An Introduction to Seepage, Slope and Settlement of Levees

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Editor

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1. FOUNDATION UNDERSEEPAGE

1.1 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.

1.2 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.

1.3 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.

1.4 LANDSIDE SEEPAGE BERMS

1.4.1 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 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.

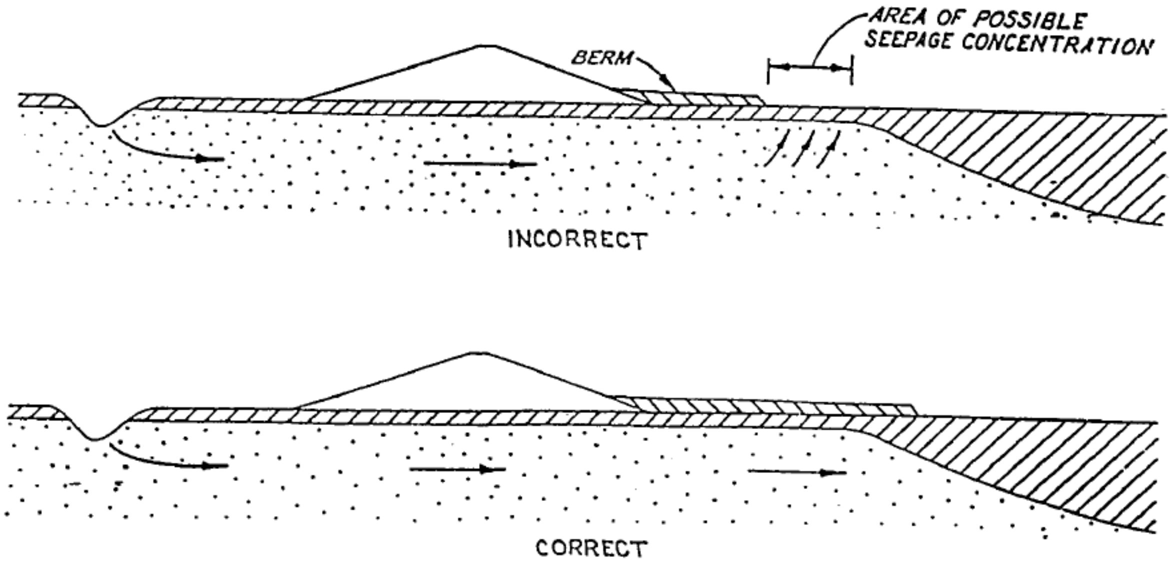


Figure 1

Example of incorrect and correct berm length per existing foundation conditions

1.4.2 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.

1.4.2.1 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.

1.4.2.2 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.

1.4.2.3 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 x 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.

1.4.2.4 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.

1.4.3 BERM DESIGN. Design equations, criteria, and examples are discussed in the technical literature.

1.4.4 COMPUTER PROGRAMS TO USE FOR SEEPAGE ANALYSIS.

1.4.4.1 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 assumptions in the technical literature could be used.

1.4.4.2 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.

1.4.4.3. 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 ≠ k_h), 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.

1.5 PERVIOUS TOE TRENCH

1.5.1 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 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 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.

