DREDGED MATERIAL RESEARCH PROGRAM

TECHNICAL REPORT D-77-9

DESIGN AND CONSTRUCTION OF RETAINING DIKES FOR CONTAINMENT OF DREDGED MATERIAL

by

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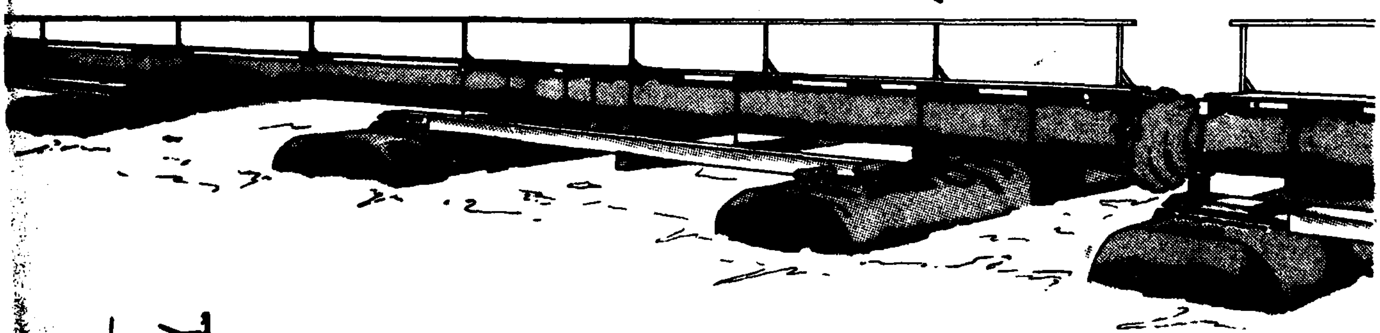

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SUBJECT: Transmittal of Technical Report D-77-9

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1. The report transmitted herein represents the results of one of the research efforts (work units) initiated to date as part of Task 2C (Containment Areas Operations Research) of the Corps of Engineers Dredged Material Research Program (DMRP). Task 2C is included as part of the Disposal Operations Project of the DMRP which among other considerations includes research into the various ways of improving the efficiency and acceptability of facilities for confining dredged material on land.

2. Confining dredged material on land is a relatively recent disposal alternative to which practically no specific design or construction improvement investigations (much less applied research) have been addressed. Being a form of a waste product disposal, dredged material placement on land has seldom been evaluated on other than purely economic grounds with emphasis nearly always on lowest possible cost. There has been a dramatic increase within the last several years in the amount of land disposal necessitated by confining dredged material classified as polluted. Attention necessarily is directed more and more to the environmental consequences of this disposal alternative and methods for minimizing adverse environmental impacts.

3. DMRP work units are in progress to investigate and improve facility design and construction and to investigate concepts for increasing facility capacities for both economic and environmental protection purposes. However, the total picture would be incomplete without considering methods for improving the performance of containment areas. To this end the investigation reported herein was accomplished by the U. S. Army Engineer District, Savannah, Soils Section. This group was selected because of the excellent theoretical background of the personnel as well as their practical experience with the design and construction of retaining dikes. Input from other Districts and Divisions was also important in reaching the goal of providing a set of usable guidelines. It is felt that the guidelines presented in

this report may be applied to provide a sound engineering basis for the design and construction of retaining dikes.

4. Guidelines and recommendations are presented in this report for the proper investigation, design, and construction of retaining dikes to aid in assuring that these dikes will be constructed with a minimum of problems and will serve their project requirements. Raising of existing dikes is covered as well as construction of new dikes. Recommendations are based on a survey of past Corps of Engineers design and construction practices for retaining dikes and current state-of-the-art design procedures for construction of earth embankments.

JOHN L. CANNON
Colonel, Corps of Engineers
Commander and Director

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Past experience has indicated a major problem exists in the area of dike construction for dredged material containment areas. Frequent dike failures have been reported along with problems associated with high maintenance requirements. These problems can usually be traced to inadequate design and construction practices compounded by very poor site conditions. Site conditions will continue to be poor, but this in itself indicates the need for more comprehensive planning and design efforts along with improved construction practices.

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20. ABSTRACT (Continued).

This report presents recommendations for proper investigation, design, and construction of retaining dikes to aid in ensuring that these dikes will be constructed with a minimum of problems and will serve their project requirements. Raising of existing dikes is covered as well as construction of new dikes. Recommendations are based on a survey of past Corps of Engineers design and construction practices for retaining dikes and current state-of-the-art design procedures for construction of earth embankments.

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PREFACE

The study from which the guidelines reported herein were developed was performed at the U. S. Army Engineer District, Savannah (SAS), under Work Unit 2C04, "Design and Construction Guidelines for Positive Dredged Material Retention." The research was sponsored by the Office, Chief of Engineers (DAEN-CWO-M), under the Civil Works Dredged Material Research Program (DMRP) being planned and implemented by the Environmental Effects Laboratory (EEL), U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss. This study was a part of the DMRP Disposal Operations Project, Mr. C. C. Calhoun, Jr., Manager, Task 2C "Containment Area Operations Research," Mr. N. C. Baker, Manager. The study was under the general supervision of Dr. John Harrison, Chief, EEL.

The work was conducted during the period November 1973-December 1976 by Mr. D. P. Hammer, presently assigned to the Research Group, Soil Mechanics Division (SMD), Soils and Pavements Laboratory (S&PL), WES (formerly Chief, Soils Section, SAS), and Mr. E. D. Blackburn, Soils Section, SAS, under the general supervision of Mr. J. G. Higgs, Chief, Engineering Division, SAS, and Mr. W. K. Thompson, Chief, Foundation and Materials Branch, SAS. The study also benefitted substantially from valuable contributions made by Mr. F. J. Weaver, Chief, Geology, Soils, and Materials (G,S,&M) Branch, Lower Mississippi Valley Division (LMVD), members of the G,S,&M Branch, Messrs. C. R. Furlow, L. H. Cave, J. A. Young, and T. R. Freeman; and Mr. J. B. Phillips, Soils Section, SAS. Also giving valuable advice were the various district offices throughout the Corps of Engineers (CE). This report was prepared by Messrs. Hammer and Blackburn.

District Engineer of the SAS during conduct of the study and preparation of the report was COL E. C. Keiser, CE. Directors of WES were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director of WES was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	2.54	centimetres
feet	0.3048	metres
cubic yards	0.7645549	cubic metres
pounds (mass)	0.4535924	kilograms
kips (mass)	0.0004535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	6894.757	pascals
pounds (force) per square foot	47.88026	pascals
kips (force) per square foot	47.88026	kilopascals
tons (force) per square foot	95.76052	kilopascals
degrees	0.01745329	radians

DESIGN AND CONSTRUCTION OF RETAINING DIKES FOR CONTAINMENT OF DREDGED MATERIAL

PART I: INTRODUCTION

Background

1. The Corps of Engineers (CE), in developing and maintaining the Nation's navigable waterways and harbors, is responsible for the dredging of large volumes of material each year. In the past, the CE has been able to deposit the material removed by dredging activities at selected open-water and land-based disposal sites near enough to the dredging site to minimize disposal costs, but in locations that had a minimum direct effect on other important activities in the area. Until recently over two-thirds of all disposal has been in open water. However, due to the effects of rapid industrialization and population concentration near many of our navigable waterways and recent environmental concerns, this practice has been sharply curtailed. As a result, there has been a significant increase in the volume of material that must be placed in land-based confined disposal sites. Much land-based disposal was placed into (a) areas formed by haphazardly constructed retaining dikes that were frequently breached or (b) natural low-lying areas. Because of rapid industrailization and population concentration and due to public concern over damage to the environment, land-based disposal methods such as these can no longer be employed. Methods must now be employed that will allow only minimal damage to the environment.

2. Recognizing the need for more information concerning the handling and disposal of dredged material, the CE was authorized in 1970 to initiate a comprehensive nationwide study concerned with dredged material. The purpose of this study was to provide more definitive information on the environmental impact of dredging and dredged material disposal operations and to develop new or improved disposal practices. The results of this study were set forth in a report[1] that presented an

assessment of the dredged material problem and outlined a research program designed to provide needed information concerning current and potential disposal practices. Among other things, the following conclusions were reached in that report:

a. There will be more land disposal of dredged material in future years.

b. Most of the materials to be disposed of on land will come from highly developed areas where land disposal sites will be difficult to obtain.

c. At least four basic problem areas associated with land disposal can be identified: the environmental impact of land disposal, problems related to obligations of local sponsors of a project, problems related to site availability, and technical problems related to design, construction, operation, and utilization of land disposal sites.

d. Substantial improvements are necessary in containment area dike design and construction to prevent expensive and environmentally damaging failures.

Purpose

3. Based on the above conclusions, a study was initiated in 1973 to develop guidelines for the design and construction of containment area retaining structures based on sound engineering principles. The purpose of this report is to present the results of that study.

Scope

4. This study was limited to land-based retaining structures, the majority of which lie above water, i.e., retaining structures constructed primarily in water were not included. Associated structures such as sluices were covered only to the extent of their effect on the primary retaining structure.

Applicability

5. This report is applicable to all CE Divisions and Districts concerned with the design and construction of land-based dredged material retaining dikes.

PART II: GENERAL CONSIDERATIONS

6. Retaining dikes used to form confined disposal facilities consist primarily of earth embankments constructed on lowland areas or near-shore islands with the principal objective of retaining solid particles within the disposal area while at the same time allowing the release of clean effluent back to natural waters. Retaining dikes are similar to flood protection levees in size and shape but differ in the following important respects: (a) a retaining dike will retain an essentially permanent pool, whereas most levees have water against them only for relatively short periods of time and (b) the location of a retaining dike will usually be established by factors other than foundation conditions and available borrow material (i.e., proximity to dredge, only land available, etc.) from which there will be little deviation.

7. The heights and geometric configurations of retaining dikes are generally dictated by containment capacity requirements, availability of construction materials, and prevailing foundation conditions. This report will be primarily concerned with the latter two items.

