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

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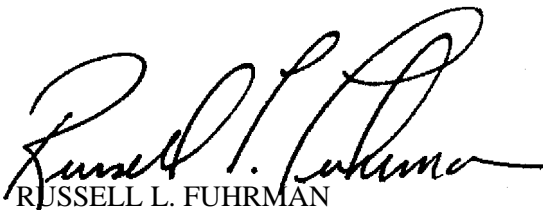
Manual  
No. 1110-2-1913

30 April 2000

**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. Distribution.** This manual is approved for public release; distribution is unlimited.
- 4. 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 COMMANDER:



RUSSELL L. FUHRMAN  
Major General, USA  
Chief of Staff

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This manual supersedes EM 1110-2-1913, dated 31 March 1978.



EM 1110-2-1913  
30 April 2000

**US Army Corps  
of Engineers  
ENGINEERING AND DESIGN**

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# **Design and Construction of Levees**

**ENGINEER MANUAL**

## **AVAILABILITY**

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Engineering and Design  
**DESIGN AND CONSTRUCTION OF LEVEES**

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## 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

Appendix A contains a list of required and related publications pertaining to this manual. Unless otherwise noted, all references are available on interlibrary loan from the Research Library, ATTN: CEWES-IM-MI-R, U.S. Army Engineer Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

### 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 1-1. Information for implementing this procedure is presented in subsequent chapters.

**Table 1-1**  
**Major and Minimum Requirements**

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: <ul style="list-style-type: none"><li>a. Additional information on soil profiles.</li><li>b. Undisturbed strengths of foundation materials.</li><li>c. More detailed information on borrow areas and other required excavations.</li></ul>
4	Using the information obtained in Step 3: <ul style="list-style-type: none"><li>a. Determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas.</li><li>b. Compute rough quantities of suitable material and refine borrow area locations.</li></ul>
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: <ul style="list-style-type: none"><li>a. Underseepage and through seepage.</li><li>b. Slope stability.</li><li>c. Settlement.</li><li>d. Trafficability of the levee surface.</li></ul>
7	Design special treatment to preclude any problems as determined from Step 6. Determine surfacing requirements for the levee based on its expected future use.
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.

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

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**Table 1-2**  
**Classification of Levees According to Use**

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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.

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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.

## Chapter 2 Field Investigations

### 2-1. Preliminary and Final Stage

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. Since 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 depositional environment of 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.

**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 exceed 3 m (10 ft)
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

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**Table 2-2**  
**Stages of Field Investigations**

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1. Investigation or analysis produced by field reconnaissance and discussion with knowledgeable people is adequate for design where:
    - a. Levees are 3 m (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:*
    - 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:* Required for all cases except those in 1 above. Use to decide the need for and scope of subsurface exploration and field testing:
    - 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 (e.g., seismic or electrical resistivity) or cone penetrometer test 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.
- 

## 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. The GIS (Geographic Information System) became used extensively in major range of projects. It is capable of compiling large multi-layered data bases, interactively analyzing and manipulating those data bases, and generating and displaying resultant thematic maps and statistics to aid in engineering management decisions. Federal, state, and private organizations provide free internet access to such systems. Table 2-3 shows some of the contour maps GIS systems provide.

**Table 2-3**  
**Types of Contour Maps**

Contour Type	Uses	
Geologic Structure Elevation Maps	Contour maps in which each line represents the elevation of the top of a geological material or facies	GIS can produce these maps based on the selection of one of four structure parameters
Geologic formations	Contours the top of a user-defined geologic formation	
Blow counts	Contours the top of a structure identified by the first, second, or third occurrence of a specified range of blow counts	A blow count is defined as the number of standard blows required to advance a sampling device into 150 mm (6 in.) of soil
Soil units	Contours the top of a structure identified by the first, second, or third occurrence of one or more soil types	
Fluid level elevation - water table contour maps	Show elevation data (hydraulic head) from unconfined water bearing units where the fluid surface is in equilibrium with atmospheric pressure	Help to evaluate the direction of ground water flow and the energy gradient under which it is flowing
Fluid level elevation - potentiometric surface maps	Show elevation data from confined water bearing units where the fluid surface is under pressure because of the presence of a confining geologic unit	
Hydraulic conductivity	Show the rate of water flow through soil under a unit gradient per unit area	GIS stores vertical and horizontal conductivity data for up to five water bearing zones
	Portray the variations in the water-bearing properties of materials which comprise each water bearing zone	Necessary parameter for computing ground water flow rates, which is important since groundwater velocity exerts a major control on plume shape

## 2-4. Field Survey

The field survey is commenced after becoming familiar with the area through the office study. Walking the proposed alignment and visiting proposed borrow areas are 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. All maps should be to the same scale, and overlays of maps, e.g., topography and soil type, aerial photograph and topography, etc., to facilitate information correlation is desirable. The development of a project GIS will simplify and expedite consistently georeferenced map products.

*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 geology of the area, 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 2-4, but may also include geophysical surveys which are discussed later.

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**Table 2-4**  
**Phase I Boring and Sampling Techniques**

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Technique	Remarks
1. Disturbed sample borings	
a. Split-spoon or standard penetration test	1-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 EM 1110-1-1906  Preferred for general exploration of levee foundations; indicates need and locations for undisturbed samples
b. Auger borings	1-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

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## 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 127-mm- (5-in.-) diameter samples are taken in cohesive materials and 76.2-mm- (3-in.-) diameter samples are taken in cohesionless materials. EM 1110-1-1906 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 60 to 300 m (nominally 200 to 1,000 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 proposed levee at its highest point but not less than 3 m (nominally 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 3 m (nominally 10 ft) below the bottom of the proposed excavation.

## 2-10. Geophysical Exploration

*a.* It is important to understand the capabilities of the different geophysical methods, so that they may be used to full advantage for subsurface investigations. Table 2-5 summarizes those geophysical methods most appropriate to levee exploration. These methods are a fairly inexpensive means of exploration and are very useful for correlating information 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. 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. Applicable geophysical ground surface exploration methods include: (1) seismic methods, (2) electrical resistivity, (3) natural potential (SP) methods, (4) electromagnetic induction methods, and (5) ground penetrating radar. Information obtained from seismic 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 low resistivity such as clayey strata. Both methods require differences in properties of levee and/or foundation materials in order to be effective. The resistivity method requires a resistivity contrast between materials being located, while the seismic method requires contrast in wave transmission velocities. Furthermore, the seismic refraction method requires that any underlying stratum transmit waves

**Table 2-5**  
**Applicable Geophysical Methods of Exploration<sup>a</sup>**

	Top of Bedrock	Fault Detection	Suspected Voids or Cavity Detection	In Situ Elastic Moduli (Velocities)	Material Boundaries, Dip, ...	Subsurface Conduits and Vessels	Landfill Boundaries
Seismic Refraction	W	S		W	S		
Seismic Reflection	S	S	S		W		
Natural Potential (SP)						S	
DC Resistivity	S	S	S		S	S	W
Electro- Magnetics		S			S	W	W
Ground Penetrating Radar	S	S	S		S	S	S
Gravity		S	S		S		
Magnetics		S					S

W - works well in most materials and natural configurations.

S - works under special circumstances of favorable materials or configurations.

Blank - not recommended.

<sup>a</sup> After EM 1110-1-1802.

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 1110-1-1802 describes the use of both seismic refraction and electrical resistivity. Telford et al. (1990) is a valuable, general text on geophysical exploration. Applicable geophysical exploration methods based on operation from the ground surface are summarized in Table 2-5. A resistivity survey measures variations in potential of an electrical field within the earth by a surface applied current. Variation of resistivity with depth is studied by changing electrode spacing. The data is then interpreted as electrical resistivity expressed as a function of depth. (Telford et al. 1990; EM 1110-1-1802)

c. Downhole geophysical logging 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 not only to allow cost savings, but the speed, efficiency and often much more reliable information without lessening the quality of the information obtained. Electromagnetic (EM) induction surveys use EM transmitters that generate currents in subsurface materials. These currents produce secondary magnetic fields detectable at the surface. Simple interpretation techniques are advantages of these methods, making EM induction techniques particularly suitable for horizontal profiling. EM horizontal profiling surveys are useful for detecting anomalous conditions along the centerline of proposed levee construction or along existing levees. Self potential (SP) methods are based on change of potential of ground by human action or alteration of original condition. Four electric potentials due to fluid flow, electrokinetic or streaming, liquid junction or diffusion, mineralization, and solution differing concentration, are known.