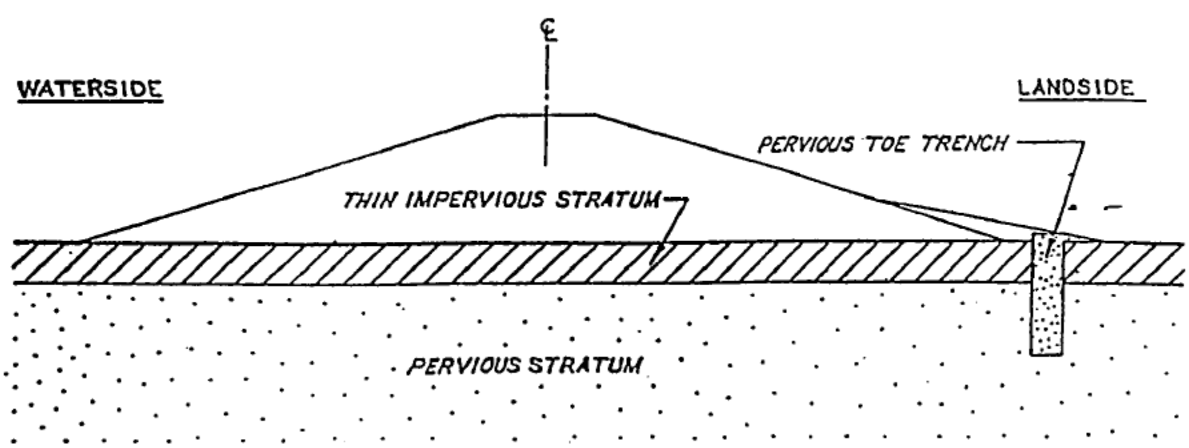


Figure 2

Typical partially penetrating pervious toe trench

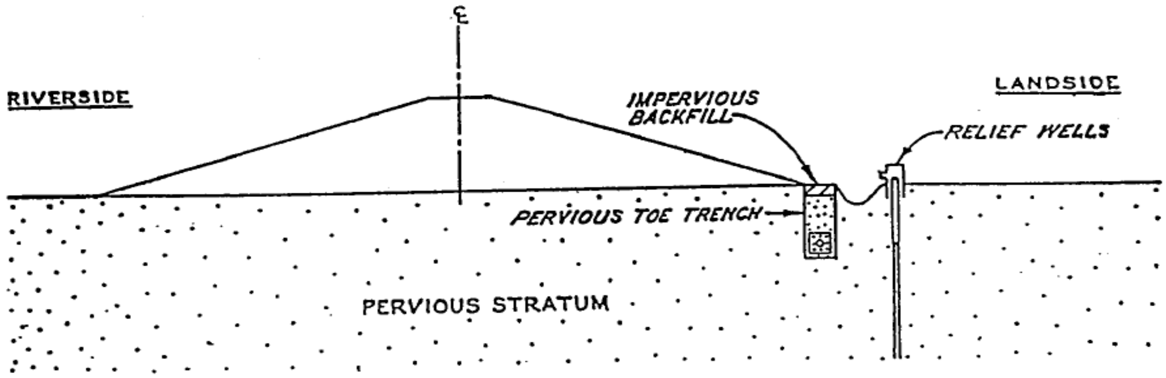


Figure 3

Typical pervious toe trench with collector pipe (Figure 5 shows trench details)

1.5.2 LOCATION. As seen in Figures 2 and 3, pervious drainage trenches are generally located at the levee toe, but are sometimes constructed beneath the downstream levee slope as shown in Figure 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.

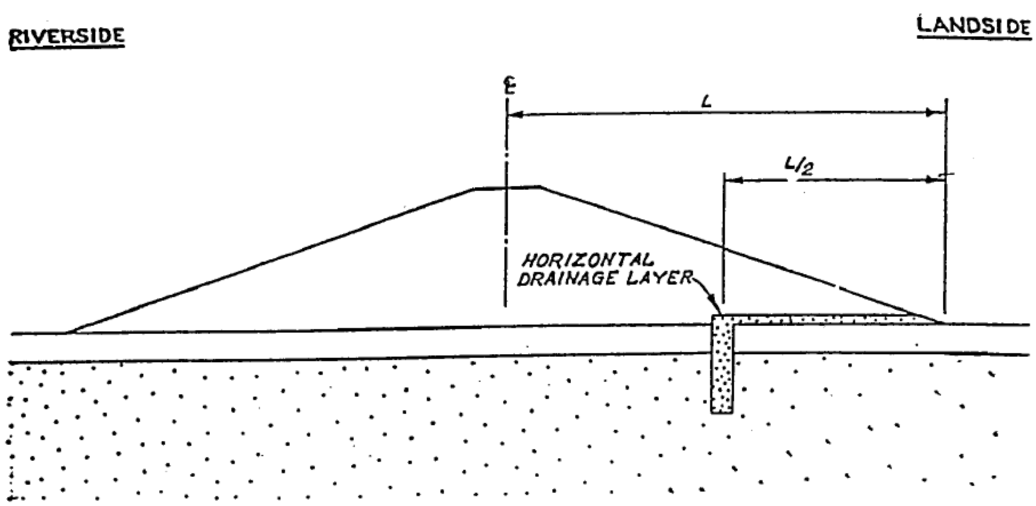


Figure 4

Pervious toe trench located beneath landward slope

1.5.3 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 6ft) 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, which is a vibrating-plate type of compactor specially made to fit on the boom of a backhoe, has apparently performed satisfactorily.

1.5.4 BACKFILL. The sand backfill for trenches must be designed as a filter material in accordance with criteria given. 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. 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.

1.6. PRESSURE RELIEF WELLS

1.6.1 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 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.

