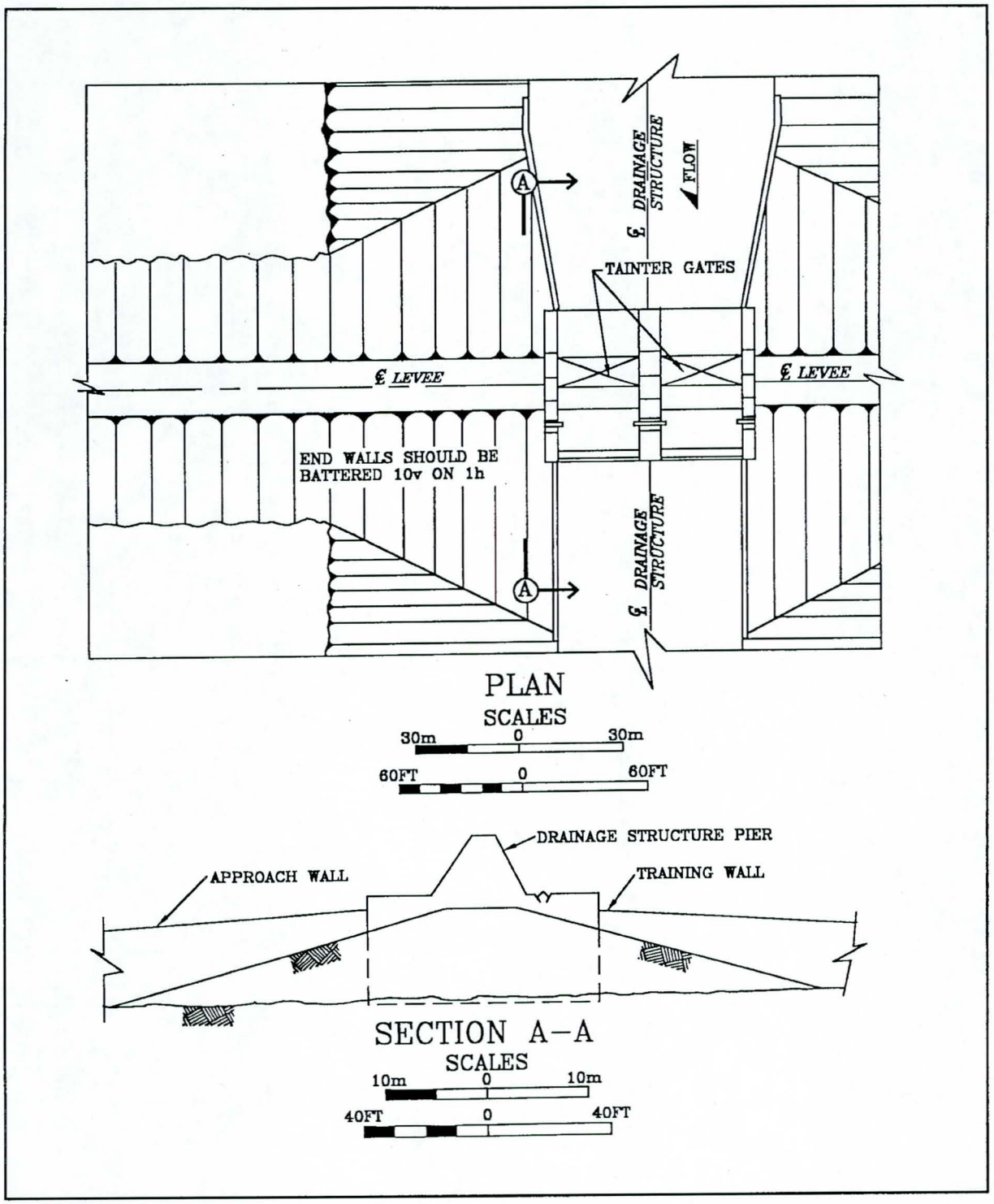


Figure 8-9. Junction of levee and drainage structure

8-16. Construction of Ditches Landside of Levee

Sometimes requests are made to locate irrigation and/or drainage ditches in close proximity to the landside levee toe. Such ditches may lead to serious seepage and/or slope stability problems. The location and depth of proposed ditches should be established by seepage and stability analyses. This requires information on foundation soil conditions, river stages and geometry of the proposed ditch.