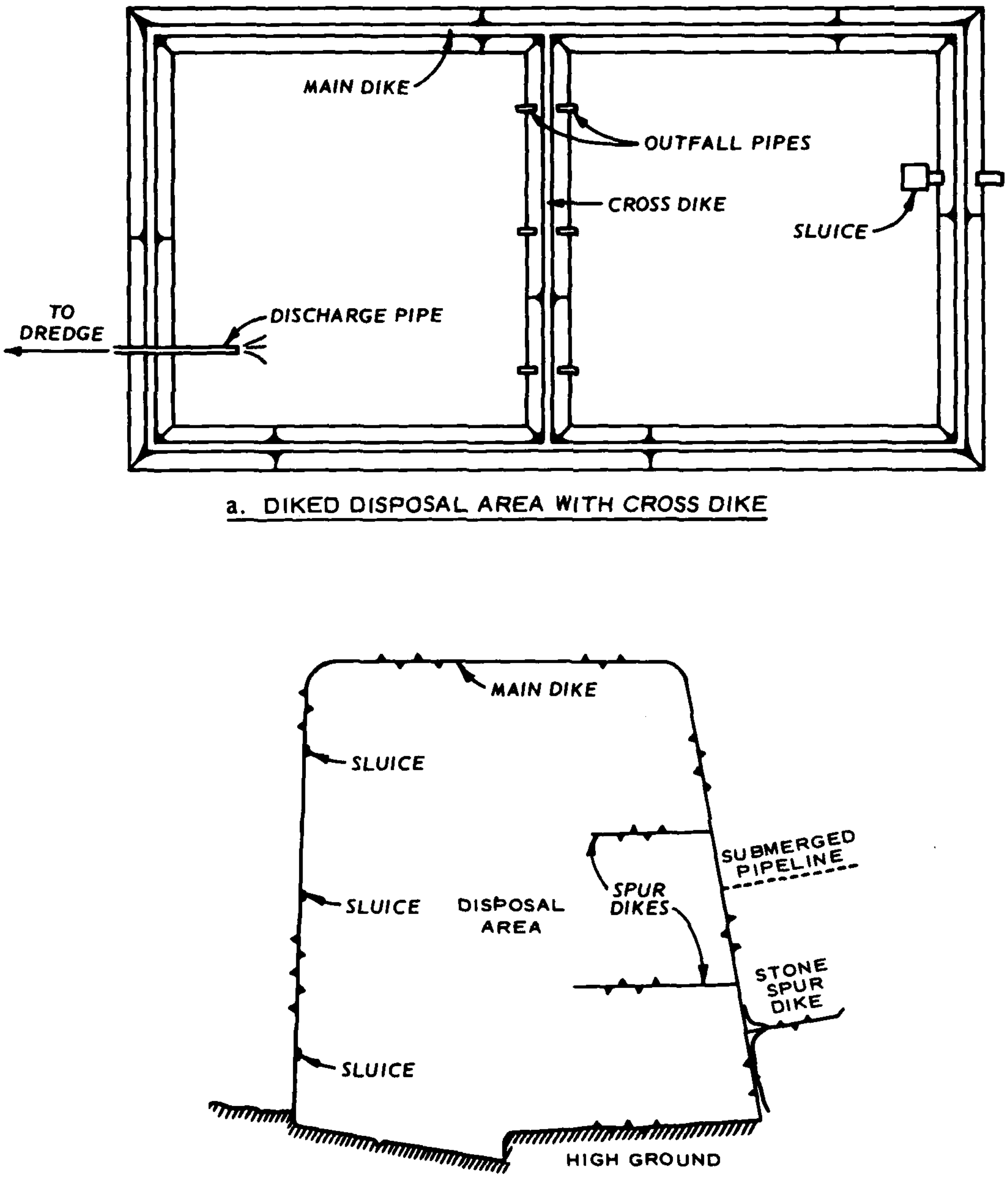
Types of Retaining Dikes

Main dike

8. The most predominant retaining structure in a containment facility extends around the outer perimeter of the containment area and is referred to as the main dike. Except as otherwise noted, all discussion in this report applies to the main dike. The main dike, along with two other type dikes that serve primarily as operational support structures for the main dike, is shown in Figure 1.

Cross dike

9. A cross or lateral dike (Figure 1) is a dike placed across the interior of the containment area connecting two sides of the main dike. The purpose of a cross dike is to separate the facility into two areas so that the slurry in one area is subjected to initial settling prior



b. DIKED DISPOSAL AREA WITH SPUR DIKES

Figure 1. Examples of cross and spur dikes

to passing over or through the cross dike to the other area. In order to accomplish this, the cross dike is placed between the dredge discharge point and the sluice discharge. Cross dikes can also be used with Y-discharge lines to divide an area into two or more areas, each receiving a portion of the incoming dredged material.

Spur dike

10. Spur or finger dikes protrude into, but not completely across, the disposal area from the main dike (Figure 1). They are used mainly to prevent channelization by breaking up a preferred flow path and dispersing the slurry into the disposal area. Spur dikes are also used to allow simultaneous discharge from two or more dredges by preventing coalescing of the two dredged material inputs and thereby discouraging an otherwise large quantity of slurry from reaching flow velocities necessary for channelization.

Dike Failures

11. Retaining dike failures in the past have been largely the result of a combination of factors: foundation conditions, construction materials, and, in some cases, construction methods and disposal practices.[2] Consequently, all of these factors must be taken into account during dike design.

12. For many containment facilities at unpopulated locations, there has been a tendency for less effort and expense to be applied to dike design and construction. Consequently, dike failures have been more frequent at these locations and resulted in the flow of dredged material onto tidal flats or marshes or into nearby rivers and streams. Not all failures have been confined to unpopulated or otherwise open areas, however. Damage to warehouses, a railroad embankment, a sewage treatment plant, and pastureland and even flooding of a subdivision have been reported.[2] In addition to property damage, there is usually the expense of redredging and repair of the dike.

Available Materials

13. Available material at a site to serve as a foundation and/or of which the embankment will be composed is probably the single most important factor that affects dike design and construction. This is because dike design must generally be adapted to the most economically available materials compatible with prevailing foundation conditions. Available disposal sites are normally lands not economically suited for private development, often being composed of soft clays and silts of varying organic content. In fact, many future confined disposal sites will undoubtedly have been used in the past for unconfined disposal, thereby forcing dikes to be constructed on previously deposited dredged material often consisting of soils having very poor engineering qualities.

14. Since dike construction requiring the use of material from inside the disposal area and/or immediately adjacent borrow areas is often an economic necessity, initial dike heights may be limited or the use of rather large embankment sections may result, expensive foundation treatment may be required, or expensive construction methods may be dictated. In some cases where more desirable borrow is available, its use can result in a lower construction cost if one or more of the above items can be eliminated (i.e., a smaller section, less expensive required foundation treatment, etc.). However, the use of select borrow does not alleviate instability problems to any great degree if the foundation is of poor quality and extends to depths that make simple foundation treatment such as excavation and replacement impracticable. In fact, poor foundation conditions are much more difficult to deal with than poor embankment materials. Both conditions are dealt with in detail in this report.

Construction Method

15. The method used to construct the dike must also be given thorough consideration because each type of construction has

characteristics inherent within itself that can strongly affect the desired dike section. The selection of a construction method, even though based largely on economics, must also be compatible with available materials and the geometry of the final dike section, as well as environmental considerations. The different types of construction, advantages and disadvantages of each, and their effects on the dike section are all discussed in detail in Part VIII.

Factors Affecting the Extent of Field Investigations and Design

16. The extent to which field investigations and design are carried out is dependent on the desired degree of safety against failure. This decision will usually be made by the local design agency and, of course, involves many factors peculiar to the particular project. However, Table 1 lists some general factors based on past practice that can be used as general guidelines in the planning stages of a project.

Table 1

Factors Affecting the Extent of Field Investigations and Design Studies

Factor	Field Investigations and Design Studies Should be More Extensive Where
Construction experience	There is little or no construction experience in the area, particularly with respect to dikes
Consequence of failure	Consequences of failure involving life, property, or damage to the environment are great
Dike height	Dike heights are substantial
Foundation conditions	Foundation deposits are weak and compressible Foundation deposits are highly variable along the alignment Underseepage and/or settlement problems are severe
Borrow materials	Available borrow is of poor quality, water contents are high, or borrow materials are variable along the alignment
Structures in dikes	Sluices or other structures are incorporated into the dike embankment and/or foundation
Utility crossings	Diked area is traversed by utility lines

PART III: FIELD INVESTIGATIONS

17. Before a dike can be adequately designed, a reasonably representative concept of the arrangement and physical properties of the foundation and embankment materials must be attained. In the past, many dike failures have been the direct result of subsurface conditions that were not discovered during design because of inadequate soils investigations. These failures were commonly characterized by embankment slides, excessive settlement, detrimental seepage, and other phenomena. Even though it is recognized that no matter how complete an exploration may be, there is always a certain degree of uncertainty concerning the exact nature of subsurface conditions at a given site, an adequately designed exploration program can reduce this uncertainty significantly and place it within limits commensurate with sound engineering practices.

18. For simplicity, subsurface investigations for dredged material retaining dikes (or most other embankments, for that matter) can be broken down into two stages. The first or preliminary stage includes a review of all available information concerning the geological and subsurface conditions at or near the site and general geological reconnaissance with only limited subsurface exploration and simple soil tests. These tests are intended to classify the soil and to determine the location of the groundwater table. With this information, the number, location, and type of additional borings needed can be most economically determined. The final or design stage includes these additional borings as well as more extensive geological investigations, field tests, and observations. Table 2 summarizes, in general, the features of geologic and subsurface investigations.

19. It should be emphasized that these stages of exploration do not necessarily have to be carried out as distinct entities but, conditions permitting, some portions may be conducted with a degree of overlap. Also, depending on the conditions at hand, some portions may be partially or completely eliminated, but this is not recommended under most circumstances since many dikes are constructed in areas typified by poor embankment and foundation materials. Ideally, an exploration

Table 2

Stages of Field Investigation

Stage	Features
Preliminary geological investigation	a. Office study. Collection and study of: Topographic, soil, and geological maps Aerial photographs Boring logs and well data Information on existing engineering projects b. Field survey. Observations and geology of area, documented by written notes and photographs, including such features as: Riverbank and coastal slopes, rock outcrops, earth and rock cuts or fills Surface materials Poorly drained areas Evidences of instability of foundations and slopes Emerging seepage and/or soft spots Natural and man-made physiographic features
Subsurface exploration and field testing and more detailed geologic study	a. Preliminary phase. Widely but not uniformly spaced disturbed sample borings (may include split spoon penetration tests) Test pits excavated by backhoes, farm tractors, or dozers Geophysical surveys to interpolate between widely spaced borings Borehole geophysical tests b. Final phase. Additional disturbed sample borings including split spoon penetration tests Undisturbed sample borings Field vane shear tests for soft materials Water table observations

program should be carried out in the sequence given, with one stage immediately following the other. This will often reduce mobilization costs for exploration equipment, but requires that an engineer be on the job full time to digest all data as they are obtained. It should also be emphasized that no adequate exploration program can be fully established beforehand. Rather, the program should be flexible and developed on a step-by-step basis as information accumulates. By this procedure the maximum amount of information can be obtained for a given amount of funds.

20. The magnitude and type of exploration programs cannot be definitely established beforehand since they will vary according to the individual characteristics of each specific project. The following lists some of these characteristics: (a) size of the project; (b) uniformity and nature of foundation materials; (c) consequences of failure; (d) local experience with similar construction; and (e) familiarity with local subsurface conditions. Hence, the information given in the following paragraphs is general in nature and may be modified to fit the individual project, but should not be modified to an extent that the effectiveness of the exploration program itself is compromised. In this respect, it should be noted that experience is often cited as reason for reducing the magnitude of an exploration program and, in some cases, may be justified. However, misapplied experience has often caused many problems on dike projects that would not have arisen had an adequate exploration program been employed. One should never rely on experience alone but should use it as a guide and supplement to an exploration program, especially in areas of erratic or soft foundations.

Geological Reconnaissance

21. A geological reconnaissance usually consists of an office study of all available geological information within the area of interest and an on-site survey. The primary purpose of the reconnaissance is to establish the nature of the deposits underlying the site. If the types of soils likely to be encountered can be determined, the best

methods of underground exploration can be selected before the actual field exploration is begun. Under favorable conditions, geophysical methods have been used successfully to determine the boundaries between different soil strata and, in some cases, the physical properties of these soils. However, since these methods are indirect, the results may be misleading and should be relied upon only when the findings are substantiated by borings or other direct means of investigation.

Office study

22. The reconnaissance should begin with an office study and a search for all information regarding foundation conditions in the area. Such information usually includes topographic, soil, and geological maps, as well as aerial photographs. Pertinent information concerning past construction in the area should also be obtained. This includes design, construction, and performance data on highways, dikes, levees, railroads, and hydraulic structures. Available boring logs should be secured. Federal, State, county, and local agencies should be contacted for information.

Field survey

23. The field survey should begin only after becoming thoroughly familiar with the area through the office study. Walking the proposed dike alignment and the adjacent area is always an excellent means of obtaining valuable information. Physical features to be observed are noted in Table 2. These items and any others of significance should be documented by detailed notes and supplemented with photographs. Local people or organizations in the area with knowledge of foundation conditions should be interviewed.

Subsurface Exploration

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

25. The subsurface exploration for the design stage is generally

broken down into two phases, which may be accomplished separately, in sequence, or concurrently. The main purpose of Phase I is to more accurately define soil types and to develop general ideas of soil strength, compressibility, and permeability. Phase 2 provides additional information on soil types present and usually includes the taking of undisturbed samples for testing.