The qualitative application of this method is relatively simple and serves best for detection of anomalous seepage through, under, or around levees (Butler and Llopis, 19909; EM 1110-1-1802).

### 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 2-6.

**Table 2-6**  
**Preliminary Appraisal of Foundation Strengths**

Method	Remarks
1. Split-spoon penetration resistance	1-a. Unconfined compressive strength in hundreds kPa (or tons per square foot), of clay is about 1/8 of number of blows per 0.3 m (1 ft), 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 soil 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 may underestimate actual values ; useful only for preliminary strength classifications

## 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 undrained shear strength. The apparatus and procedure for performing this test are described in ASTM D 2573. The results from this test may be greatly in error where shells or fibrous organic material are present. Also, test results in high plasticity clays must be corrected using empirical correction factors as given by Bjerrum (1972) (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. 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.

## **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. 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 gives procedures for performing field pumping tests.

## Chapter 3

### Laboratory Testing

#### 3-1. General

*a.* Reference should be made to EM 1110-1-1906 for current soil testing procedures, and to EM 1110-2-1902 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.

**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: <ul style="list-style-type: none"> <li><i>a.</i> Foundation clays are highly compressible</li> <li><i>b.</i> Foundations under high levees are somewhat compressible</li> <li><i>c.</i> Settlement of structures within levee systems must be accurately estimated</li> </ul> <p>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.</p>
Compaction	<ul style="list-style-type: none"> <li><i>a.</i> Required only for compacted or semi-compacted levees</li> <li><i>b.</i> Where embankment is to be fully compacted, perform standard 25-blow compaction tests</li> <li><i>c.</i> Where embankment is to be semi-compacted, perform 15-blow compaction tests</li> </ul>
Shear strength	<ul style="list-style-type: none"> <li><i>a.</i> Unconfined compression tests on saturated foundation clays without joints or slickensides</li> <li><i>b.</i> Q triaxial tests appropriate for foundation clays, as undrained strength generally governs stability</li> <li><i>c.</i> 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</li> <li><i>d.</i> Q, R, and S tests on fill materials compacted at appropriate water contents to densities resulting from the expected field compaction effort</li> </ul>

**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 $\phi'$ can be assumed based on S tests on similar soils. In seismically active areas, cyclic triaxial tests may be performed

### 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 (Figure 3-1), 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 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.

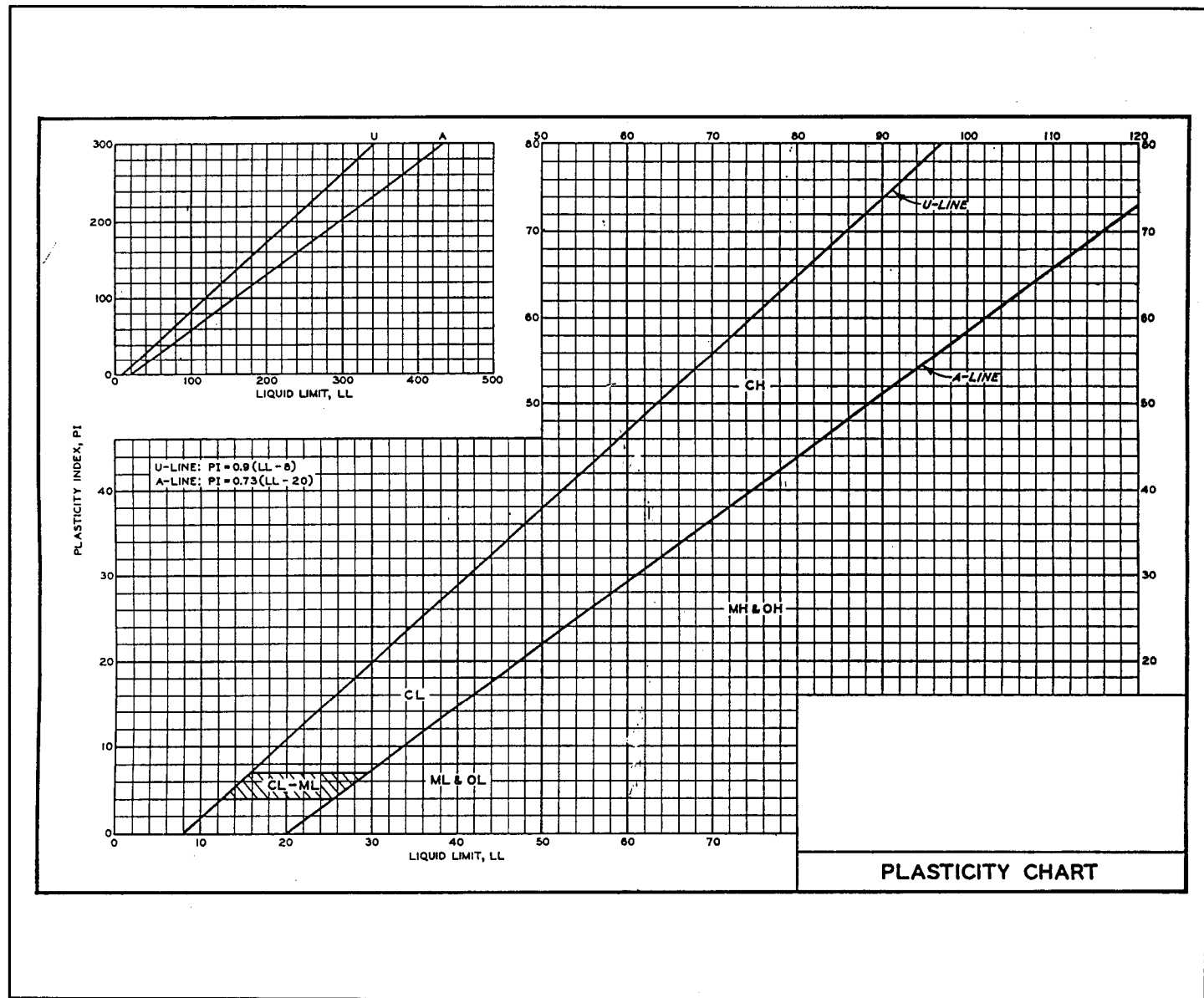
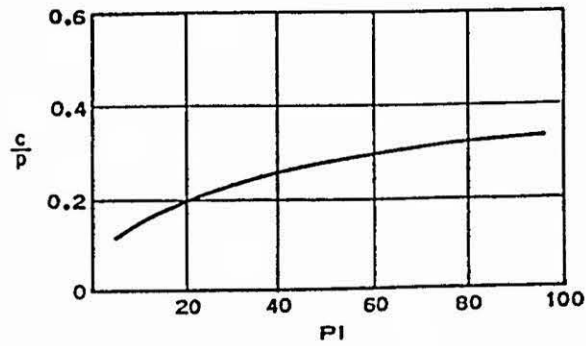
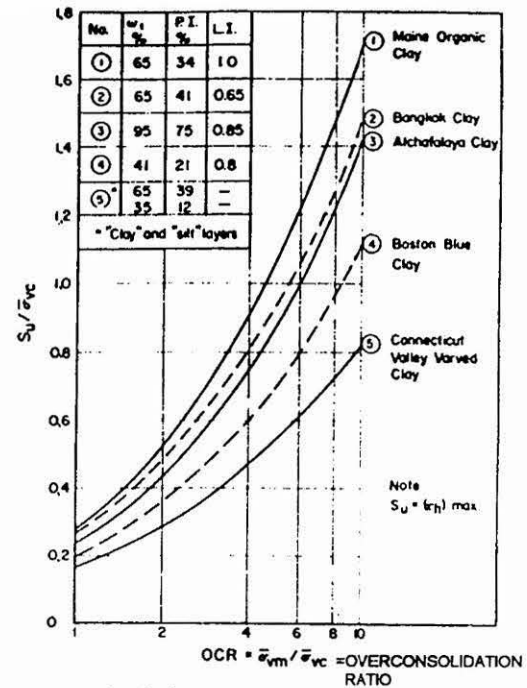