Drainage ditches should be located such that the exit gradient in the bottom of the ditch does not exceed 0.5 at the landside levee toe and does not exceed 0.8 at a distance 45.72 m (150 ft) landward of the landside levee toe and beyond. Between the landside levee toe and 45.72 m (150 ft) landward of the landside levee toe, the maximum allowable exit gradient in the bottom of the ditch should increase linearly from 0.5 to 0.8. The exit gradient should be computed assuming the water level in the ditch is at the bottom of the ditch.

8-17. Levee Vegetation Management

To protect or enhance esthetic values and natural resources, vegetation on a levee and its surrounding areas (trees, bushes and grasses) is an important part of design considerations. Vegetation can be incorporated in the project as long as it will not diminish the integrity and the functionality of the embankment system or impede ongoing operations, maintenance and floodfighting capability. A multidiscipline team including structural and geotechnical engineers, biologists and planners should evaluate the vegetation design or proposal. Coordination with local governments, states and Native American tribes may be needed during the design process. EM 1110-2-301 and ER 500-1-1 are two documents covering the vegetation policy applicable to both federal levees and non-federal levees under the PL-84-99 program.

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	2.5	182.9	
												5.7[c]			
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	
														365.8	
	>3.05											2.4[c]	1.0		
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	0.05	1219.2[e]	
												0.4[c,f]		or L_1	

[a] Values of x_1 computed from observed values of x_t and adjusted to a condition where $L_1 = \infty$.
[b] Does not include Hole-in-the-Wall where values of S and x_1 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)

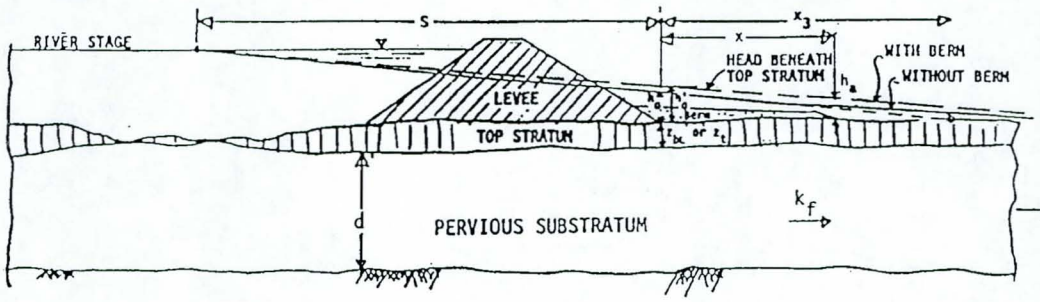
Blanket in Riverside Borrow Pit		Numer of Piezometer	S, ft			x_1, ft[a]			k_{br} x 10^{-4} cm/sec			Suggested Design Values		
Soil Type	Thickness in ft	Lines from Which Data Were Obtained	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	k_{br}	x_1 (ft)	
Sand --	3	1080		800	960	480	200	370	--		--	--	--	250
Silty sand[b]	<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 and adjusted to a condition where $L_1 = \infty$.
[b] Does not include Hole-in-the-Wall where values of S and x_1 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

$C = \sqrt{\frac{k_{bl}}{k_f z_{bl} d}}$

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 = $\frac{Hx_3}{s + x_3}$

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, $k_t = 0$

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

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

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

USE $F \geq 2.8$

Semipervious Berm, $k_t = k_{bl}$

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

where:

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

$r = i_o/i_1$

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

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

Sand Berm

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

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

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

Pervious Berm with Collector

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

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

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

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

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.

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

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.