Phase 1

26. Phase 1 exploration consists almost entirely of general (or disturbed) sample borings, but may also include geophysical surveys as will be discussed later. Table 3 briefly remarks on some types of techniques employed in Phase 1 exploration. For details regarding methods, equipment, and procedures for disturbed soil sampling, References 3 and 4 should be consulted.

Phase 2

27. Phase 2 of subsurface exploration combines undisturbed samples with undisturbed borings and may also include geophysical methods. Undisturbed samples are obtained most often by rotary and push-type drilling methods, employing the thin-walled Shelby tube sampler in most soils.

Boring and sampling

28. Type. There are several procedures in common use today for drilling exploratory holes and extracting representative samples for identification and/or testing. The choice of which method to use depends on the type of material and information required. Detailed descriptions of different drilling and sampling techniques as well as guidance on method selection are contained in References 3 and 4.

29. Location and spacing. The location and spacing of borings for Phase 1 exploration should be based on an examination of air photos and geological conditions determined in the preliminary stage or known from prior experience in the area, and on the nature of the project. Initial spacing of borings usually varies from 200 to 1000 ft* along the dike alignment, being closer spaced in expected problem areas or areas

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units can be found on page 9.

Table 3

Phase 1 Boring and Sampling Techniques

Technique	Remarks
Disturbed sample borings	
Standard penetration test	Primarily for soil identification, but also permits estimate of shear strength and density parameters Preferred for general exploration of dike foundations; indicates need and locations for undisturbed samples
Auger borings	Generally made in borrow areas; bag samples can be obtained for testing
Test pits	Generally made only in borrow areas; not usually required. Use backhoes, dozers, farm tractors
Trenches	Useful in borrow areas and dike foundations

of erratic foundation conditions and wider spaced in nonproblem and more uniform areas. The spacing of borings should not be arbitrarily uniform, but should be based on available geologic information. At least one boring should be located at every major structure during Phase 1.

30. During Phase 2 exploration, the locations of additional general sample borings are selected on the basis of Phase 1 results. Undisturbed sample borings are located where soil shear strength and compressibility characteristics are most needed. Usually the best procedure is to group the foundation profiles developed during Phase 1 into reaches of similar conditions and then locate undisturbed sample borings so as to define soil properties in critical reaches.

31. One feature that has consistently caused problems at dike projects in the past is old sloughs filled with either very soft cohesive material or pervious granular material. These features are often undetected by boring programs due to their narrow extent; hence, the possibility of their existence, especially in swampy and coastal areas, should always be kept in mind during the formulation of both Phase 1 and Phase 2 exploration programs.

32. Depth. Like location and spacing of borings, no definite guidelines can be given for the depth of exploratory borings. Only general guides can be given along with factors that affect boring depths. The depth to which borings should be taken depends largely on the size of the dike and foundation conditions as reported by the geological reconnaissance or as the boring program progresses. Where soft soils are encountered, boring depths should extend to the maximum depth within which the stress caused by the dike could conceivably produce excessive settlement. This depth may be established on the basis of approximate stress and settlement calculations, the procedures for which can be found in most any soil mechanics text.

33. Borings should also be deep enough to provide sufficient data for stability analyses of the dike with respect to both foundation shear failure and foundation seepage problems. Where pervious or soft materials are encountered, borings should extend through the permeable material to impermeable material or through the soft material to firm

material unless the impermeable or firm material exists at great depths, in which case only a few borings to these depths are required with the remainder within the zone of influence of the dike being more shallow.

34. In borrow areas, the depth of exploration should extend several feet below the practicable or allowable borrow depth or groundwater table. If borrow is to be obtained from below the groundwater table by dredging, dragline, or other means, borings should be taken to a depth at least 5 ft below the bottom of the proposed excavation.

Geophysical exploration

35. Geophysical methods of exploration are often quite useful as part of the foundation investigation due to the long, relatively narrow areas to be explored and the increasing cost of borings. The relatively inexpensive geophysical methods are useful for interpolating between borings that, for reasons of economy, are spaced at fairly wide intervals. EM 1110-2-1802[5] (currently under revision) provides guidance in the use of geophysical methods of exploration.

36. There are several methods of geophysical exploration available to the engineer; however, the most commonly used methods are seismic, electrical resistivity, and borehole surveying. Since magnetic methods have a limited application and the continuous vibration method is in the development stages, they will not be discussed here.

37. Portable seismic and resistivity equipment allows exploration to be carried out often economically and rapidly over large areas. Under some circumstances, the use of both types of equipment may facilitate interpretation. It is, however, advisable to check the results of a geophysical survey by at least a few borings.

38. Seismic method. The seismic method of geological exploration is best adapted for determining the depth to rock although it may also be of use in defining boundaries between a clay or silt top stratum and an underlying sand and gravel substratum where relatively uniform top stratum and substratum materials are present. It is a fairly reliable method, provided the thickness of the weathered top layer is small and the rock surface is not too uneven. The location of the groundwater table in pervious soils can be determined since the velocity of seismic

waves is greater in saturated than in unsaturated soils. In some cases, the depth to a stiff or hard deposit beneath soft overlying material can be determined. On the other hand, the presence of a soft layer below a stiffer one ordinarily cannot be detected.

39. Resistivity method. The resistivity method has been found to be useful in defining the boundaries between soils of low resistivity, such as soft clay and soft organic deposits, and materials of higher resistivity, such as sand, gravel, and bedrock. Low resistivity materials can be detected even if they underlie those of higher resistivities. The surface of a body of water can be found using this method also. Boundaries between strata of similar resistivity, such as organic soil and soft clay or loose sand and coarse-grained sandstone, usually cannot be detected. In all applications, the interpretation requires calibration of the equipment over known materials in the immediate area.

40. Borehole surveying. Recent developments in the use of downhole surveying devices have shown that these tools can be successful in correlating subsurface soil and rock stratification and in providing quantitative engineering properties such as porosity, density, water content, and elastic moduli. Once a boring has been made, the cost of using the tools in the borehole is small relative to the cost of the boring.

41. The ultimate goal in using these devices is to allow cost savings to be made in the exploration program without lessening the quality of information obtained. This can be done by reducing the number of borings required to determine the subsurface stratification and by sampling only in those zones where samples are necessary for laboratory testing, thus reducing the number of undisturbed samples.

Field Testing

42. It is often desirable to estimate foundation strengths during the preliminary stage of the exploration program. These are several available methods of doing this and some are listed in Table 4.

Table 4

Preliminary Appraisal of Foundation Strengths

Method	Remarks
Penetration resistance from standard penetration test	In clays, provides data helpful in a relative sense; i.e., in comparing different deposits. Generally not helpful where number of blows per foot N* is low
	In sand, N-values less than about 15 indicate low relative densities
Natural water content of disturbed or general type samples	Useful when considered with soil classification and previous experience is available
Hand examination of disturbed samples	Useful where experienced personnel are available who are skilled in estimating soil shear strengths
Position of natural water contents relative to liquid limit (LL) and plastic limits (PL)	Useful where previous experience is available
	If natural water content is close to PL, foundation shear strength should be high
	Natural water contents near LL indicate sensitive soils with low shear strengths
Torvane or pocket penetrometer tests on intact portions of general samples	Easily performed and inexpensive, but results may be excessively low; useful for preliminary strength estimates

* The letter N is also used later in this report (Appendix A) to denote normal force.

Vane shear test

43. It is well known that any so-called "undisturbed" sampling technique results in some degree of disturbance to the sample. Also, it is often very difficult, if not impossible, to retain samples of very soft clay upon which many retaining dikes are built. For this reason and for economical reasons, the use of the field vane shear method of testing has become very popular. Briefly, the field vane shear test consists of pushing a set of vanes into the soil and rotating them to failure. The soil shear strength measured by this test is known as the unconsolidated-undrained strength and is applied to what is termed the end-of-construction condition, which, for most retaining dikes, is the critical condition of stability. The apparatus and procedure for performing this test are described in EM 1110-2-1907.[4]

44. Even though the field vane shear test has proved to be a valuable tool for the determination of the undrained shear strength of soft materials, its proper uses and limitations must be realized and allowances made in order to obtain reliable results. Some of these aspects are discussed in the following paragraphs.

45. Although it is felt by some that the vane shear test is easily standardized and the results reproducible,[6] experience has shown that this is the case only if the tests are run by experienced personnel who are totally familiar with proper test procedures. Thus, one cannot expect to utilize a test crew who perform this test only occasionally, without employing strict supervision during testing and providing a cautious review of the results. One example of a problem not readily evident is the fact that the use of warm weather grease in cold weather can appreciably alter the results. Problems such as these point out the need for someone thoroughly familiar with the test procedure to be present at all times.

46. For many years the field vane shear strength has been assumed to be equal to actual field strengths. However, it is now known that a considerable discrepancy can exist between actual and measured vane shear strengths. The following factors attributable to this difference have been described by Bjerrum:[6] rate of loading, anisotropy, and

progressive failure. In order to at least partially allow for these effects, Bjerrum proposed a correction factor μ* based on the plasticity index (PI) of the soil (Figure 2). The use of this chart will, in most cases, allow a more accurate determination of the actual shear strength to be made. In any event, field vane shear test results should never be relied upon alone, but should be liberally supplemented with results from unconfined and triaxial Q-tests (shear tests representing unconsolidated-undrained conditions).

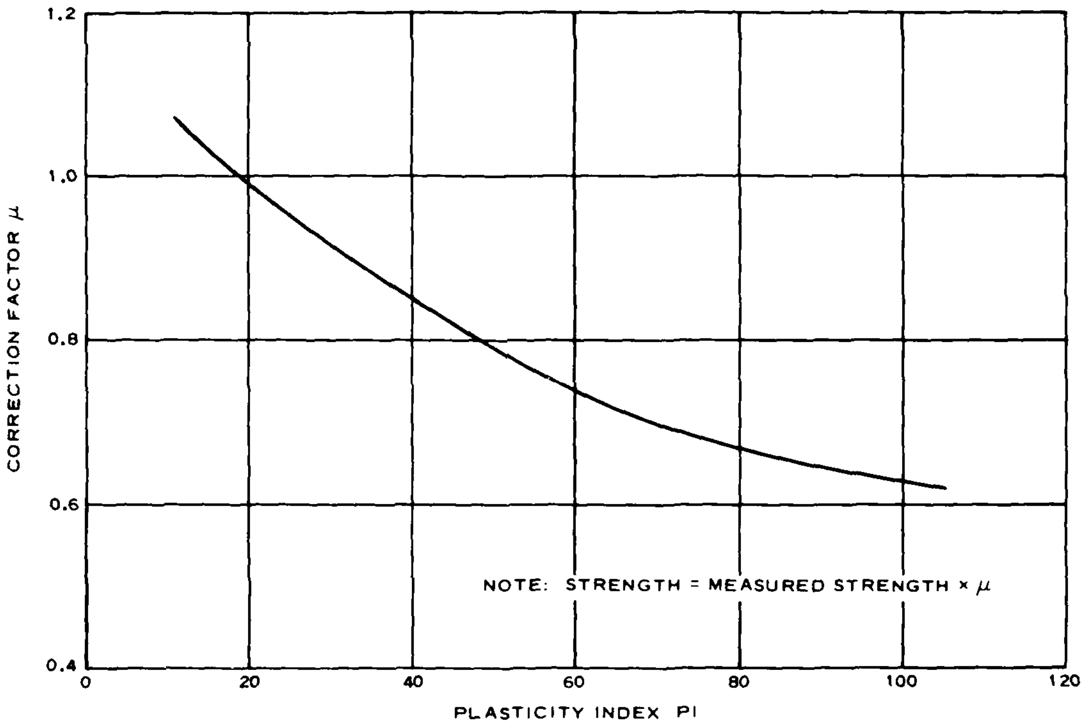

Figure 2. Vane shear correction chart (after Bjerrum)[6]

Standard penetration test

47. One of the most widely used methods for determining the relative consistency of cohesive soil and relative density of granular

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix B).

soils is the standard penetration test. This test provides a quick and economical method to check the resistance offered by the soil to penetration by a sampling spoon. The standard penetration test gives a means of combining subsurface investigation and preliminary soil testing with little additional expense. Correlations between blow counts (N) from this test and various soil properties have been made by several authorities[7-10] and are found in many soil mechanics texts. Also given in these references are the defects and limitations inherent in the use of this test that one must be aware of before attempting use of the results. Correlations normally used by the CE for making preliminary estimates are given in Table 5. Procedures for performing the standard penetration test are given in EM 1110-2-1907.[4]

Permeability

48. The permeability of pervious material can usually be estimated with sufficient accuracy using existing correlations with grain-size determinations (see Part IV). Field pumping tests are the most accurate means of determining permeabilities of stratified deposits; however, they are expensive and will rarely be justified for dike projects.