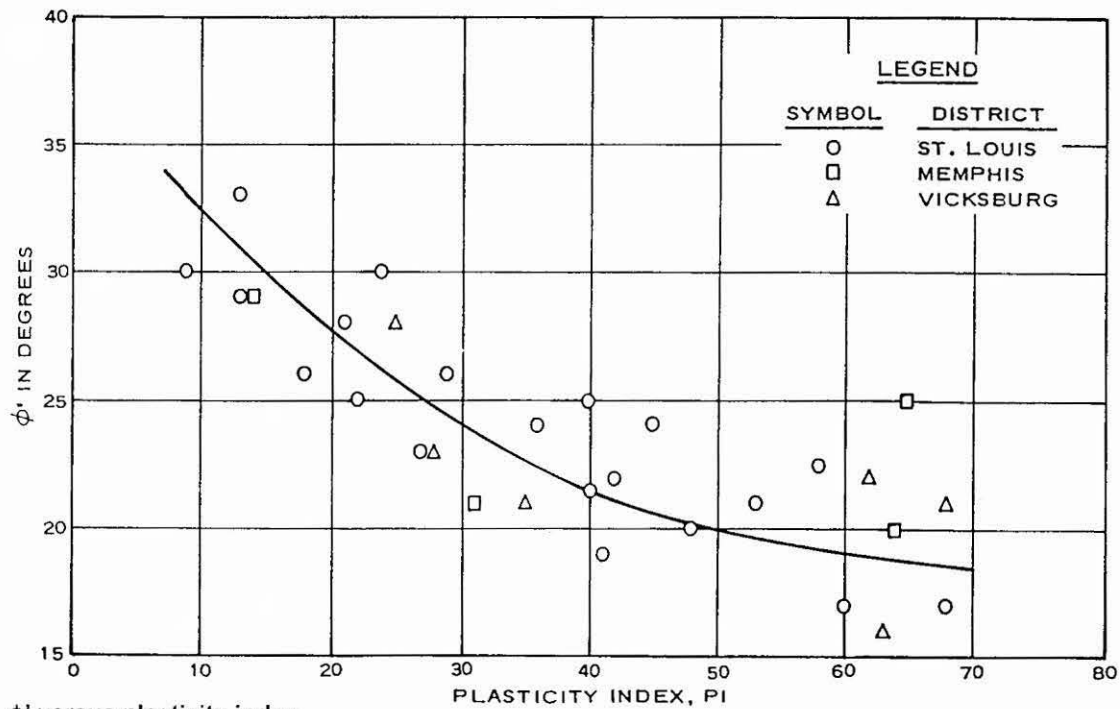
Figure 3-1. Plasticity chart (ENG Form 4334)



a.  $c/p$  versus plasticity index for normally consolidated soils (after Bjerrum, ref. A-2)

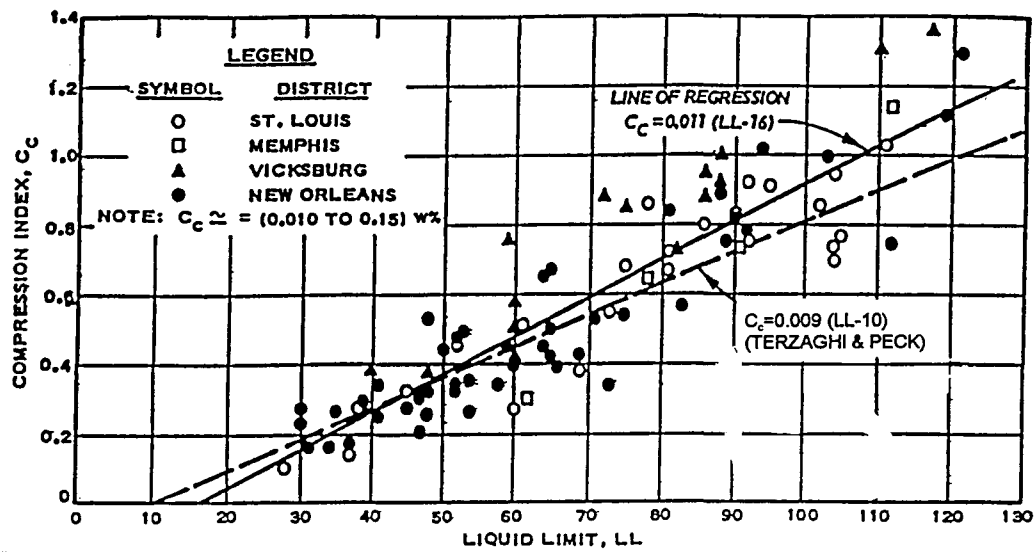


b.  $S_u/\sigma_{vc}$  versus overconsolidation ratio (after Ladd and Foott, 1974, Ref. A-2)