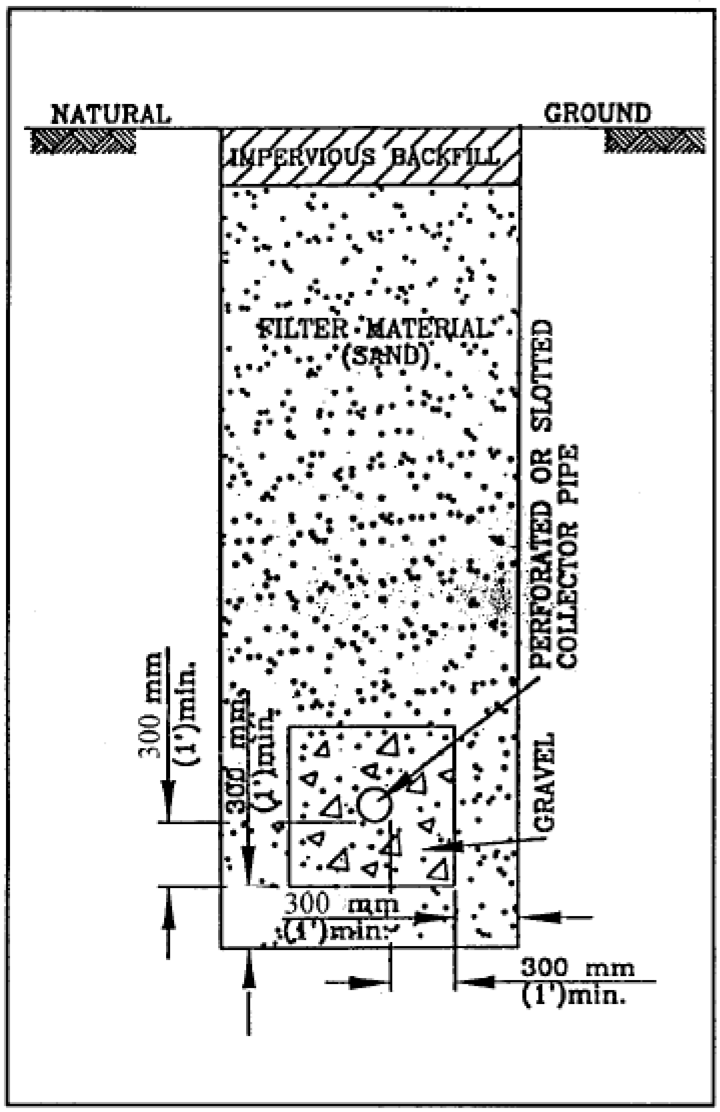


Figure 5

Pervious toe trench with collector pipe

1.6.2 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

1.6.2.1 DEPTH, STRATIFICATION, and permeability of foundation soils,

1.6.2.2 DISTANCE TO THE EFFECTIVE source of seepage,

1.6.2.3 CHARACTERISTICS OF THE LANDSIDE top stratum, if any, and degree of pressure relief desired. 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/m3 (115 pet)). 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.

1.6.3 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 6.

1.6.3.1 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.