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

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

$k_f = 1{,}000 \times 10^{-4}$ cm/sec

$d = 30.5$ m

$k_{bl} = 3 \times 10^{-4}$ cm/sec

$s = 304.8$ m

$z_{bl} = z_f = 1.83$ m

$i_o = 0.30$

$i_1 = 0.80$

$\gamma' = 840.5$ kg/m^3

$x_3 = 137.2$ m

$\gamma' = 840.5$ kg/m^3 for impervious berms

$\gamma' = 920.6$ kg/m^3 for sand berm or pervious berm with collector, $F = 1.6$

$F = 1.6$ for impervious berm

$L_3 = \infty$

Type Berm	Required Berm Width X, m	Required Berm Thickness[a] t, m	Required Berm h_o'[b] m	Suggested Design Dimensions Thickness at Berm Crown m	Suggested Design Dimensions Berm Width X, m	Suggested Design Dimensions Berm Slope	Approximate Thickness Levee Toe m	Suggested Construction Dimensions Thickness[c] at Berm Crown m	Suggested Construction Dimensions Berm Width X, m	Suggested Construction Dimensions Thickness[c] at Levee Toe m	Approximate[d] Material Required 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	$\tilde{a}'$	=	52.5 pcf for impervious berms
k_f	=	$1{,}000 \times 10^{-4}$ cm/sec	i_o	=	0.30	$\tilde{a}'$	=	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	$\tilde{a}'$	=	52.5 pcf	L_3	=	∞
s	=	1,000 ft	x_3	=	450 ft			

	Required Berm			Suggested Design Dimensions				Suggested Construction Dimensions			Approximate[d] Material
Type Berm	Width X, ft	Thickness[a] t, ft	h_o' ft	Thickness at Berm Crown ft	Berm Width X, ft	Berm[b] Slope	Approximate Thickness Levee Toe ft	Thickness[c] at Berm Crown ft	Berm Width X, ft	Thickness at Levee Toe ft	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 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