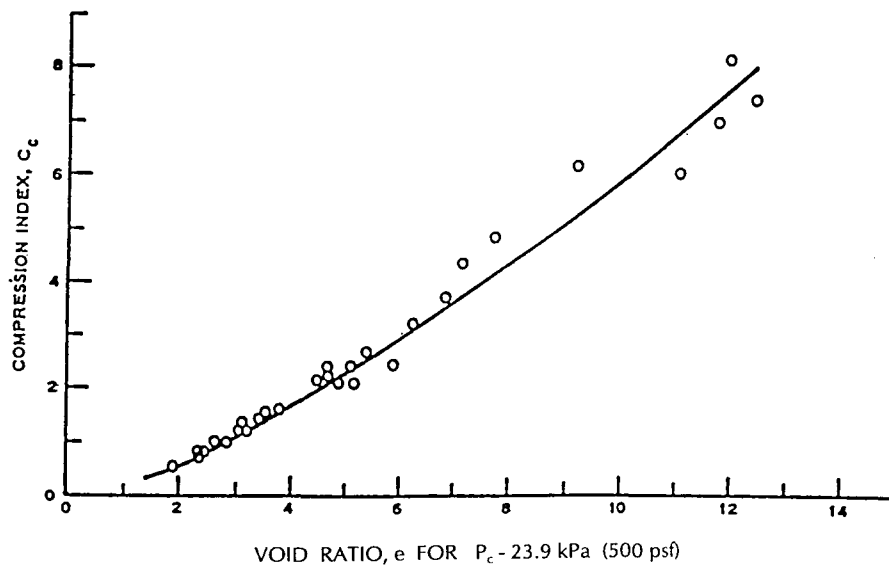


c.  $\phi'$  versus plasticity index

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

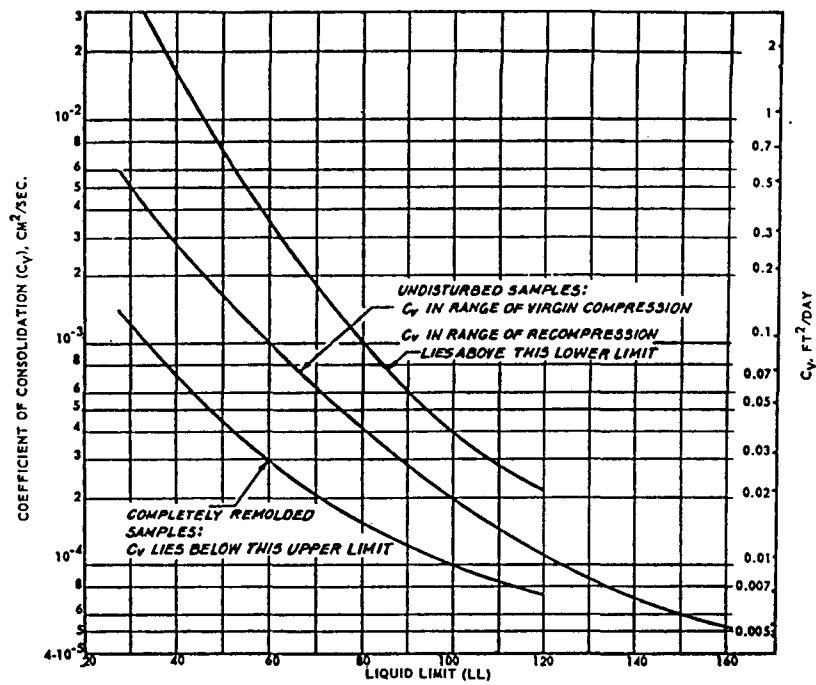


a. Compression index versus liquid limit for normally consolidated soils

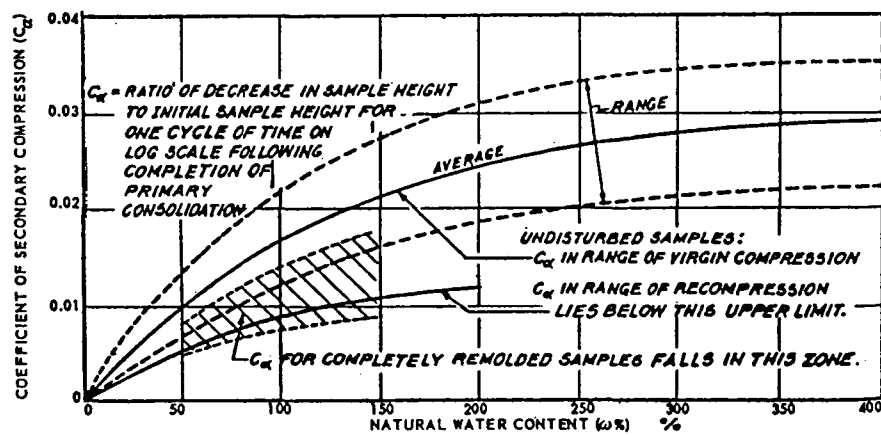


b. Compression index versus initial void ratio for tidal marsh

Figure 3-3. Example correlations for consolidation characteristics of fine-grained soils (after Kapp, ref. A-2)



c. Coefficient of consolidation versus liquid limit (from NAVFAC DM-7 ref. A-1)



d. Coefficient of secondary compression versus water content (from NAVFAC DM-7 ref. A-1)

Figure 3-3. (Concluded)

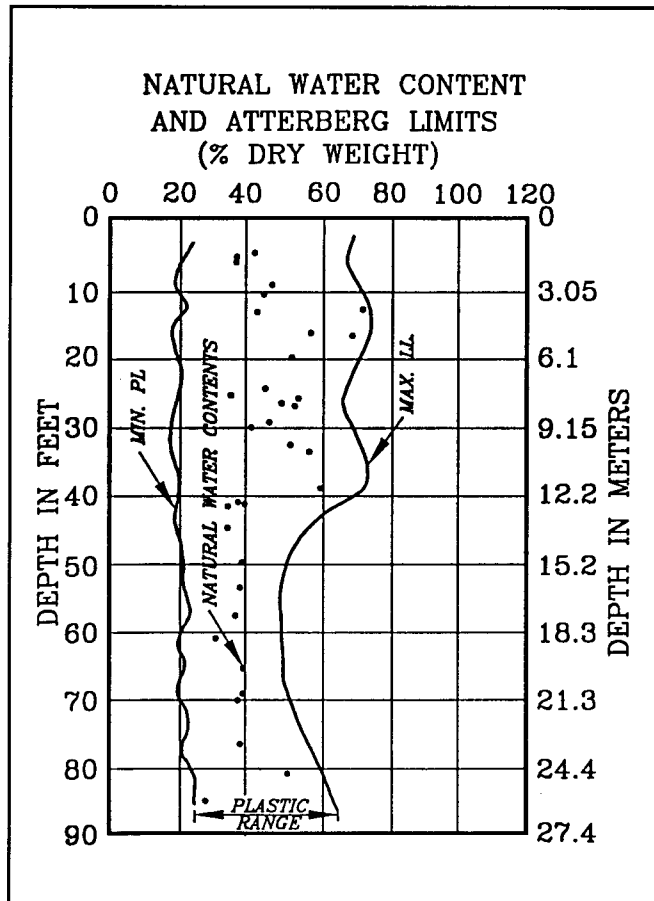


Figure 3-4. Comparisons of Atterberg limits and natural water contents

tions 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 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.

### 3-4. Shear Strength

Approximate shear strengths of fine-grained cohesive soils can be rapidly 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

*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 ( $\phi$ ) 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 density tests on available pervious borrow materials should be performed in accordance with ASTM D 4253 so that relative compaction requirements for pervious fills may be checked in the field when required by the specification. Due to the inconsistencies in duplicating minimum densities (ASTM D 4254), relative density may not be used. Factors such as (but not limited to) site specific materials, availability of testing equipment and local practice may make it more practical to utilize methods other than ASTM D 4253 and ASTM D 4254 to control the degree of compaction of cohesionless material. The other methods used include comparison of in-place density to either the maximum Proctor density or the maximum density obtained by ASTM 4253 (if vibratory table is available).

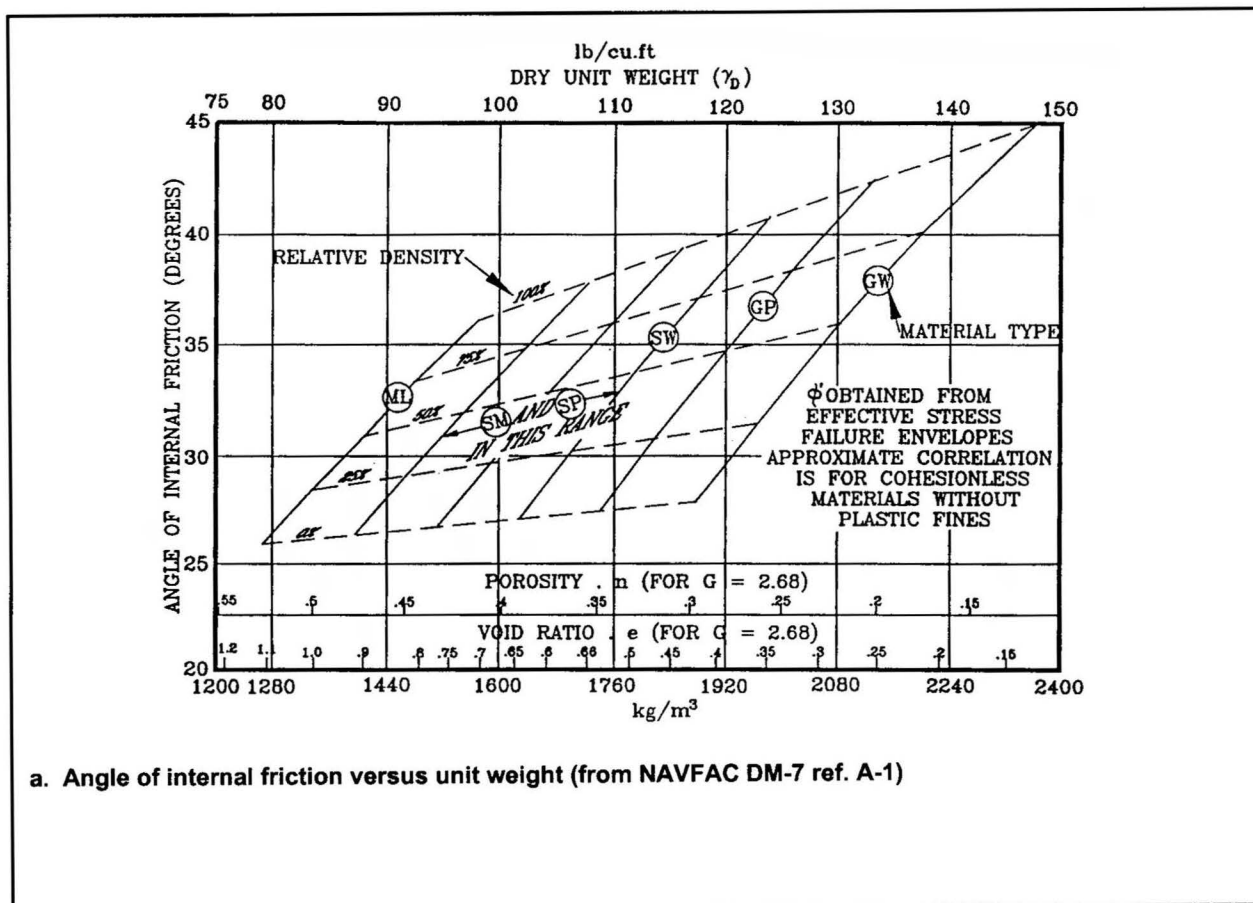
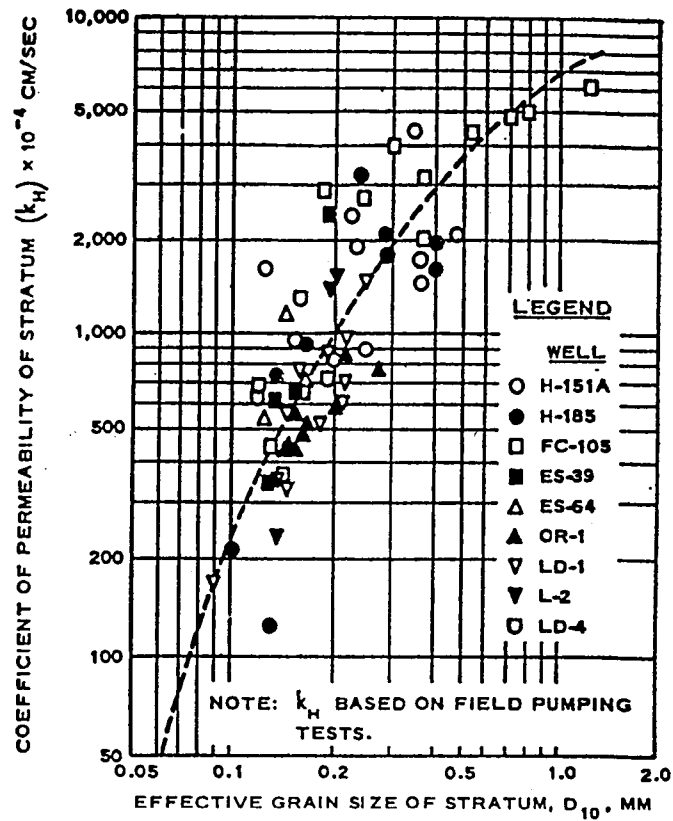