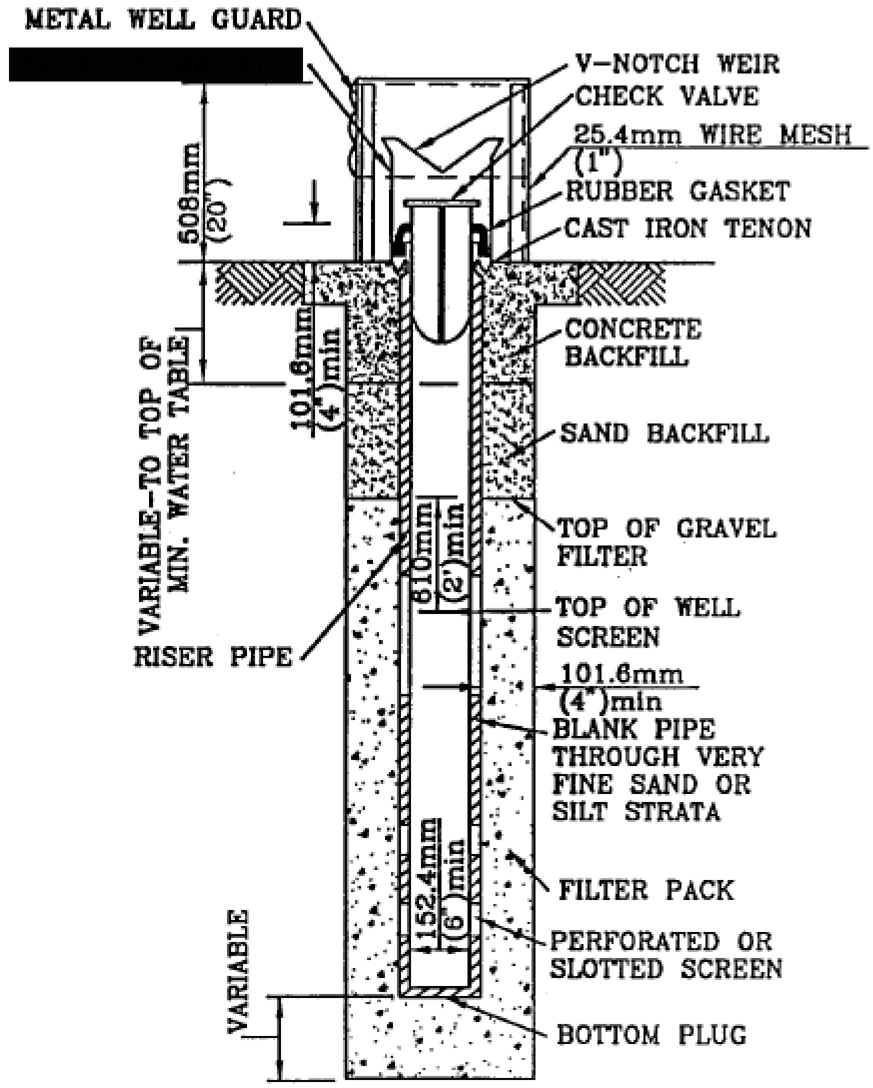


Figure 6

Typical relief well

1.6.3.2 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 of610.8 mm (2ft) 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.

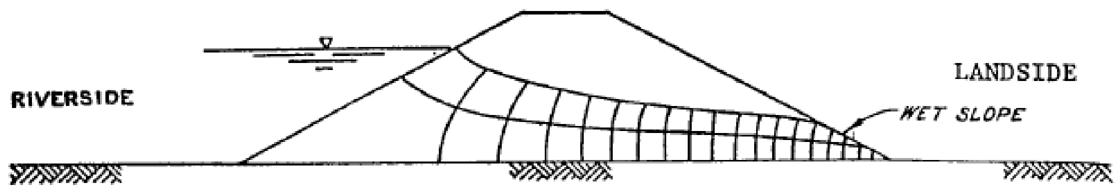
1.6.3.3 WELL APPURTENANCES. In selecting well appurtenances, consideration must be given to ease of maintenance, protection against contamination from back flooding, damage by debris, and vandalism. To prevent wells from becoming backflooded with muddy surface water, which greatly impairs their efficiency when they are not flowing, an aluminum check valve, rubber gasket, and plastic standpipe, as shown in Figure 6, 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.

1.6.3.4 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.

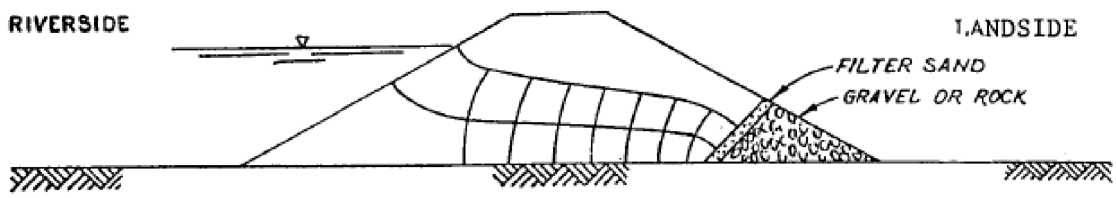
2. SEEPAGE THROUGH EMBANKMENTS

2.1 GENERAL. Should through seepage in an embankment emerge on the landside slope (Figure 7a), it can soften finegrained 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 slope. On the other hand, if no berms are needed, lands ide 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.

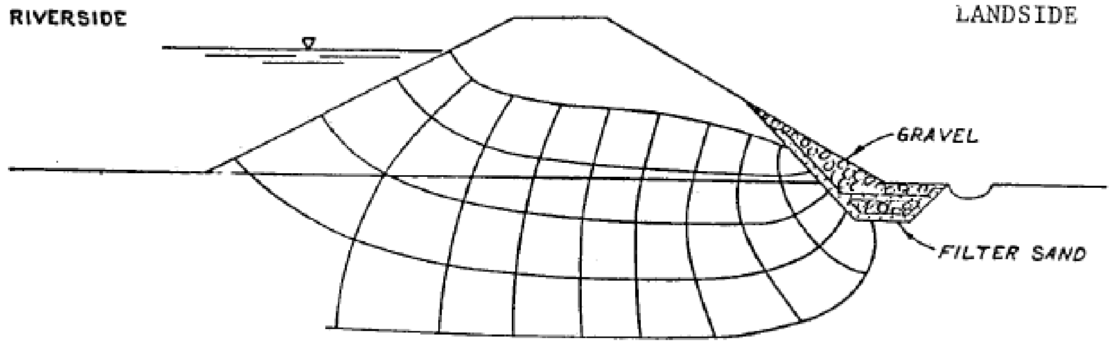
2.2 PERVIOUS TOE DRAIN. A pervious toe (Figure 7b) will provide a ready exit for seepage through the embankment and can lower the phreatic surface sufficiently so that no seepage will emerge on the landside slope. A pervious toe can also be combined with partially penetrating toe trenches, which have previously been discussed, as a method for controlling shallow underseepage. Such a configuration is shown in Figure 7c.