Table 5

Relationships Between Standard Penetration Test Results and Soil Density or Consistency

Soil Type*	Density or Consistency	Range of Standard Penetration Resistance**
Cohesionless	Very loose	<4
	Loose	4 to 10
	Medium dense	10 to 30
	Dense	30 to 50
	Very dense	>50
Cohesive	Very soft	<2
	Soft	2 to 4
	Medium stiff	4 to 8
	Stiff	8 to 15
	Very stiff	15 to 30
	Hard	>30

* The basic soil types are described in accordance with the Unified Soil Classification System.[11,12]

** Number of blows from a 140-lb weight falling 30 in. required to drive a 2-in.-OD, 1-3/8-in.-ID sampler a distance of 1 ft.

PART IV: LABORATORY INVESTIGATIONS

49. This part describes laboratory tests considered appropriate in establishing the engineering properties of foundation soils and embankment materials for use in the design of retaining dikes. Basically, these laboratory tests are essentially the same as would be performed for a rational design of any earthen embankment. The scope and magnitude of the laboratory test program will, however, depend on the nature and importance of the project, the complexity of foundation and borrow conditions and how well they are known, and the extent to which previous experience and correlations are applicable. The number and types of laboratory tests to be performed should be determined only after a careful study of the boring profiles in order to determine the parameters likely to control the design.

50. Like exploration programs, laboratory testing programs are costly and will increase the initial cost of the project, but the findings therefrom will result in safer, more suitable dikes with fewer failures and, because of this, may very well result in a lower overall project cost when viewed from both a construction and maintenance standpoint.

51. Current soil testing procedures are fully described in EM 1110-2-1906, "Laboratory Soils Testing."[13] EM 1110-2-1902, "Stability of Earth and Rock-Fill Dams,"[14] outlines the applicability of the various laboratory strength tests to appropriate field loading conditions. The subject of various field loading conditions and how they relate to appropriate laboratory tests is also further discussed in Part VII of this report.

Laboratory Testing Programs

52. A laboratory testing program can generally be divided into two parts. The first part consists essentially of index tests, the purpose of which is to classify the soils and thereby develop the boring log with the end result of establishing soil profiles, i.e.,

determining what type of soils exist where. Index tests include visual classification, water content, Atterberg limits, and mechanical analysis (gradation) tests. The second part consists of tests intended to determine the engineering properties of soils with respect to shear strength, consolidation, and sometimes permeability. It is these values that provide the input parameters for design analyses.

53. Soils are generally divided into two broad classifications: fine-grained soils and coarse-grained soils. Fine-grained soils are soils composed of particles of which more than half are smaller than the No. 200 sieve size. These soils are primarily silt and clay. Coarse-grained soils, primarily sand and gravel, are soils composed of particles of which less than half are smaller than the No. 200 sieve size. Coarse-grained soils with less than about 5 percent passing the No. 200 sieve are usually termed free draining. These soils are known as "clean" sand and gravel. Tables 6 and 7 contain the various tests that may be included in a laboratory testing program for fine-grained and coarse-grained soils, respectively. Also included in Tables 6 and 7 are pertinent remarks concerning the purposes and scope of testing.

Index Property Tests

54. Index tests are used to classify soil in accordance with the Unified Soil Classification System (Table 8), to develop accurate foundation soil profiles, and to aid in correlating and extrapolating the results of engineering property tests to areas of similar soil conditions. Both general (disturbed) and undisturbed soil samples should be subjected to index-type tests. Index tests should be initiated, if possible, during the course of field investigations. All samples furnished to the laboratory should be visually classified and natural water content determinations made; however, no water content tests need be run on clean sands or gravels. Mechanical analyses (gradations) of a large number of samples are not usually required for identification purposes. Atterberg limits tests should be performed discriminately and should be reserved for representative fine-grained

Table 6

Laboratory Testing of Fine-Grained Cohesive Soils

Type Test	Purpose	Scope of Testing
Visual classification	To visually classify the soil in accordance with the Unified Soil Classification System	All samples
Water content	To determine the water content of the soil in order to better define soil profiles, variation with depth, and behavioral characteristics	All samples
Atterberg limits	Foundation soils: for classification, comparison with natural water contents, or correlation with shear or consolidation parameters Borrow soils: for classification, comparison with natural water contents, or correlations with optimum water content and maximum dry densities	Representative samples of foundation and borrow soils. Sufficient samples should be tested to develop a good profile with depth
Compaction	To establish maximum dry density and optimum water content	Representative samples of all borrow soils for compacted or semicompacted dikes: Compacted - perform standard 25-blow test Semicompacted - perform 15-blow test
Consolidation	To determine parameters necessary to estimate settlement of dike and/or foundation and time-rate of settlement. Also, to determine whether soils are normally consolidated or over-consolidated and to aid in estimating strength gain with time	Representative samples of compacted borrow where consolidation of dike embankment itself is expected to be significant. Representative samples of foundation soils where such soils are anticipated to be compressible On samples of fine-grained adjacent and/or underlying materials at structure locations
Permeability	To estimate the perviousness of borrow and/or foundation soils in order to calculate seepage losses and time-rate of settlement	Generally not required for fine-grained cohesive soils as such soils can be assumed to be essentially impervious in seepage analyses. Can be computed from consolidation tests
Shear strength	To provide parameters necessary for input into stability analysis Pocket penetrometer, miniature vane, unconfined compression, and Q-tests to determine unconsolidated-undrained strengths R-tests to determine consolidated-undrained strengths S-tests to determine consolidated-drained strengths	Pocket penetrometer and miniature vane (Torvane) for rough estimates Unconfined compression tests on saturated foundation clays without joints, fissures, or slickensides Appropriate Q- and R-triaxial and S-direct shear tests on representative samples of both foundation and compacted borrow soils

Table 7

Laboratory Testing of Coarse-Grained Noncohesive Soils

Test	Purpose	Scope of Testing
Visual classification	To visually classify the soil in accordance with the Unified Soil Classification System	All samples
Gradation	Determine grain-size distribution for classification and correlation with permeability and/or shear strength parameters	Representative samples of foundation and borrow materials
Relative density or compaction	Determine minimum-maximum density values or maximum density and optimum water content values; should use the test which gives greatest values of maximum density	Representative samples of all borrow materials
Consolidation	To provide parameters necessary for settlement analysis	Not generally required as pervious soils consolidate rapidly under load and post-construction magnitude is usually such as to be insignificant
Permeability	To provide parameters necessary for seepage analysis	Not usually performed as correlations with grain size are normally of sufficient accuracy. Where underseepage problems are very serious, best to use results from field pumping test

(Continued)

Table 7 (Concluded)

Test	Purpose	Scope of Testing
Shear strength	To provide parameters necessary for stability analysis	Representative samples of compacted borrow and foundation soils. Consolidated-drained strengths from S-direct shear or triaxial tests are appropriate for free-draining pervious soils Conservative values of ϕ can usually be assumed based on S-test results from similar soils

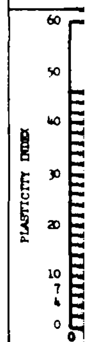
Table 8

UNIFIED SOIL CLASSIFICATION
(Including Identification and Description)

Major Divisions (1)	Major Divisions (2)		Group Symbols (3)	Typical Names (4)	Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights) (5)			Information Required for Describing Soils (6)
Coarse-grained Soils More than half of material is larger than No. 200 sieve size.	Gravels More than half of coarse fraction is larger than No. 4 sieve size. (For visual classification, the 1/4-in. size may be used as equivalent to the No. 4 sieve size)	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.			For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.			
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixture.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).			Give typical name; indicate approximate percentages of sand and gravel, maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).			
	Sands More than half of coarse fraction is smaller than No. 4 sieve size.	Clean Sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain size and substantial amounts of all intermediate particle sizes.			
			SP	Poorly graded sands or gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.			Example: Silty sand, gravelly; about 20% hard, angular gravel particles 1/2-in. maximum size; rounded and subangular sand grains, coarse to fine, about 15% nonplastic fines with low dry strength, well compacted and moist in place, alluvial sand; (SM).
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).			
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).			
The No. 200 sieve size is about the smallest particle visible to the naked eye.					Identification Procedures on Fraction Smaller than No. 40 Sieve Size			
					Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)	
Fine-grained Soils More than half of material is smaller than No. 200 sieve size.	Silts and Clays Liquid limit is less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium	
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight	Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name and other pertinent descriptive information; and symbol in parentheses.
	Silts and Clays Liquid limit is greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium	
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High	
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium	Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.			

Use grain-size curve in identifying the fractions as given under field identification.

Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows:



(1) Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purpo[ses] screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air-drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (consis[tency]

After particles
about one-hal[f]
dry, water
thin layer
is rolled out
one-eighth in
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is reached.
After the thread
action conti[nues]
The tougher the
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Weakness of
lump below
materials
Highly organic

Adopted by Corps of Engineers and Bureau of Reclamation, January 1952.

Table 8

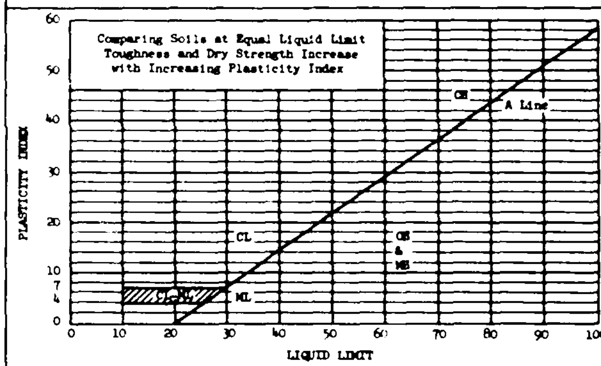
UNIFIED SOIL CLASSIFICATION
(Including Identification and Description)

...ical Names	Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)	Information Required for Describing Soils	Laboratory Classification Criteria	
4	5	6	7	
...ls, gravel-sand mixtures, ...nes.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
...vels or gravel-sand mixtures, ...nes.	Predominantly one size or a range of sizes with some intermediate sizes missing.		Not meeting all gradation requirements for GW	
...vel-sand-silt mixture.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).	Give typical name; indicate approximate percentages of sand and gravel, maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
...ravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		Atterberg limits above "A" line with PI greater than 7	
... gravelly sands, little or	Wide range in grain size and substantial amounts of all intermediate particle sizes.		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
...s or gravelly sands, little	Predominantly one size or a range of sizes with some intermediate sizes missing.		Not meeting all gradation requirements for SW	
...silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).	Example: Silty sand, gravelly; about 20% hard, angular gravel particles 1/2-in. maximum size; rounded and subangular sand grains, coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
...clay mixtures.	Plastic fines (for identification procedures see CL below).		Atterberg limits above "A" line with PI greater than 7	

Use grain-size curve in identifying the fractions as given under field identification.

Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows:
Less than 5% = GW, GP, SW, SP,
More than 12% = GM, GC, SM, SC.
5% to 12% = Borderline cases requiring use of dual symbols.