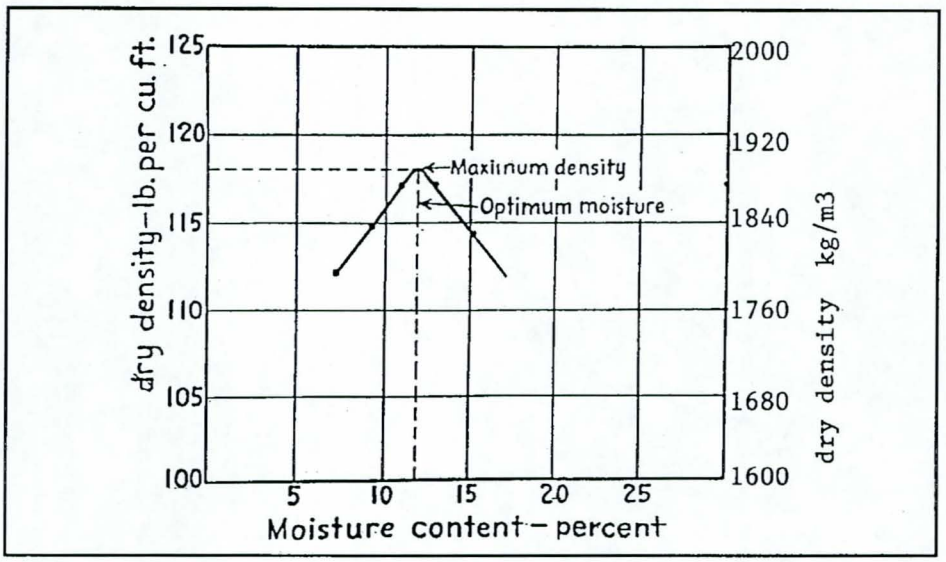


Figure G-1. Typical moisture-density curve

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

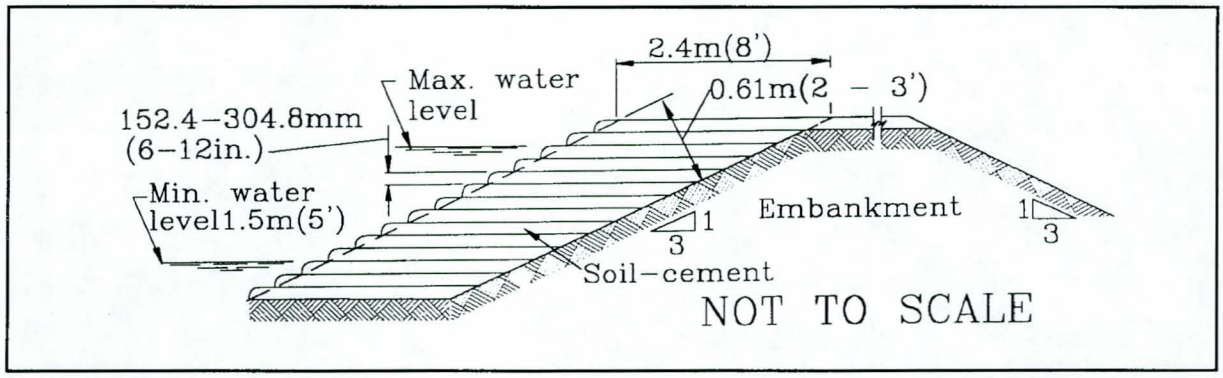


Figure G-2. Stair-step method of slope protection

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.

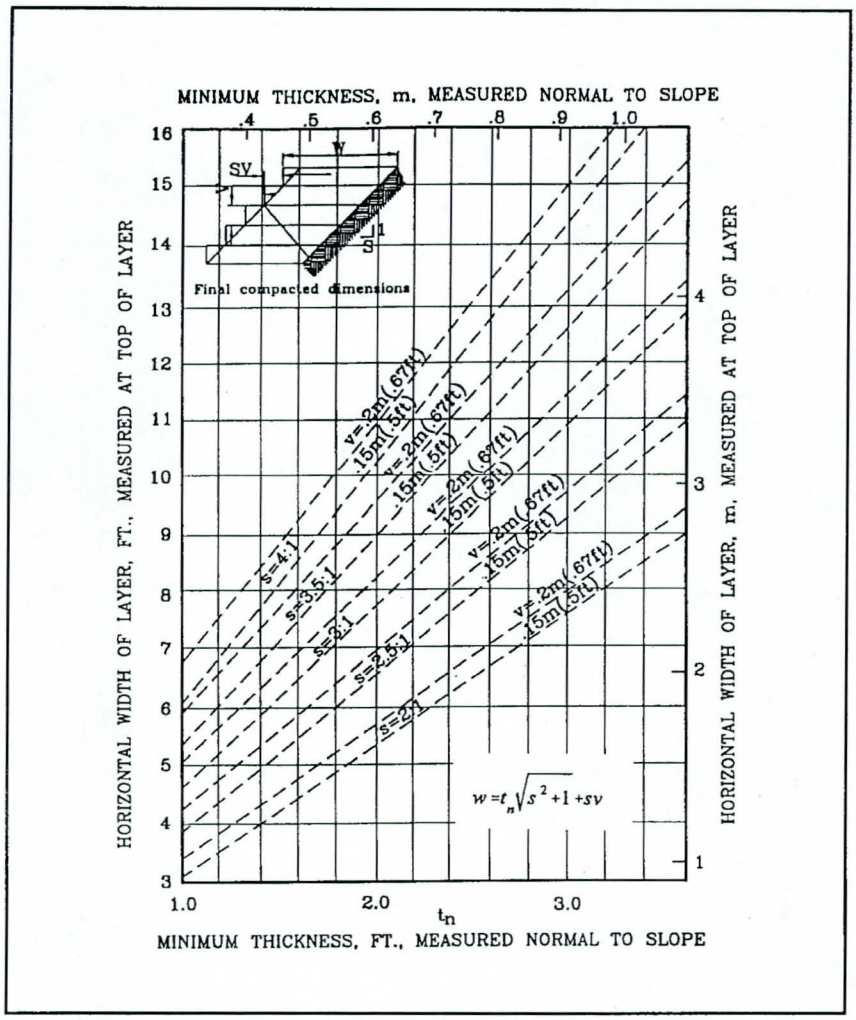


Figure G-3. Relationship of slope, facing thickness, layer thickness, and horizontal layer width

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.