a. Homogeneous section on impervious foundation seepage emerging on landside slope



b. Section with pervious toe



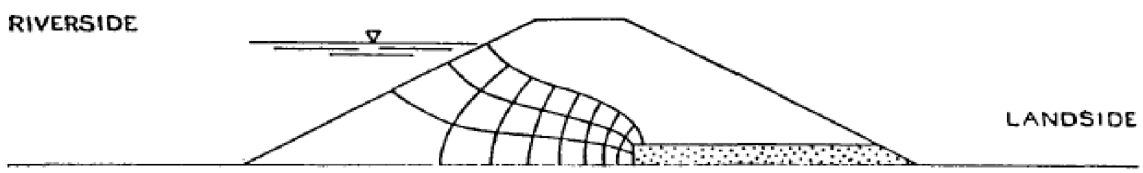
c. Pervious toe combined with partially penetrating toe trencn

Figure 7

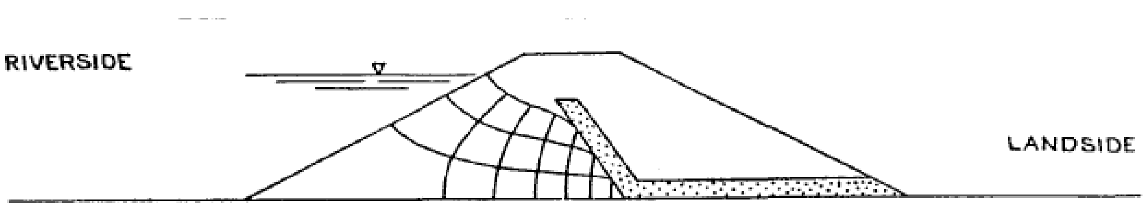
Embankment with through seepage

2.3 HORIZONTAL DRAINAGE LAYERS. Horizontal drainage layers, as shown in Figure 8a, 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.

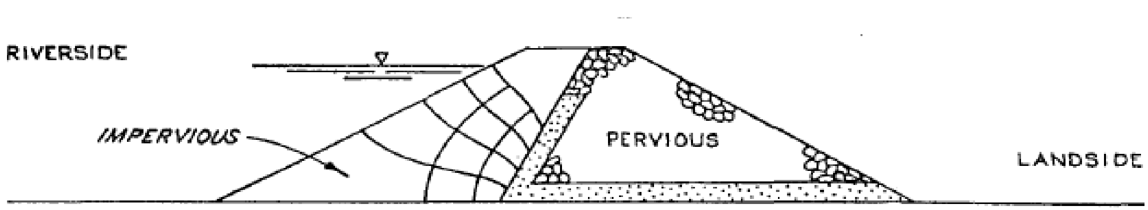
2.4 INCLINED DRAINAGE LAYERS. An inclined drainage layer as shown in Figure 8b is one of the more positive means of controlling internal seepage and is used extensively in earth dams. It is rarely used in levee construction because of the added cost, but might be justified for short levee reaches in 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 8c), it also serves as a filter to prevent migration of impervious fines into the outer shell.



a. Horizontal drainage layer



b. Inclined drainage layer-homogeneous embankment



c. Inclined drainage layer-zoned embankment

Figure 8

Use of horizontal and inclined drainage layers to control seepage via an embankment

If the difference in gradation between the impervious and pervious material is great, the drain may have to be designed as a graded filter. Inclined drains must be tied into horizontal drainage layers to provide an exit for the collected seepage as shown in Figures 9b and 9c.

2.5 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.

2.6 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.

3. EMBANKMENT STABILITY

3.1 EMBANKMENT GEOMETRY

3.1.1 SLOPES. For levees of significant height or when there is concern about the adequacy of available embankment materials or foundation conditions, embankment design requires detailed analysis. Low levees and levees to be built of good material resting on proven foundations may not require extensive stability analysis. For these cases, practical considerations such as type and ease of construction, maintenance, seepage and slope protection criteria control the selection of levee slopes.

3.1.1.1 TYPE OF CONSTRUCTION. Fully compacted levees generally enable the use of steeper slopes than those of levees constructed by semicompacted or hydraulic means. In fact, space limitations in urban areas often dictate minimum levee sections requiring select material and proper compaction to obtain a stable section.

3.1.1.2 EASE OF CONSTRUCTION. A 1V on 2H slope is generally accepted as the steepest slope that can easily be constructed and ensure stability of any riprap layers.