	Identification Procedures on Fraction Smaller than No. 40 Sieve Size			
	Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)	
...d very fine sands, rock ...clayey fine sands or ...h slight plasticity.	None to slight	Quick to slow	None	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
... low to medium plasticity, ...sandy clays, silty clays,	Medium to high	None to very slow	Medium	
...organic silty clays of low	Slight to medium	Slow	Slight	Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains, color in wet condition; odor, if any, local or geologic name and other pertinent descriptive information; and symbol in parentheses.
...caceous or diatomaceous ...ty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium	
... high plasticity, fat clays.	High to very high	None	High	
...dium to high plasticity,	Medium to high	None to very slow	Slight to medium	Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).
...ly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.			



PLASTICITY CHART

For laboratory classification of fine-grained soils

...es of two groups are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder. (2) All sieve sizes on this chart are U. S. standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

...re a pat of moist
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Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air-drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (consistency near plastic limit)

After particles larger than the No. 40 sieve size are removed, a specimen of soil about one-half inch cube in size, is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

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samples selected after evaluation of the boring profile. For selected borings, Atterberg limits should be determined at frequent intervals on the same samples for which natural water contents are determined.

55. Normally, Atterberg limits determinations and mechanical analyses are performed on a sufficient number of representative samples from preliminary borings to establish the general variation of these properties within the foundation, borrow, or existing fill soils. A typical boring log (in the recommended method for presenting the results of index tests) is shown in Figure 3.

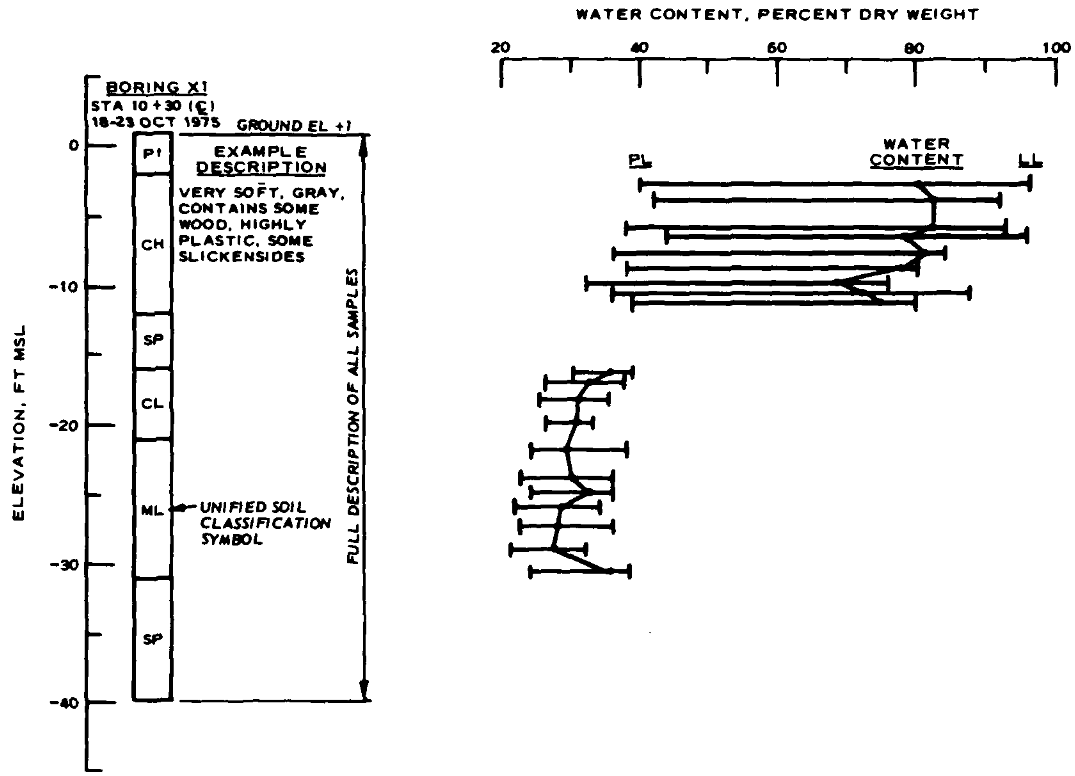


Figure 3. Typical boring log with results of Atterberg limits and water content tests

Engineering Property Tests

Compaction

56. Compaction tests are performed primarily on fine-grained materials (except as discussed in the next paragraph) and serve to define the maximum dry density and optimum water content of a soil. The type and number of compaction tests will be dependent on the proposed method of construction and variability of available borrow materials. There are several compaction tests used today, each intended to match a certain field compaction effort. The types of compaction tests required for different methods of construction are summarized in Table 6 and further discussed in Part VIII. The minimum number of compaction tests should consist of one test for each type of borrow material to be used in the dike.

Relative density

57. Since standard impact compaction tests on clean coarse-grained materials do not normally yield well-defined values of maximum dry density and optimum water content, the relative density test, which results in a minimum and maximum density, is utilized for these materials. However, for coarse-grained materials with significant amounts of fines (i.e. percent smaller than the No. 200 sieve size), the impact compaction test may yield a greater value of maximum density. If such is the case, then the compaction test should be employed. For borderline soils, both tests should be run to determine which test method results in the greater value of dry density. The test yielding the greater value should be adopted for all subsequent tests on materials with similar amounts of fines.

Permeability

58. Fine-grained soils. There is generally no need for laboratory permeability tests on fine-grained fill material or surface clay overlying pervious foundation deposits. In underseepage analyses, simplifying assumptions must be made relative to thickness and soil types of fine-grained surface blankets. Furthermore, stratification, animal burrows, root channels, and other discontinuities in fine-grained

materials can significantly affect seepage patterns. Therefore, an average value of the coefficient of permeability based on the dominant soil type is generally of sufficient accuracy for use in underseepage analyses, thus negating the need for laboratory tests. Most homogeneous clay dikes and positive clay cutoffs can be considered impervious.

59. Coarse-grained soils. The problem of foundation underseepage and dike through-seepage requires reasonable estimates of permeability of coarse-grained pervious deposits. However, because of the difficulty and expense in obtaining undisturbed samples of sand and gravel, laboratory permeability tests are rarely performed on foundation deposits. Instead, correlations developed between grain size and coefficient of permeability (such as that shown in Figure 4) are generally utilized.

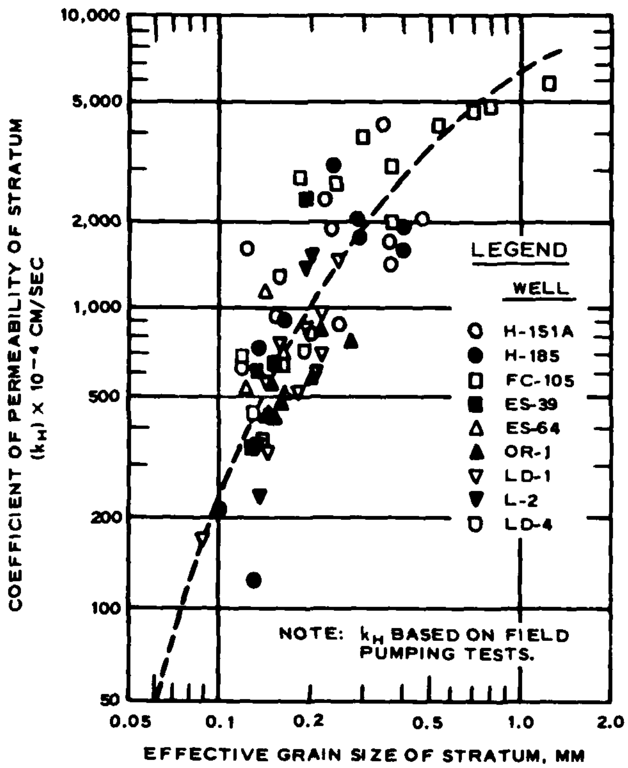


Figure 4. Effective grain size of stratum versus in situ coefficient of permeability. Based on data collected in the Mississippi River Valley and Arkansas River Valley (after TM 5-818-5[15])

This correlation explains the need for performing gradation tests on pervious materials where underseepage problems are indicated. For seepage analyses of the dikes themselves, permeability tests on compacted representative samples can be utilized or correlations may be employed.

Consolidation

60. If settlement of the retaining dike is considered to be a significant factor in design, consolidation tests should be performed on selected samples representative of the principal compressible foundation strata. Consolidation tests are normally not required for the dike materials themselves, unless the dike is to be extremely high, or for coarse-grained foundation materials, because consolidation of such materials can usually be assumed to occur simultaneously with loading.

61. Consolidation tests require high quality undisturbed samples as sample disturbance can influence the results considerably. Test loads should be sufficiently high to define the straightline or virgin compression portion of a semilogarithmic plot of the pressure-void ratio curve (Figure 5) in order that the maximum past effective vertical stress $\bar{P}_c$ may be determined. A sufficient number of consolidation tests should be performed within a selected boring or borings to develop a good definition of the variation of $\bar{P}_c$ with depth.

62. In evaluating normalized soil parameters (NSP)[16] for use in obtaining a reliable undrained shear strength variation with depth for cohesive soils (discussed in Part VII), it is necessary to have an accurate picture of $\bar{P}_c$ (or $\bar{\sigma}_{vm}$ as it is termed in Reference 16) in order that the overconsolidation ratio (OCR) may be defined with depth. For these purposes, it has been found by experience that plotting the consolidation test results as strain (rather than void ratio) versus log pressure at the end of primary consolidation instead of at the end of the 24-hr standard load increment yields better values of $\bar{\sigma}_{vm}$.

Shear strength

63. There are three primary types of shear strength tests, each representing a certain loading condition. The Q-test represents unconsolidated-undrained conditions; the R-test, consolidated-undrained conditions; and the S-test, consolidated-drained conditions. For dike

