CECW-EG Engineer Manual 1110-2-1913	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-2-1913 31 March 1978 (Original) 30 June 1996 (Change 1)
	Engineering and Design DESIGN AND CONSTRUCTION OF LEVEES	
	Distribution Restriction Statement Approved for public release; distribution is unlimited.	

ENGINEER MANUAL

EM 1110-2-1913
31 March 1978

ENGINEERING AND DESIGN

DESIGN AND CONSTRUCTION OF LEVEES

DEPARTMENT OF THE ARMY
OFFICE OF THE CHIEF OF ENGINEERS
WASHINGTON, D.C. 20314

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

EM 1110-2-1913
Change 1

CECW-EG-S

Manual
No. 1110-2-1913

30 June 1996

Engineering and Design
DESIGN AND CONSTRUCTION OF LEVEES

1. This change replaces paragraph 6-2. Standard Levee Section of EM 1110-2-1913, dated 31 March 1978.

2. File this change sheet in front of the publication for reference purposes.

FOR THE COMMANDER:

Tim Stewt
for ROBERT H. GRIFFIN
Colonel, Corps of Engineers
Chief of Staff

DEPARTMENT OF THE ARMY
Office of the Chief of Engineers
Washington, D.C. 20314

EM 1110-2-1913

DAEN-CWE-S

Engineer Manual
No. 1110-2-1913

31 March 1978

ENGINEERING AND DESIGN
Design and Construction of Levees

1. Purpose. The purpose of this manual is to present basic principles used in the design and construction of earth levees.

2. Applicability. This manual applies to all Corps of Engineers Divisions and Districts having responsibility for the design and construction of levees.

3. General. This manual is intended as a guide for designing and constructing levees and not intended to replace the judgment of the design engineer on a particular project.

FOR THE CHIEF OF ENGINEERS:

James N. Ellis

JAMES N. ELLIS
Colonel, Corps of Engineers
Executive Director, Engineer Staff

INITIAL EDITOR

DEPARTMENT OF THE ARMY
Office of the Chief of Engineers
Washington, D. C. 20314

EM 1110-2—1913

DAEN—CWE—S
Engineer Manual
No. 1110—2-1913

31 March 1978

Engineering and Design
DESIGN AND CONSTRUCTION OF LEVEES

Table of Contents

CHAPTER 1

INTRODUCTION

1-1. Purpose. The purpose of this manual is to present basic principles used in the design and construction of earth levees.

1-2. Applicability. This manual applies to all Corps of Engineers Divisions and Districts having responsibility for designing and constructing levees.

1-3. References. Applicable references are listed in Appendix A.

1-4. Objective. The objective of this manual is to develop a guide for design and construction of levees. The manual is general in nature and not intended to supplant the judgment of the design engineer on a particular project.

1-5. General Considerations.

a. General.

(1) The term levee as used herein is defined as an embankment whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks a year. Embankments that are subject to water loading for prolonged periods (longer than normal flood protection requirements) or permanently should be designed in accordance with earth dam criteria rather than the levee criteria given herein.

(2) Even though levees are similar to small earth dams they differ from earth dams in the following important respects: (a) a levee embankment may become saturated for only a short period of time beyond the limit of capillary saturation, (b) levee alignment is dictated primarily by flood protection requirements, which often results in construction on poor foundations, and (c) borrow is generally obtained from shallow pits or from channels excavated adjacent to the levee, which produce fill material that is often heterogeneous and far from ideal. Selection of the levee section is often based on the properties of the poorest material that must be used.

(3) Numerous factors must be considered in levee design. These factors may vary from project to project, and no specific step-by-step procedure covering details of a particular project can be established. However, it is possible to present general, logical steps based on

successful past projects that can be followed in levee design and can be used as a base for developing more specific procedures for any particular project. Such a procedure is given in table l-l. Information for implementing this procedure is presented in subsequent chapters.

(4) The method of construction must also be considered. In the

Table 1-1. General Design Procedure

Step	Procedure
1	Conduct geological study based on a thorough review of available data including analysis of aerial photographs. Initiate preliminary subsurface explorations.
2	Analyze preliminary exploration data and from this analysis establish preliminary soil profiles, borrow locations, and embankment sections.
3	Initiate final exploration to provide: a. Additional information on soil profiles. b. Undisturbed strengths of foundation materials. c. More detailed information on borrow areas and other required excavations.
4	Using the information obtained in Step 3: a. Determine both embankment and foundation soil parameters and refinepreliminary sections where needed, noting all possible problem areas. b. Compute rough quantities of suitable material and refine borrow area locations.
5	Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.
6	Analyze each trial section as needed for: a. Underseepage and through seepage. b. Slope stability. c. Settlement.
7	Design special treatment to preclude any problems as determined from Step 6.
8	Based on the results of Step 7, establish final sections for each reach.
9	Compute final quantities needed; determine final borrow area locations.
10	Design embankment slope protection.

past levees have been built by methods of compaction varying from none to carefully controlled compaction. The local economic situation also affects the selection of a levee section. Traditionally, in areas of high property values, high land use, and good foundation conditions, levees have been built with relatively steep slopes using controlled compaction, while in areas of lower property values, poor foundations, or high rainfall during the construction season, uncompacted or semi-compacted levees with flatter slopes are more typical. This is evident by comparing the steep slopes of levees along the industrialized Ohio River Valley with levees along the Lower Mississippi River which have much broader sections with gentler slopes. Levees built with smaller sections and steeper slopes generally require more comprehensive investigation and analysis than do levees with broad sections and flatter slopes whose design is more empirical. Where rainfall and foundation conditions permit, the trend in design of levees is toward sections with steeper slopes. Levee maintenance is another factor that often has considerable influence on the selection of a levee section.

b. Levee Types According to Location. Levees are broadly classified according to the area they protect as either urban or agricultural levees because of different requirements for each. As used in this manual, urban and agricultural levees are defined as follows:

(1) Urban levees. Levees that provide protection from flooding in communities, including their industrial, commercial, and residential facilities.

(2) Agricultural levees. Levees that provide protection from flooding in lands used for agricultural purposes.

c. Levee Types According to Use. Some of the more common terms used for levees serving a specific purpose in connection with their overall purpose of flood protection are given in table 1-2.

d. Causes of Levee Failures. The principal causes of levee failure are

(1) Overtopping.

(2) Surface erosion.

(3) Internal erosion (piping).

(4) Slides within the levee embankment or the foundation soils.

Table 1-2. Classification of Levees According to Use

Type	Definition
Mainline and tributary levees	Levees that lie along a mainstream and its tributaries, respectively.
Ring levees	Levees that completely encircle or "ring" an area subject to inundation from all directions.
Setback levees	Levees that are built landward of existing levees, usually because the existing levees have suffered distress or are in some way being endangered, as by river migration.
Sublevees	Levees built for the purpose of underseepage control. Sublevees encircle areas behind the main levee which are subject, during high-water stages, to high uplift pressures and possibly the development of sand boils. They normally tie into the main levee, thus providing a basin that can be flooded during high-water stages, thereby counterbalancing excess head beneath the top stratum within the basin. Sublevees are rarely employed as the use of relief wells or seepage berms make them unnecessary except in emergencies.
Spur levees	Levees that project from the main levee and serve to protect the main levee from the erosive action of stream currents. Spur levees are not true levees but training dikes.

CHAPTER 2

FIELD INVESTIGATIONS

2-1. Preliminary and Final Stages. Many field investigations are conducted in two stages: a preliminary stage and a final (design) stage. Normally, a field investigation in the preliminary stage is not extensive since its purpose is simply to provide general information for project feasibility studies. It will usually consist of a general geological reconnaissance with only limited subsurface exploration and simple soil tests. In the design stage, more comprehensive exploration is usually necessary, with more extensive geological reconnaissance, borings, test pits, and possibly geophysical studies. The extent of the field investigation depends on several factors. Table 2-1 lists these factors together with conditions requiring extensive field investigations and design studies. Sometimes field tests such as vane shear tests, groundwater observations, and field pumping tests are necessary. Table 2-2 summarizes, in general, the broad features of geologic and subsurface investigations.

Section I. Geological Study

2-2. Scope. A geological study usually consists of an office review of all available geological information on the area of interest, and an on-site (field) survey. As most levees are located in alluvial floodplains, the distribution and engineering characteristics of alluvial deposits in the vicinity of proposed levees must be evaluated. The general distribution, nature, and types of floodplain deposits are directly related to changes in the river and its tributaries. Each local area in the floodplain bears traces of river action, and the alluvial deposits there may vary widely from those in adjacent areas. The general nature and distribution of sediments can be determined through a study of the pattern of local river changes as a basis for selection of boring locations.

2-3. Office Study. The office study begins with a search of available information, such as topographic, soil, and geological maps and aerial photographs. Pertinent information on existing construction in the area should be obtained. This includes design, construction, and performance data on utilities, highways, railroads, and hydraulic structures. Available boring logs should be secured. Federal, state, county, and local agencies and private organizations should be contacted for information.

2-4. Field Survey. The field survey is commenced after becoming familiar with the area through the office study. Walking the proposed

Table 2-1. Factors Requiring Intensive Field Investigations and Design Studies

Factor	Field Investigations and Design Studies Should Be More Extensive Where:
Previous experience	There is little or no previous experience in the area particularly with respect to levee performance
Consequences of failure	Consequences of failure involving life and property are great (urban areas for instance)
Levee height	Levee heights are great
Foundation conditions	Foundation soils are weak and compressible
	Foundation soils are highly variable along the alignment
	Potential underseepage problems are severe
	Foundation sands may be liquefaction susceptible
Duration of high water	High water levels against the levee exist over relatively long periods
Borrow materials	Available borrow is of low quality, water contents are high, or borrow materials are variable along the alignment
Structure in levees	Reaches of levees are adjacent to concrete structures

Table 2-2. Stages of Field Investigations

1. Investigation or analysis produced by rapid field reconnaissance and discussion with knowledgeable people is adequate for design where:

   a. Levees are 10 ft or less in height.
   b. Experience has shown foundations to be stable and presenting no underseepage problems.

   Use standard levee section developed through experience.

2. Preliminary geological investigation: Required for all cases except those in 1 above. Use to decide the need for and scope of subsurface exploration and field testing:

   a. Office study: Collection and study of

      (1) Topographic, soil, and geological maps.
      (2) Aerial photographs.
      (3) Boring logs and well data.
      (4) Information on existing engineering projects.

   b. Field survey: Observations and geology of area, documented by written notes and photographs, including such features as:

      (1) Riverbank slopes, rock outcrops, earth and rock cuts or fills.
      (2) Surface materials.
      (3) Poorly drained areas.
      (4) Evidence of instability of foundations and slopes.
      (5) Emerging seepage.
      (6) Natural and man-made physiographic features.

3. Subsurface exploration and field testing and more detailed geologic study:

   a. Preliminary phase:

      (1) Widely but not necessarily uniformly spaced disturbed sample borings (may include split-spoon penetration tests).
      (2) Test pits excavated by backhoes, dozers, or farm tractors.
      (3) Geophysical surveys (seismic or electrical resistivity) to interpolate between widely spaced borings.
      (4) Borehole geophysical tests.

   b. Final phase:

      (1) Additional disturbed sample borings.
      (2) Undisturbed sample borings.
      (3) Field vane shear tests for special purposes.
      (4) Field pumping tests (primarily in vicinity of structures).
      (5) Water table observations (using piezometers) in foundations and borrow areas.

alignment is always an excellent means of obtaining useful information. Physical features to be observed are listed in table 2-2. These items and any others of significance should be documented by detailed notes, supplemented by photographs. Local people or organizations having knowledge of foundation conditions in the area should be interviewed.

2-5. Report. When all available information has been gathered and assimilated, a report should be written that in essence constitutes a geological, foundation, and materials evaluation report for the proposed levee. All significant factors that might affect the alignment and/or design should be clearly pointed out and any desirable changes in alignment suggested.

Section II. Subsurface Exploration

2-6. General.

a. Because preliminary field investigations usually involve only limited subsurface exploration, only portions of the following discussion may be applicable to the preliminary stage, depending on the nature of the project.

b. The subsurface exploration for the design stage generally is accomplished in two phases, which may be separate, in sequence, or concurrent: (1) Phase 1, the main purpose of which is to better define the soil types present and to develop general ideas of soil strengths and permeabilities; (2) Phase 2, provides additional information on soil types present and usually includes the taking of undisturbed samples for testing purposes.

2-7. Phase 1 Exploration. Phase 1 exploration consists almost entirely of disturbed sample borings and perhaps test pits excavated with backhoes, dozers, farm tractors, etc., as summarized in table 2-3, but may also include geophysical surveys which are discussed later.

2-8. Phase 2 Exploration. Phase 2 subsurface exploration consists of both disturbed and undisturbed sample borings and also may include geophysical methods. Undisturbed samples for testing purposes are sometimes obtained by handcarving block samples from test pits but more usually by rotary and push-type drilling methods (using samplers such as the Denison sampler in extremely hard soils or the thin-walled Shelby tube fixed piston sampler in most soils). Samples for determining consolidation and shear strength characteristics and values of density and permeability should be obtained using undisturbed borings in which 5-in.-diameter samples are taken in cohesive materials and

Table 2-3. Phase I Boring and Sampling Techniques

Technique	Remarks
1. Disturbed sample borings	
a. Split-spoon or standard penetration test	l-a. Primarily for soil identification but permits estimate of shear strength of clays and crude estimate of density of sands; see paragraph 5-3d of EN 1110-2-1907 (ref. A-3a(8))
	Preferred for general exploration of levee foundations; indicates need and locations for undisturbed samples
b. Auger borings	l-b. Bag and jar samples can be obtained for testing
2. Test pits	2. Use backhoes, dozers, and farm tractors
3. Trenches	3. Occasionally useful in borrow areas and levee foundations

3-in,-diameter samples are taken in cohesionless materials. EM 1110-2-1907 (ref. A3-a(8)) gives details of drilling and sampling techniques,

2-9. Borings.

a. Location and Spacing. The spacing of borings and test pits in Phase 1 is based on examination of airphotos and geological conditions determined in the preliminary stage or known from prior experience in the area, and by the nature of the project. Initial spacing of borings usually varies from 200 to 1000 ft along the alignment, being closer spaced in expected problem areas and wider spaced in nonproblem areas. The spacing of borings should not be arbitrarily uniform but rather should be based on available geologic information. Borings are normally laid out along the levee centerline but can be staggered along the alignment in order to cover more area and to provide some data on nearby borrow materials. At least one boring should be located at every major structure during Phase 1. In Phase 2, the locations of additional general sample borings are selected based on Phase 1 results. Undisturbed sample borings are located where data on soil shear strength are most needed. The best procedure is to group the foundation profiles developed on the basis of geological studies and exploration into reaches of similar conditions and then locate undisturbed sample borings so as to define soil properties in critical reaches.

b. Depth. Depth of borings along the alignment should be at least equal to the height of levee but not less than 10 ft. Boring depths should always be deep enough to provide data for stability analyses of the levee and foundation. This is especially important when the levee is located near the riverbank where borings must provide data for stability analyses involving both levee foundation and riverbank. Where pervious or soft materials are encountered, borings should extend through the permeable material to impervious material or through the soft material to firm material. Borings at structure locations should extend well below invert or foundation elevations and below the zone of significant influence created by the load. The borings must be deep enough to permit analysis of approach and exit channel stability and of underseepage conditions at the structure. In borrow areas, the depth of exploration should extend several feet below the practicable or allowable borrow depth or to the groundwater table. If borrow is to be obtained from below the groundwater table by dredging or other means, borings should be at least 10 ft below the bottom of the proposed excavation.

2-10. Geophysical Exploration.

a. Use of geophysical methods of subsurface exploration is expected to increase as a part of foundation exploration for levees because of the long, relatively narrow areas to be explored and the increasing cost of borings. Table 2-4 summarizes those geophysical methods most appropriate to levee exploration. These methods are a fairly inexpensive means of exploration and are very useful for interpolating between borings which, for reasons of economy, are spaced at fairly wide intervals. Geophysical data must be interpreted in conjunction with borings and by qualified experienced personnel or misleading information is almost certain to result. Because there have been significant improvements in geophysical instrumentation and interpretation techniques in recent years, more consideration should be given to their use.

b. Currently available geophysical methods can be broadly subdivided into two classes: those accomplished entirely from the ground surface and those which are accomplished from subsurface borings.

Table 2-4. Applicable Geophysical Methods of Exploration

Name	Principle	Primary Use
1. Seismic methods	Based on time required for seismic waves to travel from source to points on ground surface, as measured by geophones spaced at intervals on the surface	
a. Refraction	Refraction of seismic waves at the interface between different strata gives a pattern of arrival times versus distance at a line of geophones	Utilized to determine depth to rock or other lower stratum substantially different in wave velocity than the overlying material. Generally limited to depths of 5 to 500 ft. Used only where wave velocity in successive layers becomes greater with depth
b. Continuous vibration	The travel time of transverse or shear waves generated by a mechanical vibrator consisting of a pair of eccentrically weighted disks is recorded by seismic detectors placed at specific distances from the vibrator	Velocity of wave travel gives indication of soil type. Travel time plotted as a function of distance indicates depths or thicknesses of substrata. Useful in determining dynamic modulus of subgrade reaction and obtaining information

(Continued)

Table 2-4. (Continued)

Name	Principle	Primary Use
b. Continuous vibration (Continued)		for the natural period of vibration for foundations of vibrating structures
2. Electrical methods		
a. Resistivity	Based on the difference in electrical conductivity or resistivity of strata. Resistivity of subsoils at various depths is determined by passing a known current between two electrodes and measuring the potential difference between two intermediate electrodes. Resistivity is correlated to material type	Used to determine depths up to 100 ft of horizontal subsurface strata. Principal applications for investigating foundations of dams, levees, and other large structures, particularly in exploring granular river channel deposits or bedrock surfaces. Also used to measure depth to saturated zones or aquifers
b. Equipotential mapping	Location of lines of equal potential around a current electrode	Delineation of vertical boundaries and zones of limited horizontal extent. Can trace lines of water flow or locate bodies such as clay plugs

Exploration from the ground surface will generally be adequate for levee design purposes. Geophysical ground surface exploration can involve: (1) seismic refraction and (2) electrical resistivity. Information obtained from seismic refraction surveys includes material velocities, delineation of interfaces between zones of differing velocities, and the depths to these interfaces. The electrical resistivity survey is used to locate and define zones of different electrical properties such as pervious and impervious zones or zones of higher ionic activity such as clayey strata. Both methods require distinct differences in properties of foundation strata materials in order to be effective. The resistivity method requires a high resistivity contrast between materials being located, while the seismic method requires high contrast in wave transmission velocities. Furthermore, the seismic method requires that any underlying stratum transmit waves at a higher velocity than the overlying stratum. Difficulties arise in the use of the seismic method if the surface terrain and/or layer interfaces are steeply sloping or irregular instead of relatively horizontal and smooth. Therefore, in order to use these methods, one must be fully aware of what they can and cannot do. EM 1110-2-1802 (ref. A-3a(3)) describes the use of both seismic refraction and electrical resistivity. Dobrin (ref. A-5c) is a valuable, general text on geophysical exploration. Applicable geophysical exploration methods based on operation from the ground surface are summarized in table 2-4.

c. Recent developments in the use of downhole logging devices have shown that these tools can be used with success in correlating subsurface soil and rock stratification and in providing quantitative engineering parameters such as porosity, density, water content, and moduli. They also provide valuable data for interpreting surface geophysical data. The purpose in using these methods is to allow cost savings to be made in the exploration program without lessening the quality of the information obtained. This can be done by reducing the number of borings required to determine subsurface stratification and by allowing sampling to be done only in those zones where samples are necessary for laboratory testing, thus reducing the number of undisturbed samples.

Section III. Field Testing

2-11. Preliminary Strength Estimates. It is often desirable to estimate foundation strengths during Phase 1 of the exploration program. Various methods of preliminary appraisal are listed in table 2-5.

2-12. Vane Shear Tests. Where undisturbed samples are not being obtained or where samples of acceptable quality are difficult to obtain, in situ vane shear tests may be utilized as a means of obtaining

Table 2-5. Preliminary Appraisal of Foundation Strengths

Method	Remarks
1. Split-spoon penetration resistance	1-a. Unconfined compressive strength, tsf, of clay is about 1/8 of number of blows per foot, or N/8 , but considerable scatter must be expected. Generally not helpful where N is low
	1-b. In sands, N values less than about 15 indicate low relative densities. N values should not be used to estimate relative densities for earthquake design
2. Natural water content of disturbed or general type samples	2. Useful when considered with soil classification, and previous experience is available
3. Hand examination of disturbed samples	3. Useful where experienced personnel are available who are skilled in estimating soil shear strengths
4. Position of natural water contents relative to liquid and plastic limits	4-a. Useful where previous experience is available
	4-b. If natural water content is close to plastic limit foundation shear strength should be high
	4-c. Natural water contents near liquid limit indicate sensitive soils usually with low shear strengths
5. Torvane or pocket penetrometer tests on intact portions of general samples or on walls of test trenches	5. Easily performed and inexpensive but results may be low; useful for preliminary strength classifications

undrained shear strength. The apparatus and procedure for performing this test are described in Appendix D of EM 1110-2-1907 (ref. A-3a(8)). The results from this test may be greatly in error where shells or fibrous organic material are present. Also, test results in fat clays must be corrected using empirical correction factors as given by Bjerrum (ref. A-5a) but these are not always conservative.

2-13. Groundwater and Pore Pressure Observations. Piezometers to observe groundwater fluctuations are rarely installed solely for design purposes but should always be installed in areas of potential underseepage problems. The use and installation of piezometers are described in EM 1110-2-1908 (ref. A-3a(9)). Permeability tests should always be made after installation of the piezometers; these tests provide information on foundation permeability and show if piezometers are functioning. Testing and interpretation procedures are described in EM 1110-2-1908 (ref. A-3a(9)).

2-14. Field Pumping Tests. The permeability of pervious foundation materials can often be estimated with sufficient accuracy by using existing correlations with grain-size determination; see TM 5-818-5 (ref. A-2)). However, field pumping tests are the most accurate means of determining permeabilities of stratified in situ deposits. Field pumping tests are expensive and usually justified only at sites of important structures and where extensive pressure relief well installations are planned. The general procedure is to install a well and piezometers at various distances from the well to monitor the resulting drawdown during pumping of the well. Appendix III of TM 5-818-5 (ref. A-2)) gives procedures for performing field pumping tests.

CHAPTER 3

LABORATORY TESTING

3-1. General.

a. Reference should be made to EM 1110-2-1906 (ref. A-3a(7)) for current soil testing procedures, and to EM 1110-2-1902 (ref. A-3a(4)) for applicability of the various shear strength tests in stability analyses.

b. Laboratory testing programs for levees will vary from minimal to extensive, depending on the nature and importance of the project and on the foundation conditions, how well they are known, and whether existing experience and correlations are applicable. Since shear and other tests to determine the engineering properties of soils are expensive and time-consuming, testing programs generally consist of water content and identification tests on most samples and shear, consolidation, and compaction tests only on representative samples of foundation and borrow materials. It is imperative to use all available data such as geological and geophysical studies, when selecting representative samples for testing. Soil tests that may be included in laboratory testing programs are listed in table 3-1 for fine-grained cohesive soils and in table 3-2 for pervious soils, together with pertinent remarks on purposes and scope of testing.

3-2. Classification and Water Content Determinations. After soil samples have been obtained in subsurface exploration of levee foundations and borrow areas, the first and essential step is to make visual classifications and water content determinations on all samples (except that water content determinations should not be made on clean sands and gravels). These samples may be jar or bag samples obtained from test pits, disturbed or undisturbed drive samples, or auger samples. Field descriptions, laboratory classifications, and water content values are used in preparing graphic representations of boring logs. After examining these data, samples of fine-grained soils are selected for Atterberg limits tests, and samples of coarse-grained soils for gradation tests.