3.1.1.3 MAINTENANCE. A 1V on 3H slope is the steepest slope that can be conveniently traversed with conventional mowing equipment and walked on during inspections.

3.1.1.4 SEEPAGE. For sand levees, a 1V on 5H landside slope is considered flat enough to prevent damage from seepage exiting on the landside slope.

3.1.1.5 SLOPE PROTECTION. Riverside slopes flatter than those required for stability may have to be specified to provide protection from damage by wave action.

3.1.2 FINAL LEVEE GRADE. In the past, freeboard was used to account for hydraulic, geotechnical, construction, operation and maintenance uncertainties. The term and concept of freeboard to account for these uncertainties is no longer used in the design of levee projects. The risk-based analysis directly accounts for hydraulic uncertainties and establishes a nominal top of protection. Deterministic analysis using physical properties of the foundation and embankment materials should be used to set the final levee grade to account for settlement, shrinkage, cracking, geologic subsidence, and construction tolerances.

3.1.3 CROWN WIDTH. The width of the levee crown depends primarily on roadway requirements and future emergency needs. To provide access for normal maintenance operations and floodfighting operations, minimum widths of 3.05 to 3.66 m (10 to 12 ft) are commonly used with wider turnaround areas provided at specified intervals; these widths are about the minimum feasible for construction using modern heavy earthmoving equipment and should always be used for safety concerns. Where the levee crown is to be used as a higher class road, its width is usually established by the responsible agency.

3.2 STANDARD LEVEE SECTIONS AND MINIMUM LEVEE SECTION

3.2.1 MANY DISTRICTS have established standard levee-sections for particular levee systems, which have proven satisfactory over the years for the general stream regime, foundation conditions prevailing in those areas, and for soils available for levee construction. For a given levee system, several different standard sections may be established depending on the type of construction to be used (compacted, semicompacted, uncompacted, or hydraulic fill). The use of standard sections is generally limited to levees of moderate height (say less than 7.62 m (25 ft)) in reaches where there are no serious underseepage problems, weak foundation soils, or undesirable borrow materials (very wet or very organic). In many cases the standard levee section has more than the minimum allowable factor of safety relative to slope stability, its slopes being established primarily on the basis of construction and maintenance considerations. Where high levees or levees on foundations presenting special underseepage or stability problems are to be built, the uppermost riverside and landside slopes of the levee are often the same as those of the standard section, with the lower slopes flattened or stability berms provided as needed.

3.2.2 THE ADOPTION OF STANDARD LEVEE SECTIONS does not imply that stability and underseepage analyses are not made. However, when borings for a new levee clearly demonstrate foundation and borrow conditions similar to those at existing levees, such analyses may be very simple and made only to the extent necessary to demonstrate unquestioned levee stability. In addition to being used in levee design, the standard levee sections are applicable to initial cost estimate, emergency and maintenance repairs.

3.2.3 THE MINIMUM LEVEE SECTION shall have a crown width of at least 3.05 m (10 ft) and a side slope flatter than or equal to 1V on 2H, regardless of the levee height or the possibly less requirements indicated in the results of stability and seepage analyses. The required dimensions of the minimum levee section is to provide an access road for flood-fighting, maintenance, inspection and for general safety conditions.

3.3 EFFECTS OF FILL CHARACTERISTICS AND COMPACTION

3.3.1 COMPACTED FILLS. The types of compaction, water content control, and fill materials govern the steepness of levee slopes from the stability aspect if foundations have adequate strength. Where foundations are weak and compressible, high quality fill construction is not justified, since these foundations can support only levees with flat slopes. In such cases uncompacted or semicompacted fill, as defined in paragraph 1-5, is appropriate. Semicompacted fill is also used where fine-grained borrow soils are considerably wet of optimum or in construction of very low levees where other considerations dictate flatter levee slopes than needed for stability. Uncompacted fill is generally used where the only available borrow is very wet and frequently has high organic content and where rainfall is very high during the construction season. When foundations have adequate strength and where space is limited in urban areas both with respect to quantity of borrow and levee geometry, compacted levee fill construction by earth dam procedures is frequently selected. This involves the use of select material, water content control, and compaction procedures.

3.3.2 HYDRAULIC FILL. Hydraulic fill consists mostly of pervious sands built with one or two end-discharge or bottom-discharging pipes. Tracked or rubber-tired dozers or front-end loaders are used to move the sand to shape the embankment slopes. Because a levee constructed of hydraulic fill would be very pervious and have a low density, it would require a large levee footprint and would be susceptible to soil liquefaction. Hydraulic fill would also quickly erode upon overtopping or where an impervious covering was penetrated. For these reasons, hydraulic fill may be used for stability berms, pit fills and seepage berms but shall not normally be used in constructing levee embankments. However, hydraulic fill may be used for levees protecting agricultural areas whose failure would not endanger human life and for zoned embankments that include impervious seepage barriers.