Figure 3-5. Example correlations for properties of coarse-grained soils



b. Effective grain size,  $D_{10}$  versus coefficient of permeability,  $k_h$  (from WES TM No. 3-424, ref. A-1)

Figure 3-5. (Concluded)

## 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 may be considered for portions of levees. 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.* Where possible, 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. Minimum berm widths used frequently in the past are 12.2 m (40 ft) riverside and 30.5 m (100 ft) landside, but berm widths should be the maximum practicable since borrow areas may 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 0.91 m (3 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 4.6 to 6.1 m (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

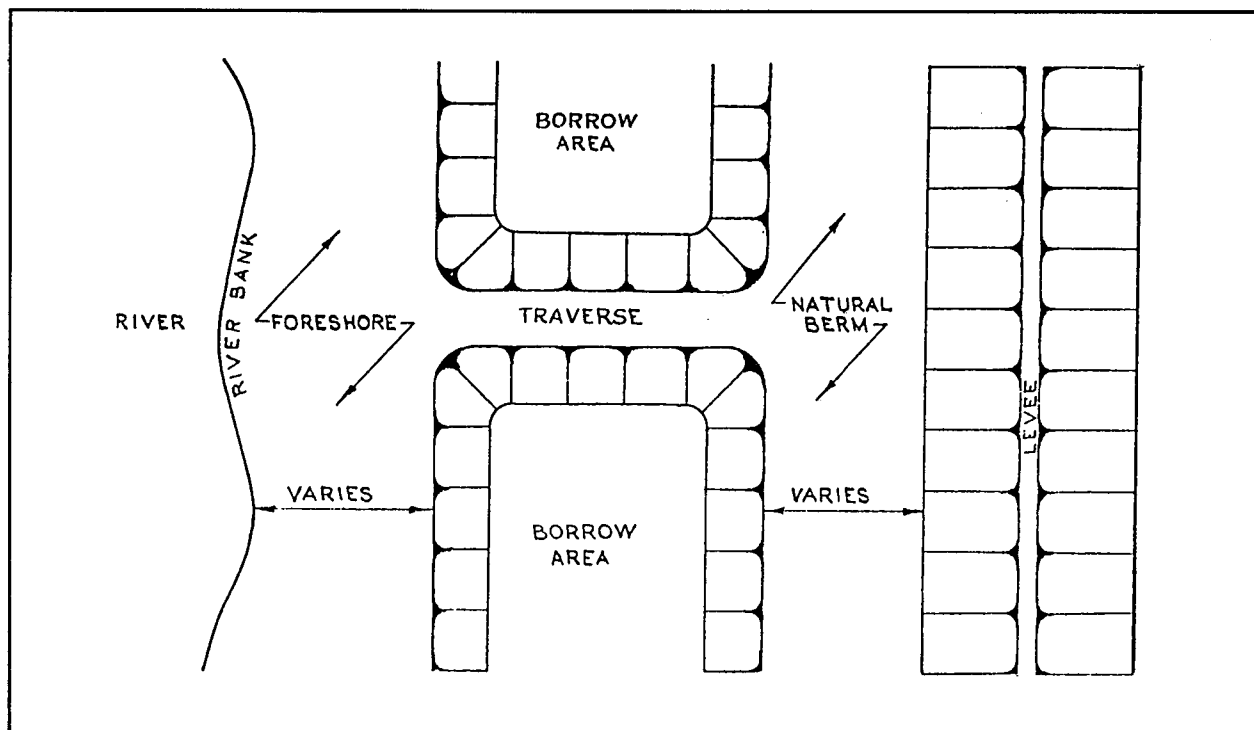
*a. Slopes.* Excavation slopes of borrow areas should be designed to assure stability. This is particularly important for slopes adjacent to the levee but could also be important for any slope whose top is near the right-of-way limits. Borrow area slopes must also be flat enough to allow mowing, if required. 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 foreshore is specified (i.e., the borrow excavation is not to be cut into the riverbank), it should have a substantial width, say 61 m (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



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

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 become standard 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 practicable, 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. Guidelines for landscape planting are given in EM 1110-2-301.

*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 guidance is given in Appendixes B and C. Turnbull and Mansur (1959) have proposed control measures for underseepage also. Additional information on seepage control in earth foundations including cutoffs, impervious blankets, seepage berms, relief wells and trench drains is given in EM 1110-2-1901 and EM 1110-2-1914.

#### **5-2. Cutoffs**

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

#### **5-3. Riverside Blankets**

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

#### 5-4. Landside Seepage Berms

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

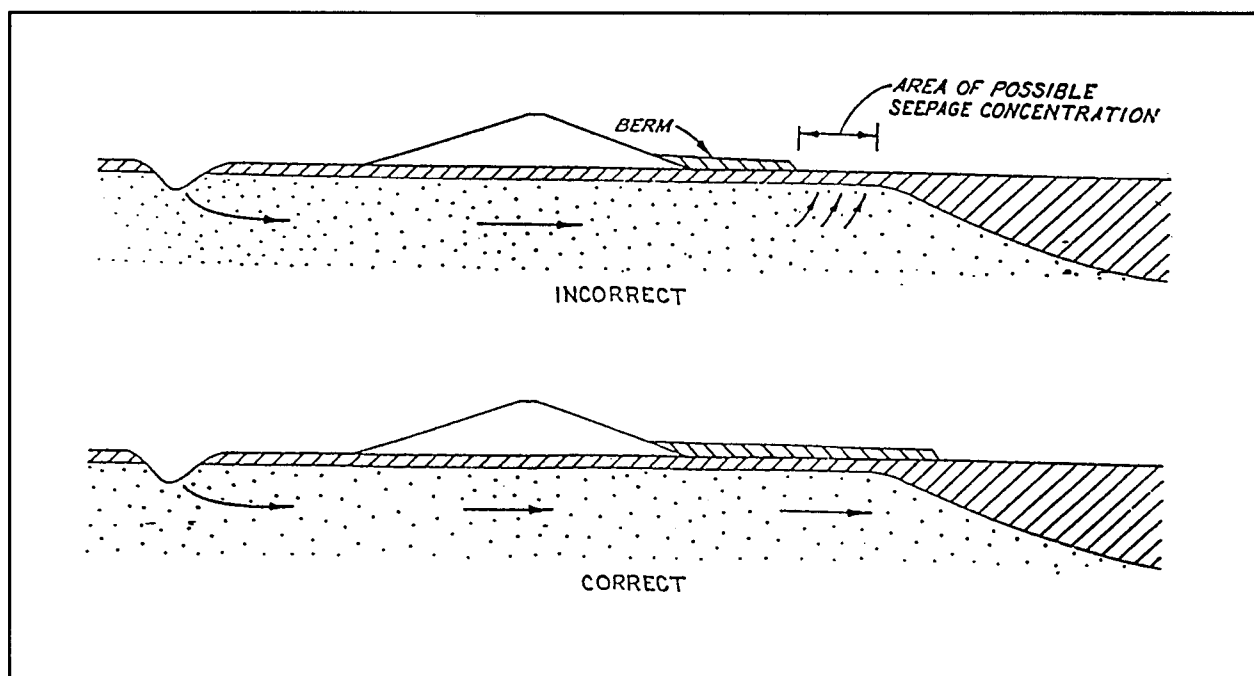


Figure 5-1. Example of incorrect and correct berm length according to existing foundation conditions