Section I. Fine-Grained Soils

3-3. Use of Correlations. Comparisons of Atterberg limits values with natural water contents of foundation soils and use of the plasticity chart itself (fig. 3-l), together with split-spoon driving resistance, geological studies, and previous experience often will indicate potentially weak and compressible fine-grained foundation strata and thus the

Table 3-1. Laboratory Testing of Fine-Grained Cohesive Soils

Test	Remarks
Visual classification and water content determinations	On all samples
Atterberg limits	On representative samples of foundation deposits for correlation with shear or consolidation parameters, and borrow soils for comparison with natural water contents, or correlations with optimum water content and maximum densities
Permeability	Not required; soils can be assumed to be essentially impervious in seepage analyses
Consolidation	Generally performed on undisturbed foundation samples only where: a. Foundation clays are highly compressible b. Foundations under high levees are somewhat compressible c. Settlement of structures within levee systems must be accurately estimated Not generally performed on levee fill; instead use allowances for settlement within levees based on type of compaction. Sometimes satisfactory correlations of Atterberg limits with coefficient of consolidation can be used. Compression index can usually be estimated from water content.
Compaction	a. Required only for compacted or semicompacted levees b. Where embankment is to be fully compacted, perform standard 25-blow compaction tests

(Continued)

Table 3-1. (Continued)

Test	Remarks
Compaction (Continued)	c. Where embankment is to be semi-compacted, perform 15-blow compaction tests
Shear strength	a. Pocket penetrometer, laboratory vane, and miniature vane (Torvane) for rough estimates b. Unconfined compression tests on saturated foundation clays without joints or slickensides c. Q triaxial tests appropriate for foundation clays, as undrained strength generally governs stability d. R triaxial and S direct shear: Generally required only when levees are high and/or foundations are weak, or at locations where structures exist in levees e. Q, R, and S tests on fill materials compacted at appropriate water contents to densities resulting from the expected field compaction effort

Table 3-2. Laboratory Testing of Pervious Materials

Test	Remarks
Visual classification	Of all jar samples
In situ density determinations	Of Shelby-tube samples of foundation sands where liquefaction susceptibility must be evaluated
Relative density	Maximum and minimum density tests should be performed in seismically active areas to determine in situ relative densities of foundation sands and to establish density control of sand fills
Gradation	On representative foundation sands: a. For correlating grain-size parameters with permeability or shear strength b. For size and distribution classifications pertinent to liquefaction potential
Permeability	Not usually performed. Correlations of grain-size parameters with permeability or shear strength used. Where underseepage problems are serious, best guidance obtained by field pumping tests
Consolidation	Not usually necessary as consolidation under load is insignificant and occurs rapidly
Shear strength	For loading conditions other than dynamic, drained shear strength is appropriate. Conservative values of $\bar{\phi}$ can be assumed based on S tests on similar soils. In seismically active areas, cyclic triaxial tests may be performed

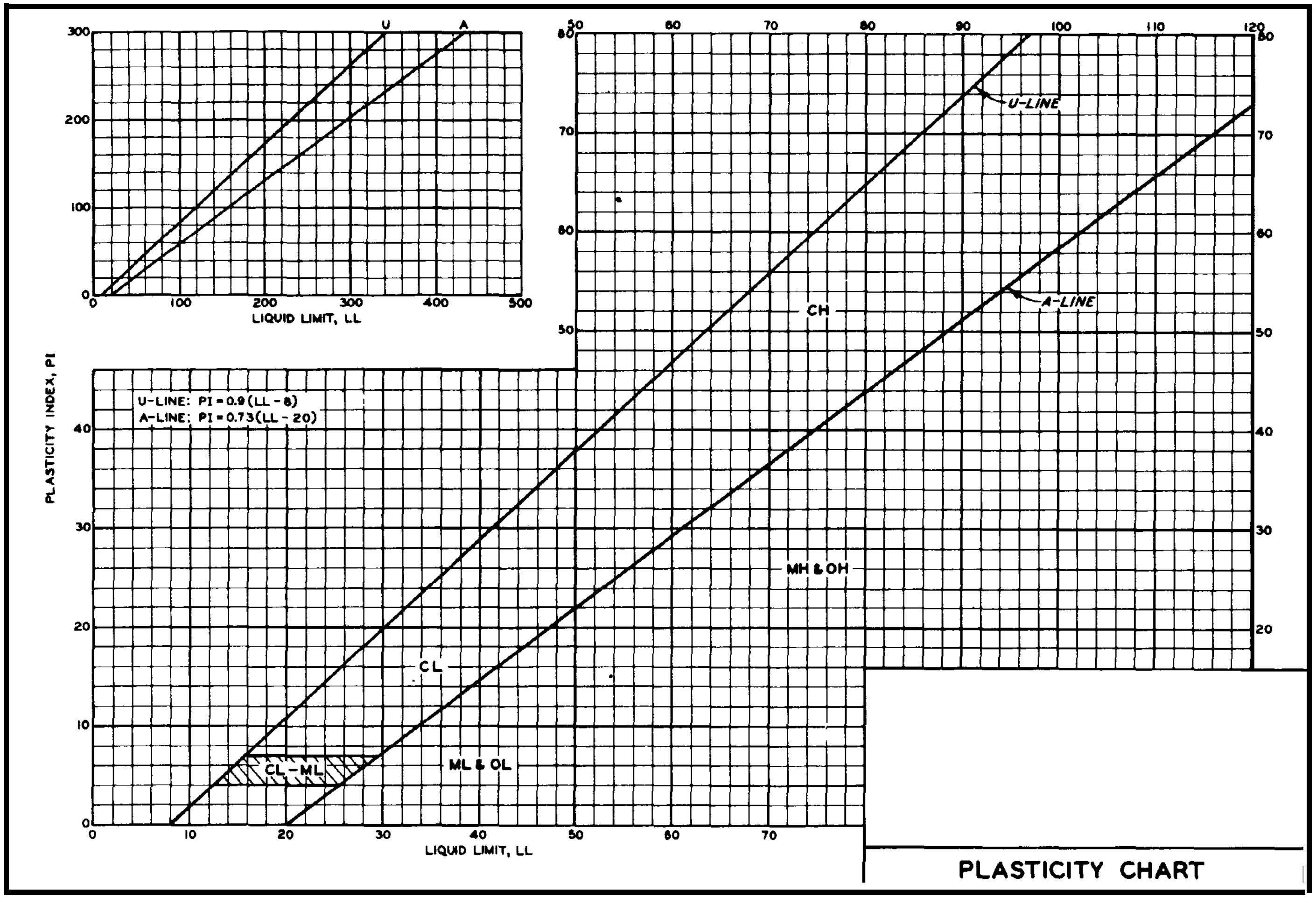


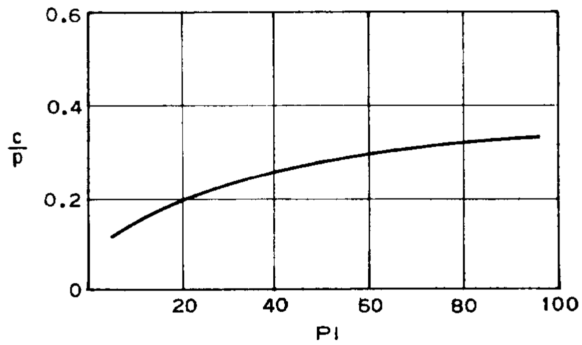
Figure 3-1. Plasticity chart (ENG Form 4334)

need for shear and perhaps consolidation tests. In some cases, in the design of low levees on familiar foundation deposits for example, correlations between Atterberg limits values and consolidation or shear strength characteristics may be all that is necessary to evaluate these characteristics. Examples of correlations among Atterberg limits values, natural water content, shear strength and consolidation characteristics are shown in figures 3-2 and 3-3. Correlations based on local soil types and which distinguish between normally and overconsolidated conditions are preferable. Such correlations may also be used to reduce the number of tests required for design of higher levees. As optimum water content may in some cases be correlated with Atterberg limits, comparisons of Atterberg limits and natural water contents of borrow soils as shown in figure 3-4 can indicate whether the borrow materials are suitable for obtaining adequate compaction.

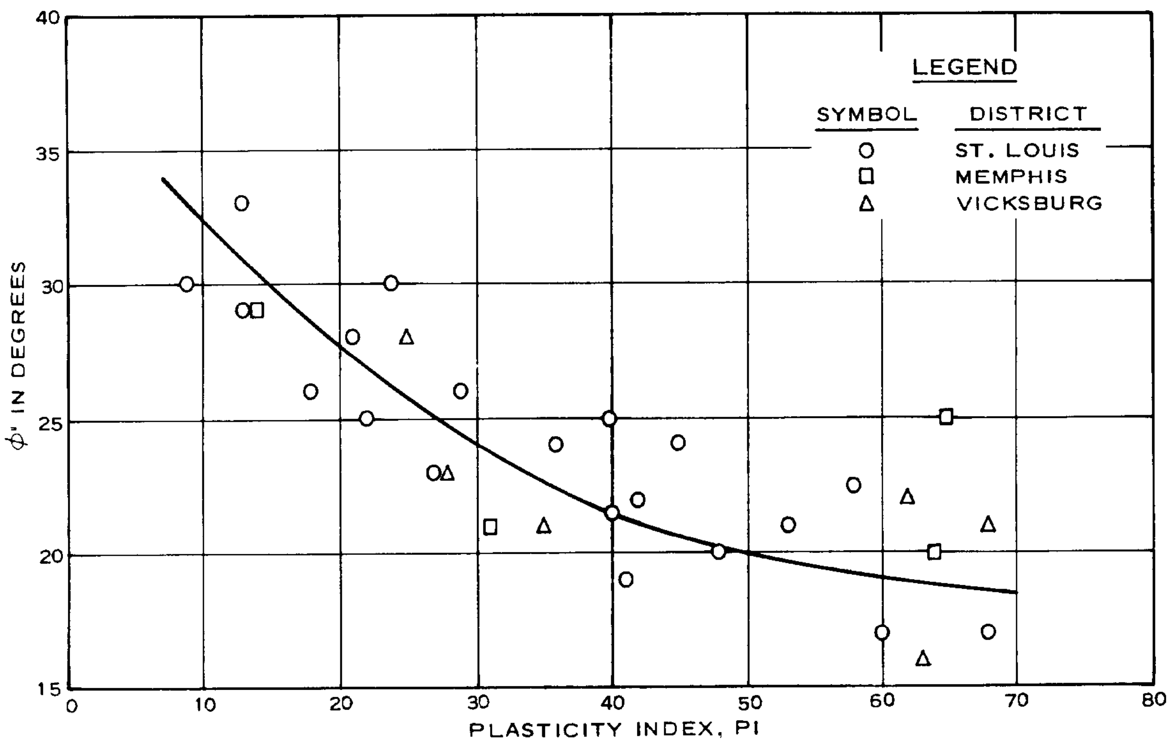
3-4. Shear Strength. Approximate shear strengths of fine-grained cohesive soils can be rapdily determined on undisturbed foundation samples, and occasionally on reasonably intact samples from disturbed drive sampling, using simple devices such as the pocket penetrometer, laboratory vane shear device, or the miniature vane shear device (Torvane). To establish the reliability of these tests, it is desirable to correlate them with unconfined compression tests. Unconfined compression tests are somewhat simpler to perform than Q triaxial compression tests, but test results exhibit more scatter. Unconfined compression tests are appropriate primarily for testing saturated clays which are not jointed or slickensided. Of the triaxial compression tests, the Q test is the one most commonly performed on foundation clays, since the in situ undrained shear strength generally controls embankment design on such soils. However, where embankments are high, stage construction is being considered, or important structures are located in a levee system, R triaxial compression tests and S direct shear tests should also be performed.

3-5. Consolidation. Consolidation tests are performed for those cases listed in table 3-1. In some locations correlations of liquid limit and natural water content with coefficient of consolidation, compression index, and coefficient of secondary compression can be used satisfactorily for making estimates of consolidation of foundation clays under load.

3-6. Permeability. Generally there is no need for laboratory permeability tests on fine-grained fill materials, nor on surface clays overlying pervious foundation deposits. In underseepage analyses, simplifying assumptions must be made relative to thickness and soil type of fine-grained surface blankets. Furthermore, animal burrows, root

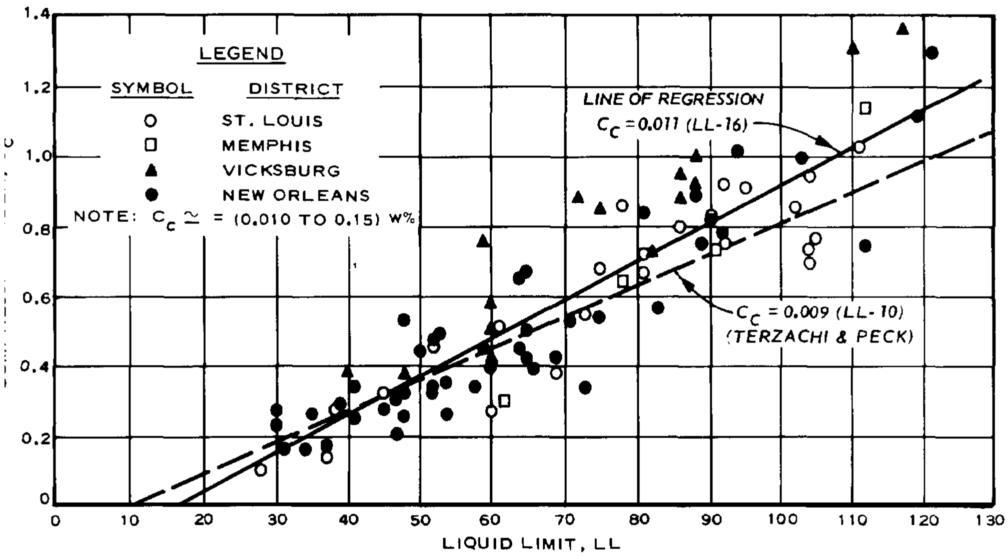


(a) c/p versus plasticity index for normally consolidated soils (after Bjerrum, ref. A-5a)

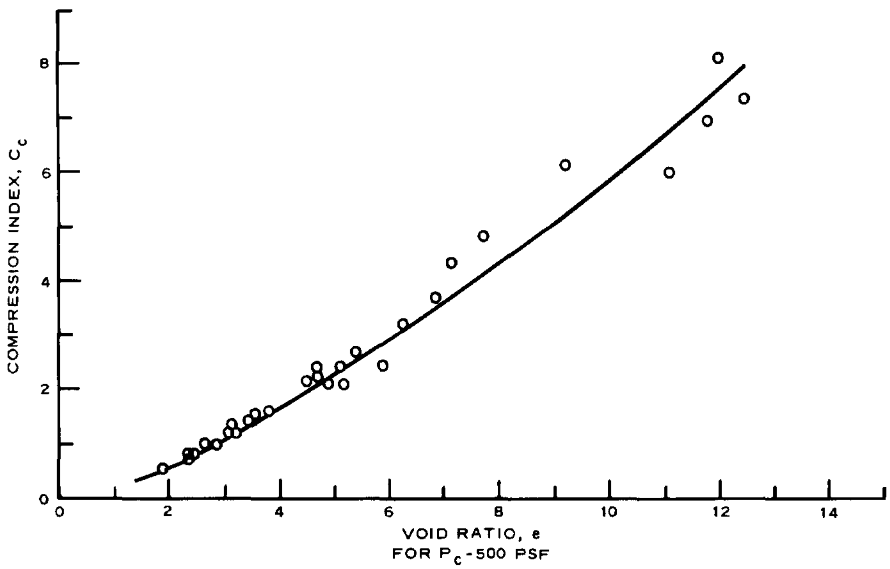


(b) Ø' versus plasticity index

Figure 3-2. Example correlations of strength characteristics for fine-grained soils

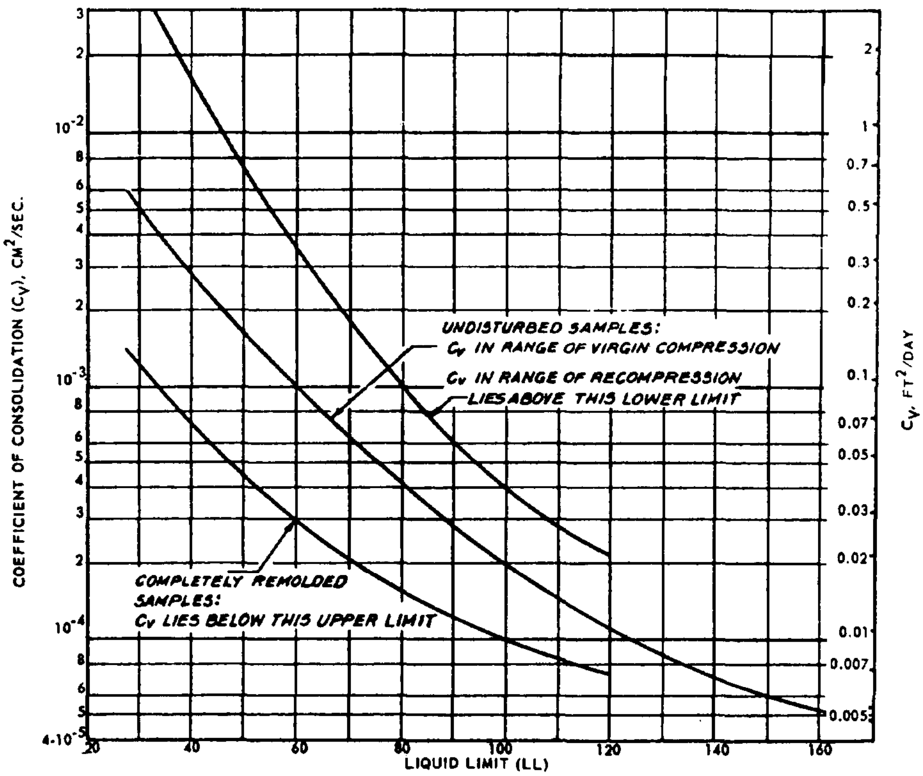


(a) Compression index versus liquid limit
for normally consolidated soils

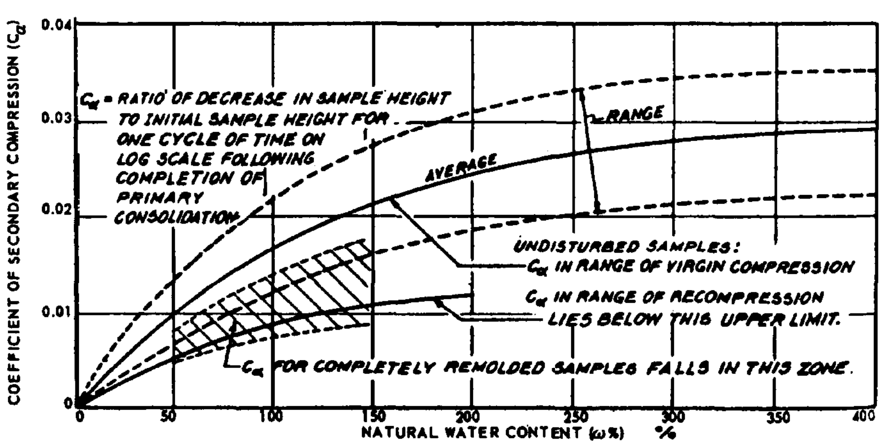


(b) Compression index versus initial
void ratio for tidal marsh

Figure 3-3. Example correlations for consolidation characteristics
of fine-grained soils (after Kapp, ref. A-5e)



(c) Coefficient of consolidation versus
liquid limit (from NAVFAC DM-7 ref. A-4)



(d) Coefficient of secondary compression versus
water content (from NAVFAC DM-7 ref. A-4)

Figure 3-3. (Continued)

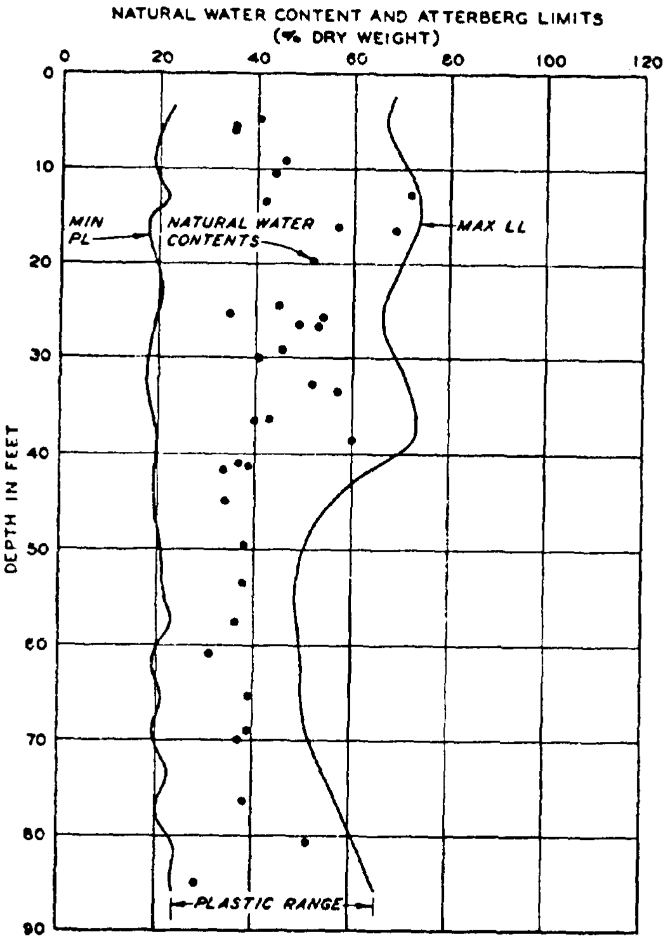


Figure 3-4. Comparisons of Atterberg limits and natural water contents

channels, and other discontinuities in surface blankets can significantly affect the overall effective permeability. Therefore, an average value of the coefficient of permeability based on the dominant soil type (Appendix B) is generally of sufficient accuracy for use in underseepage analyses, and laboratory tests are not essential.

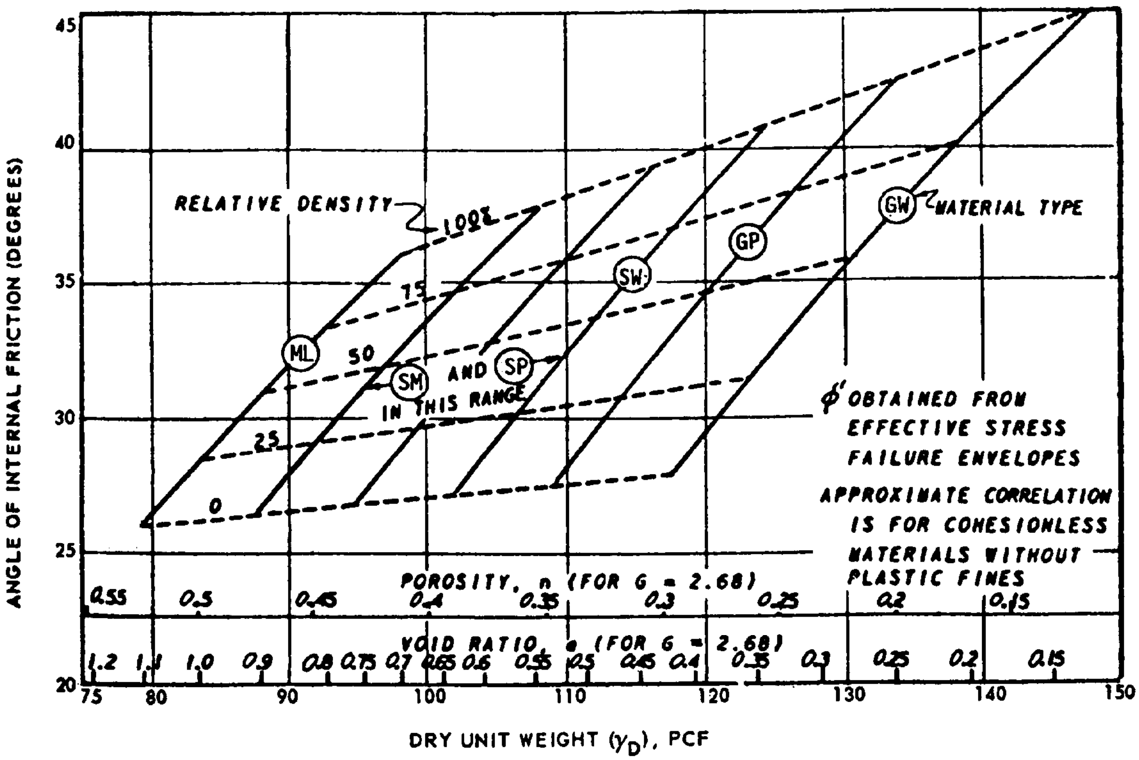
3-7. Compaction Tests. The type and number of compaction tests will be influenced by the method of construction and the variability of available borrow materials. The types of compaction tests required are summarized in table 3-1.

Section II. Coarse-Grained Soils

3-8. Shear Strength. When coarse-grained soils contain few fines, the consolidated drained shear strength is appropriate for use in all types of analyses. In most cases, conservative values of the angle of internal friction (ø) can be assumed from correlations such as those shown in figure 3-5, and no shear tests will be needed.