4. METHODS OF ANALYSIS. The principal methods used to analyze levee embankments for stability against shear failure assume either (a) a sliding surface having the shape of a circular arc within the foundation and/or the embankment or (b) a composite failure surface composed of a long horizontal plane in a relatively weak foundation or thin foundation stratum connecting with diagonal plane surfaces up through the foundation and embankment to the ground surface.

5. CONDITIONS REQUIRING ANALYSIS. The various loading conditions to which a levee and its foundation may be subjected and which should be considered in analyses are designated as follows: Case I, end of construction; Case II, sudden drawdown from full flood stage; Case III, steady seepage from full flood stage, fully developed phreatic surface; Case IV, earthquake. Each case is discussed briefly in the following paragraphs and the applicable type of design shear strength is given.

5.1 CASE I - END OF CONSTRUCTION. This case represents undrained conditions for impervious embankment and foundation soils; i.e., excess pore water pressure is present because the soil has not had time to drain since being loaded. Results from laboratory Q (unconsolidated-undrained) tests are applicable to fine-grained soils loaded under this condition while results of S (consolidated-drained) tests can be used for pervious soils that drain fast enough during loading so that no excess pore water pressure is present at the end of construction. The end of construction condition is applicable to both the riverside and landside slopes.

5.2 CASE II - SUDDEN DRAWDOWN. This case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream embankment portion and then falls faster than the soil can drain. This causes the development of excess pore water pressure which may result in the upstream slope becoming unstable. For the selection of the shear strengths see Table 1.

5.3 CASE III - STEADY SEEPAGE FROM FULL FLOOD STAGE (FULLY DEVELOPED PHREATIC SURFACE). This condition occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition may be critical for landside slope stability. Design shear strengths should be based on Table 1.

5.4 CASE IV - EARTHQUAKE. Earthquake loadings are not normally considered in analyzing the stability of levees because of the low probability of earthquake coinciding with periods of high water. Levees constructed of loose cohesionless materials or founded on loose cohesionless materials are particularly susceptible to failure due to liquefaction during earthquakes. Depending on the severity of the expected earthquake and the importance of the levee, seismic analyses to determine liquefaction susceptibility may be required.

Analysis Condition	Shear Strength[a]	Pore Water Pressure
During and End-of-Construction	Free draining soils - use effective stresses	Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).
	Low permeability soils - use undrained strengths and total stresses[b]	Low permeability soils - Total stresses are used; pore water pressures are set to zero in the slope stability computations.
Steady State Seepage Conditions	Use effective stresses. Residual strengths should be used where previous shear deformation or sliding has occurred.	Estimated from field measurements of pore water pressures, hydrostatic pressure computations for no flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).
Sudden Drawdown Conditions	Free draining soils - use effective stresses	Free draining soils - First stage computations (before drawdown) - steady-state seepage pore pressures as described for steady state seepage condition. Second and third stage computations (after drawdown) - pore water pressures estimated using same techniques as for steady seepage, except with lowered water levels.
	Low permeability soils - Three stage computations: First stage use effective stresses; second stage use undrained shear strengths and total stresses; third stage use drained strengths (effective stresses) or undrained strengths (total stresses) depending on which strength is lower - this will vary along the assumed shear surface.	Low permeability soils - First stage computations - steady-state seepage pore pressures as described for steady state seepage condition. Second stage computations - Total stresses are used pore water pressures are set to zero. Third stage computations - Use same pore pressures as free draining soils if drained strengths are being used; where undrained strengths are used pore water pressures are set to zero.

[a] Effective stress parameters can be obtained from consolidated-drained (CD, S) tests (either direct shear or triaxial) or consolidated-undrained (CU, R) triaxial tests on saturated specimens with pore water pressure measurements. Direct shear or Bromhead ring shear tests should be used to measure residual strengths. Undrained strengths can be obtained from unconsolidated-undrained (UU, Q) tests. Undrained shear strengths can also be estimated using consolidated-undrained (CU, R) tests on specimens consolidated to appropriate stress conditions representative of field conditions; however, the “R” or “total stress” envelope and associated c and ö, from CU, R tests should not be used.

[b] For saturated soils use ö = 0; total stress envelope with ö > 0 is only applicable to partially saturated soils.

Table 1

Summary of Design Conditions

6. MINIMUM ACCEPTABLE FACTORS OF SAFETY. The minimum required safety factors for the preceding design conditions along with the portion of the embankment for which analyses are required and applicable shear test data are shown in Table 2.