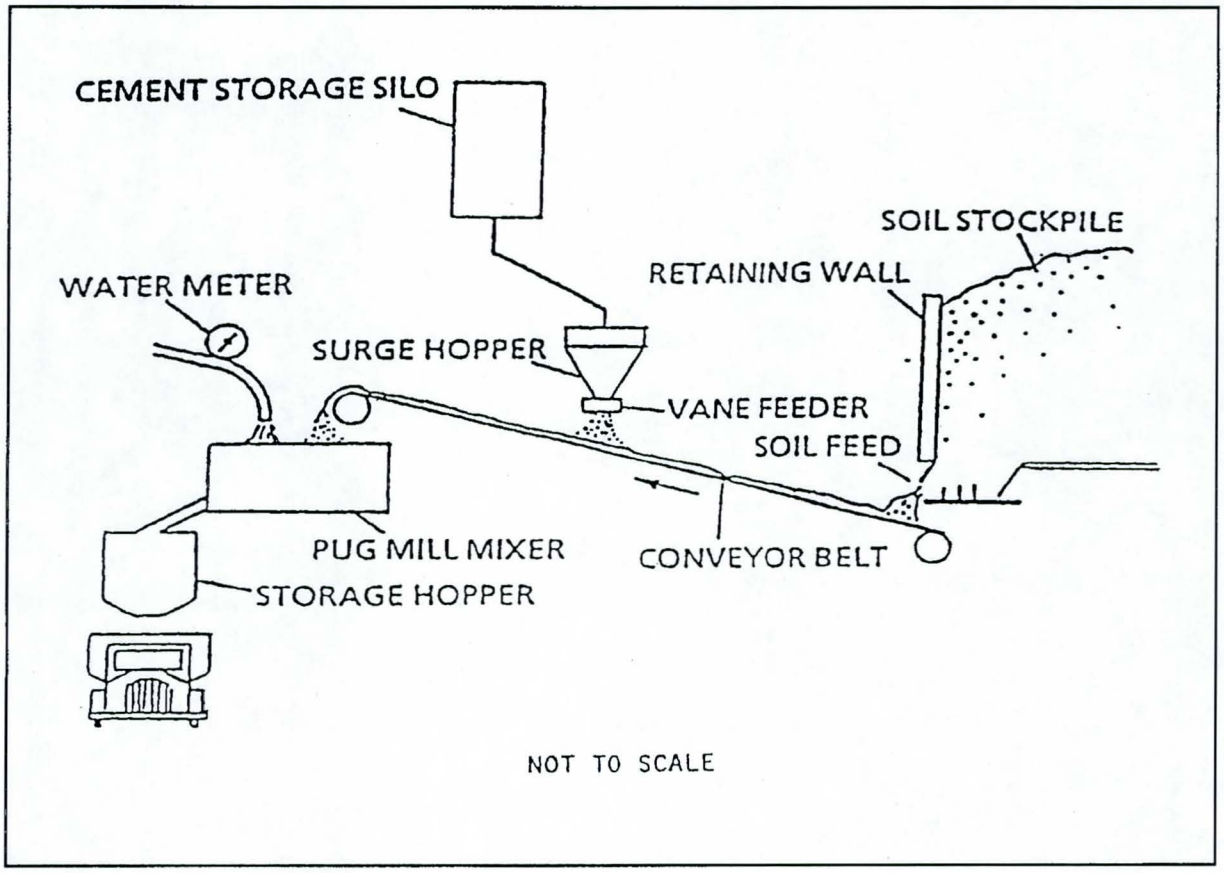


Figure G-4. Typical pug mill central plant

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

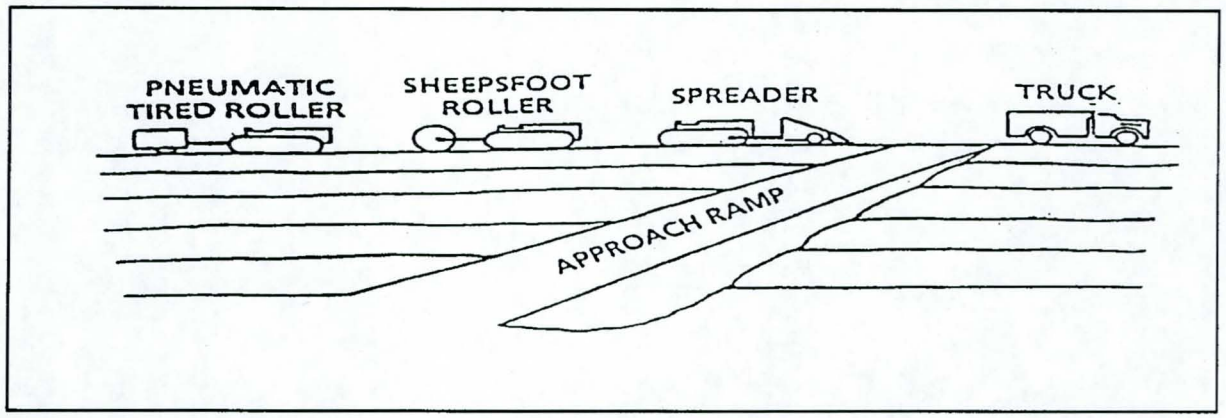


Figure G-6. Typical construction sequence

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.

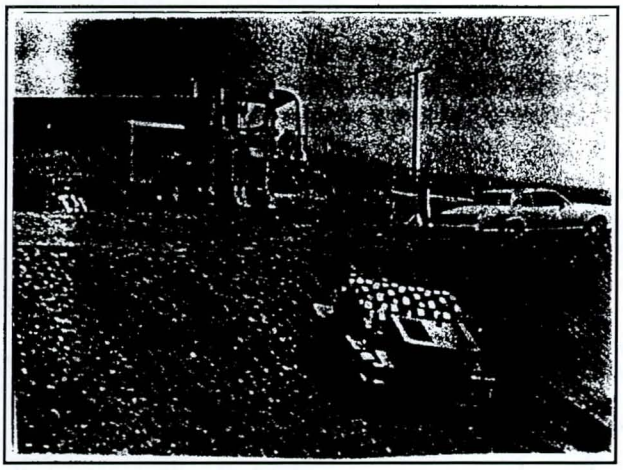


Figure G-7. "Deadman" pulling vibratory sheepsfoot roller up the slope