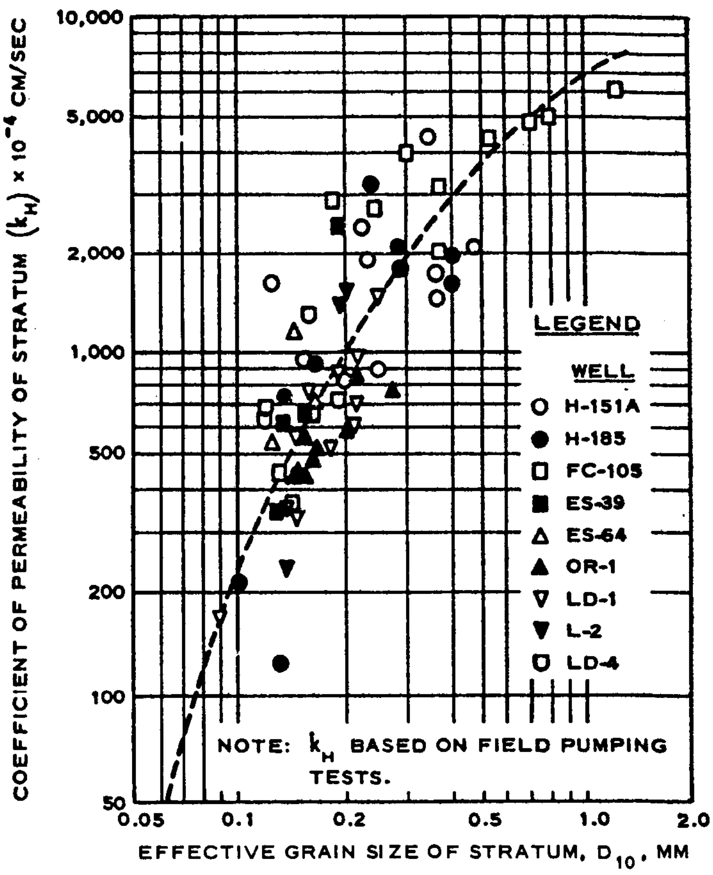
3-9. Permeability. To solve the problem of underseepage in levee foundations, reasonable estimates of permeability of pervious foundation deposits are required. However, because of difficulty and expense in obtaining undisturbed samples of sands and gravels, laboratory permeability tests are rarely performed on foundation sands. Instead, field pumping tests or correlations such as that of figure 3-5 developed between a grain-size parameter (such as D_{10}) and the coefficient of permeability, k , are generally utilized.

3-10. Density Testing of Pervious Fill. Maximum and minimum density tests on available pervious borrow materials should be performed in accordance with procedures described in EM 1110-2-1906 (ref. A-3a(7)) so that relative density requirements for pervious fills may be checked in the field when required by the specifications.



(a) Angle of internal friction versus unit weight (from NAVFAC DM-7 ref. A-4)

Figure 3-5. Example correlations for properties of coarse-grained soils



(b) Effective grain size, D_{10} , versus coefficient of permeability, k_H (from WES TM No. 3-424, ref. A-3b(2)

Figure 3-5. (Continued)

CHAPTER 4

BORROW AREAS

4-1. General. In the past borrow areas were selected largely on the basis of material types and quantities and haul distances. Today, borrow areas receive much more attention and must be carefully planned and designed, because of considerations such as environmental aspects, increasing land values, and greater recognition of the effects of borrow areas with respect to underseepage, uplift pressures, overall levee stability, and erosion. The following paragraphs discuss some factors involved in locating and using borrow areas.

4-2. Available Borrow Material.

a. Material Type. Almost any soil is suitable for constructing levees, except very wet, fine-grained soils or highly organic soils. In some cases, though, even these soils must be used. Accessibility and proximity are often controlling factors in selecting borrow areas, although the availability of better borrow materials involving somewhat longer haul distances may sometimes lead to the rejection of poorer but more readily available borrow.

b. Natural Water Content. Where compacted levees are planned, it is necessary to obtain borrow material with water content low enough to allow placement and adequate compaction. The cost of drying borrow material to suitable water contents can be very high, in many cases exceeding the cost of longer haul distances to obtain material that can be placed without drying. Borrow soils undergo seasonal water content variations; hence water content data should be based on samples obtained from borrow areas in that season of the year when levee construction is planned. Possible variation of water contents during the construction season should also be considered.

4-3. General Layout. Generally, the most economical borrow scheme is to establish pits parallel and adjacent to the levee. If a levee is adjacent to required channel excavation, levee construction can often utilize material from channel excavation. Large centralized borrow areas are normally established only for the construction of urban levees, where adjacent borrow areas are unavailable. Long, shallow borrow areas along the levee alignment are more suitable, not only because of the shorter haul distance involved, but also because they better satisfy environmental considerations.

a. Location. Borrow area locations on the river side of a levee are preferable as borrow pits. Borrow area locations within the protected area are less desirable environmentally, as well as generally being more expensive. Riverside borrow locations in some areas will be filled eventually by siltation, thereby obliterating the man-made changes in the landscape. While riverside borrow is generally preferable, required landside borrow from ponding areas, ditches, and other excavations should be used wherever possible. A berm should be left in place between the levee toe and the near edge of the borrow area. The berm width depends primarily on foundation conditions, levee height, and amount of land available. Its width should be established by seepage analyses where pervious foundation material is close to the bottom of the borrow pit and by stability analyses where the excavation slope is near the levee. Generally, the width of this berm should be about 2-1/2 times greater for landside berms than for riverside berms. Minimum berm widths used frequently in the past are 40 ft riverside and 100 ft landside, but berm widths should be the maximum possible since riverside borrow areas increase the severity of underseepage effects. In borrow area excavation, an adequate thickness of impervious cover should be left over underlying pervious material. For riverside pits a minimum of 2 ft of cover should be left in place, and for landside pits the cover thickness should be adequate to prevent the formation of boils under expected hydraulic heads. Topsoil from borrow and levee foundation stripping can be stockpiled and spread over the excavated area after borrow excavation has been completed. This reinforces the impervious cover and provides a good base for vegetative growth.

b. Size and Shape. It is generally preferable to have riverside borrow areas "wide and shallow" as opposed to "narrow and deep." While this may require extra right-of-way and a longer haul distance, the benefits derived from improved underseepage, hydraulic, and environmental conditions usually outweigh the extra cost. In computing required fill quantities, a shrinkage factor of at least 25 percent should be applied (i.e., borrow area volumes should be at least 125 percent of the levee cross-section volume). This will allow for material shrinkage, and hauling and other losses. Right-of-way requirements should be established about 15 to 20 ft beyond the top of the planned outer slope of the borrow pit. This extra right-of-way will allow for flattening or caving of the borrow slopes, and can provide maintenance borrow if needed later.

4-4. Design and Utilization.

a. Slopes. Excavation slopes of borrow areas should be designed to assure stability. This is particularly important for slopes adjacent to the levee but could also be important for any slope whose top is near the right-of-way limits. Borrow area slopes must also be flat enough to allow mowing, if required. Also, where landside pits are to be placed back into cultivation, changes in grade must be gentle enough to allow farm equipment to operate safely. The slopes of the upstream and downstream ends of riverside pits should be flat enough to avoid erosion when subjected to flow at high water stages.

b. Depths. Depths to which borrow areas are excavated will depend upon factors such as (1) groundwater elevation, (2) changes at depth to undesirable material, (3) preservation of adequate thickness of riverside blanket, and (4) environmental considerations.

c. Foreshore. The foreshore is that area between the riverside edge of the borrow area and the riverbank as shown in figure 4-1. If a

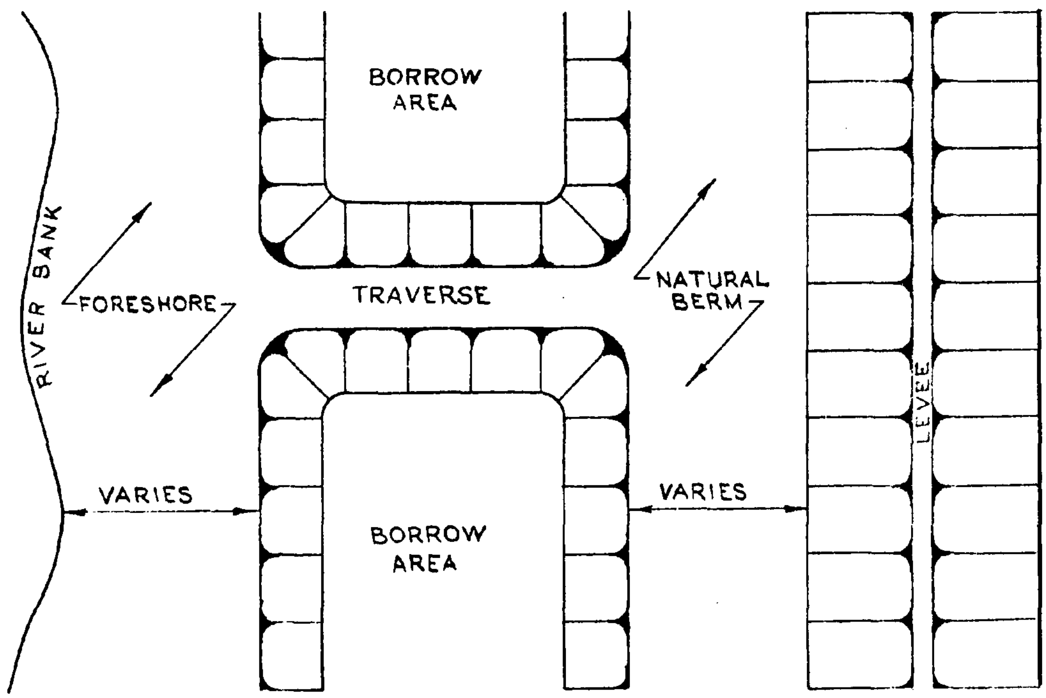


Figure 4-1. Plan of typical levee and borrow areas with traverse and foreshore

foreshore is specified (i.e., the borrow excavation is not be cut into the riverbank), it should have a substantial width, say 200 ft or more, to help prevent migration of the river channel into the borrow area.

d. Traverse. A traverse is an unexcavated zone left in place at intervals across the borrow area (figure 4-1). Traverses provide roadways across the borrow area, provide foundations for transmission towers and utility lines, prevent less than bank-full flows from coursing unchecked through the borrow area, and encourage material deposition in the borrow area during high water. Experience has shown that when traverses are overtopped or breached, severe scour damage can result unless proper measures are taken in their design. Traverse heights should be kept as low as possible above the bottom of the pit when they will be used primarily as haul roads. In all cases, flat downstream slopes (on the order of 1V and 6H to 10H) should be specified to minimize scour from overtopping. If the traverse carries a utility line or a public road, even flatter slopes and possibly stone protection should be considered.

e. Drainage. Riverside borrow areas should be so located and excavated that they will fill slowly on a rising river and drain fully on a falling river. This will minimize scour in the pit when overbank river stages occur, promote the growth of vegetation, and encourage silting where reclamation is possible. The bottom of riverside pits should be sloped to drain away from the levee. Culvert pipes should be provided through traverses, and foreshore areas should be ditched through to the river as needed for proper drainage. Landside pits should be sloped to drain away from or parallel to the levee with ditches provided-as necessary to outlet points. Gravity outlets or pump stations should be located so as to minimize lengths of flow paths within the pit area.

f. Flow Conditions. To avoid damage from confined or restricted flow through the riverside borrow areas, obstructions or impediments to smooth and uniform flow should be removed if possible, or else protective measures must be taken. Riverside borrow areas should be made as uniform in width and grade as possible, avoiding abrupt changes. Removal of obstructions that could cause concentrated flow includes degradation of old levee remnants and of narrow high ground ridges beyond the borrow area, as well as removal of timber from traverses and from foreshore areas immediately adjacent to the borrow area. Obstructions to flow that cannot be removed include transmission towers, bridge piers, and other permanent structures near the levee. In such areas, stone protection should be provided for the levee or borrow area slopes if scour damage is considered probable.

g. Environmental Aspects. The treatment of borrow areas after excavation to satisfy aesthetic and environmental considerations has in the last few years become standard operating practice. The extent of treatment will vary according to the type and location of a project. Generally, projects near urban areas or where recreational areas are to be developed will require more elaborate treatment than those in sparsely populated agricultural areas. Minimum treatment should include proper drainage, topographic smoothing, and the promotion of conditions conducive to vegetative growth. Insofar as possible, borrow areas should be planted to conform to the surrounding landscape. Stands of trees should be left remaining on landside borrow areas if at all possible, and excavation procedures should not leave holes, trenches, or abrupt slopes. Restoration of vegetative growth is important for both landside and riverside pits as it is not only pleasing aesthetically but serves as protection against erosion. Willow trees can aid considerably in drying out boggy areas. Riverside pits should not be excavated so deep that restored grass cover will be drowned out by long submergence. Agencies responsible for maintenance of completed levees should be encouraged to plant and maintain vegetation, including timber, in the borrow areas. It is desirable that riverside borrow pits be filled in by natural processes, and frequent cultivation of these areas should be discouraged or prohibited, if possible, until this has been achieved.

h. Clearing, Grubbing, and Stripping. Borrow areas should be cleared and grubbed to the extent needed to obtain fill material free of objectionable matter, such as trees, brush, vegetation, stumps, and roots. Subareas within borrow areas may be specified to remain untouched to preserve standing trees and existing vegetation. Topsoil with low vegetative cover may be stripped and stockpiled for later placement on outer landside slopes of levees and seepage berms.

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, construction, and maintenance guidance is given in Appendixes B, C, and D. Turnbull and Mansur (ref. A-5f and A-5g) have proposed control measures for underseepage also.

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 40 ft. Steel sheet piling is not entirely watertight due to leakage at the interlocks but can significantly reduce the possiblity 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 (ref. A-3b(4) and A-3b(5)) 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.

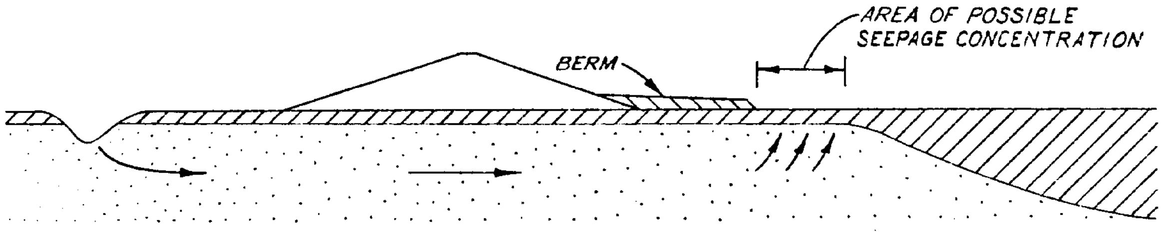
5-3. Riverside Blankets. Levees are frequently situated on foundations having natural covers of relatively fine-grained impervious to semipervious soils overlying pervious sands and gravels. These surface strata constitute impervious or semipervious blankets when considered in connection with seepage 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, 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.

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



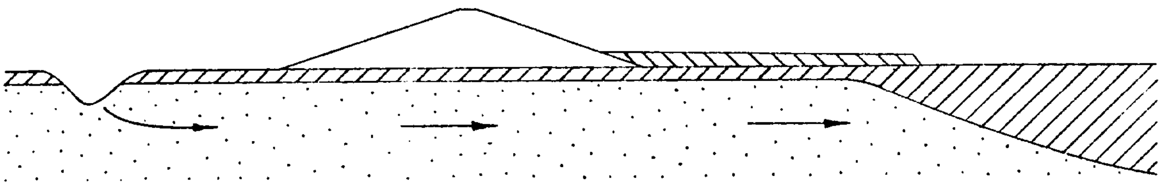


Figure 5-1. Example of incorrect and correct berm length according to existing foundation conditions

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 or 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.

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

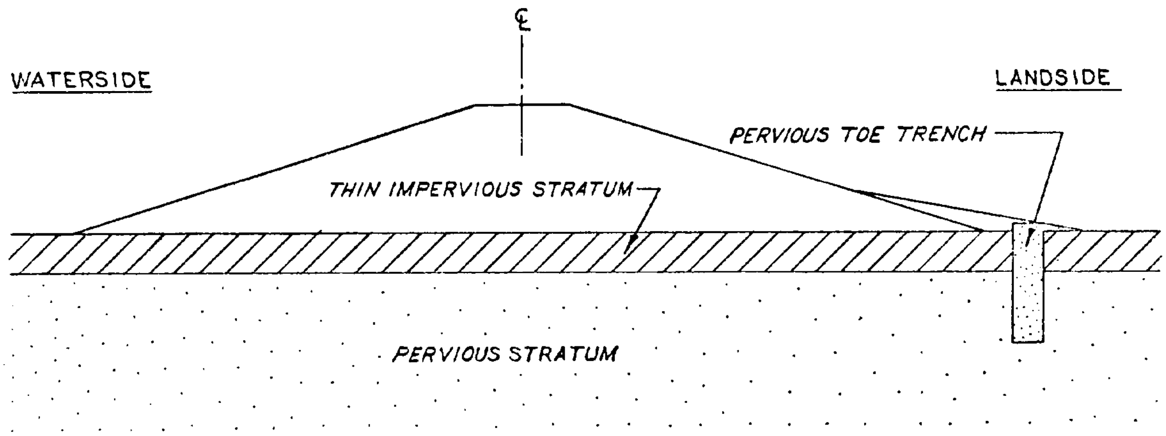


Figure 5-2. Typical partially penetrating pervious toe trench

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 are often 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.

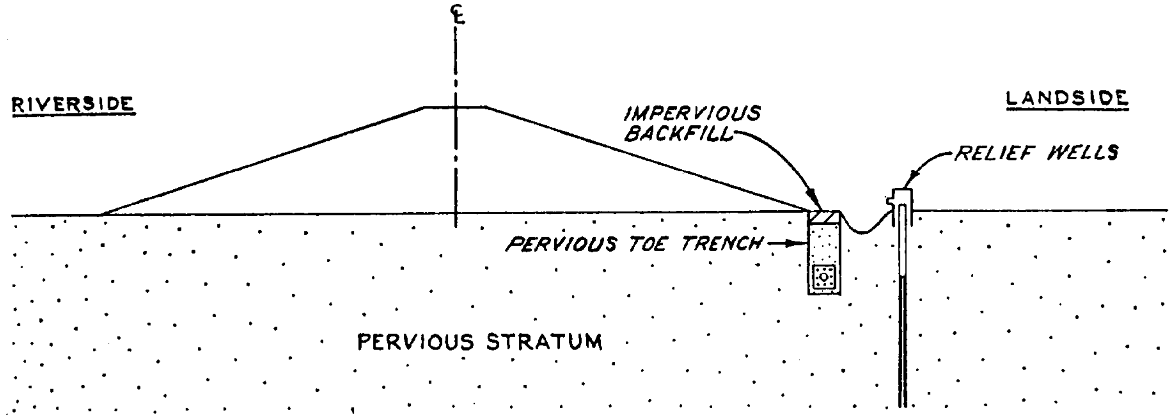


Figure 5-3. Typical pervious toe trench with collector pipe (Figure 5-6 shows trench details)

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.

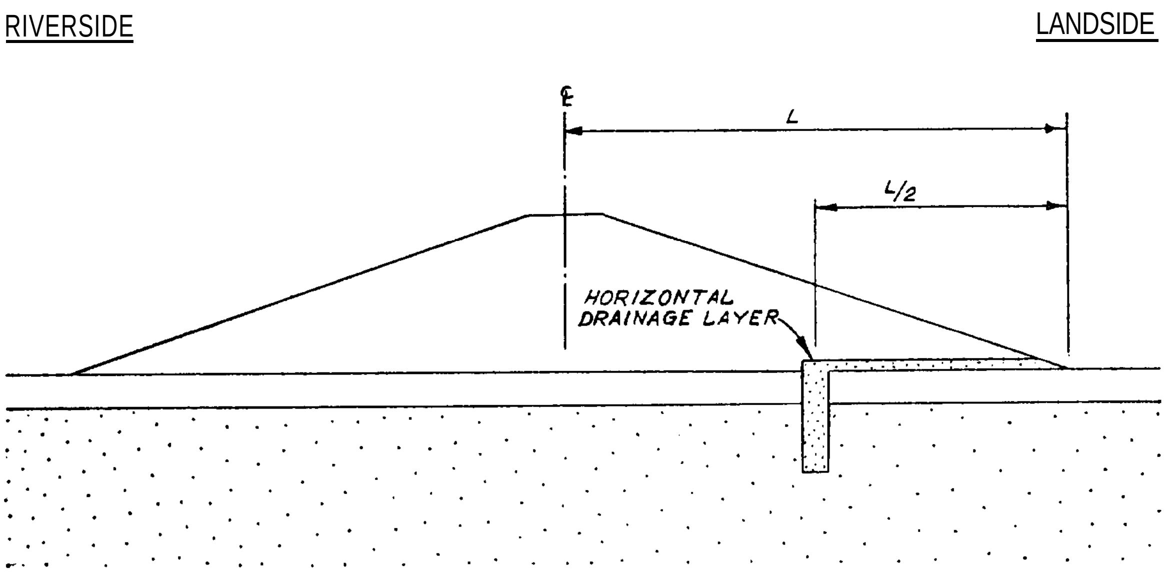


Figure 5-4. Pervious toe trench located beneath landward slope

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 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 (fig. 5-5), which is a vibrating-plate type of compactor specially made to fit on the boom of a backhoe, has apparently performed satisfactorily.

d. Backfill. The sand backfill for trenches must be designed as a filter material in accordance with criteria given in Appendix E. If a collector pipe is used, the pipe should be surrounded by about a 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.

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 too deep or too thick to be penetrated by cutoffs or toe drains. 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 the 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 other seepage control

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

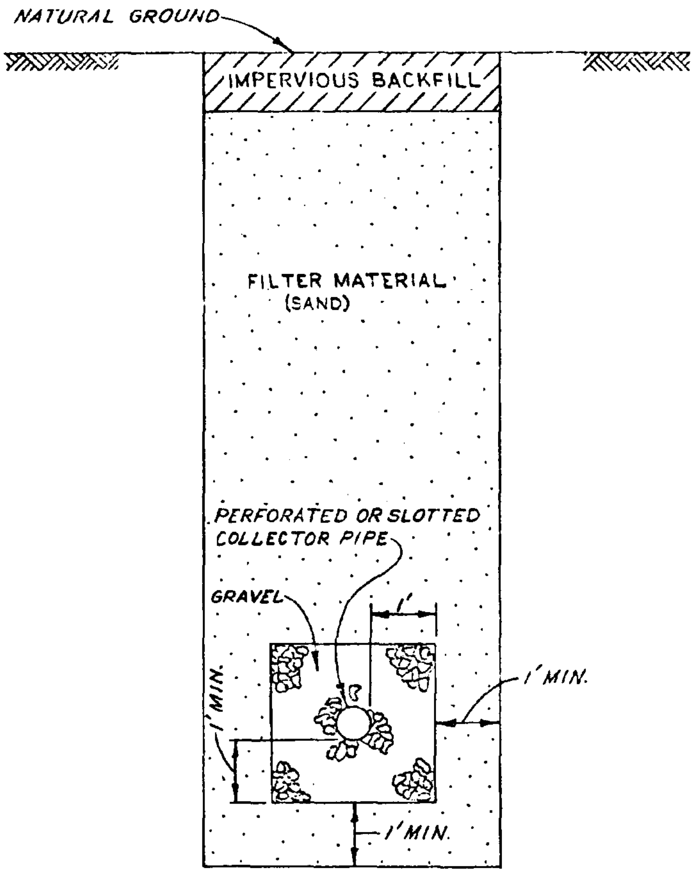


Figure 5-6. Pervious toe trench with collector pipe

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 and penetration is contained in EM 1110-2-1905 (ref. A-3a(6)), and U. S. Army Engineer Waterways Experiment Station TM No. 3-424 (ref. A-3b(2)). 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 well spacing for a given reach of levee has been determined, the location of each well should be established in the office and field to ensure that the wells will be located at critical seepage points and will fit natural topographic features.

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, wood and stainless steel are the most widely used, primarily because of their corrosion-resistant properties. Figure 5-7 shows a typical well using a wooden riser pipe and screen. Wood will not deteriorate as long as it is permanently submerged but will deteriorate when subjected to alternate wetting and drying. For this reason that portion of the riser above the lowest expected water table should be surrounded with concrete.