	Applicable Stability Conditions and Required Factors of Safety			
Type of Slope	End-of-Construction	Long-Term (Steady Seepage)	Rapid Drawdown [a]	Earthquake[b]
New Levees	1.3	1.4	1.0 to 1.2	(see below)
Existing Levees	--	1.4[c]	1.0 to 1.2	(see below)
Other Embankments and dikes[d]	1.3[e,f]	1.4[c,f]	1.0 to 1.2[f]	(see below)

[a] Sudden drawdown analyses. F. S. = 1.0 applies to pool levels prior to drawdown for conditions where these water levels are unlikely to persist for long periods preceding drawdown. F. S. = 1.2 applies to pool level, likely to persist for long periods prior to drawdown.

[b] See ER 1110-2-1806 for guidance. An EM for seismic stability analysis is under preparation.

[c] For existing slopes where either sliding or large deformation have occurred previously and back analyses have been performed to establish design shear strengths lower factors of safety may be used. In such cases probabilistic analyses may be useful in supporting the use of lower factors of safety for design.

[d] Includes slopes which are part of cofferdams, retention dikes, stockpiles, navigation channels, breakwater, river banks, and excavation slopes.

[e] Temporary excavated slopes are sometimes designed for only short-term stability with the knowledge that long-term stability is not adequate. In such cases higher factors of safety may be required for end-of-construction to ensure stability during the time the excavation is to remain open. Special care is required in design of temporary slopes, which do not have adequate stability for the long-term (steady seepage) condition.

[f] Lower factors of safety may be appropriate when the consequences of failure in terms of safety, environmental damage and economic losses are small.

Table 2

Minimum Factors of Safety - Levee Slope Stability

7. MEASURES TO INCREASE STABILITY. Means for improving weak and compressible foundations to enable stable embankments to be constructed thereon are discussed elsewhere. Methods of improving embankment stability by changes in embankment section are presented in the following paragraphs.

7.1 FLATTEN EMBANKMENT SLOPES. Flattening embankment slopes will usually increase the stability of an embankment against a shallow foundation type failure that takes place entirely within the embankment. Flattening embankment slopes reduces gravity forces tending to cause failure, and increases the length of potential failure surfaces (and therefore increases resistance to sliding).

7.2 STABILITY BERMS. Berms essentially provide the same effect as flattening embankment slopes but are generally more effective because of concentrating additional weight where it is needed most and by forcing a substantial increase in the failure path. Thus, berms can be an effective means of stabilization not only for shallow foundation and embankment type failures but for more deep-seated foundation failures as well. Berm thickness and width should be determined from stability analyses and the length should be great enough to encompass the entire problem area, the extent of which is determined from the soil profile. Foundation failures are normally preceded by lateral displacement of material beneath the embankment toe and by noticeable heave of material just beyond the toe. When such a condition is noticed, berms are often used as an emergency measure to stabilize the embankment and prevent further movement.

8. SURFACE SLIDES. Experience indicates that shallow slides may occur in levee slopes after heavy rainfall. Failure generally occurs in very plastic clay slopes. They are probably the result of shrinkage during dry weather and moisture gain during wet weather with a resulting loss in shear strength due to a net increase in water content, plus additional driving force from water in cracks. These failures require maintenance and could be eliminated or reduced in frequency by using less plastic soils near the surface of the slopes or by chemical stabilization of the surface soils.

9. SETTLEMENT

9.1 GENERAL. Evaluation of the amount of postconstruction settlement that can occur from consolidation of both embankment and foundation may be important if the settlement would result in loss of freeboard of the levee or damage to structures in the embankment. Many districts overbuild a levee by a given percent of its height to take into account anticipated settlement both of the foundation and within the levee fill itself. Common allowances are 0 to 5 percent for compacted fill, 5 to 10 percent for semicompacted fill, 15 percent for uncompacted fill, and 5 to 10 percent for hydraulic fill. Overbuilding does however increase the severity of stability problems and may be impracticable or undesirable for some foundations.

10. SETTLEMENT ANALYSES. Settlement estimates can be made by theoretical analysis. Detailed settlement analyses should be made when significant consolidation is expected, as under high embankment loads, embankments of highly compressible soil, embankments on compressible foundations, and beneath steel and concrete structures in levee systems founded on compressible soils. Where foundation and embankment soils are pervious or semipervious, most of the settlement will occur during construction. For impervious soils it is usually conservatively assumed that all the calculated settlement of a levee built by a normal sequence of construction operations will occur after construction. Where analyses indicate that more foundation settlement would occur than can be tolerated, partial or complete removal of compressible foundation material may be necessary from both stability and settlement viewpoints. When the depth of excavation required to accomplish this is too great for economical construction, other methods of control such as stage construction or vertical sand drains may have to be employed, although they seldom are justified for this purpose.