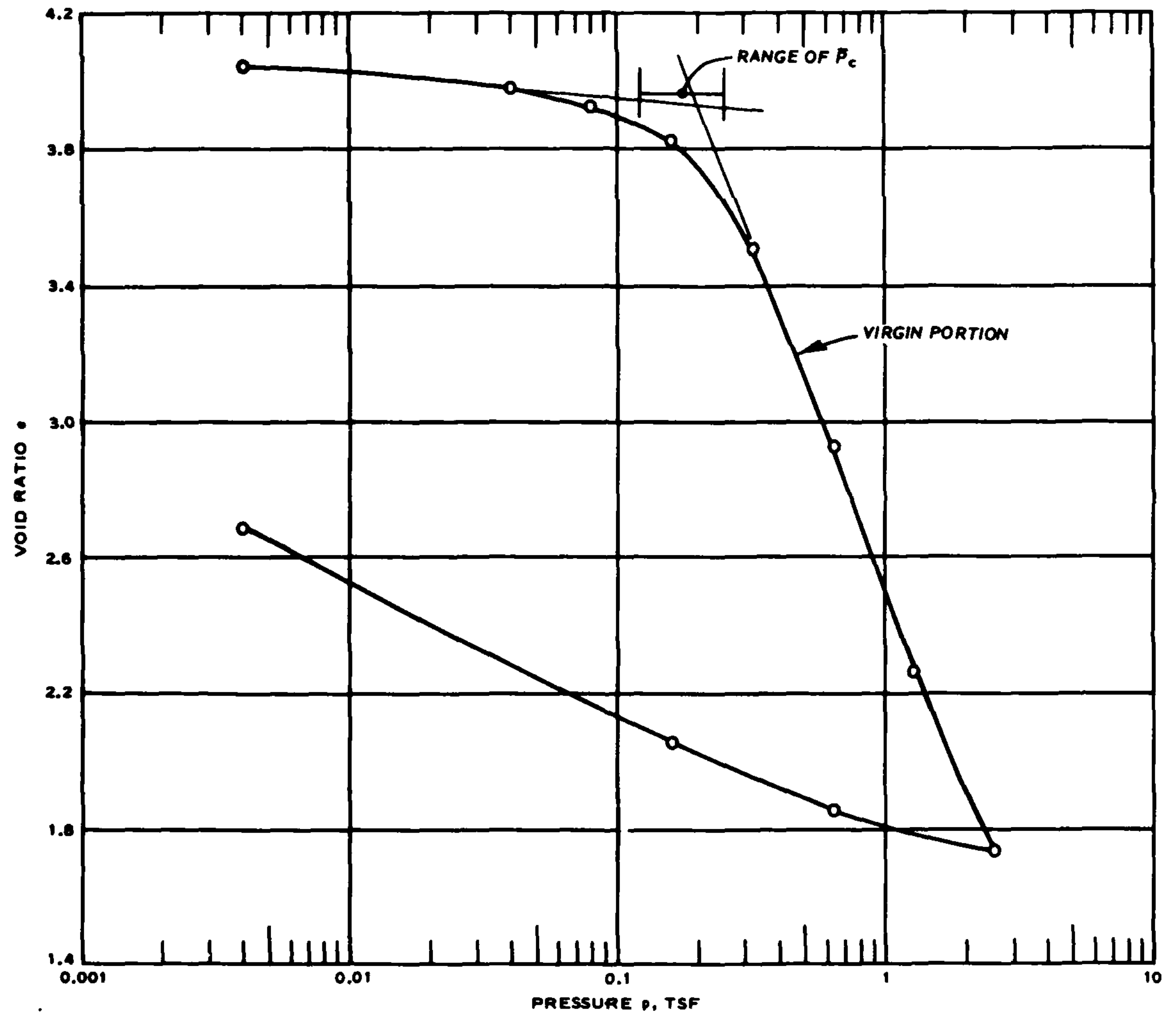


Figure 5. Typical void ratio-pressure curve

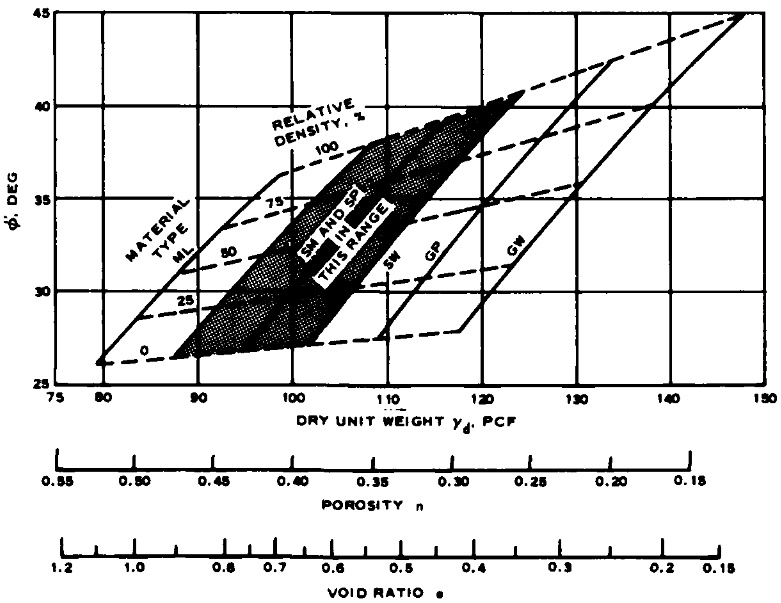
design, the most common of these will be the Q-test since the in situ undrained strength generally governs the design of embankments on soft to medium clay deposits. R-tests are generally not needed for most dike designs unless the embankment is very high (i.e., over about 30 ft) or if stage construction is planned and estimates of strength gain with time are needed. S-tests are more commonly used for dikes and are employed where long-term stability is to be checked, the soil to be tested is free-draining, or effective stress analyses are to be performed.

64. Q- and R-tests are performed in triaxial testing devices while S-tests are usually performed in triaxial devices for sand and direct shear devices for silt and clay. The unconfined compression (UC) test is a special case of the Q-test in that it also represents unconsolidated-undrained conditions but is run with no confining pressure. UC tests should only be performed on saturated clays that are not jointed, fissured, or slickensided. Also, rough estimates of unconsolidated-undrained strength of clay can be obtained through the use of simple hand devices such as the pocket penetrometer or Torvane. However, these devices should be correlated with the results of Q- and UC-tests.

65. The following discussion relates the applicability of each type test to the different general soil types. The applicability of the results of the different shear tests to field loading conditions and the different cases of stability are discussed in Part VII.

66. Sand. Since consolidation of sand can be considered as occurring simultaneously with loading, the appropriate shear strength of sands for use in stability analyses is the consolidated-drained S-strength. However, the shear strength of sand, either in the foundation or embankment (regardless of the method of placement), is not normally a critical or controlling factor in dike stability. Therefore, a comprehensive laboratory testing program to determine the shear strength of sand is usually not warranted. The use of a design shear strength where the angle of internal friction ϕ equals 30 deg and the soil cohesion c equals 0 is considered acceptable for both naturally

occurring foundation sand as well as embankment sand placed with relative densities as low as 40 to 50 percent. Satisfactory approximations of ϕ for most sand can also be made from correlations with standard penetration resistances and relative densities. Such correlations can be found in most standard engineering texts on soil mechanics. One example is shown in Figure 6.



NOTE: ϕ' OBTAINED FROM EFFECTIVE STRESS FAILURE ENVELOPES.
APPROXIMATE CORRELATION IS FOR COHESIONLESS MATERIALS WITHOUT PLASTIC FINES.
SPECIFIC GRAVITY OF 2.68 WAS ASSUMED TO COMPUTE POROSITY AND VOID RATIO.

Figure 6. Angle of internal friction versus density for coarse-grained soils (after Reference 17)

67. <u>Clay and high plasticity silt.</u> The undrained shear strength parameters should be determined for all fine-grained materials in the dike foundation and any existing fine-grained dredged material that may affect the dike design. In areas of soft, weak cohesive foundations, it is imperative that an adequate shear testing program be accomplished to establish the variation in unconsolidated-undrained shear strength with depth within the foundation (usually expressed as the ratio of undrained shear strength S_u to overburden pressure p_o) as shown in Figure 7. A sufficient number of Q-tests, supplemented by UC tests, where

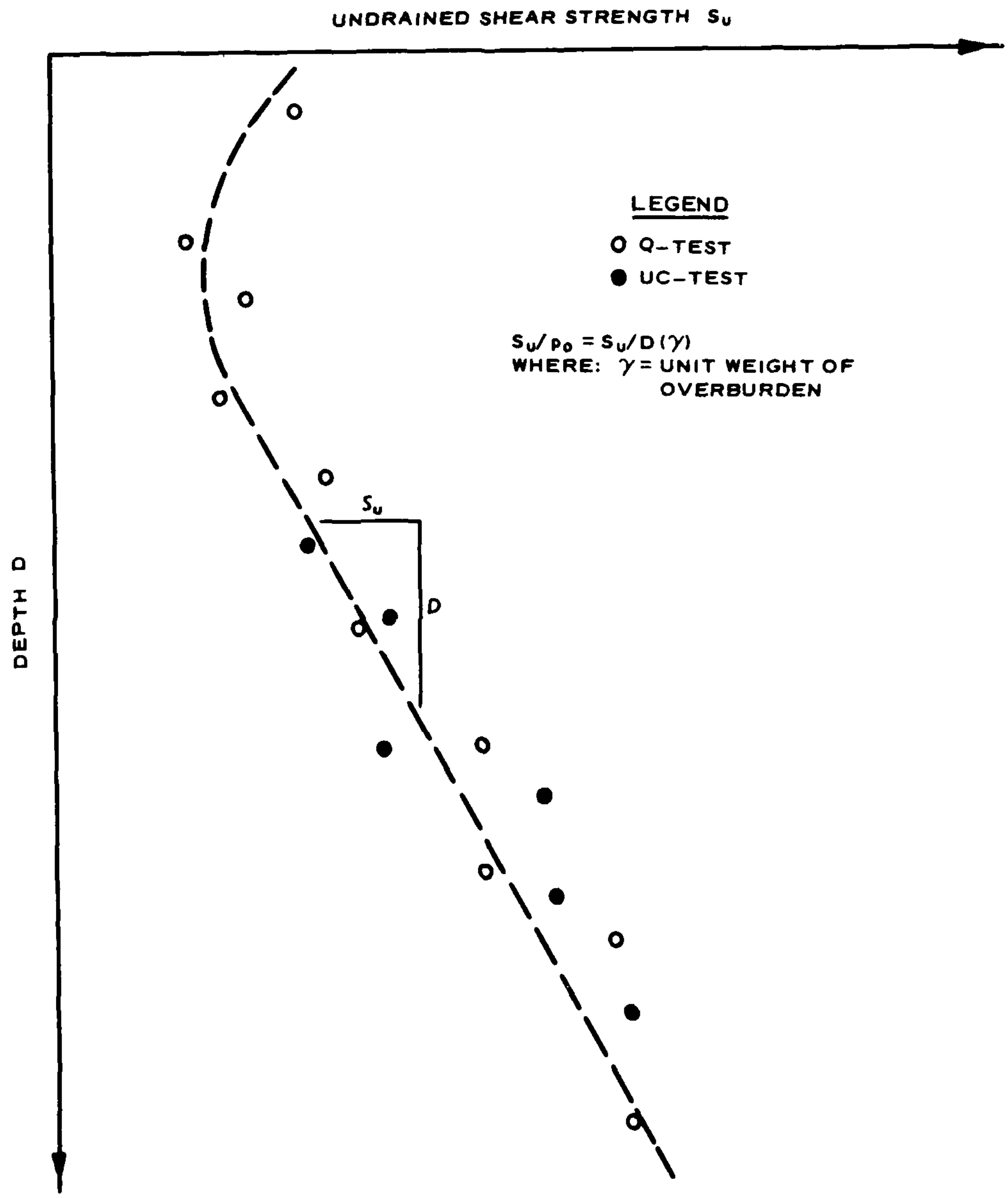


Figure 7. Typical plot showing variation of unconsolidated-undrained shear strength with depth

appropriate, should be performed throughout the critical foundation stratum or strata. Data obtained from any field vane shear strength tests may also be helpful in establishing this variation.

68. R-tests can be extremely helpful in establishing the variation in undrained shear strength with depth, in evaluating long-term stability, and in determining the increase in undrained shear strength with increased effective consolidation stress, which may be necessary in estimating the gain in shear strength with time after loading for stage construction.

69. The results of S-tests are used in evaluating the long-term stability of dikes and are extremely helpful in judging the stability of embankments where pore pressure data, such as that obtained from piezometers, are available.

70. Low plasticity silt. The rate of drainage and consequently the rate of increase in effective stress with loading for low plasticity silt is intermediate to that of sand and clay. Therefore, it is often questioned if the Q- or R-shear strength should be used in design for the undrained or after-construction type analyses. However, considering the fact that the rate of applied loading in the construction of most retaining dikes will not be too rapid, it is considered appropriate to use the R shear strength in design for low plasticity silt for the undrained loading cases. This assumes that the soil will consolidate under the applied load, but may shear in an undrained condition. Unfortunately, the determination of appropriate or realistic R shear strengths for low plasticity silt deposits from laboratory shear tests is often difficult. This is due to the dilative nature of many silts and silty soils that results in the development of large induced negative pore pressures during undrained shear and consequently unusually high apparent shear strengths that may not in reality develop in the ground. As a result of this, R-tests on these types of soils should be performed with pore pressure measurements to indicate if dilatency is occurring and to determine its magnitude and influence on the measured undrained shear strength. Any portion of the undrained shear strength that is derived from induced negative pore pressures should not be used

in design. For medium to high plasticity silts that drain slowly, the Q-test is considered appropriate.

71. Based on numerous R-tests on low plasticity silt in the U. S. Army Engineer Lower Mississippi Valley Division (LMVD) that tended to dilate during shear and produce large induced negative pore pressures, an R shear strength of ϕ equals 20° and c equals 300 psf has been developed as the maximum value that should be used for silts in design work performed in the LMVD. If R-tests on this type of silt do not show a tendency of the silt to dilate significantly during shear, the measured strength values should be considered valid and used in design.