(2) Filter. The filter that surrounds the screen must be designed in accordance with criteria given in Appendix E 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 6 in. of filter material should surround the screen and the filter should extend a minimum of 2 ft above the top and 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 appurtances. 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

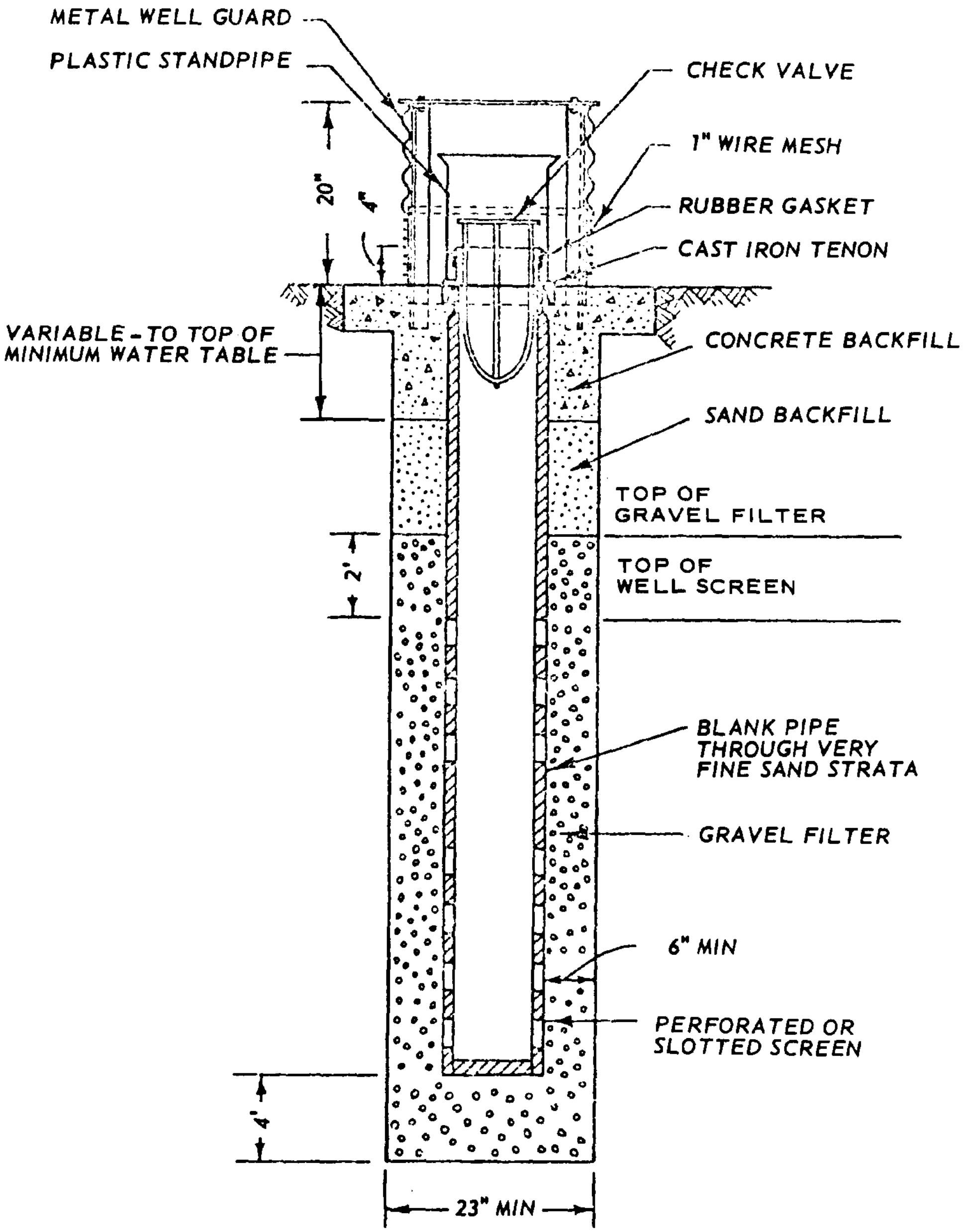


Figure 5-7. Typical relief well

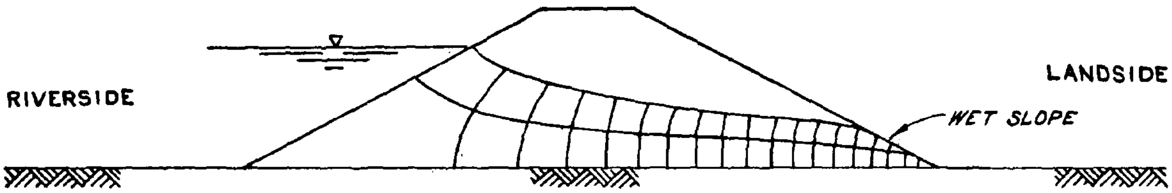
valve, rubber gasket, and plastic standpipe, as shown in figure 5-7, can be installed on each well. To safeguard against vandalism, accidental damage and the entrance of debris, the tops of the wells should be provided with a metal screen or flat gate. The elevation of the top of any protective standpipes 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 Appendix D of this manual.

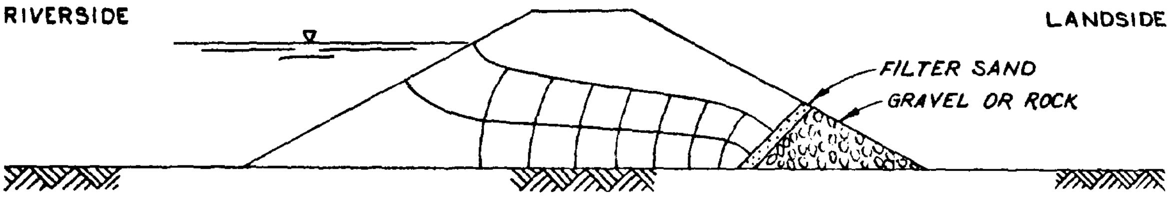
Section II. Seepage Through Embankments

5-7. General. Should through seepage in an embankment emerge on the landside slope (fig. 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. 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, 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.

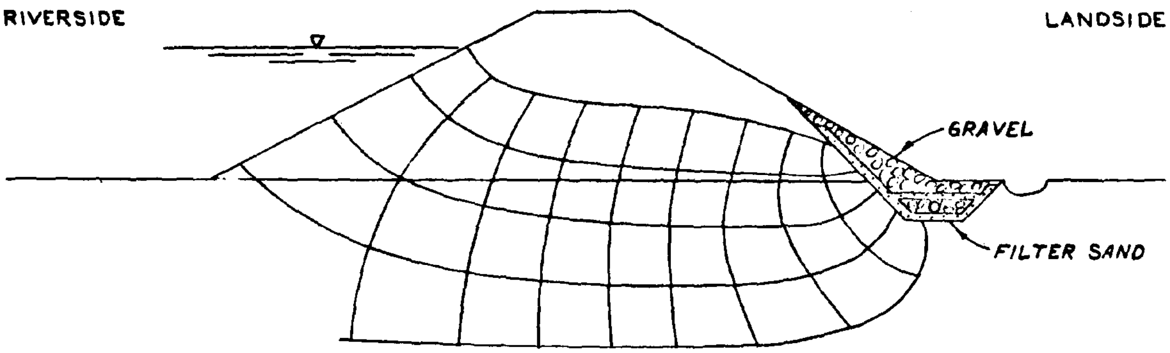
5-8. Pervious Toe Drain. A pervious toe (fig. 5-8b) will provide a ready exit for seepage through the embankment and can lower the phreatic surface sufficiently so that no seepage will emerge on the landside slope. A pervious toe can also be combined with partially penetrating toe trenches, which have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



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



b. Section with pervious toe

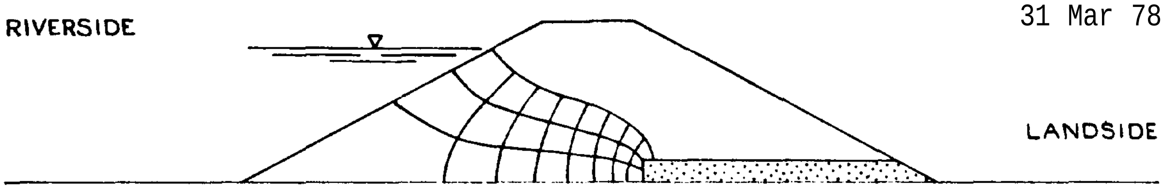


c. Pervious toe combined with partially penetrating toe trench

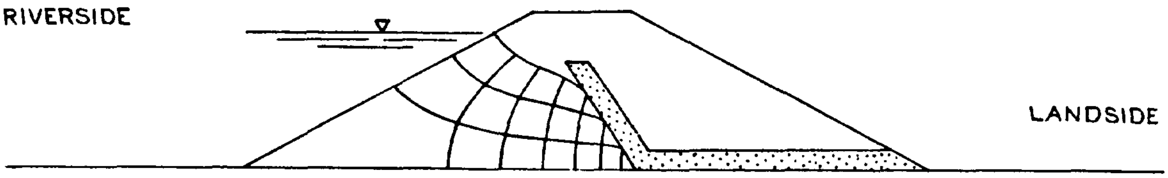
Figure 5-8. Embankment with through seepage

5-9. <u>Horizontal Drainage Layers.</u> 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 located some distance under the embankment as shown previously in figure 5-4.

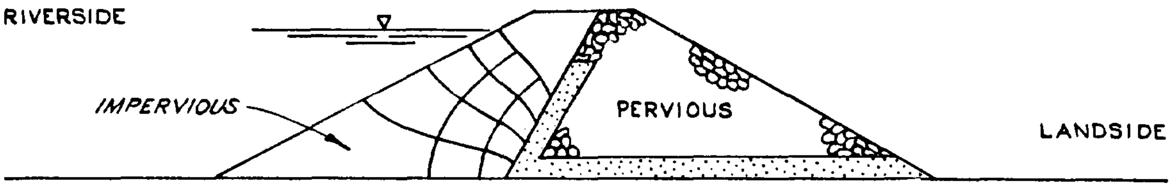
5-10. <u>Inclined Drainage Layers.</u> An inclined drainage layer as shown in figure 5-9b is one of the more positive means of controlling internal



a. Horizontal drainage layer



b. Inclined drainage layer-homogeneous embankment



c. Inclined drainage layer-zoned embankment

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

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 (fig. 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 E). 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 E for filter design. Horizontal drainage layers should have a minimum thickness of 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. 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.

CHAPTER 6

SLOPE DESIGN AND SETTLEMENT

Section I. Embankment Stability

6-1. Embankment Geometry.

a. Slopes. 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, and operation, and slope protection criteria control the selection of levee slopes. When there is concern about the adequacy of available embankment materials or foundation conditions, embankment design requires detailed analysis.

(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 and select material must be obtained and compacted properly to obtain a stable section.

(2) Ease of construction. A 1V on 2H slope is generally accepted as the steepest slope that will permit machine placement of riprap and also the steepest slopes that will ensure stability of the riprap blanket.

(3) Maintenance. A 2V on 5H slope is the steepest slope that can be conveniently traversed with conventional mowing equipment.

(4) Slope protection. Riverside slopes flatter than those required for stability may have to be specified to provide protection from damage by wave action.

(5) Floodfighting. Some districts specify a somewhat flatter upper landside slope than necessary for stability to provide a ready source of additional material should emergency raising of the levee grade become necessary.

b. Crown Elevation. The levee grade is established by the design flood profile computations plus allowances for settlement and freeboard. The purpose of a freeboard allowance is to provide for those factors that cannot be rationally accounted for in design flood profile computations. Freeboard allowances for levees have not been strictly standardized but minimum values most commonly used are 2 ft for agricultural levees and 3 ft for urban levees. Experience in actual flood and results of hydraulic model studies have indicated the need for additional freeboard in the following locations.

(1) The upstream end of a levee segment. Overtopping of levees near their upstream ends during the rising stage of a flood when the river level at the downstream portion was still a few feet below levee crest generally has caused greater damage than in the downstream reach as a result of higher initial current velocities and greater depths and durations of flooding. An additional freeboard of 0.5 ft is commonly specified at the upstream end tapering to zero at the downstream end.

(2) Drainage structure locations. To provide additional protection against overtopping in the vicinity of structures in levees, additional freeboard of 1 ft is commonly specified to extend 100 ft on either side of a structure.

(3) Near bridges and other constricted areas. Overtopping can occur at these areas because of debris accumulations. An additional 1 ft of freeboard at these areas extending 50 to 100 ft either side should be specified.

(4) Wave action. An additional freeboard allowance may be needed to protect against wave action during design flood stage if severe wave action is likely.

c. Crown Width. The width of the levee crown depends primarily on roadway requirements. To provide access for normal maintenance operations and floodfighting operations, minimum widths of 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. Where the levee crown is to be used as a higher class road, its width is usually established by the responsible agency.

* 6-2. Standard Levee Sections and Minimum Levee Section.*

a. 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 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.

* b. 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. *

* c. The minimum levee section shall have a crown width of at least 10 feet and a side slope flatter than or equal to one (vertical) on two (horizontal), 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, and inspection. *

6-3. Effects of Fill Characteristics and Compaction.

a. 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 as described in paragraph 1-5.

b. Hydraulic Fill.

(1) Hydraulic fill consisting largely of pervious sands can result in satisfactory levees. Sand levees can be built with one or two end-discharge or bottom-discharge pipes. One pipe should be at grade along

the centerline and the second pipe, if used, should be near the landside toe so that increased amounts of coarse material will be deposited in this area. Tracked or rubber-tired dozers or front-end loaders are used to move the sand to shape the levee slopes. Gradation of the sands should generally be controlled so that at least 70 percent passes the No. 4 sieve and no more than about 5 percent passes the No. 200 sieve. In agricultural areas, slopes are generally 1V on 4H riverside and 1V on 5H landside. With these slopes and for levees up to 20 ft high, special provisions for control of through seepage are not required. When initially exposed to floodwaters, seepage through the levee may be large, but with a 1V on 5H landside slope, surface erosion is relatively minor and in a few days river sediments deposited on the upstream face will tend to significantly reduce the flow.

(2) In urban areas, space is generally limited and construction procedures and the levee section may be altered. Normally the hydraulicked sand will be stockpiled and then moved to the site with trucks or scrapers. Hauled material may be placed in lifts and compacted with tracked or rubber-tired rollers to about 95 percent of standard effort density. When the landside slope is steeper than 1V on 5H, either an impervious blanket on the riverside slope or an underdrain to control seepage is required; the choice depends on cost and availability of suitable material. The impervious blanket is normally 10 to 12 ft wide which will produce an effective thickness of about 3 to 4 ft normal to the slope surface. Riverside slopes as steep as 1V on 3H or greater are commonly used; slopes as steep as 1V on 3H are normally protected with sod, or riprap if river currents as great as 3 fps are anticipated. Landside slopes generally do not require protection, although native grass will normally appear naturally; sometimes 6 in. of gravel or 12 in. of top soil and sod will be used to reduce dust and improve appearance.

(3) Hydraulic fill consisting of fine-grained soils is usually restricted to construction of stability or seepage berms with a central levee zone constructed of hauled borrow by the semicompacted method.

(4) There may be situations in which it may be desirable to utilize one type of construction for the central (and therefore higher) portion of the levee, and utilize less desirable material and lesser compaction quality in the flanking zones or berms. For example, if material is in short supply and space permits flat levee slopes, semicompacted fill may be specified for the central portion and uncompacted fill, utilizing poorer quality borrow or clay hydraulic fill, for the outer zones.

Section II. Stability Analyses

6-4. 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. Analyses assuming a circular arc failure surface are made either using the Modified Swedish Method described in EM 1110-2-1902 (ref. A-3a(4)) (which considers forces on the sides of slices), or the simpler Swedish Slide Method (Method of Slices), described in Appendix D of U. S. Army Engineer Waterways Experiment Station TM No. 3-777 (ref. A-3b(1)) (which assumes that side forces are equal in magnitude and parallel to the base of each slice). The wedge method for planar sliding surfaces is described in EM 1110-2-1902 (ref. A-3a(4)). The wedge method is appropriate for weak foundations requiring flat levee slopes or for an otherwise strong foundation containing a thin weak stratum. Computer programs are available for these analyses, with the various loading cases described in EM 1110-2-1902 (ref. A-3a(4)), so the effort of making such analyses is greatly reduced, and primary attention can be devoted to the more important problems of defining the shear strengths, unit weights, geometry, and limits of possible sliding surfaces.

6-5. 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, critical flood stage; Case IV, steady seepage from full flood stage, fully developed phreatic surface; Case V, steady seepage from full flood stage, partially developed phreatic surface; Case VI, earthquake. Each case is discussed briefly in the following paragraphs and the applicable type of design shear strength is given. For more detailed information on applicable shear strengths, methods of analysis, and assumptions made for each case refer to EM 1110-2-1902 (ref. A-3a(4)).

a. 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 conditions is applicable to both the riverside and landside slopes.

b. 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. Design shear strengths of impervious soils for this case should be based on the minimum of the combined S and R (consolidated-undrained) envelopes. For free-draining cohesionless soils the S envelope alone should be used.

c. Case III - Critical Flood Stage. This case refers to the condition whereby some intermediate prolonged flood stage saturates the embankment and a condition of steady seepage is established. This case is the same as the partial pool case for earth dams as given in EM 1110-2-1902 (ref. A-3a(4)), and the analysis is the same as is described therein. The design shear strength of impervious soils should correspond to a strength envelope midway between the R and S envelopes where the S strength is greater than the R strength and to the S envelope where the S strength is less than the R strength. The design strength of free-draining cohesionless material should correspond to the S envelope.

d. Case IV - 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 downstream slope stability. Design shear strengths should be based on the same envelopes as previously described for Case III.

e. Case V - Steady Seepage from Full Flood Stage (Partially Developed Phreatic Surface). This case is essentially the same as Case IV except that the flood stage remains on the embankment long enough to cause only partial saturation of the embankment therefore resulting in a steady seepage condition over only a portion of the embankment. This case does require an estimate of how much of the embankment is subjected to steady seepage (i.e. determining the location of the partially developed phreatic surface). This estimate should be based on (1) duration of flood stage, (2) permeability and effective porosity of the embankment material, and (3) embankment geometry and zonation. If an embankment is analyzed for Case IV, then it need not be analyzed for this case and vice versa. However, if this case is analyzed in lieu of Case IV it must be demonstrated that Case IV cannot occur.

f. Case VI - Earthquake. Earthquake loadings are not normally considered in analyzing the stability of levee 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.

6-6. 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 6-1.

Table 6-1. Minimum Factors of Safety - Levee Slope Stability

Case No.[a]	Design Condition	Slope Analyzed	Shear Strength	Minimum Factor of Safety
I(I)	End of construction	Riverside and landside[b]	Q or S[c]	1.3
II(II)	Sudden drawdown	Riverside	S where < R R where < S[d]	1.0
III(IV)	Intermediate river stage	Riverside	S where < R $\frac{R+S}{2}$ where R < S[d]	1.4
IV(V)	Steady seepage from full flood stage	Landside	S where < R $\frac{R+S}{2}$ where R < S[d]	1.4
IV(VII)	Earthquake: Cases I, III, and IV with seismic loading	Riverside and landside	e	1.0

[a] Numbers in parentheses are corresponding cases described in paragraph 1-1x of EM 1110-2-1902 (ref. A-3a(4)).
[b] high water can occur while this case applies, the additional increase in driving forces due to the water must be included in analyzing the landside slope.
[c] In zones where no excess pore water pressures are anticipated, use S strength.
[d] Composite shear strength envelope.
[e] Use shear strength applicable for case analyzed.

6-7. Measures to Increase Stability. Means for improving weak and compressible foundations to enable stable embankments to be constructed thereon are discussed in Chapter 7. Methods of improving embankment stability by changes in embankment section are presented in the following paragraphs.

a. 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).

b. 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. Also, thick landside berms can often serve as a source of material for emergency repairs to the main levee embankment.

6-8. 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.

Section III. Settlement

6-9. 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.

6-10. Settlement Analyses. Settlement estimates can be made by theoretical analysis as set forth in EM 1110-2-1904 (ref. A-3a(5)). 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.

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 semi-pervious 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.

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

Category	Construction Method	Use
I. Compacted	Specification of: _a_. Water content range with respect to standard effort optimum water content _b_. Loose lift thickness _c_. Compaction equipment (sheepsfoot or rubber-tired rollers) _d_. Number of passes to attain a given percent compaction based on standard maximum density	Provides embankment section occupying minimum space. Provides strong embankments of low compressibility needed adjacent to concrete structures or forming parts of highway systems. Requires strong foundation of low compressibility and availability of borrow materials with natural water contents reasonably close to specified ranges.
II. Semicompacted	Compaction of fill materials at their natural water content (i.e., no water content control). Placed in thicker lifts than Category I (about 12 in.) and compacted either by controlled movement of hauling and spreading equipment 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 require 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. _b_. Hydraulic fill by dredge, often from channel excavation.	Levees infrequently constructed today using method _a_ except for temporary emergency. Both methods are used for construction of landside and riverside berms. Method _b_ is used in some areas to build the entire levee section. Construction results in very flat slopes, with large space requirements.

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 1-1/2 in. in diameter within the levee foundation area should be removed to a depth of 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 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 is determined by local conditions and normally varies from 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, disposal is usually accomplished by burial in suitable locations near the project such as old sloughs, ditches, and depressions outside the limits of the embankment foundation but within project rights-of-way. Debris may also be stockpiled for later burial in excavated borrow areas. Debris should never be placed in areas where it may be carried away by streamflow or where it blocks drainage of an area. After disposal, the debris should be covered with at least 3 ft of earth and a vegetative cover established.

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 waterside toe of sand levees so as to connect with waterside impervious facings. While dimensions will vary with soil conditions and embankment configurations, the trench should have 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 6 ft except for embankment heights less than 6 ft, in which case the minimum depth should equal the embankment height. In some cases they can be deepened slightly to reach impervious soil, thereby eliminating underseepage problems. Side slopes should normally not be steeper than 1V on lH, with flatter slopes if needed for stability. Backfill should be placed only after a careful inspection of the excavated trench to ensure that through-going potential seepage channels or undesirable material are not present; if they are, they should be dug out and the excavation backfilled with compacted material. 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 backfilling 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 (ref. A-2) provides guidance in dewatering system design.

h. Final Foundation Preparation. 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 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 be finally 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 (ref. A-5b) 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) With vertical sand drains. If the expected rate of consolidation under stage construction is unacceptably slow, it may be increased by the use of vertical sand drains. Such drains consist of sand columns in the compressible stratum, their purpose being to reduce the length of drainage paths, thus speeding up primary consolidation. Drains are generally 12 in. or more in diameter, and spaced on 6- to 15-ft centers. 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. Johnson (ref. A-5d) presents details on the use of vertical sand drains.

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 additional freeboard, 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 1/2 mile in length and only in a few cases are dams longer than 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 (ref. A-3a(2)). 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.

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. During high water, seepage tends to concentrate along the outer surface of pipes resulting in piping of fill or foundation material. 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. 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.

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.

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 (below freeboard) 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 below the freeboard 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 pipe 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

Table 8-1. Criteria for Pipelines Crossing Levees

		New Pipeline Installation	
Pipelines	Leaving Existing Pipeline in Foundations of Proposed Levees	In Levee Below Design High Water	Over Levee in or on Slopes and Through Levee Above Design High Water
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 anchorage or cover to prevent uplift due to buoyance			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	

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. No pipe should be allowed to penetrate a levee constructed of a pervious soil if the riverside or upstream impervious blanket is less than 5 ft thick.

8-3. General Considerations for Pipelines Crossing over Levees. Requiring a pipeline to cross over or within the freeboard reduces or eliminates many of the dangers that are inherent with pipelines crossing through the embankment or within the foundation. Problems do exist, however, with pipelines crossing over or within the freeboard of 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 (ref. A-3a(12)) 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 (ref. A-3a(12)). 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 (ref. A-3a(12)) 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-405a (ref. A-l) 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 (ref. A-3a(12)). 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 (Note 1). 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 (ref. A-3a(12)). 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.

Note 1--Pending revision of EM 1110-2-2902 designers of corrugated metal pipes installed under levees should specify that the connecting joints for annular and helical pipe should be Flexible Watertight, Rubber-Type Gasketed Joints. Paragraphs 9.3.2 and 9.3.3 of Guide Specification CE-02501 should be modified as follows to insure watertight joints.

a. The gaskets should be limited to Grade SBE 43 or SCE 43 of ASTM D 1056.

b. The width of the gasket should be 1/2 in. less than the width of the connection band required.

c. The connecting bands should be either the angle-lug or rod-and-lug type corrugated coupling bands of the same material, coating, and thickness -as the pipe specified. Bands with projections or dimples will not be permitted. The bands should provide a minimum circumferential lap of 3 in. and be formed to fit and mesh with the corrugations of the pipe to be connected.