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

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

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

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

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

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

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

*d. Computer programs to use for seepage analysis.*

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

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

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

## 5-5. Pervious Toe Trench

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

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

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

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

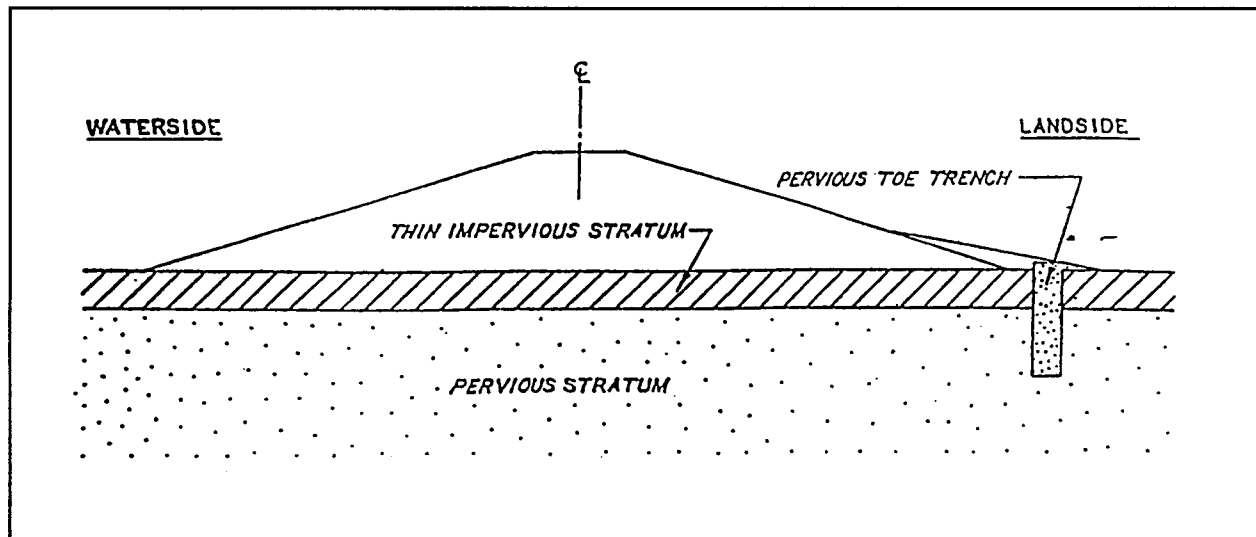


Figure 5-2. Typical partially penetrating pervious toe trench

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

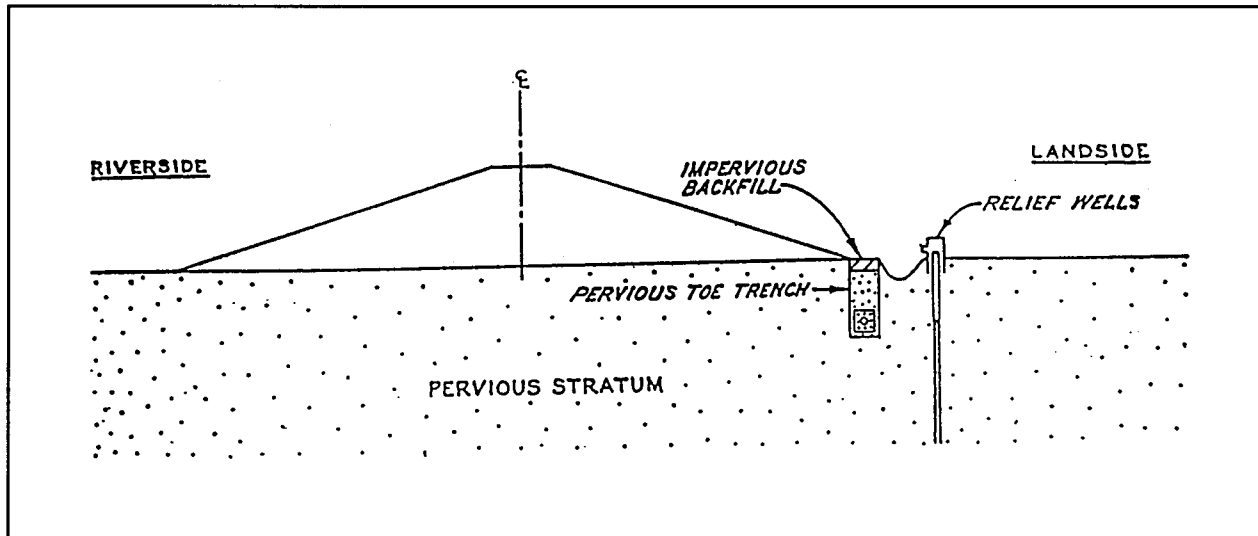


Figure 5-3. Typical pervious toe trench with collector pipe (Figure 5-6 shows trench details)

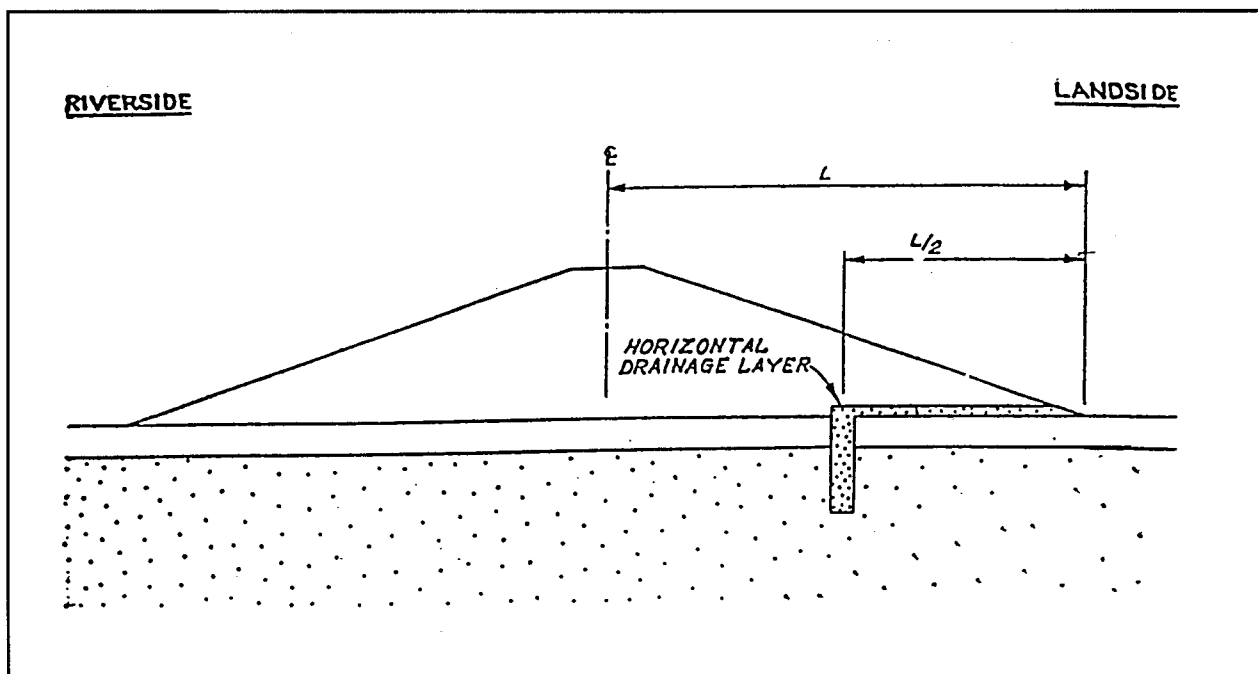


Figure 5-4. Pervious toe trench located beneath landward slope

## 5-6. Pressure Relief Wells

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



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

ward from the well line should not exceed 0.50 (equivalent to  $FS = 1.7$  for an average soil saturated unit weight of  $1840 \text{ kg/m}^3$  (115 pcf)). Many combinations of well spacing and penetration will produce the desired pressure relief; hence, the final selected spacing and penetration must be based on cost comparisons of alternative combinations. After the general well spacing for a given reach of levee has been determined, the actual location of each well should be established to ensure that the wells will be located at critical seepage points and will fit natural topographic features.