72. Procedures. Procedures for the performance of previously discussed shear tests are outlined in EM 1110-2-1906.[13] In performing these tests one should be sure that field conditions are being duplicated as closely as possible. Confining pressures for triaxial tests and normal loads for direct shear tests should be chosen such that the anticipated field pressures are bracketed by the laboratory pressures based on depth and location of sample and anticipated field loadings. All samples should be sheared at a rate of loading slow enough that there will be no significant time-rate effect. The specimen size should also be chosen such that scale effects are minimized. Standard size of samples for triaxial testing is 1.4-in. diam by 3-in. height. However, if the sample is fissured or contains an appreciable amount of large particles such as shells, gravel, etc., then a larger size sample (say 2.8-in. diam by 6-in. height) can be utilized in order to obtain valid results. Guidance on minimizing the effects of rate of loading, size, etc., is also contained in EM 1110-2-1906.[13]

PART V: BORROW AREAS

73. In the past many borrow areas have been selected for dike projects without adequate planning and proper consideration of all the many factors involved. As a result, borrow-related problems have frequently been encountered during dike construction, many of which resulted in costly design and contract modifications. Proper design of retaining dikes must, therefore, consider the borrow areas as well as the dike embankments.

74. A proper evaluation of borrow and the determination of its impact of the project should include the assessment of the following factors:

- a. Engineering desirability of each material type. This includes determination of the maximum allowable height of dikes; estimation of the required length of time between dike construction and retention area usage; and estimation of dike size, settlement, and maintenance. The predicted performance of dikes constructed of each type of available material should be in accordance with the long-range plan for the retention area.
- b. Relative economics of dikes constructed of each available material. Involved in this study will be real estate costs, costs of moving the material from the borrow area to the dike, and cost of placement and compaction, shaping, etc. The advantages of utilizing material from required excavation or increasing the retention area size by borrowing from inside the area should also be included.
- c. Environmental impact. The impact on the environment of the use of each possible source must be carefully evaluated. This consideration has become much more important today, and, although it is often difficult to put a dollar value on environmental factors, these effects must be considered to the fullest extent possible and may very well result in a higher total dollar costs of the project.

75. It must be emphasized that in order to make a proper evaluation of materials available for a project, sufficient exploration and testing of these materials must be conducted. Exploration and testing must be extensive enough so that all possible sources of borrow are located, the extent of each determined, and the type and engineering

properties of each determined. Only then can the necessary assessments be properly accomplished.

Material Sources

76. As previously mentioned, more than one source of material for dike construction is normally available. Some possible sources of borrow are briefly discussed in the following paragraphs.

Required excavation

77. Material from required excavations should be given first consideration since it is usually the most economically desirable because it must be excavated and disposed of in any event. Included in this category is material from adjacent ditches, canals, and appurtenant structures. Also included is material from inside the retention area which, although not strictly required, may be classified as such since it serves a purpose other than just providing material for the dike (i.e., the use of it increases the capacity of the retention area). The use of material from required excavation also eliminates the problem of dealing with borrow areas left permanently exposed after project completion.

Material adjacent to dike toe

78. This is probably the most common source of dike material because it involves a short or no haul distance and is conducive to dragline operation, which is an often used and economical method of construction. However, one important factor not to be overlooked when utilizing this or any source of borrow near the dike is the effect of the excavation on the stability of the dike. As shown in Figure 8, a berm should be left in place between the toe of the dike and the excavation, not only to ensure stability of the dike but also to facilitate construction. The length of this berm should be based on stability analyses of the dike and the excavation. An example of improper placement of the borrow ditch (i.e., no berm) is shown in Figure 9. The effect of nearby excavations and natural depressions and dike stability is further discussed in Part VII.

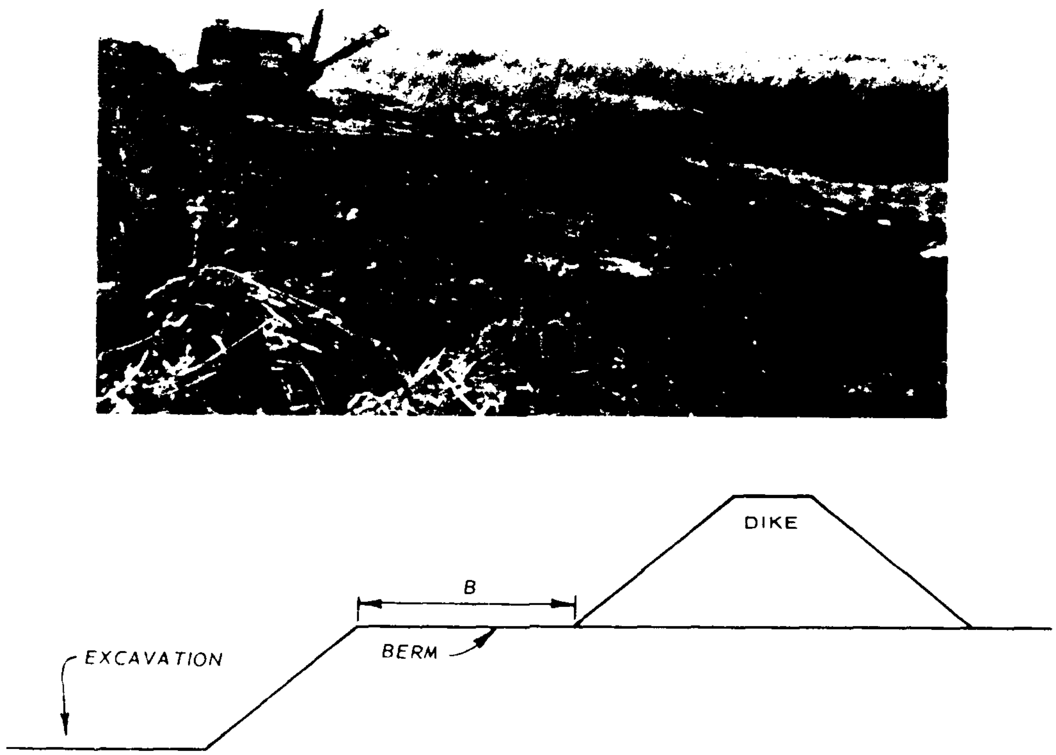


Figure 9. Excavation near dike toe

Central borrow area

[illegible]. A central borrow area as shown in Figure 10 is normally utilized when suitable material cannot be economically obtained from sources previously mentioned. Central borrow areas can be utilized for both hauled and hydraulic fill dikes. Dredging from a water-based central pit is usually an economical source of borrow where hydraulic fill dikes are to be constructed. Usually a deeper pit with a smaller surface area is the most economical since the less movement required by the dredge, the cheaper the operation. The disadvantage of hauling material from land-based central pits is the longer haul distance required.

Figure 9. Dike with borrow ditch too close to dike toe

Figure 10. Central borrow area showing use of berms, ponding areas, temporary dikes, etc., to control water

Maintenance dredging

80. Maintenance dredging is a very economical source of material since, like required excavation, the material must be disposed of anyway. However, most fine-grained material from maintenance dredging is not suitable for dike construction without considerable drying as it normally has extremely poor engineering qualities in the "dredged" state. Generally, only sands or predominately coarser grained materials from maintenance dredging are desirable for dike construction with material from new dredging work more often suited to dike construction.

Previously placed hydraulically dredged material

81. Previously placed dredged material, if sufficiently dried, can often make an adequate material for dike construction. The quality of this material may vary considerably across the retention area, however. Zones around the dredge discharge usually will provide the highest quality material. Use of previously placed dredged material has been common for the raising of existing dikes since it is so readily available and its use increases the capacity of the retention area (Figure 11).

Acceptable Materials

82. Almost any type of material can be classified as acceptable (even though not the most desirable) for construction of retaining dikes, with the exception of very wet fine-grained soils and those containing a high percentage of organic matter. Also, highly plastic clays may sometimes present a problem because of their detrimental shrink-swell properties when subjected to alternate cycles of drying and wetting.

Compacted, semicompacted, and uncompacted (cast) fill

83. The natural water content of materials used in conjunction with these methods of construction is very important. When compacted dikes are planned, it is necessary to ensure that available borrow material has a low enough water content to allow placement and compaction.

Figure 11. Use of previously placed dredged material. Photo taken from existing dike looking across containment area. Note wide, shallow excavation necessitated by desire to utilize upper dried material. Also note ditch to aid in drying of borrow

Semicompacted fill can tolerate material with higher water contents, while uncompacted cast fill can be placed at very high water contents. Since dike construction is normally in low wet areas, problems with materials being too dry are rarely encountered.

34. In the construction of retaining dikes, it is common to find that all available borrow material has high water contents. If such is the case, the material must either be dried to a water content suitable for the desired type of construction, or the design must take into account the fact that the material has a high water content. The cost of drying material is often very expensive, time consuming, and highly dependent on the weather. It is therefore recommended that, if at all possible, the design be one that incorporates the properties of the material at its natural water content or involves only a minimum of drying.

Hydraulic fill

85. Almost any material from coarse-grained sandy gravel to fine-grained clay can be dredged and pumped. However, the coarser grained the material, the quicker the material can be worked and shaped by conventional equipment after deposition by the dredge. Specific qualities of different types of pumped materials and their effect on dike construction are discussed in Part VIII.

Geometry

Slopes

86. Excavation slopes for borrow areas should be designed to ensure stability from all possible modes of failure (shear, erosion, seepage, etc.). This is of particular importance for slopes of pits that parallel the dike alignment or are a part of required excavations and will be permanently exposed, but it is also important for any slope whose top is near right-of-way limits or existing structures. Where mowing will be required, slopes should be flat enough to facilitate the operation of machinery (at least 1V on 3H). It is also advisable, especially in populated areas, to leave permanently exposed side slopes of borrow areas that will contain water flat enough to allow the victim of an accident to climb out.

Size and depth

87. It is generally preferable that pits that parallel the dike alignment be wide and shallow as opposed to narrow and deep even though narrow and deep pits are sometimes preferred from a construction standpoint, especially when using a dragline. The use of wide and shallow pits will reduce effects of the excavation on dike stability and, even though requiring a greater surface area, may make it easier to employ measures to reclaim the area from an environmental standpoint. The size of large central borrow areas will be primarily dependent on the economics of the excavation operation considering factors mentioned in the next paragraph.

88. Factors that govern the depth of excavation of borrow areas

include the depth and thickness of useable material, elevation of groundwater, effects of the excavation on dike stability, real estate cost, and environmental considerations.

Quantities

89. To avoid costly contract modifications and to reduce the possibility of claims by the contractor, it is important that sufficient material be available at the outset of construction. In order to ensure this, the theoretical quantity of material required to build the dikes should be increased by a certain amount to take care of contingencies such as material loss from handling and compaction, stumps, pockets of unuseable material, and miscellaneous use of material by the contractor to facilitate construction. The factor or number by which the theoretical required quantity should be multiplied in order to arrive at the amount of borrow required is commonly referred to as a "shrinkage factor." Shrinkage factors f for various methods of construction are given in Table 9. These values are based on the past experience of several CE Districts and are considered minimum values. Where a particular project is such that higher material losses are anticipated, higher values should be used. Also, although not reflected in Table 9, less shrinkage will generally occur with sands than with finer grained material for hauled and cast dikes.

Borrow Area Operations

Clearing, grubbing, and stripping

90. Clearing, grubbing, and stripping of borrow areas should be carried out to the extent needed to obtain fill material free from objectional matter such as trees, brush, vegetation, stumps, roots, and organic soil. In marshy areas, a considerable depth of stripping may be required due to the frequent existence of 3- to 4-ft root mats, peat, and underlying highly organic soil. However, such operations may be restricted in soft, marshy areas because of lack of support for

Table 9

Shrinkage Factors

Method of Construction	Shrinkage Factor, f*
Hauled, compacted	1.3
Hauled, semicompacted	1.3
Cast, uncompacted	1.5
Hydraulic	
Sand	1.5
Clays (medium or greater consistency)	2.0
Soft silty clays**	4.0 to 6.0
Clayey silts**	4.0 to 6.0

* Theoretical quantity needed × f = actual quantity of borrow needed to construct dike.