(1) Angle-Lug Type. The bands shall not be less than 7 in. wide for pipe 6 to 30 in. in diameter, 12 in. wide for pipe 36 to 60 in. in diameter, and 24 in. wide for pipe 66 to 120 in. in diameter. The bands should have end connection angles of not less than 2 in. by 2 in. by 3/16 in. by the width of the band minus 1 in. adequately fastened to each end and shall be secured with 1/2-in.-diameter bolts. The 7-, 12-, and 24-in. bands shall be secured with a minimum of 2, 3, and 5 bolts, respectively.

(2) Rod-and-Lug Type. The bands shall not be less than 12-in. wide for pipe 6 to 60 in. in diameter, and 24 in. wide for pipe 66 to 120 in. in diameter. Bands shall be secured with 1/2-in.-diameter circumferential rods and cast-iron, silo-type lugs. A minimum of 4 circumferential rods shall be

Note (Continued).--used per band for pipes 6 to 30 in. in diameter, 6 circumferential rods per band for pipes 36 and 60 in. in diameter, and 8 circumferential rods per band for pipes 66 to 120 in. in diameter.

d. Circumferential rods, lugs, connection angles, bolts, and nuts shall be galvanized after fabrication.

e. After installation of coupling bands, the entire exterior of each joint assembly, including bands, rods, lugs, angles, bolts, and nuts shall be given one coat of cold applied bituminous compound conforming to AASHTO M. 243-73.

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 to depths greater than the design freeboard allowance:

(1) Seepage rings or collars should not be provided for the purpose of increasing seepage resistance. 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) An 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 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.

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 water side 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 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. EM 1110-2-1410 (ref. A-3a(l)) fully describes supplemental emergency gates and other closure devices and presents details of the analysis that should be made in determining their necessity, primarily in urban areas. 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. Closure requirements for pressure conduits in urban areas are given in EM 1110-2-1410 (ref. A-3a(1)). 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. Generally, corrugated metal pipe is used if cambering is necessary. 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 should be installed in the dry and a dewatering system should be used where necessary.

b. Pipes Crossing Through or Beneath Levees.

(1) The preferred method of installing pipes within the embankment or foundation of a levee is by the open cut method. Preferably, new levees should be brought to a grade about 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 2 ft below the bottom of the pipe and at least 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.

(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 Plate 11 of EM 1110-2-2902 (ref. A-3a(12)) 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 an average relative density of 85 percent and a minimum of 80 percent.

(4) In existing levees, the cut should be made on stable slopes and the excavated material selectively stockpiled as was described for new levees. The pipe is installed as described in the previous paragraphs. Impervious material within 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 tunneling or jacking should be discouraged. It is recognized, however, that in some instances installation by the open cut method is not feasible or cannot be economically justified. Where tunneling is allowed, metal liners are required. The space around the liners should be pressure grouted for

the entire length and the space between the inner pipe and liners should be filled with concrete. Pipes installed by jacking should be either one piece steel (welded sections, if necessary), or a continuous sleeve should be used in which the pipe is placed, The annular space between the pipe and sleeve should be filled with concrete.

c. Pipes Crossing over Levees. Pipe crossings on the surface or within trenches in the embankment slopes 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 water side of the levee should have a minimum of 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. 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 and to allow traffic to move safely on the crown.

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 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 12-ft levee crown is shown in figure 8-1.

LANDSIDE

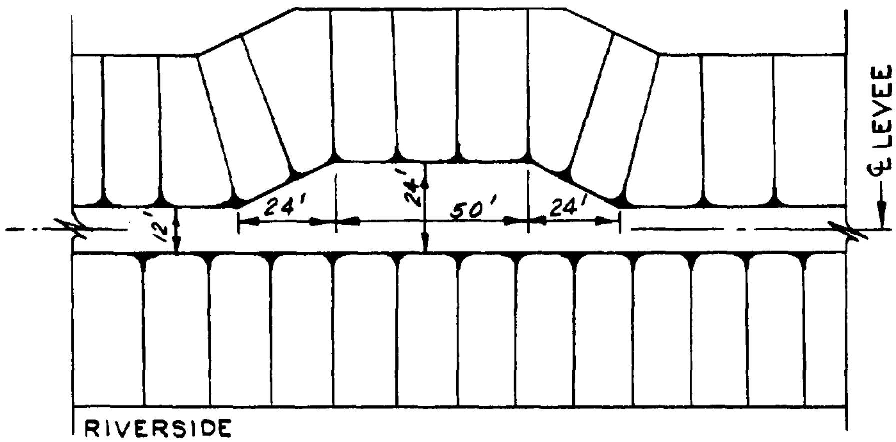


Figure 8-1. Example of levee turnout

(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 12-ft crown is shown in figure 8-2.

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

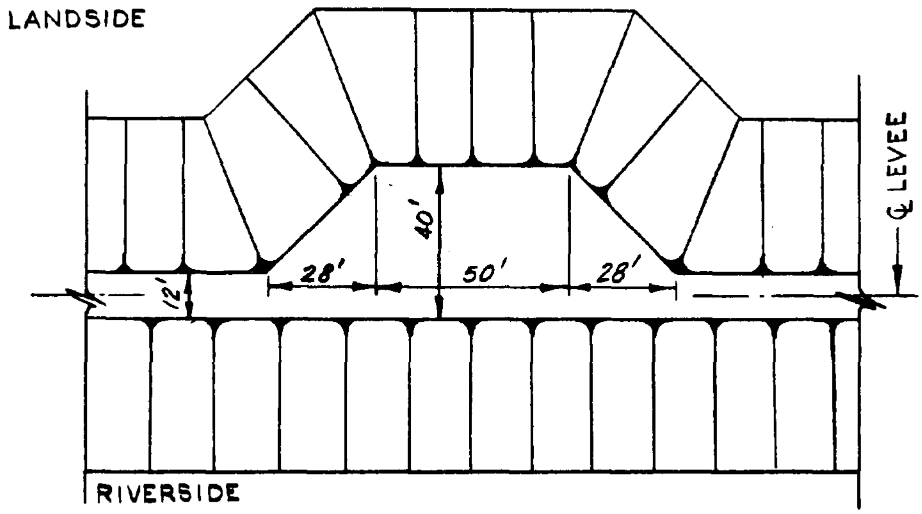


Figure 8-2. Example of levee turnaround

traverses where necessary. The actual locations of the ramps should have the approval of the local levee agency which owns and maintains the levee.

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 floodwall, "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 enlargment. 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 8-3a. 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 8-3b. 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 8-3c. Usually the riverside levee enlargement affords the greatest economy, provided sufficient and suitable riverside borrow exists. A landside enlargement should consist of material at least as pervious as the embankment and preferably more pervious. Landside levee enlargements are usually the least desirable from an economy standpoint, since additional right-of-way often has to be purchased. Another possible economic advantage of a riverside enlargement over a landside enlargement is less material may be required to construct the riverside enlargement. The reason is that on some levee systems the riverside slope of the levee is steeper than the landside slope, and the slope of the enlargement is normally made equal to or flatter than the existing slope of the levee.

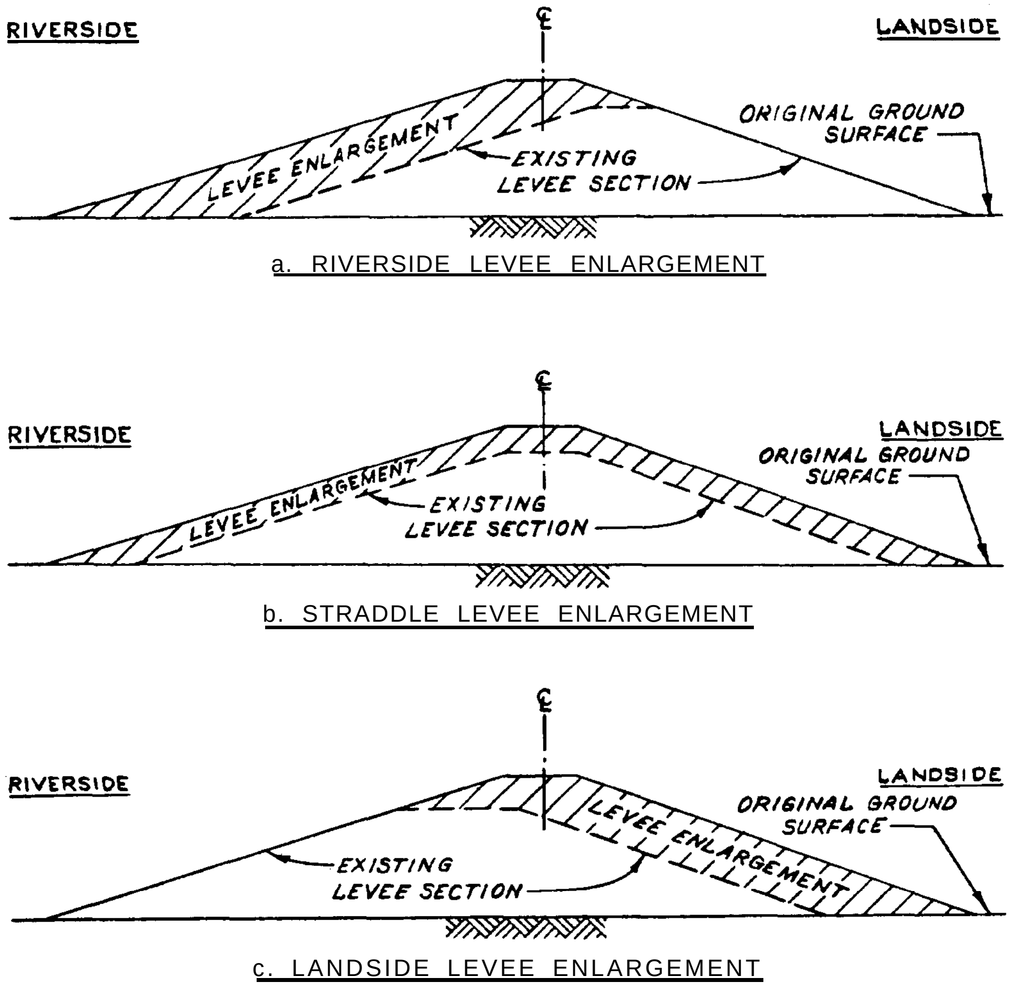


Figure 8-3. Enlargements

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.

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 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 8-4. 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 8-5. The inverted T floodwall is used to make floodwall levee enlargements when walls higher than 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 1110-2-2501 (ref. A-3a(11)).

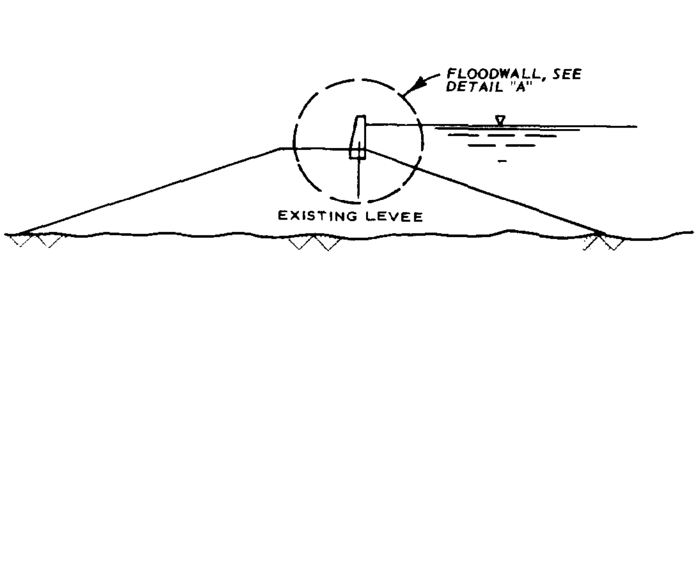


Figure 8-4. I-type floodwall-levee enlargement

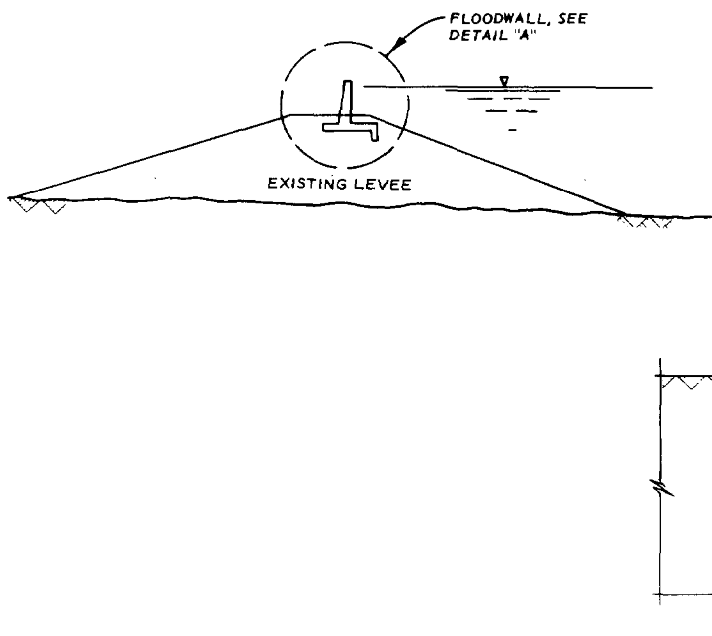


Figure 8-5. Inverted T-type floodwall-levee enlargement

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.

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 1110-2-2501 (ref. A-3a(11)) 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 8-6. 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 1110-2-2501 (ref. A-3a(11)) a transitioning procedure for a junction between a levee embankment and a floodwall is presented that minimizes the effect of differential settlement.

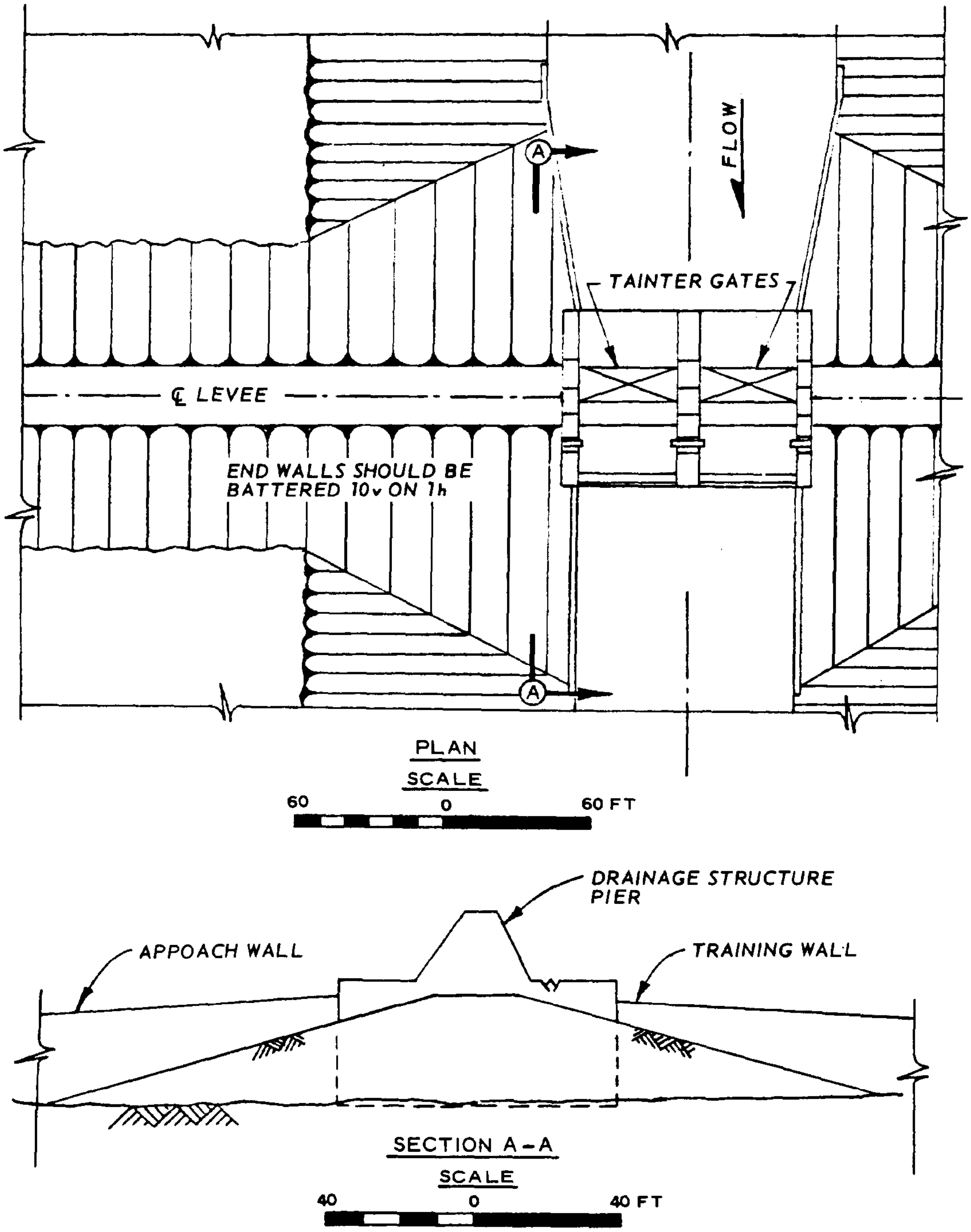


Figure 8-6. Junction of levee and drainage structure

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 (ref. A-3a(10)).

c. 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.

APPENDIX A

REFERENCES

A-1. Federal Specification.

WW-P-405B(1)	Pipe, Corrugated (Iron or Steel, Zinc Coated) U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, Pennsylvania 19120

A-2. Department of the Army Technical Manual.

TM 5-818-5	Dewatering and Groundwater Control for Deep Excavations

A-3. Department of the Army, Corps of Engineers.

a. Engineer Manuals.

(1)	EM 1110-2-1410	Interior Drainage of Leveed Urban Areas
(2)	EM 1110-2-1601	Hydraulic Design of Flood Control Channels
(3)	EM 1110-2-1802	Subsurface Exploration--Geophysical Explorations
(4)	EM 1110-2-1902	Stability of Earth and Rock-Fill Dams
(5)	EM 1110-2-1904	Soil Mechanics Design--Settlement Analysis
(6)	EM 1110-2-1905	Design of Finite Relief Well Systems
(7)	EM 1110-2-1906	Laboratory Soils Testing
(8)	EM 1110-2-1907	Soils Sampling
(9)	EM 1110-2-1908	Instrumentation of Earth and Rock-Fill Dams
(10)	EM 1110-2-1911	Construction Control for Earth and Rock-fill Dams
(11)	EM 1110-2-2501	Wall Design, Flood Wall
(12)	EM 1110-2-2902	Conduits, Culverts, and Pipes

(13) CE-02501 Storm Drainage System

b. Other Corps of Engineers Publications.

(1) U.S. Army Engineer Waterways Experiment Station, CE, "Soil Mechanics Design, Stability of Slopes and Foundations," Technical Memorandum No. 3-777, Feb 1952, P.O. Box 631, Vicksburg, Miss. 39180

(2) U.S. Army Engineer Waterways Experiment Station, CE, "Under seepage and Its Control, Lower Mississippi River Levees," Technical Memorandum No. 3-424, Vol I, Oct 1956, P.O. Box 631, Vicksburg, Miss. 39180

(3) U.S. Army Engineer Division, Lower Mississippi Valley, CE, "Design Procedures for Design of Landside Seepage Berms for Mississippi River Levees," P.O. Box 80, Vicksburg, Miss. 39180

(4) U.S. Army Engineer District, Rock Island, CE, "Engineering and Design Studies for Underseepage Control (Slurry Trench Cutoff) for Saylorville Dam, Des Moines River, Iowa," Design Memorandum, Nov 1965, Clock Tower Bldg., Rock Island, Ill. 61201

(5) U.S. Army Engineer District, Savannah, CE, "West Point Project, Chatahoochee River, Alabama and Georgia, Construction of Slurry Trench Cutoff," May 1968, P.O. Box 889, Savannah, Ga., 31402

A-4. Department of the Navy.

NAVFAC, Soil Mechanics Foundations and Earth Structures, Design Manual 7, Mar 1971. U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, Pennsylvania 19120

a. Bjerrum, L., "Embankments on Soft Ground," Proceedings, Specialty Conference on Performance of Earth and Earth-Supported Structures, Vol II, 11-14 June 1972. ASCE, 345 East 47th St., New York, New York 10017

b. Blasters' Handbook, 15th ed., E. I. DuPont de Nemours and Co., Inc., 1966, Wilmington, Delaware 19898

c. Dobrin, M. B., Geophysical Prospecting, 1960, McGraw Hill Publishing Co., 1221 Avenue of The Americas, New York, New York, 10020

d. Johnson, S. J., "Foundation Precompression with Vertical Sand Drains," Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers, Jan 1970, ASCE, 345 East 47th St., New York, New York 10017

e. Kapp, M. S. et al., "Construction on Marshland Deposits: Treatment and Results," Utilization of Sites of Soft Foundations, Highway Research Record No. 133, pp 1-22, 1966, Transportation Research Board, 2101 Constitution Avenue, N. W., Washington, D. C. 20418

f. Turnbull, W. J. and Mansur, C. I., "Design of Underseepage Control Measures for Dams and Levees," Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers, Oct. 1959, ASCE, 345 East 47th St., New York, New York 10017

g. Turnbull, W. J. and Mansur, C. I., "Construction and Maintenance of Underseepage Control Measures," Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers, Dec. 1959, ASCE, 345 East 47th St., New York, New York 10017

h. ASTM D 1056. American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pennsylvania 19103

i. AASHTO M. 243-73. American Association of State Highway and Transportation Officials, 341 National Press Bldg., Washington, D. C. 20004

APPENDIX B

MATHEMATICAL ANALYSIS OF UNDERSEEPAGE AND SUBSTRATUM PRESSURE

B-1. General. The design of seepage control measures for levees often requires an underseepage analysis without the use of piezometric data and seepage measurements. Contained within this appendix are equations by which an estimate of seepage flow and substratum pressures can be made, provided soil conditions at the site are reasonably well defined. The equations contained herein were developed during a study (reported in U. S. Army Engineer Waterways Experiment Station TM 3-424 (ref. A-3b(2)) of piezometric data and seepage measurements along the Lower Mississippi River and confirmed by model studies. It should be emphasized that the accuracy obtained from the use of equations is dependent upon the applicability of the equation to the condition being analyzed, the uniformity of soil conditions, and evaluation of the various factors involved. As is normally the case, sound engineering judgment must be exercised in determining soil profiles and soil input parameters for these analyses.

B-2. Assumptions. It is necessary to make certain simplifying assumptions before making any theoretical seepage analysis. The following is a list of such assumptions and criteria necessary to the analysis set forth in this appendix.

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

b. Flow through the top stratum is vertical.

c. Flow through the pervious substratum is horizontal.

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

e. All seepage is laminar.

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

B-3. Factors Involved in Seepage Analyses. The volume of seepage (Q_s)

that will pass beneath a levee and the artesian pressure that can develop under and landward of a levee during a sustained high water are related to the basic factors given and defined in table B-l and shown graphically in figure B-l. Other values used in the analyses are defined as they are discussed in subsequent paragraphs.

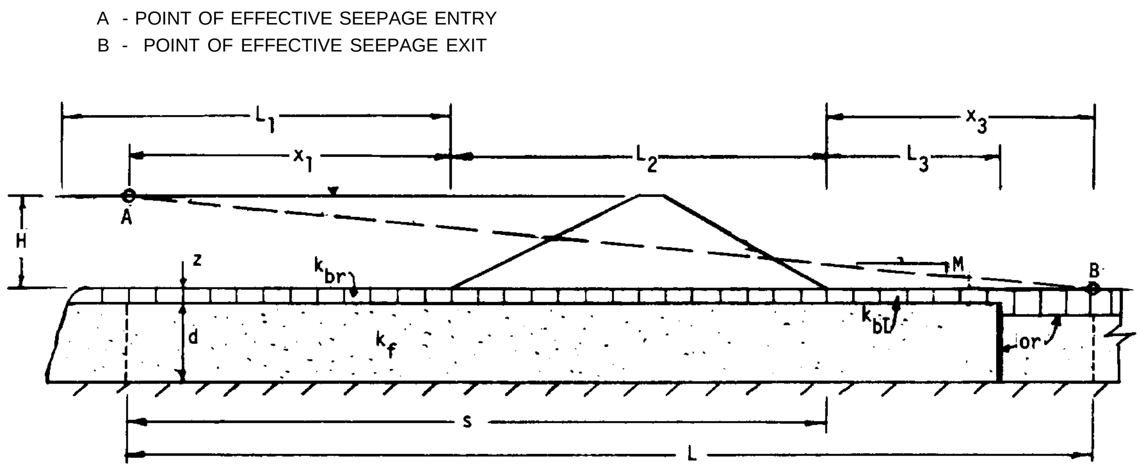


Figure B-l. Illustration of symbols used in Appendix B.