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

*b. Design of well systems.* The design of a pressure relief well system involves determination of well spacing, size, and penetration to reduce uplift between wells to allowable values. Factors to be considered are (a) depth, stratification, and permeability of foundation soils, (b) distance to the effective source of seepage, (c) characteristics of the landside top stratum, if any, and (d) degree of pressure relief desired. Guidance on the method used to determine well spacing, size, and penetration is contained in EM 1110-2-1914 and U.S. Army Engineer Waterways Experiment Station TM No. 3-424. Where no control measures are present, relief wells for agricultural and urban levees should be designed so that  $i_{\max}$  midway between the wells or land-

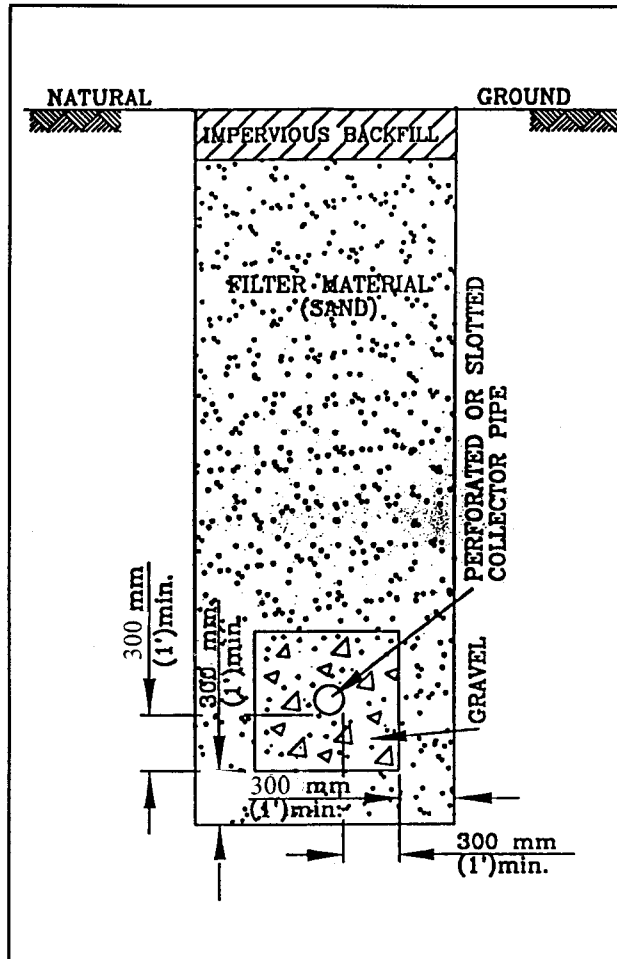


Figure 5-6. Pervious toe trench with collector pipe

and 1.2 m (4 ft) below the bottom of the well screen. Above the filter to the bottom of the concrete or impervious backfill, sand backfill may be used.

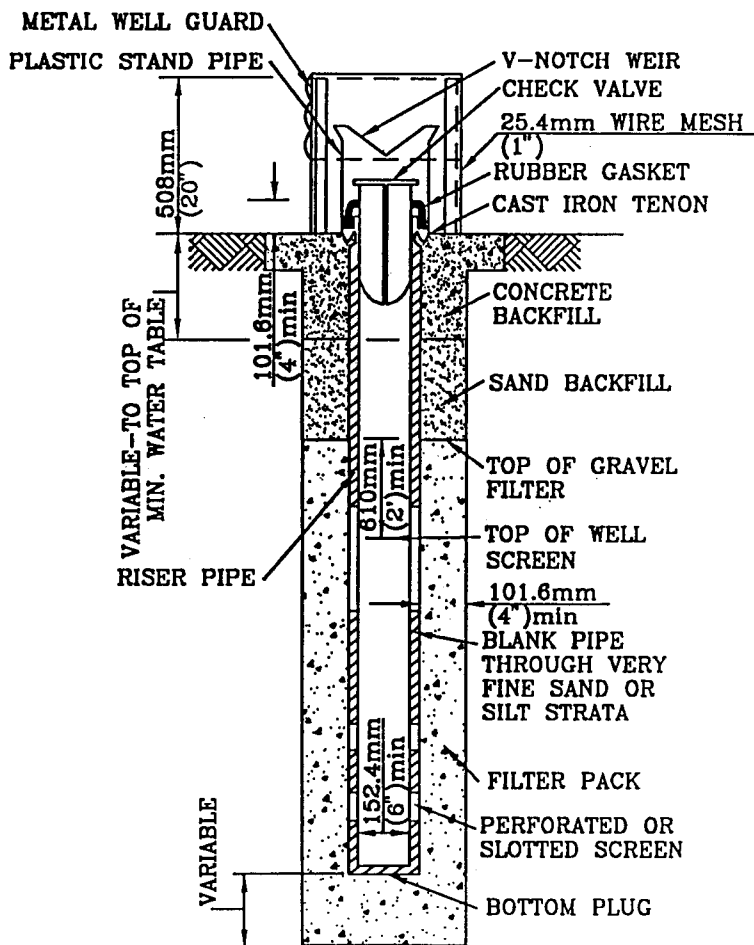
(3) Well appurtenances. In selecting well appurtenances, consideration must be given to ease of maintenance, protection against contamination from back flooding, damage by debris, and vandalism. To prevent wells from becoming backflooded with muddy surface water, which greatly impairs their efficiency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

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

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

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

(2) Filter. The filter that surrounds the screen must be designed in accordance with criteria given in Appendix D using the slot size of the screen and the gradation of surrounding pervious deposit as a basis of design. No matter what size screen is used, a minimum of 152.4 mm (6 in.) of filter material should surround the screen and the filter should extend a minimum of 610.8 mm (2 ft) above the top