(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.

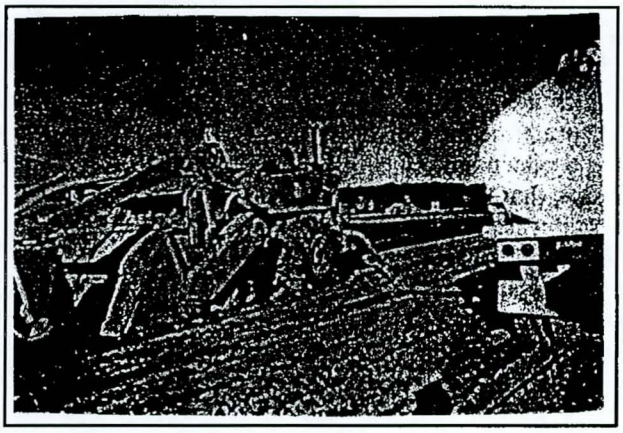


Figure G-8. Transverse single-shaft rotary mixer

(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.

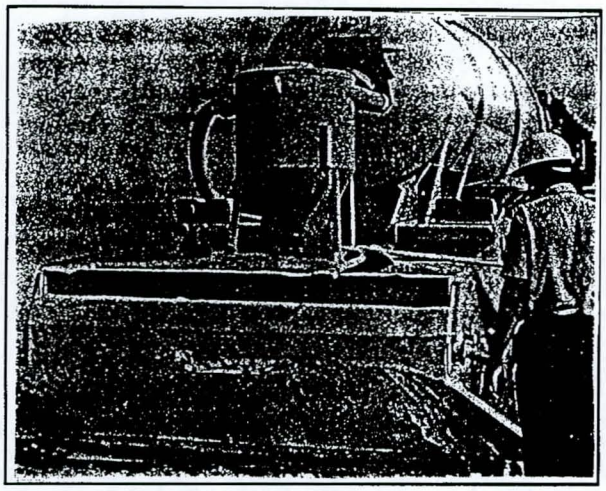


Figure G-9. Bulk mechanical spreader

(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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