B-4. Determination of Factors Involved in Seepage Analyses. Table B-2 contains a brief summary of methods normally used to determine the factors necessary to perform a seepage analysis. The determination of these factors is discussed in more detail in the following paragraphs. Many of the methods given, such as exploration and testing, have previously been mentioned in the text; however, they will be discussed herein in more detail as they apply to each specific factor. The use of piezometric data, although rarely available on new projects, is mentioned primarily because it is not infrequent for seepage analyses to be performed as a part of remedial measures to existing levees in which case piezometric data often are available.

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

b. Thickness, z , and Vertical Permeability, k_b , of Top Stratum.

(1) Exploration. The thickness of the top stratum, both riverward

Table B-1. Factors Involved in Seepage Analyses

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

Table B-2. Methods for Determination of Design Parameters

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

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

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

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

where F_t = transformation factor.

Some examples using this procedure are given in table B-3 and in figure B-2.

Table B-3. Examples of Transformation Procedure

Strata	Actual Thickness ft	Actual Permeability cm/sec	$F_t = \frac{k_b}{k_n}$	Transformed Thickness, ft for $k_b = 1 \times 10^{-4}$ cm/sec
Clay	5	1×10^{-4}	1	5.0
Sandy silt	8	2×10^{-4}	1/2	4.0
Silty sand	5	10×10^{-4}	1/10	0.5
	z = 18			z_b = 9.5

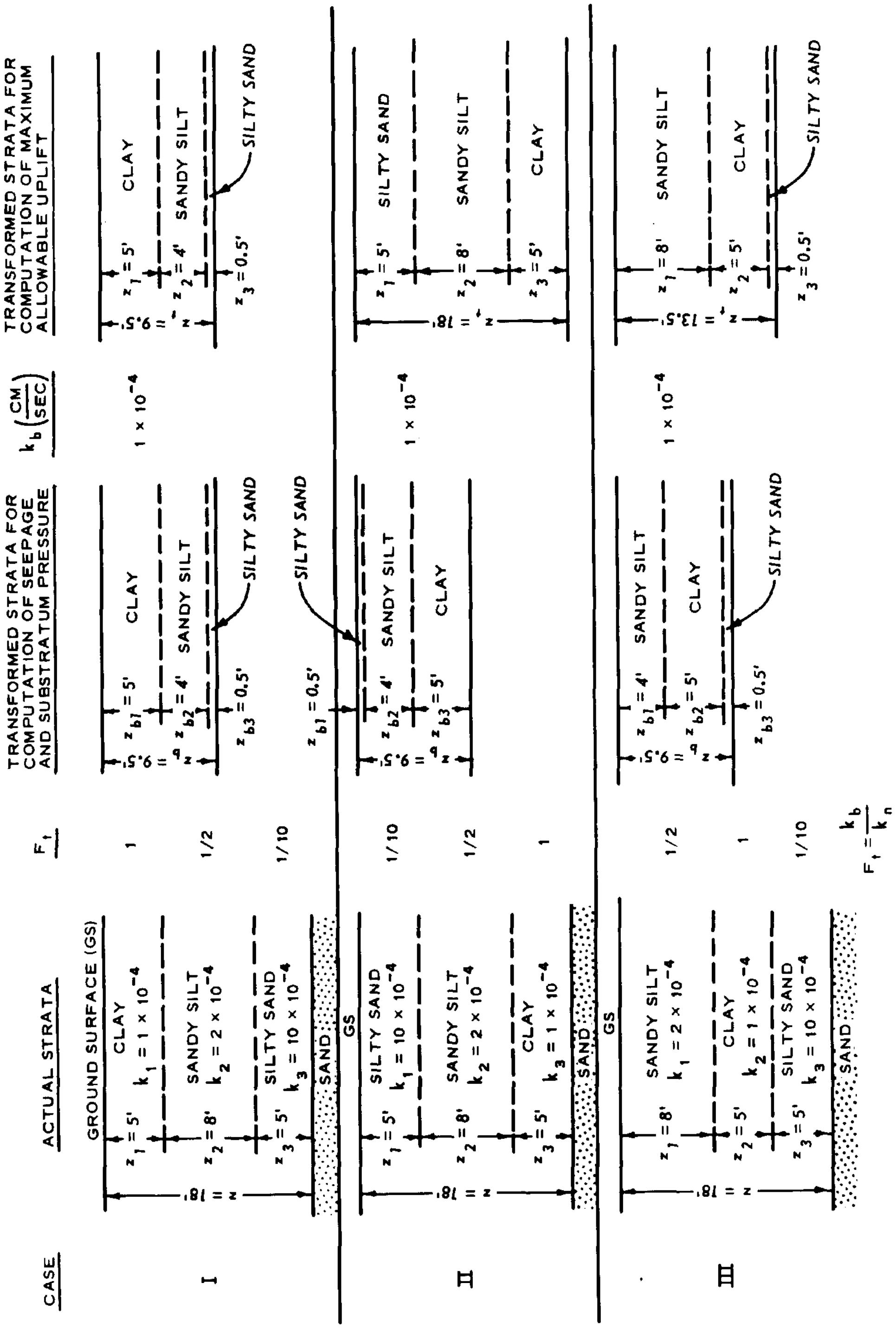


Figure B-2. Transformation of top strata.

A generalized top stratum having a uniform permeability of 1 x 10^{-4} and 9.5 ft thick would then be used in the seepage analysis for computation of the length to the effective seepage exit. However, the thickness z_b may or may not be the effective thickness of the landside top stratum z_t that should be used in determining the allowable pressure beneath the top stratum. The transformed thickness of the top stratum for estimating allowable uplift z_t equals the in situ thicknesses of all strata above the base of the least pervious stratum plus the transformed thicknesses of the underlying more pervious top strata. This means that z_b will equal z_t only when the least pervious stratum is at the ground surface. Several examples of this transformation are given in figure B-2. In making the final determination of the effective thicknesses and permeabilities of the top stratum the characteristics of the top stratum at least 200 to 300 ft landward of the levee must be considered. In addition, certain averaging assumptions are almost always required where soil conditions are reasonably similar. Thin or critical areas should be given considerable weight in arriving at such averages.

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

$$k_f = \frac{Q_s}{M_d} \tag{B-2}$$

d. Distance from Riverside Levee Toe to River, L_1. This distance can usually be estimated from topographic maps.

e. Base Width of Levee and Berm, L_2 . L_2 can be determined from anticipated dimensions of new levees or by measurement in the case of existing levees.

f. Length of Top Stratum Landward of Levee Toe, L_3. This distance can usually be determined from borings, topographic maps, and/or field reconnaissance. In determining this distance careful consideration must be given to any geological feature that may affect the seepage analysis. Of special importance are deposits of impervious materials such as clay plugs which can serve as seepage barriers and if located near the landside toe could force the emergence of seepage at their near edge, thus having a pronounced effect on the seepage analysis.

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

(1) For $L_3 = \infty$

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

where

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

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

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

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

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

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

$$x_3 = 0.1\sqrt{d}\, x_3(d = 100) \tag{B-6}$$

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

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

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

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

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

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

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

1. Critical Gradient for Landside Top Stratum, i_c. The critical

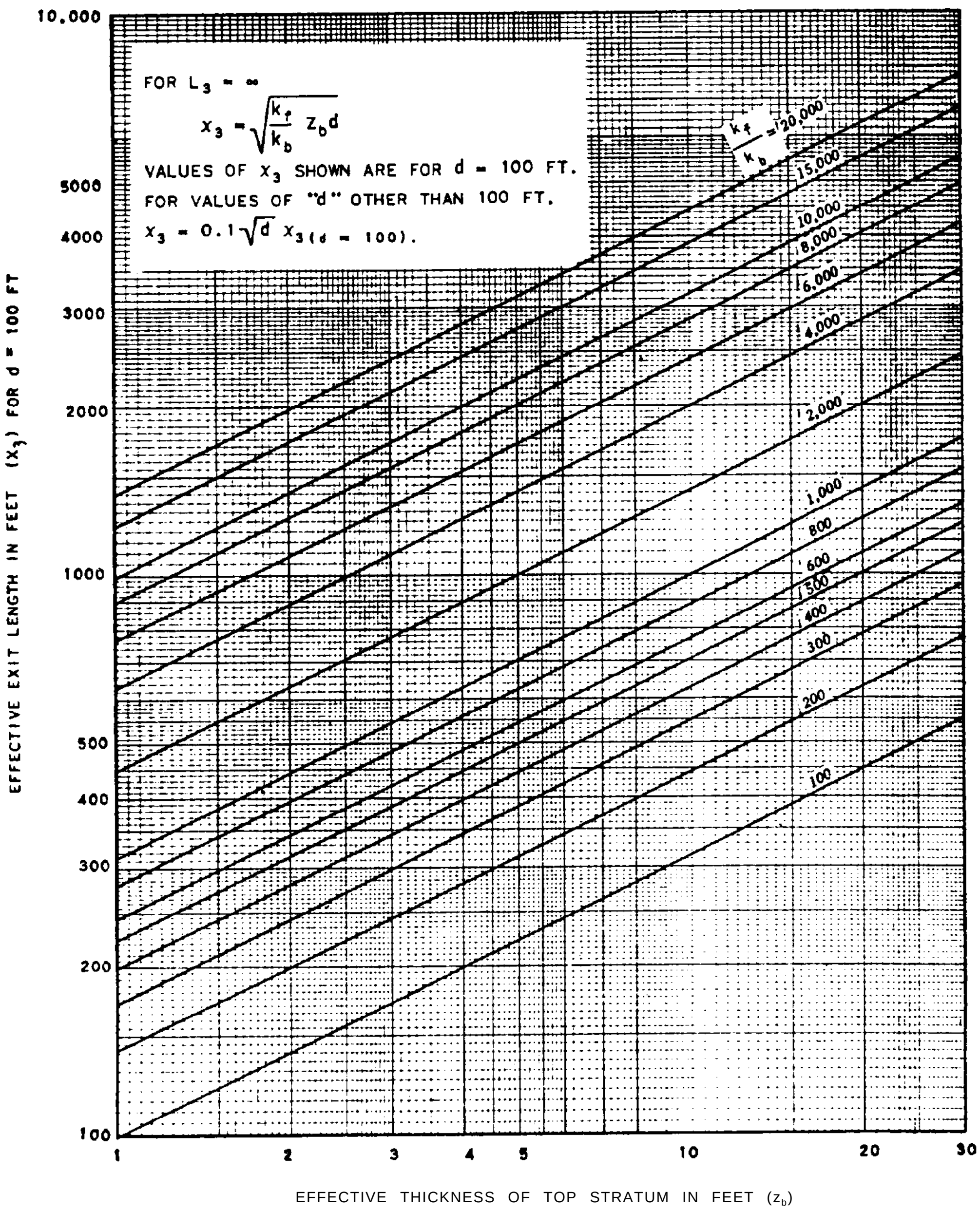


Figure B-3. Effective seepage exit length for $L_3 = \infty$ and d = 100 ft.

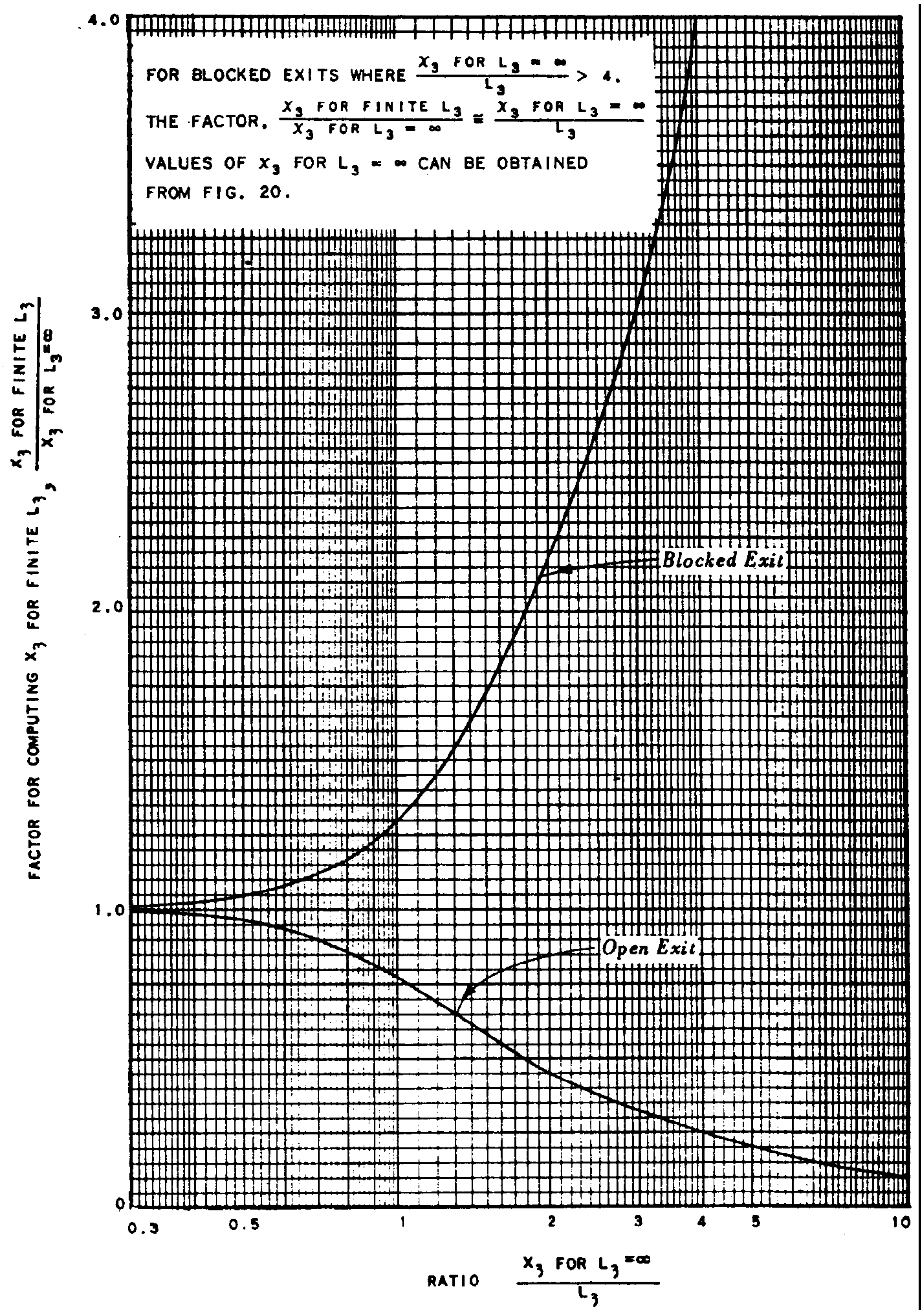


Figure B-4. Relation between x_3 for blocked or open exits and X_3 for $L_3 = \infty$

gradient is defined as the gradient required to cause boils or heaving (flotation) of the landside top stratum and is taken as the ratio of the submerged or buoyant unit weight of soil γ' comprising the top stratum and the unit weight of water γ_w or

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{G_S - 1}{1 + e} \qquad \text{(B-9)}$$

where

G_S = specific gravity of soil solids

e = void ratio

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

$$M = \frac{\Delta h}{\ell} \qquad \text{(B-10)}$$

where

Δh = the difference in piezometer readings

ℓ = the horizontal distance between piezometers.

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

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

a. General.

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

$$Q_s = \not{S} k_f H \qquad \text{(B-11)}$$

where $\not{S}$ = shape factor . This equation is valid provided the assumptions upon which Darcy's law is based are met. The mathematical expressions for the shape factor $\$$ (subsequently given in this appendix) depend upon the dimensions of the generalized cross section of the levee and foundation, the characteristics of the top stratum both riverward and landward of the levee, and the pervious substratum. Where the hydraulic grade line M is known from piezometer readings, the quantity of underseepage can be determined from equation B-12.

$$Q_s = Mk_f d \qquad \text{(B-12)}$$

(2) Excess Hydrostatic Head Beneath the Landside Top Stratum.

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

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

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

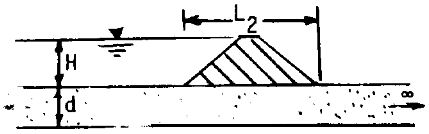
b. Case 1 - No Top Stratum. Where a levee is founded directly on pervious materials and no top stratum exists either riverward or landward of the levee (fig. B-5a), the seepage Q_s can be obtained from equation B-11 in which

$$\$ = \frac{d}{L_2 + 0.86d} \qquad \text{(B-13)}$$

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

c. Case 2 - Impervious Top Stratum Both Riverside and Landside. This case is found in nature where the levee is founded on thick

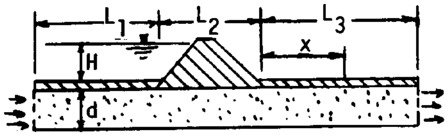
a. CASE 1 - No top stratum



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

$$h_o = h_x = 0$$

b. CASE 2 - Impervious topstratum both riverside and landside



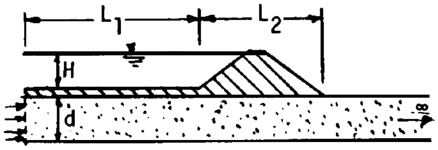
$$\$ = \frac{d}{L_1 + L_2 + L_3}$$

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

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

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

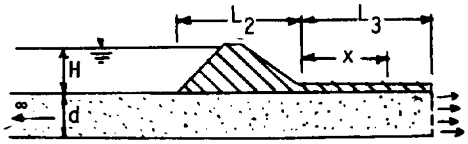
c. CASE 3 - Impervious riverside top stratum & no landside top stratum



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

$$h_o = h_x = 0$$

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



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

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

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

BASIC DEFINITIONS AND EQUATIONS

HEAD BENEATH TOP STRATUM AT LANDSIDE LEVEE TOE . . . . h_o

HEAD BENEATH TOP STRATUM AT DISTANCE x FROM LANDSIDE LEVEE TOE . . . h_x

SHAPE FACTOR FOR USE IN SEEPAGE EQUATION . . . . $\$$

SEEPAGE PER UNIT LENGTH OF LEVEE . . . . $Q_s = \$ k_f H$

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

(<l5 ft) deposits of clay or silts with clay strata. For such a condition little or no seepage can occur through the landside top stratum.

(1) If L_3 is infinite in landward extent or the pervious substratum is blocked landward of the levee, no seepage occurs beneath the levee and $Q_3 = 0$. The head beneath the levee and the landside top stratum is equal to the net head on the levee at all points so that $H = h_o = h_x$.

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

$$\$ = \frac{d}{L_1 + L_2 + L_3} \qquad \text{(B-14)}$$

The heads h_o and h_x can be computed from

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

$$h_x = h_o\left(\frac{L_3 - x}{L_3}\right) \quad \text{for} \quad x \leq L_3 \qquad \text{(B-16)}$$

$$h_x = 0 \quad \text{for } x \geq L_3$$

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

$$\$ = \frac{d}{L_1 + L_2 + 0.43d} \qquad \text{(B-17)}$$

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

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

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

$$h_o = H\left(\frac{L_3}{0.43d + L_2 + L_3}\right) \tag{B-20}$$

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

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

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

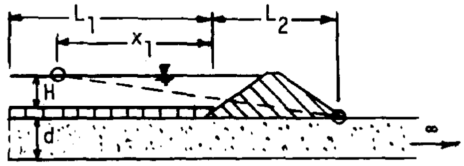
Since no landside top stratum exists, $h_o = h_x = 0$. This case is illustrated in figure B-6a.

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

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

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

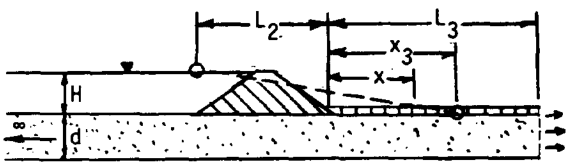
a. CASE 5 - Semipervious riverside top stratum and no landside top stratum



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

$$h_o = h_x = 0$$

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



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

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

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

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

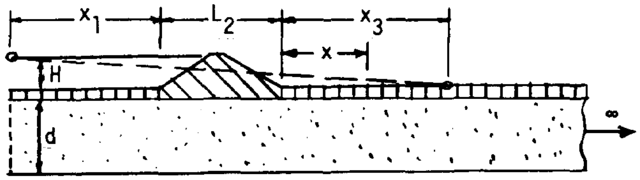
$$h_x = h_o\left(\frac{x_3 - x}{x_3}\right) \qquad \text{(B-24)}$$

h. Case 7 - Semipervious Top Strata Both Riverside and Landside. Where both the riverside and landside top strata exist and are semipervious (fig. B-7) the quantity of underseepage can be computed from equation B-11 where S is defined in equation B-25.

$$S = \frac{d}{x_1 + L_2 + x_3} \qquad \text{(B-25)}$$

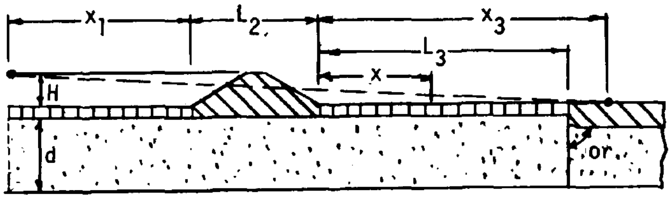
The head beneath the top stratum at the landside toe of the levee is expressed by

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



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

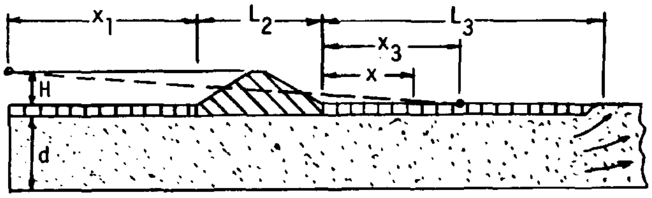
a. $L_3 = \infty$



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

$$h_{x = L_3} = \frac{h_o}{\cosh cL_3}$$

b. L_3 is finite to a seepage block



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

$$h_{x = L_3} = 0$$

c. L_3 is finite to an open seepage exit

BASIC DEFINITIONS AND EQUATIONS

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

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

THE FACTOR . . . . $c = \sqrt{\dfrac{k_{bL}}{k_f z_{bL} d}}$

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

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

(1) For

$$L_3 = \infty \; h_x = h_o e^{-cw} \qquad \text{(B-27)}$$

where $e = 2.718$.

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

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

and

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

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

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

and

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

(4) The value of c in equations B-27 through B-30 is as follows:

$$c = \sqrt{\frac{k_{bl}}{k_f z_{bl} d}} \qquad \text{(B-32)}$$

(5) In order to simplify the determination of h_x for various values of x , the relationship between h_x/h_o and x/x_3 is plotted in

figure B-8 for $L_3 = \infty$ and for various values of x_3/L_3 for both a seepage block and an open seepage exit. Once x_1, L_2, x_3, L_3, and h_o have been determined, the ratio h_x/h_o can be obtained from figure B-8 for any particular x/x_3; h_x can then be computed from h_x/h_o.