**Figure 5-7. Typical relief well**

## Section II

### Seepage Through Embankments

## 5-7. General

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

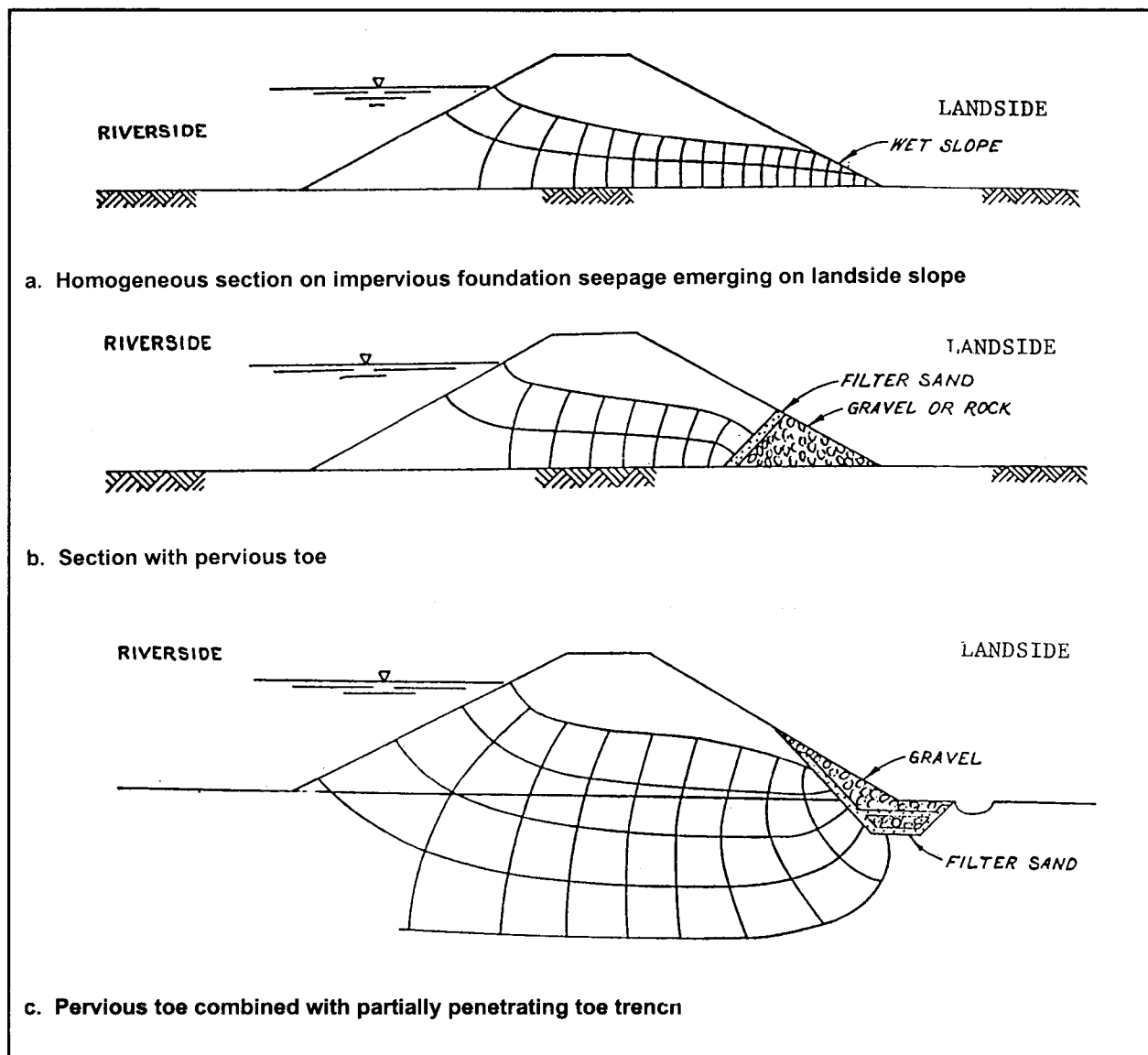


Figure 5-8. Embankment with through seepage

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

## **5-8. Pervious Toe Drain**

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

## **5-9. Horizontal Drainage Layers**

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

## **5-10. Inclined Drainage Layers**

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

## **5-11. Design of Drainage Layers**

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

## **5-12. Compaction of Drainage Layers**

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

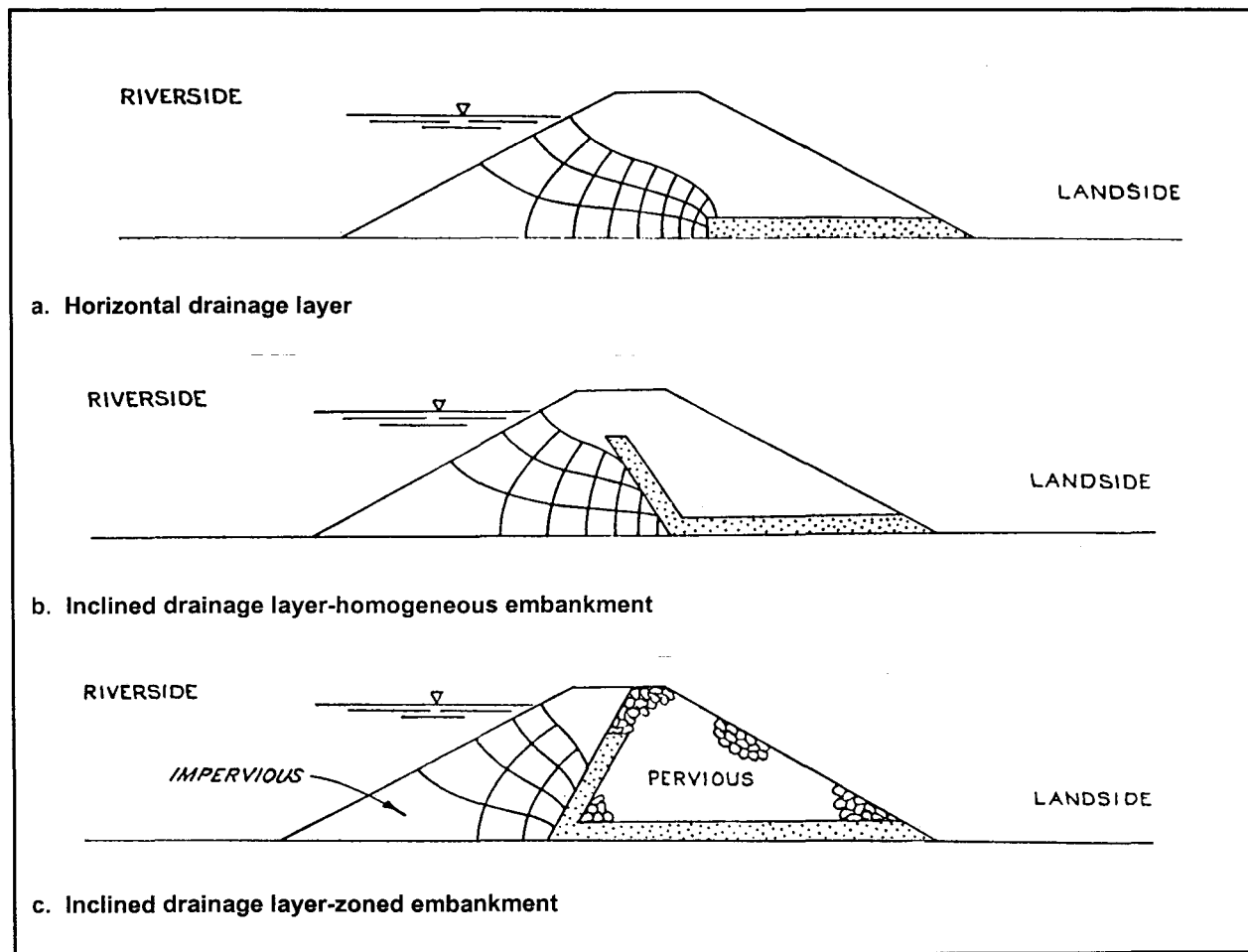


Figure 5-9. Use of horizontal and inclined drainage layers to control seepage through an embankment

## Chapter 6

### Slope Design and Settlement

#### *Section I* *Embankment Stability*

#### **6-1. Embankment Geometry**

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

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

(2) Ease of construction. A 1V on 2H slope is generally accepted as the steepest slope that can easily be constructed and ensure stability of any riprap layers.

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

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

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

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

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

#### **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 7.62 m (25 ft)) in reaches where there are no serious underseepage problems, weak foundation soils, or undesirable borrow materials (very wet or very organic). In many cases the standard levee section has more than the minimum allowable factor of safety relative to slope stability, its slopes being established primarily on the basis of construction and maintenance considerations. Where high levees or levees on foundations presenting special underseepage or stability problems are to be built, the uppermost riverside and landside slopes of the levee are often the same as those of the standard section, with the lower slopes flattened or stability berms provided as needed.

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

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

*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. Various methods of analysis are described in EM 1110-2-1902, and can be chosen for use where determined appropriate by the designer. Computer programs are available for these analyses, with the various loading cases described in EM 1110-2-1902, 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, steady seepage from full flood stage, fully developed phreatic surface; Case IV, earthquake. Each case is discussed briefly in the following paragraphs and the applicable type of design shear strength is given. For more detailed information on applicable shear strengths, methods of analysis, and assumptions made for each case refer to EM 1110-2-1902.

*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 condition 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. For the selection of the shear strengths see Table 6-1a.

*c. Case III - Steady seepage from full flood stage (fully developed phreatic surface).* This condition occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition may be critical for landside slope stability. Design shear strengths should be based on Table 6-1a.

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

**Table 6-1a**  
**Summary of Design Conditions**

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

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

<sup>b</sup> For saturated soils use  $\phi = 0$ ; total stress envelope with  $\phi > 0$  is only applicable to partially saturated soils.

## 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 6-1b.

## 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).

**Table 6-1b**  
**Minimum Factors of Safety - Levee Slope Stability**

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

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

<sup>b</sup> See ER 1110-2-1806 for guidance. An EM for seismic stability analysis is under preparation.

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

<sup>d</sup> Includes slopes which are part of cofferdams, retention dikes, stockpiles, navigation channels, breakwater, river banks, and excavation slopes.

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

<sup>f</sup> Lower factors of safety may be appropriate when the consequences of failure in terms of safety, environmental damage and economic losses are small.

*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.

## 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 1110-1-1904. 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.