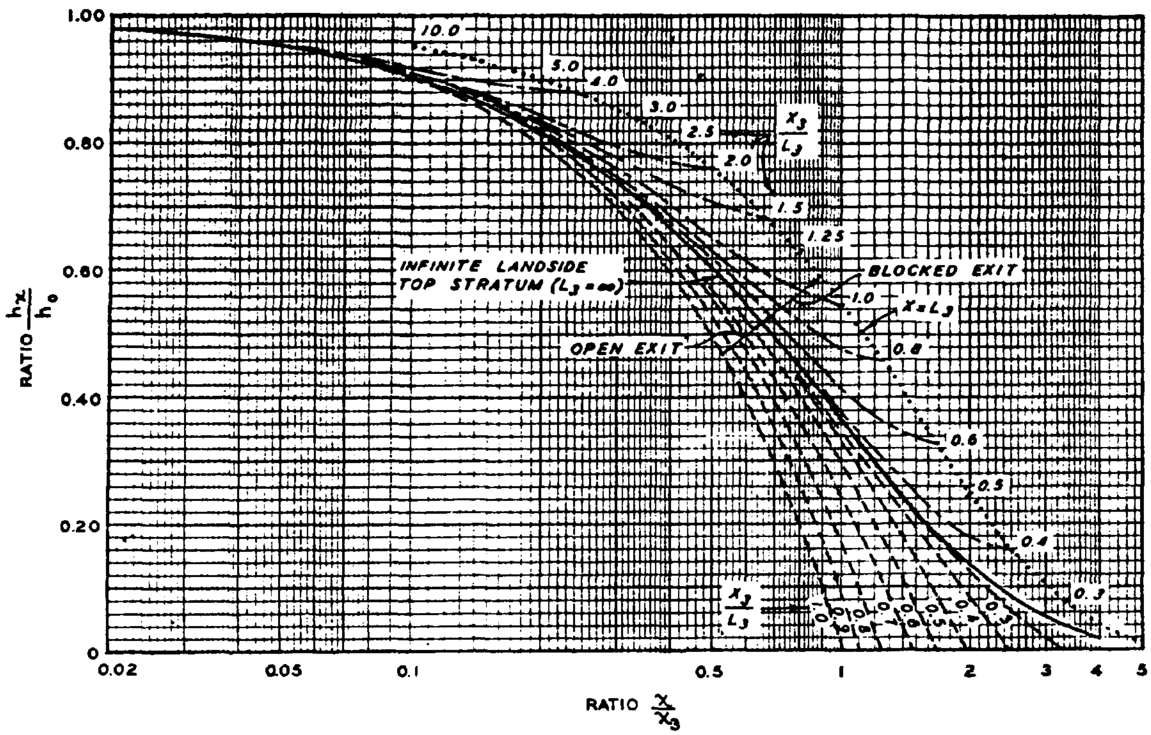


Figure B-8. Ratio between head landward of levee and head at landside toe of levee for levees founded on semipervious top stratum underlain by a pervious substratum

(6) Values of h_o and h_x resulting from the equations above are actually hydrostatic heads at the middle of the pervious substratum; where the ratio k_f/k_b is less than 100 to 500, values of h_o and h_x immediately beneath the top stratum will be slightly less than those computed because of the head loss resulting from upward seepage through the sand stratum.

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 (ref. A-3b(2)). Design procedures are taken from TM 3-424 also and from procedures developed by the Lower Mississippi Valley Division (ref. A-3b(3)).

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 must be considered in determining x_1. The thickness of the riverside blanket z_{br} should be the same thickness as the blanket in the borrow pit where a riverside borrow pit exists. 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-l. 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.

Table C-1. 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

Blanket in Riverside Borrow Pit		Number of Piezometer Lines from Which Data Were Obtained	S, ft			x_1, ft[a]			$k_{br} \times 10^{-4}$ cm/sec			Suggested Design Values	
Soil Type	Thickness in ft		Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	k_{br}	x_1
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 & 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[d]	0.34[d]	0.79[d]	0.8	600
	5 to 10	2	1720	1520	1620	1270	1070	1170	1.3[d]	0.86[d]	1.08[d]	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]

[a] Values of x_1 computed from observed values of x_1 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.

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 to determine d and k_f .

d. The permeability k_{b1} and effective thickness Z_{b1} 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_b1 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_{b1} 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).

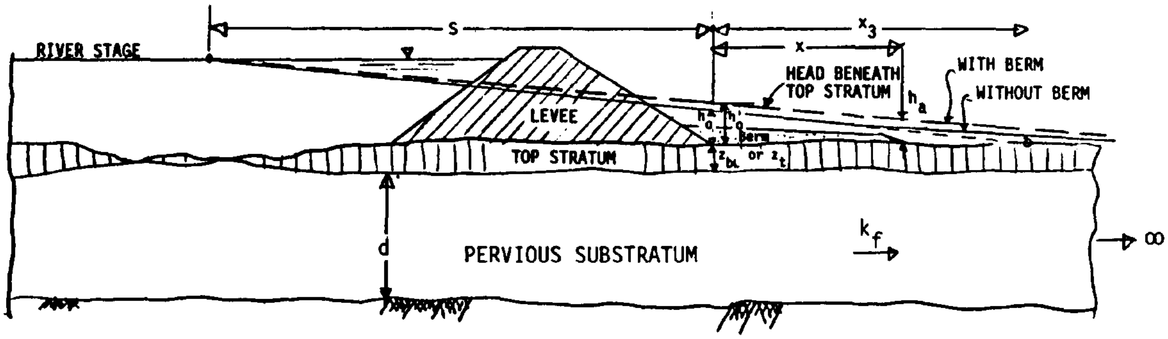
e. The seepage exit length x_3 can be calculated from equations presented in Appendix B, paragraph B-4g. These equations are applicable to conditions where the length of the landside blanket L_3 is either infinite or finite.

C-3. <u>Design Equations and Criteria.</u>

a. <u>Design Equations.</u> Equations for the design of landside seepage berms for the four major berm types are presented in figure C-l. 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.

b. <u>Design Criteria.</u>

(1) Where seepage berms overlay a landside blanket (landside top stratum) and the computed head beneath the blanket at the landside toe of the levee is greater than 0.8 of the project design flow line, the seepage berm should be designed with an allowable upward gradient i_o through the blanket and berm at the landside toe of the levee of 0.3 and an allowable upward gradient i_1 at the landside toe of the berm of 0.8. All berms should have minimum thickness of 5 ft at the levee toe, a minimum thickness of 2 ft at the berm crown, and a minimum width of 150 ft. The thickness of the berm should be increased 25 percent to allow for shrinkage, foundation settlements, and variations in design factors.



Notations

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

γ'_z = submerged unit weight of top stratum

γ'_t = submerged unit weight of berm

γ_w = unit weight of water

H = total net head on levee

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

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

i_o = allowable upward gradient at landside toe of levee

i_1 = allowable upward gradient at toe of berm

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

X = required berm width

t = required thickness of berm at toe of levee

k_t = vertical permeability of berm

F = factor of safety against uplift at toe of levee

Q_b = flow into berm per foot of levee

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

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

d = effective thickness of pervious substratum

Formulas

Impervious Berm
$k_t = 0$

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

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

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

USE $F \geq 1.5$

Semipervious Berm, $k_t = k_{bl}$

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

where:

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

$r = i_o/i_1$

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

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

Sand Berm

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

$$h'_o = h_a\left[1 + cx_3 + \left(\frac{2 + r}{6}\right)\left(cX_s\right)^2\right]$$

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

Pervious Berm with Collector

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

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

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

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

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

(2) 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 (1) 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.

(3) The width of the berm is usually limited to about 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_l 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.

(4) 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.

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

In the above equation $\bar{D}$ 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_l is the effective length of impervious blanket riverside of the levee. If no riverside blanket exists, the value of x_l is assumed to be 0.43 $\bar{D}$. The rate of seepage Q_s below the levee per unit length of levee can be determined using equation C-2.

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

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

(5) 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.

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-2. **Examples of Design of Seepage Berms**

Designs based on following conditions:

$H = 25$ ft	$z_{bl} = z_t = 6.0$ ft	$\gamma' = 52.5$ pcf for impervious berms
$k_f = 1000 \times 10^{-4}$ cm/sec	$i_o = 0.50$	$\gamma' = 57.5$ 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
$k_{bl} = 3 \times 10^{-4}$ cm/sec	$\gamma' = 52.5$ pcf	$L_3 = \infty$
$s = 1000$ ft	$x_3 = 450$ ft	

	Required Berm			Suggested Design Dimensions				Suggested Construction Dimensions			
Type Berm	Width X ft	Thickness[a] t, ft	h'_o[b] ft	Thickness at Berm Crown ft	Berm Width X ft	Berm Slope	Approximate Thickness Levee Toe ft	Thickness[c] at Berm Crown ft	Berm Width X ft	Thickness[c] at Levee Toe ft	Approximate[d] Material Required yd^3 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 sand	280	3.8	8.6	2.0	275	1 on 75	5.7	2.5	275	7.1	5,200
	260	3.3	8.3	2.0	250	1 on 75	5.3	2.5	250	6.6	4,500
Pervious with collector	215	2.9	7.7	2.0	200	1 on 75	4.7	2.5	200	5.9	3,300[f]

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

APPENDIX D

RELIEF WELL INSTALLATION

D-1. General.

a. Scope and Theory. The following material provides guidance for drilling personnel, design engineers, and inspectors responsible for adherence to specifications and acceptance of completed installations. Wells are installed for a variety of purposes, including potable and nonpotable water supply, dewatering, and pressure relief. While the specifications for all types of wells are overlapping, the designed purpose of the well dictates at least part of the content of the specifications for the installation. This appendix is applicable to, and will treat only, permanent-type relief well installations. Hydrostatic pressures build up within the foundations of dams and levees and beneath some structures not associated with the retention of surface water, However, pressure relief wells are extremely compatible with, and applicable to, the protection of dams and levees. The pressures within a foundation can cause uplift and break-through, causing sand boils, loss of foundation materials, and ultimate loss of structures. This water must flow to relieve the pressures. Consequently, the whole purpose of a relief well is to control the venting of these pressures to the surface and prevent the loss of foundation materials. Because of frictional resistance to flow through foundation materials, pressure diminishes with distance from the source. Therefore, areas of low resistance to uplift and nearest the source are potentially dangerous. Figure D-l shows schematically the flow characteristics of water below a levee indicating that the point most susceptible to sand boils is immediately adjacent to the landside toe.

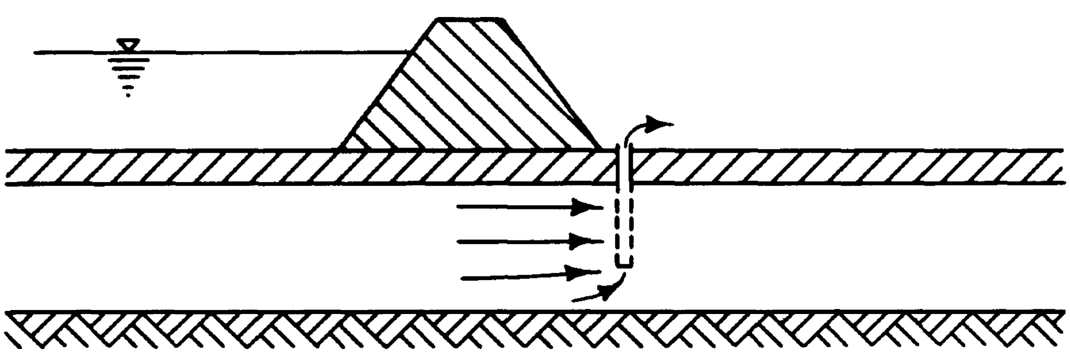


Figure D-1 Flow characteristics at pressure relief well.

b. Description of Well. While the specific materials used in the construction vary and the dimensions and methods of installation differ, relief wells are basically very similar. They consist of a boring to facilitate the installation; a screen or slotted pipe section to allow entrance of groundwater; a filter to prevent entrance and ultimate loss of foundation material; a riser to conduct the water to the ground surface; a check valve to allow escape of water and prevent backflooding and entrance of foreign material detrimental to the installation; backfill to prevent recharge of the formation by surface water; a cover and some type of barricade protection to prevent vandalism and damage to the top of the well by maintenance crews, livestock, etc. Figure 5-7 shows a typical relief well installation. The boring is drilled large enough to facilitate a minimum of 6-in. filter thickness continuously around the screen. The boring is overdrilled in depth to receive segregated filter material. The extent of overdrilling required is dependent upon the size of tremie pipe used for filter placement and the total depth of the well, but as shown in figure 5-7, a minimum of 4 ft is suggested. The backfill indicated as sand in figure 5-7 would normally consist of contaminated, segregated, or otherwise excess filter material. Its only function is to fill the annular space around the riser pipe to prevent collapse of the boring; these granular materials are easily placed and require a minimum of compaction. The backfill indicated as concrete in figure 5-7 forms a seal to prevent inflow of surface water from rains and floods. The concrete backfill should contain an additive to prevent or compensate for shrinkage. Finely ground bentonite, commercially available as a drilling fluid component, added at a rate of 10 lb per sack of cement, has been proven to compensate for shrinkage.

c. Materials for Wells. Since pressure-relief wells are designed and installed to protect the foundations of structures, selection of materials for the well should be based on cost, performance, life expectancy, and ease of maintenance. If possible, the life expectancy of the well should be equal to or greater than the life expectancy of the structure which it protects. Commercially available well screens and riser pipes are fabricated from a variety of materials such as wood, black iron, galvanized iron, stainless steel, brass, bronze, concrete, fiberglass, polyvinyl chloride (PVC), and other materials. How well a material lasts depends upon its strength, resistance to damage by servicing techniques, and resistance to attack by the chemical constituents of the water. Wood has proven to be very stable in most environments in well installations, as long as it is continuously submerged in water. Stainless steel is apparently a very stable material in most environments and has an advantage over wood, since a more efficient screen can be fabricated. Stainless steel screens made up on galvanized-iron pipe, or using galvanized-iron joints, are only slightly more durable than galvanized screens and should be avoided in permanent installations.

Fiberglass is a promising material; however, its performance history is relatively short. Brass and bronze are extremely expensive and are not completely stable in some acid environments. Porous concrete is difficult to install and is easily deteriorated by chemical action. PVC is almost completely stable and easy to handle and install; however, it is a relatively weak material and easily damaged. The life of iron screens is extended by galvanizing, but neither can be considered permanent. Ferrous and nonferrous metals should never be placed in direct contact with each other, such as the case of a brass screen and a steel riser; the direct contact of these dissimilar metals causes an electrolysis and a resultant deterioration of the material.

d. Slot Type. A variety of slot types are available in the various types of well screens. The wooden and PVC screens are limited to open slots of varying dimensions consisting of a series of saw cuts. Metal and fiberglass screens can be open slots, louvered or otherwise shielded slots, or "continuous slot." The "continuous slot" screens consist of a skeleton of vertical rods wrapped with a continuous spiral of wire. The wire can be a variety of cross-sectional shapes. The trapezoidal-shape wire provides a slot that is progressively larger toward the inside of the screen. This shape allows any filter gravel that enters the slot to fall into the well rather than clog the screen. The open-type slots are advantageous in development of filter (paragraph D-3). They allow the successful use of a water jet, whereas, shielded slots deflect the water jet and reduce or destroy its effectiveness in the filter.

e. Dimensions. The size of the individual openings in a well screen is dictated by the grain size within the foundation. The individual slots should be as wide as possible, yet compatible with a single-stage filter fine enough to retain the foundation materials (Appendix E). The anticipated maximum flow of the well dictates both the minimum total open-slot area of the screen (the spacing and length of slots) and the minimum diameter of the well. The diameter must be large enough to conduct the maximum anticipated flow to the ground surface and facilitate testing and servicing of the well after installation. Head loss in the well should also be taken into consideration in selecting a well diameter.

f. Filter. The filter should consist of natural material made up of hard, durable particles. Crushed carbonate aggregates should be avoided because they tend to break down and lose permeability. The grain sizes should be reasonably well distributed over the specified range, with no sizes missing. The range of sizes is designed to meet standard criteria, as described in Appendix E. It is often difficult to purchase material that meets the required gradation and it becomes necessary to have the materials specially blended. The special blends are expensive and sometimes difficult to acquire, but essential to the installation of acceptable permanent relief wells.

D-2. Installation of Relief Wells.

a. General. Proper installation of relief wells is the key to their proper performance. Before installation is begun, all equipment and materials required for completion of the work should be on hand at the site. The well screen and riser should be checked for proper material, length, diameter, and slot openings. The filter material should be checked against gradation specifications. Successful installation of a well is often dependent upon orderly progression of each step; many installations have been aborted because of delays.

b. Drilling. An open boring of sufficient size and depth is necessary to facilitate the installation of a well. The diameter should allow approximately 6 to 8 in. of annular space continuously around the screen for an effective filter. An annular space less than 6 in. is not acceptable because insufficient room exists to properly place the filter material. The methods of providing an open boring in the ground are numerous. However, not all are acceptable for the installation of permanent relief wells. Drilling methods considered acceptable are discussed in the following paragraphs.

c. Reverse Circulation Rotary. This method is considered to provide the most acceptable boring and, consequently, the most efficient well. This method should be used whenever possible for the installation of permanent relief wells. Standard rotary drilling consists of rotating a cutter bit against the bottom of a boring, while a fluid is pumped down through the drill pipe to cool and lubricate the bit and return the cuttings up the open hole to the ground surface. Reverse circulation rotary is a similar cutting process, except the drilling fluid is pulled up through the drill pipe by vacuum and the drilling fluid reenters the top of the open boring. Since the cross-sectional area of the boring is many times larger than that of the drill pipe, the slow downward velocity of the fluid acting against the open boring does not erode the walls. The drilling fluid consists of water and, unavoidably, a small amount of the finer fraction of the natural material being drilled. The capacity of a drilling fluid to carry the cuttings is directly proportional to its velocity and/or viscosity. The reverse rotary method generally employs a low viscosity fluid (water plus a small amount of soil) but a high velocity is attained with the fluid returning up through the drill pipe. The boring walls are stabilized by the excess hydrostatic head of the column of water in the boring which normally eliminates the need for casing. Figure D-2 shows schematically the circulating system for reverse-circulation rotary drilling.

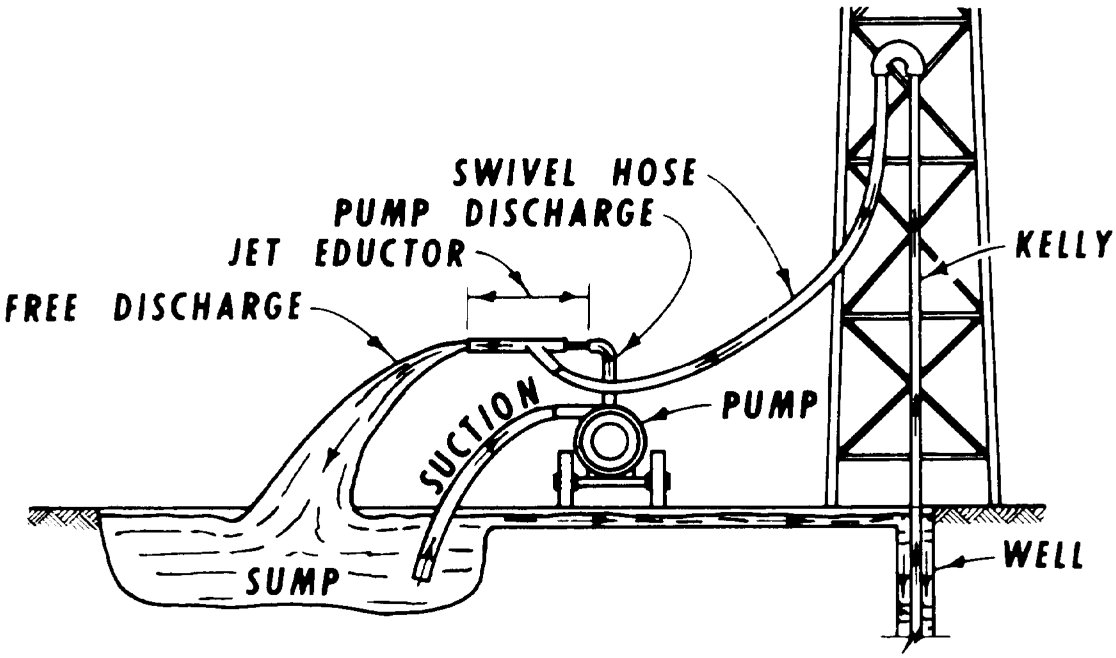


Figure D-2. Schematic diagram of circulatory system.

(1) Equipment. Reverse-circulation rotary drilling requires somewhat specialized equipment, most of which is commercially available or easily fabricated. Any rotary-type drill rig large enough to handle the load of drill tools and having sufficient torque capability to rotate the bit required can be adapted to this drilling method. As can be noted in figure D-2, the drilling fluid pumped from the boring is not conducted through the pump. A high-pressure centrifugal pump is used to circulate water through an eductor to create a vacuum on the drill pipe. Drill pipe and hoses should be of a constant inside diameter throughout the system to assure that material entering the system can be circulated completely through it. The cutter or drill bit can be fabricated by local welding shops. In alluvial deposits, a drag-type bit similar to the cutter head for a dredge is sufficient. However, roller-type cutters are commercially available for use in consolidated deposits. The eductor consists only of a pipe Y with a nozzle fitted into one end of the Y. Optimum circulation rate depends on the size of system. However, once it is accomplished, it is capable of subjecting the drill column to about 28 in. mercury vacuum.

(2) Problems.

(a) It is necessary to maintain an excess hydrostatic pressure on the boring to stabilize the boring walls. This requires that the water column in the open boring be higher than the static water level in the formation. In most materials, a minimum excess head of 7 ft is required, and an even larger excess head is desirable. When the static water level is very near the ground surface, it has been achieved by lowering the water level with well points. If the pressure is derived from a deeper, artesian source, it is necessary to lower the pressure in the aquifer with deep wells. Of course, extensive measures to lower the groundwater may not be economically justified. In such a case, one of the other available drilling methods could be used.

(b) Since the formation in which a well is installed consists predominantly of granular material that is generally very permeable, the loss of water into the formation presents a problem during drilling. A large supply of water is necessary to maintain a completely filled, open boring. A sump should be dug near the hole and connected to it to provide a gravity flow of water. During the drilling, all cuttings brought up through the drill pipe are deposited in the sump, gradually reducing the volume of available water. A sump three times the anticipated volume of the completed boring is adequate. An instantaneous loss of water resulting in loss of excess head can cause failure of the boring walls. Often, during loss of water, if the rotation of the drill bit is stopped the water loss is greatly reduced. The boring must be kept full of water until the well screen, riser, and filter are installed.

(c) In standard-rotary drilling, the flow of drilling fluid can be directed to the leading edges of the drill bit. However, in reverse-circulation rotary drilling this is not possible. Consequently, in zones and layers of cohesive materials the bit can become clogged, greatly reducing its efficiency. If the material is not too sticky, shaking the bit by raising and lowering will clear it. In very sticky materials, it is sometimes necessary to pull the drill string and clean the bit.

d. Bailing and Casing. In cases where reverse-circulation rotary drilling is not successful, an equally acceptable method of drilling consists of bailing a hole while driving a steel casing into the hole to stabilize the boring walls. It does not inject deleterious materials into the formation. Loose to medium dense, clean, granular materials can be bailed economically. If the granular materials are overlain by a cohesive overburden which does not yield easily to a bailer, it is more economical to auger through the overburden.

(1) Equipment. A drill rig with a wire line hoist is adaptable to this method of well installation. It should be remembered that large casing, heavy enough to be successfully driven, presents a sizeable load

to be handled by the drill rig. As will be discussed in the following paragraph, the casing often presents difficulties. The casing should be flush-joint, or welded-joint steel pipe which is smooth on both the inside and outside surfaces. Two types of bailers commonly used are shown in figure D-3. The bailer is operated on a wire line by lowering

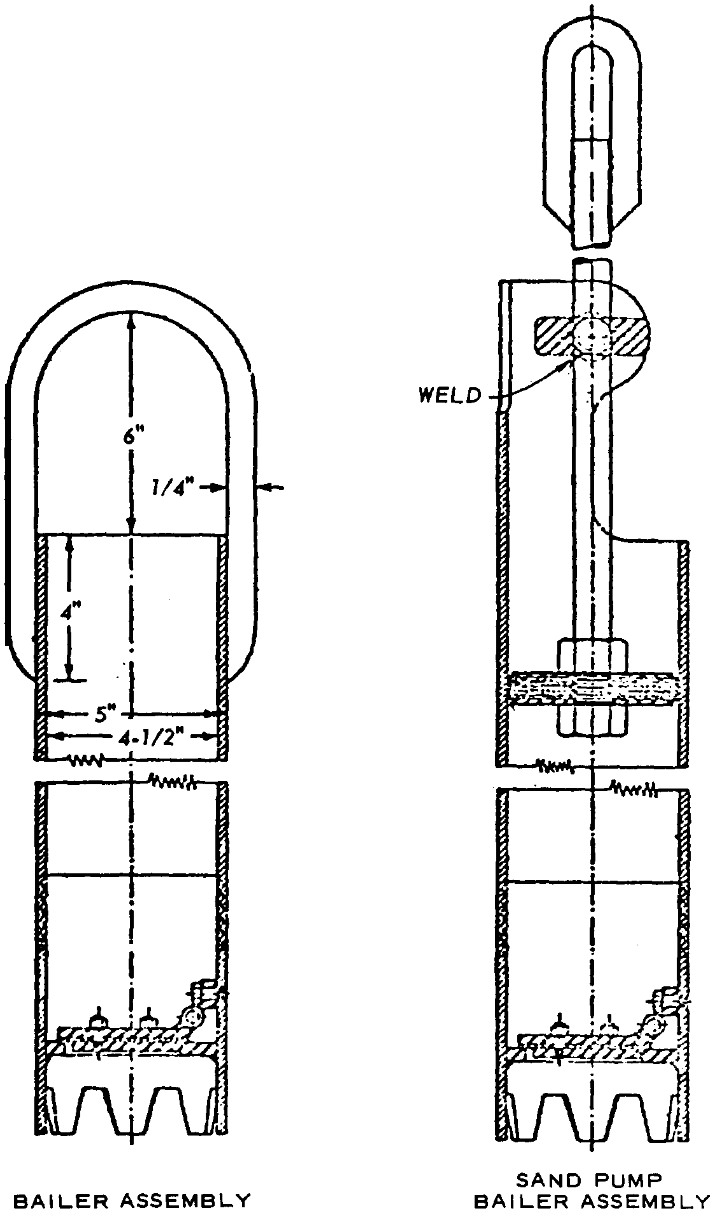


Figure D-3. Bailer and sand pump assemblies.

to the bottom of the boring and quickly jigging it a number of times to fill it.

(2) Problems. This method of drilling produces good results but often presents problems in operations. Thin layers of cohesive materials or cemented materials within the formation can preclude bailing. Driving a casing requires heavy equipment. It is difficult to advance the casing in hard zones. The penetration is also retarded by friction of the granular formation against the outside of the casing. After the casing is set, the boring completed, and the well installed, the casing is removed. Since weight of the casing and the friction of the formation make it difficult to pull the casing, often upward driving must be used. The casing should be pulled as the filter material is placed to avoid the friction of the filter material inside the casing. Casing may also be used with rotary wash-boring methods.

e. Standard Rotary. A recently developed self-destructing, organic drilling fluid additive has made standard-rotary drilling practicable for relief well installation and has gained popularity in the well drilling industry. No bentonite clays are used in the drilling fluid. The standard-rotary drilling method is described in paragraph D-2c. The self-destructing drilling fluid is marketed under the trade name "Revert." The name is derived from the action of the fluid which reverts to the viscosity of water, normally in about three days. The time to reverting can be speeded or retarded by the addition of different chemicals. The potential loss of a hole from loss of drilling fluid and the potential balling of clay in the drill bit are greatly reduced by use of this method.

(1) Equipment. .A rotary-type drill rig of sufficient hoisting and torque capacity is required. The cutter or drill bit can be either of the drag or roller design. The drill pipe should be as large as practicable to increase the volume of fluid to the drill bit and, consequently, increase the velocity of the fluid returning up the open hole.

(2) Problems. The reverting process of the drilling fluid leaves a small amount of slimy ash which, unavoidably, is mixed into the filter material. However, a large percentage of this ash is removed during development of the well. Testing to determine the extent of the detrimental effects caused by this ash residue has not been sufficient to properly evaluate the process for permanent relief wells.

f. Installation of Well Screen and Riser Pipes. Once the boring is completed and the tools withdrawn, the boring should be sounded to assure an open hole to the proper depth. The well screen and riser pipe are fabricated at the factory in lengths varying from 4 ft to approximately 30 ft. The bottom joint of the well screen should be fitted with a cap or plug to seal the bottom of the screen. The lengths of screen are connected together as they are lowered into the hole. The made-up lengths of riser and screen must be determined if the bottom of the screen is to be set accurately at the designed depth. The method of connecting the lengths of screen and riser vary: metal screens and risers have mechanical, bolted, or welded joints; plastic and fiberglass screens have either mechanical or glued joints; wooden screens have mortise and tenon-type joints. Each joint should be made up securely to prevent separation of the well during installation and servicing activities. Each joint should be kept as straight as possible to facilitate ease of servicing and testing. The screen section of the well should be centered in the boring to ensure a continuous filter around the well screen. This can be accomplished by attaching centering spiders to the outside of the screen. Occasionally, during the drilling of a well, a zone of fine silt will be encountered which could penetrate the filter material. In these zones the screen should be replaced by a solid pipe or blank screen to prevent contamination of the well by intrusion of this material. Immediately after installation of the well screen and riser, the total inside depth should be measured to check against the accumulated measurements during installation. The exact inside depth of the well must be known to prevent damage during development and servicing of the well. The equipment required for installation of the screen and riser depends upon the type of material from which it is fabricated, but consists primarily of hoisting equipment.

g. Filter Placement. Caution in design, control of manufacture, and transport of filter materials to the job site can be completely negated by improper placement in the well. Acceptable construction of permanent relief wells demands that the filter be placed without segregation. If the filter consists of varying sizes, it tends to segregate as it passes through water, with coarse particles falling faster than fine particles. A tremie has been used to successfully place filter material without significant segregation. The tremie pipe is placed to the bottom of the open boring and outside the well screen and riser pipe. Once the tremie pipe is completely filled with filter material, it is kept full until filter placement is completed. The first material placed in the tremie pipe falls free through the water and is segregated. For this reason, the hole is drilled several feet deeper than the design elevation of the bottom of the screen to be sure that the segregated filter material is deposited below the bottom of the screen.

(1) Equipment. The hoisting line on the drill rig can be used to handle the tremie pipe. The tremie pipe should consist of steel pipe large enough to facilitate the placement of filter, yet small enough to fit between the screen and the boring wall. For this purpose, 4-in.

line pipe is satisfactory. For convenience it should be cut into short pieces approximately 5 ft in length. If the pipe is slotted to allow free movement of water, it aids in placement of the filter. A small hopper attached to the top of the tremie pipe facilitates the use of a small end-loader to handle the filter material. A mechanical vibrator attached to the tremie pipe aids considerably in the movement of the filter gravel through the pipe.

(2) Operation. After the tremie pipe is made up and the hopper attached, the pipe and hopper are filled with filter material and the tremie pipe is raised off the bottom of the boring. Often the material does not flow freely and it is necessary to shake the tremie pipe by raising and lowering slightly. A mechanical vibrator helps to keep the material flowing. As the hopper runs low on filter material, the tremie should be lowered enough to stop the flow. After the hopper is recharged, the process is continued until the filter material is brought up to a specified distance above the top of the well screen. The filter material should stand above the top of the well screen several feet, depending upon the length of the well screen, to compensate for settlement during development of the well. To prevent segregation, care must be taken in the placement to ensure that the tremie pipe is continuously full. Once a filter is placed with segregated zones, the well will likely produce foundation sands beyond tolerable limits. Should this occur, the well would have to be grouted and abandoned.

D-3. Development of Relief Wells.

a. General. Development of the well removes some of the finer fraction of the filter material close to the well screen and grades the filter from coarse to fine away from the screen.

b. Method. Several methods have been devised for the development of wells. The effectiveness of the method depends on the inside diameter and type slots of the screen and the thickness and gradation of the filter material.

(1) Surging block. The use of surging block, consisting of a heavy steel weight with a diameter slightly smaller than the well screen, is the most effective development method for screens that have louvered or protected openings. The surging block is also effective in screens that have open slots. It is operated on a wire line. The surging block is lowered to the bottom and raised to the top of the well screen repeatedly with a speed of l-1/2 to 3 fps. The block acts as a piston pushing the water out of the screen and through the filter on the downstroke. During the upstroke of the block, water is drawn back through the filter and into the well, bringing some of the finer fraction of the filter

with it. The surging should be stopped at regular intervals and the depth of sand inside the well screen sounded to determine how much material has been brought into the well. When the material inside the well amounts to 1 ft or more, it should be bailed out to allow continued development of the lower part of the filter.

(2) Water jet. A water jet is very effective in developing filters around continuous slot-type, wire-wrapped screens. The water jet equipment consists of small nozzles at the end of a pipe and can be fabricated in local welding shops. The nozzles are directed toward the screen slots. After the jet tip is lowered into the screen, water is pumped down and out through the nozzles at a high velocity. The size and number of nozzles must be consistent with the size and length of the pipe through which the water is pumped to ensure a high-pressure and high-velocity jetting action. This action requires a high-pressure, relatively high-volume water pump. Normally, development with a water jet is started at the bottom of the screen. Jetting is accomplished at one place for a fixed period of time. The jet is raised approximately 0.5 ft, rotated slightly, and jetting is continued for another fixed period of time. This process is continued until the entire well screen has been jetted. Jetting can be repeated a number of times to obtain the optimum amount of development of the well.

(3) Compressed air. Allowing compressed air to escape near the bottom of the well screen results in a surging action through the well and filter. This method is best suited for a well with relatively thin filter thickness and an open-slot screen. The equipment consists of a compressor (approximately 200-cfs capacity) connected to a small (1/2-in.) pipe extended to the bottom of the well. The advantage of using compressed air in this way is that it can be continued unattended over long periods of time and cannot damage the well.

(4) Pumping. Pumping should be accomplished at a sufficient rate to produce drawdown in the well to near the top of the screen. The water passing from the formation through the filter into the well removes part of the finer fraction of the filter material. The pumping equipment required depends on the size, yield, and anticipated drawdown in the well. Pumping, continued over a long period of time, is a reasonably effective method of well development. One of the major disadvantages of this method is that granular material brought into the well will pass through the pump.

c. Regardless of the method of development, a properly developed well will not produce an appreciable amount of sand, and entrance losses through the filter will be reduced to an absolute minimum. In each of the methods discussed above, the actual amount of development must be recorded: the length, diameter, speed, and number of cycles of a surging block; the volume, pressure, and diameter of water jets; the rate of pumping and length of time pumped; compressed air pressure and length of time employed. In addition, the amount of filter and foundation materials brought into the well and bailed out should be recorded. Upon completion of the development of the well, all material infiltrated into the well should be bailed out. It is not always possible to know when a well is properly developed, until it is tested and observations such as drawdown, flow, and infiltration of sand are obtained. Often, it is necessary to resume development procedures after initial testing of the well has indicated a less than satisfactory efficiency. A permanent relief well that continues to produce sand in excess of the specified tolerance (on the order of 2 pt per hr) is not acceptable.

D-4. Testing of Relief Wells.

a. General. Performance of relief wells is determined by pumping tests. The pumping test is used primarily to determine the specific yield of the well and the amount of sand infiltration produced by the pumping. The information from this test will not only determine the acceptability of the well, but will also be the basis for evaluating any changes in performance or loss of efficiency with time. The results of this pumping test must be made a part of the permanent record concerning the well.

b. Equipment. The equipment required for a pumping test consists of a pump of adequate size to achieve a substantial drawdown. If the water level in the well is near enough to the ground surface and the specific yield of the well is high enough to produce a substantial flow (300 gpm) with a small drawdown, a centrifugal pump may be used. If the water level in the well is more than about 18 to 20 ft below ground surface, a deep-well pump will be required to affect substantial drawdown. A flow meter is required to measure the pumping rate. A flat-bottom sounding device and a steel tape are required to determine the amount of sand infiltration deposited in the bottom of the well. A suitably baffled stilling basin is used to determine the amount of sand in the effluent. A sounding device suitable for determining the depth to the top of water is needed to find the exact drawdown in the well. A well flow meter is desirable to measure the amount of flow at various depths within the well to define flow from various zones.

c. Pumping. The well must be pumped to obtain a specified drawdown or flow rate. Drawdown measurements in the well should be made to the nearest 0.01 ft and recorded with the flow rate at 15-min intervals throughout the duration of the tests. Sufficient sand infiltration determinations are necessary to establish an infiltration rate for each hour of the pumping test. The length of time that the pumping test must be continued is normally specified for the particular project.

d. Records. Permanent records of the installation, development, and testing of a permanent relief well must be kept for evaluation of future testing. To monitor the efficiency and performance of the installation, the record must include identification of the well, method of drilling, type and size of well screen, and slot size. The filter should be defined as to type, depth, and thickness. Elevation of the top of the well and the ground surface should be recorded. An abbreviated log of the boring should be included to define the depth to granular material, the thickness of that material, and the percent penetration of the well. Development data should include the method of development, the amount of effort expended in development, and the amount of material pulled into the well during development. The pumping test data should include the rate of pumping, the amount of drawdown, the length of time the pumping test was conducted, and the amount of sand infiltration during pumping. Forms should be filled in completely at the time each operation is completed and any additional observations should be recorded in a "remarks" section.

APPENIDX E

FILTER DESIGN

E-1. A filter material must meet two basic requirements: (a) The filter material must be fine enough to prevent infiltration of the material from which drainage is occurring, and (b) the filter material must be much more permeable than the material being drained. These requirements are referred to as the "stability" and "permeability" criteria, respectively. Through the years criteria have evolved from laboratory tests and field experience that allow the designer to confidently design in most instances a filter material that will meet the two basic requirements. Where it is not possible to meet the criteria given in the following paragraphs, the design must be based on carefully controlled laboratory filtration tests.

E-2. To prevent infiltration of the material being drained into the filter material, the following conditions should be met:

Stability

$$\frac{\text{15 percent size of filter material}}{\text{85 percent size of material being drained}} \leq 5 \qquad \text{(E-1)}$$

and

$$\frac{\text{50 percent size of filter material}}{\text{50 percent size of material being drained}} \leq 25 \qquad \text{(E-2)}$$

To assure the filter material is much more permeable than the material being drained, the following condition should be met:

Permeability

$$\frac{\text{15 percent size of filter material}}{\text{15 percent size of material being drained}} \geq 5 \qquad \text{(E-3)}$$

The permeability of a soil is approximately proportional to the square of its 15 percent size. Therefore, the criterion given by equation E-3 assures the permeability of the filter material is at least 25 times the permeability of the material being drained. Normally, such differences in permeability allow water to drain freely into the filter and then safely discharge into a ditch or some other type of collector system. However, where the inflow of seepage is great, hydraulic gradients available for removing the water are small, and areas for discharging the flow from the filter are limited (or any one or combination of these factors exist), the water-removing capacity of the system should be analyzed. This generally requires estimating the quantity of seepage and then determining the required thickness and permeability of filter material from flow net analyses.

E-3. The previously given filter criteria are applicable for all soils with gradation curves approximately parallel except for CL or CH soils. Where the gradation curves are not approximately parallel, the filter design should be based on filtration tests. For CL and CH soils without sand or silt partings, the 15 percent size of the filter in equation E-l may be as great as 0.4 mm and equation E-2 may be disregarded. However, if the drained material should contain partings or strata of uniform nonplastic fine sand and silt sizes, the filter must be designed to meet the stability and permeability criteria.

E-4. The following criteria are allicable for preventing infiltration of filter material into perforated pipe, screens, etc.:

Circular openings

$$\frac{\text{50 percent size of filter material}}{\text{hole diameter}} \geq 1.0 \qquad \text{(E-4)}$$

Slotted openings

$$\frac{\text{50 percent size of filter material}}{\text{slot width}} \geq 1.2 \qquad \text{(E-5)}$$

In many instances a filter material meeting the criteria given by equations E-l through E-3 relative to the material being drained is too fine to meet the criteria given by equations E-4 and E-5. In these instances multilayered or "graded" filters are required. In a graded filter each layer meets the requirements given by equations E-l through E-3 with respect to the previous layer with the final layer also meeting the requirements given by equation E-4 or E-5 when a collector pipe is used. Graded filter systems may also be needed when transitioning from fine to coarse materials in a zoned embankment or where coarse material is required for improving the water-carrying capacity of the system.

E-5. The preceding criteria cannot, in most instances, be applied directly to protect severely gap- or skip-graded soils. In a gap-graded soil such as that shown in figure E-l, the coarse material simply floats in the matrix of fines. Consequently, the scattered coarse particles will not deter the migration of fines as they do in a well-graded material. For such gap-graded soils, the filter should be designed to protect the fine matrix rather than the total range of particle sizes. This is illustrated in figure E-l. The 85 percent size of the total sample is 5.2 mm. Considering only the matrix material, the 85 percent

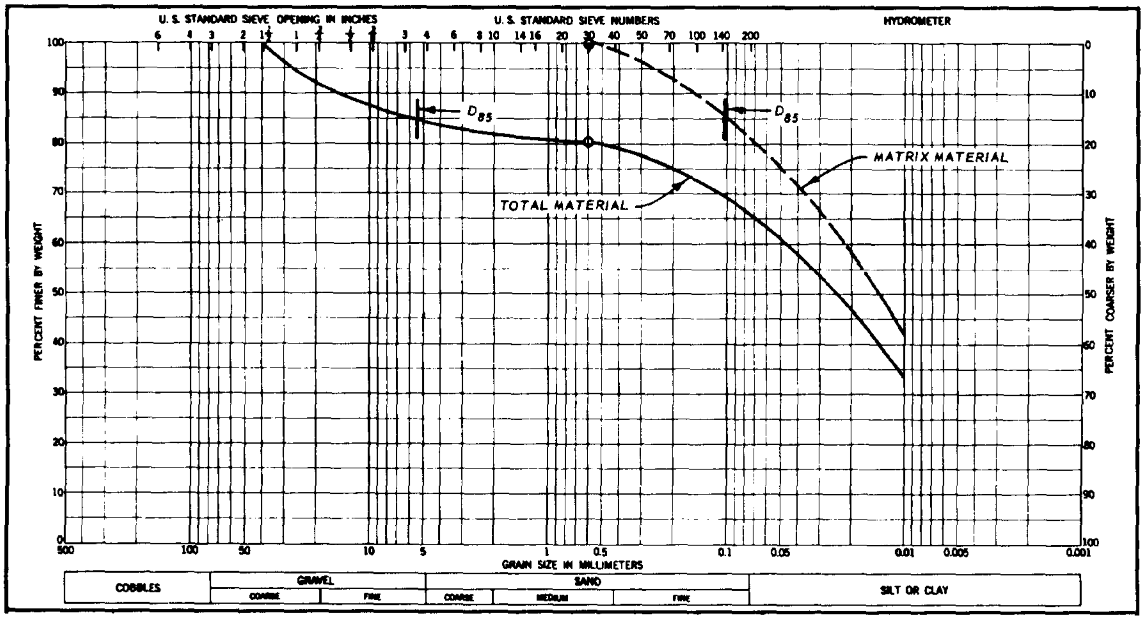


Figure E-1. Analysis of gap-graded material.

size would be 0.1 mm resulting in a much finer filter material being required. This procedure may also be followed in some instances where the material being drained has a very wide range of particle sizes (e.g., materials graded from coarse gravels to significant percentages of silt or clay). For major structures such a design should be checked with filtration tests.

E-6. A gap-graded filter material must never be specified or allowed since it will consist of either the coarse particles floating in the finer material or the fine material having no stability within the voids produced by the coarse material. In the former case the material may not be permeable enough to provide adequate drainage. The latter case is particularly dangerous since piping of the protected material can easily occur through the relatively large, loosely filled voids provided by the coarse material.

E-7. The designer should specify that the filter material must not become segregated or contaminated prior to, during, and after installation. Segregation results in zones of material too fine to meet the permeability requirements and other zones too coarse to meet the stability requirements. Contamination of the filter material from muddy water, dust, etc., can clog the voids in the material rendering the drainage system useless. In the event the filter material becomes segregated or contaminated, it should be removed and replaced with acceptable material.

E-8. Seldom, if ever, is a single gradation curve representative of a given material. A material is generally represented by a gradation band which encompasses all the individual gradation curves. Likewise, the required gradation for the filter material is also given as a band. The design of a graded filter which shows the application of the filter criteria where the gradations are represented by bands is illustrated in figure E-2. Tests on the soil being drained indicate it varies from a CL to a CH material. Therefore, the criterion that the 15 percent size

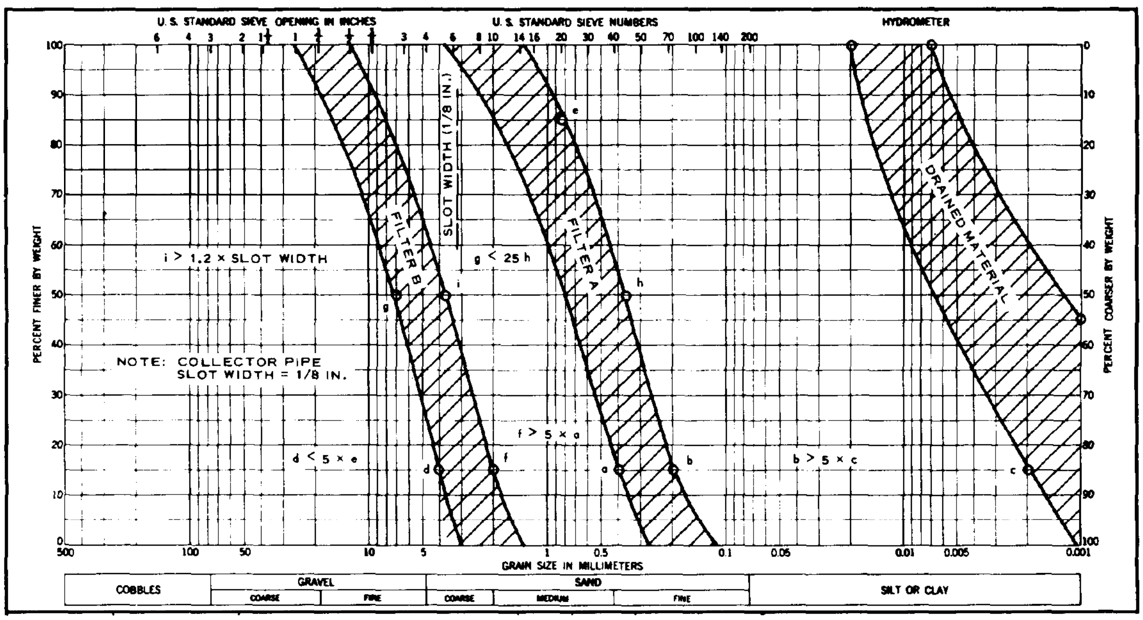


Figure E-2. Illustration of the design of a graded filter.

of the filter material cannot exceed 0.4 mm applies and point a is established in figure E-2. Filter material graded within a band such as that shown for Filter Material A in figure E-2 is acceptable based on the stability criteria. (The fine limit of the band was arbitrarily draw-n and in this example is intended to represent the gradation of a readily available material.) A check is then made to assure the 15 percent size of the fine limit of the filter material band (point b) is equal to or greater than 5 times the 15 percent size of the coarse limit of the drained material band (point c). Filter Material A meets both the stability and permeability requirements and is a suitable filter material for protecting the drained material.

E-9. Although Filter Material A meets the filter criteria with respect to the material being drained, it does not meet the criteria with respect to the circular openings (1/4 in.) in the collector pipe and consequently a graded filter is required. The second filter material must meet the criteria given by equations E-1 through E-3 with respect to Filter Material A. For stability, the 15 percent size of the coarse limit of the gradation band for the second filter (point d) cannot be greater than 5 times the 85 percent size of the fine limit of the gradation band for Filter Material A (point e). For permeability, the 15 percent size of the fine limit (point f) must be at least 5 times greater than the 15 percent size of the coarse limit for Filter Material A (point a). With points d and f established, the fine and coarse limits for Filter Material B may be established by drawing curves through the points approximately parallel to the respective limits for Filter Material A. A check is then made to assure the 50 percent size of the coarse limit for Filter Material B (point g) is no greater than 25 times the 50 percent size of the fine limit of Filter Material A (point h). Filter Material B is also coarse enough to prevent infiltration into the pipe since the 50 percent size of the fine limit is equal to or greater than the l/8-in. opening of the pipe perforation.

E-10. Figure E-2 is intended to show only the principles of filter design. A two-layer filter system is normally all that is ever required. The cost of a three-layer system would be excessive and should be avoided if at all possible.

APPENDIX F

NOTATION

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

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

Symbol	Term
L_3	Length of top stratum landward of levee toe
M_d	Slope of hydraulic grade line
Q	Shear test for specimen tested at constant water content (unconsolidated-undrained)
Q_s	Total amount of seepage passing beneath levee
R	Shear test for specimen consolidated and then sheared at constant water content (consolidated-undrained)
s	(a) distance from the landside toe of the levee to the point of effective seepage entry (b) shear test for specimen consolidated and sheared without restriction of change in water content (consolidated-drained)
x_1	Effective length of riverside blanket
x_3	Distance from landside levee toe to effective seepage exit
z_b	Effective thickness of stratum
z_t	Transformed thickness of top stratum
z_{bl}	Effective thickness of landside top stratum
z_{br}	Effective thickness of riverside top stratum
z_{bt}	Effective thickness of top stratum
γ_t	Wet unit weight of soil
γ_w	Unit weight of water
γ'	Submerged or buoyant unit weight of soil
ϕ'	Angle of internal friction based on effective stresses
$\emptyset$	Shape factor to generalized cross section of the levee and foundation

INDEX

*U.S. GOVERNMENT PRINTING OFFICE : 1978 o-721-613/782