** Use higher values of f for materials that will be completely dispersed in the slurry.

equipment. All stripped organic material should be wasted in low areas or, where useable as topsoil, stockpiled for later placement on outer dike slopes, berms, exposed borrow slopes, or other areas where vegetative growth is desired.

<u>Excavation</u>

91. In order that borrow areas may be utilized to the fullest extent possible, excavation operations should be carefully planned with consideration given to proximity of areas to the dike, topography, location of groundwater table, possible excavation methods and equipment, and surface drainage. The excavation operation should be provided with experienced personnel and close supervision of the contractor's operations should be maintained. Excavation techniques and overall methods of operations should be utilized such that no useable areas will become inaccessible, thereby causing a reduction in obtainable quantities.

<u>Drainage</u>

92. Proper drainage of borrow areas (entailing control of surface and groundwater) is necessary to achieve a satisfactory degree of utilization. Past experience has shown this one item probably has more effect on borrow operations than any other single item and can often be the difference in a good job and a poor one. Proper drainage of borrow areas can often be achieved by working the area in accordance with natural topography and drainage patterns. Many times, however, natural drainage is poor and the only choice is to begin at the lowest point and work toward the higher areas, thus creating a sump to aid in draining the work area. In some cases pumping of sumps or low areas may be necessary.

93. Maximum utilization of ditches, especially in shallow borrow areas, should be made, as ditches provide a cheap method of controlling water and drying material (Figure 12). It is sometimes amazing how a series of properly placed ditches can aid in controlling surface water and groundwater. The Philadelphia District has successfully used ditching techniques in previously dredged material to the extent that placement properties of the material were measurably improved. It is felt this success is due primarily to the fact that previously dredged

fine-grained material contains numerous desiccation cracks (Figure 13) that aid significantly in drainage, evaporation, and, consequently,

Figure 12. Example of ditching to aid in drying borrow. Existing dike was breached to allow escape of water from perimeter ditches

Figure 13. Desiccation cracks in previously placed dredged material

the drying process. It is desirable that the ditching be done well ahead of the excavation, especially in fine-grained soils. This will allow maximum drying of the material prior to excavation. Proper placement and timing of excavation of ditches should not be left up to the contractor but should be a part of the contract plans and specifications:

Environmental Considerations

94. The treatment of permanently exposed borrow areas to satisfy aesthetic and environmental considerations has, in the past few years, become standard operating practice. Generally, projects near heavily populated or industrial areas will require more elaborate treatment than those in sparsely populated areas. Minimum treatment should include proper drainage, topographic smoothing and blending, and promotion of conditions conducive to vegetative growth. Insofar as possible, borrow areas should be planted to conform to the surrounding landscape. Restoration of vegetative growth is important because it is not only aesthetically pleasing but serves as protection against erosion and promotes wildlife habitation. Mann et al.[18] should be consulted for more detailed information concerning landscaping techniques.

PART VI: DESIGN AND CONSTRUCTION CONSIDERATIONS

95. In their review of CE design and construction procedures for retaining dikes, Murphy and Zeigler[2] concluded that there is normally little effort expended in the design of most retaining dikes. It was found that, in most cases, no special effort was made to improve foundation conditions and that construction materials were normally borrowed from within the containment area even though such materials often possessed very poor engineering properties. The method of construction generally was established through past practice and was not likely to be altered due to any particular foundation and/or dike material properties. Consequently, the selection of dike dimensions and construction methods was based largely on a review of previous dike construction experience. Dike heights, side slopes, and crown widths were chosen to match those of similarly constructed dikes that performed satisfactorily. In many cases a successful and stable dike was obtained; however, where foundation and/or dike materials were poor or dikes were constructed to appreciable heights, frequent failures occurred and continual maintenance was required. It should be noted that the above conclusions, which were based on a survey made in 1972-73, did concern the majority of CE Districts, but not all; more recently, extensive and detailed design studies have been conducted on a fairly regular basis by a number of CE Districts. In fact, much of the information in this report is based on the work of these Districts.

96. Past experience indicates that the occurrence of dike failures can be related to the amount of design effort expended on the dike; i.e. as the dike design effort increased, the occurrence of dike failure decreased. Small dikes constructed in areas where design experience has been gained through actual dike construction will obviously require less design consideration than large dikes to be constructed in unfamiliar areas. The factors that affect the extent of design effort were given in Table 1. It should be noted that design effort is not limited to selection of a dike section, but must include a thorough study of construction methods and techniques and their effects on the final

desired product. This is because the method of construction is such an integral part of dike construction and is very critical to the success of the overall project.

97. Factors that should be considered in the design and construction of retaining dikes are foundation conditions; dike materials; dike stability with respect to shear strength, seepage, settlement, and erosion; and construction methods. The importance of proper determination and evaluation of foundation conditions and dike materials has previously been discussed. The purpose of Part VI is to present some of the remaining items to be considered in the design and construction of retaining dikes.

Dike Geometry

98. Dike geometry refers to crown width, height, and side slopes. These variables are primarily dependent on foundation conditions, embankment materials, construction methods, and project objectives. This being the case, it is not possible to arrive at many specific recommendations relative to the geometry of dike cross sections that can be generally applied to all dikes regardless of site conditions and project requirements.

99. The crown width will be dependent on required usage for maintenance and emergency operations and, to some degree, stability requirements. However, a minimum crown width of 6 to 8 ft is recommended. Dike height will be established by retention area capacity requirements and freeboard. In the past, freeboard for most dikes has ranged from 1 to 3 ft. A minimum freeboard of 2 ft is recommended with greater amounts for dikes in areas subjected to considerable wave action or weathering and where failure consequences are severe. Overbuilding for anticipated settlement is not included in the above freeboard figures and, if required, should be determined from settlement analyses. Dike slopes will be established on the basis of stability of the dike with respect to shear strength, seepage, and settlement, and on the method of construction. However, as stated previously, if any mowing of

exterior slopes is required, slopes should be no steeper than 1V on 3H.

100. A summary of some dike cross sections constructed in various CE districts is given in Table 10. An analysis of these data reveals the impracticality of generalizations relative to dike geometry. When foundation conditions and dike materials are similar to those where dike systems have been successfully constructed previously, slopes and sections previously used are a valid basis for approximating initial section geometry. However, unless it can be shown that these conditions are very similar to those being designed for, design sections should be determined only by detailed stability analyses that include all of the specifics of each individual section. Requirements and descriptions for necessary stability analyses are presented in Part VII.

Effect of Dike Materials and Foundation Conditions

101. The types of materials available to build a retention dike of and on play the most important role of all variables in the selection of a dike section. Available materials not only affect the design of a dike from the stability standpoint but usually also dictate the method of construction. For example, where materials with suitable engineering properties for dike construction are either unavailable in the immediate vicinity of the disposal area or are not accessible to conventional types of hauling or casting equipment, hydraulic dredging of materials over a long distance may be the only practical means of construction available. In such cases, the dike may possibly have a high factor of safety with respect to stability because of the very flat side slopes, but still be more economical than a smaller section constructed by other methods. In other cases where adequate borrow material is available, construction of a dike system utilizing draglines or hauling equipment may be the most economical.

102. Where a competent dike foundation exists, considerable flexibility is available for selection of the dike section. However, as the adequacy of the foundation decreases, the flexibility in selection of the section and method of construction also decreases. For

Table 10

Summary of Corps of Engineers Dike Sections

District	Foundation Material	Dike Material	Height, ft Initial	Height, ft Increment	Height, ft Maximum	Crown Width, ft	Side Slopes	Remarks
Galveston	Soft marsh clay and dredged sandy silt	Dredged clay and silty sand from disposal area	4 to 5	2 to 4	10	4 to 8	1V on 2H to 1V on 4H	Dike placed by dragline with some compaction by dozers
	Soft to stiff clay with some silt and sand	Dredged sandy clay from disposal area	(Interior height of 6 to 10 ft)		15 to 25	8	1V on 3H	Semicompacted fill raising existing dikes
	Dredged silt, sand, and clay	Dredged clay and clayey sand from disposal area	(Interior height of 10 ft)		17	8	1V on 3H	Semicompacted fill raising existing dikes
Charleston	Dredged silty sand and clay overlying soft marsh deposits	Dredged sandy silt from disposal area	(Interior height of 3 to 5 ft)		10 to 15	8	1V on 1.5H	Dike placed by dragline with no compaction. Failures occurred in two reaches
	Soft organic silt and clay	Fine silty sand with high organic clay and shell content	--	--	--	10	1V on 1.5H	Dike placed by dragline with no compaction. Raised existing dikes
	Dredged silty sand and clay overlying soft marsh deposits	Dredged sandy silt from disposal area	7	--	7	10	Broken 1V on 8H then 1V on 25H	Hydraulic fill shaped with dozers
Philadelphia	Dredged silt and clay underlain by marsh	Dredged silt and clay from disposal area	10 to 12	--	10 to 12	8	1V on 1.5H	Dike compacted by hauling equipment
	Dredged silty sand underlain by marsh	Dredged silty sand from disposal area	10	--	10	10	1V on 1.5H	Dike compacted by hauling equipment
Wilmington	Dredged clay, silt, and sand	Sand and clayey sand	20 to 25	--	20 to 25	15	1V on 2H	Dike compacted by hauling equipment
Norfolk	Dredged silt and sand with variable organic content	Dredged silt and sand from disposal area	6 to 8	--	6 to 8	3	1V on 1H	Dike placed by dragline with no compaction
Detroit	Dredged organic silt and clay and sandy silt	Clay with some sand and gravel	--	--	15 to 18	25	1V on 2H	Dike compacted by routing construction equipment
Sacramento	Sand	Silt, clay, and sand	10 to 15	--	10 to 15	12	1V on 2H	Semicompacted fill

instance, it is impractical, if not impossible, to construct a steep-sloped compacted dike on a soft foundation. Conversely, it is usually unnecessary to specify a compacted dike where a soft foundation dictates a section with flat slopes; rather, it would be more reasonable to specify a method of construction which, by its very use, results in flatter slopes such as traffic compaction or hydraulic fill. The important thing is to make all of the variables involved mesh together. Only when this is accomplished will a sound design result.

Effect of Method of Construction

103. Dike embankments, classified according to general construction methods, are listed in Table 11. The choice of construction will be governed by available materials, foundation conditions, and economics. As can be seen in Table 11, there are basically three types of embankments with respect to material placement and compaction: compacted, semicompacted, and uncompacted. Classification by these means does not necessarily refer to the end quality of the embankment, rather it specifically refers to how much compaction effort and water content control was applied in construction of the embankment. For instance, both a cast dike and a hydraulic fill dike are classified as uncompacted. However, a hydraulic fill sand dike will have a higher density than will a cast dike built of previously dredged material. The classifications given in Table 11 merely provide a convenient means of grouping dikes according to construction methods. Basically, though, the dike section will increase in size as one goes from a compacted to an uncompacted dike. One exception to this is a low cast dike that is often built with fairly steep side slopes. From a stability point of view, however, these are the least desirable types of dikes. Methods of construction are discussed in more detail in Part VIII.

Basic Design Concepts for Slope Stability

104. There are three basic concepts of dike design for slope stability. These are shown in Figure 14 and are termed floating,

Table 11

Dike Classification According to Method of Construction

Type Compaction	Method of Construction	Requirements, Use, and Remarks
Compacted	Hauled, spread, and compacted with compaction equipment Requires specification of: Water content with respect to optimum Loose-lift thickness Type compaction equipment and number of passes	Requirements: a. Strong foundation of low compressibility b. Fill materials with natural water content reasonably close to specified ranges Provides: a. Steep-sloped embankment, occupying minimum space b. Strong embankment of low compressibility
Semicompacted	Hauled or cast with draglines Compacted with fewer passes of light roller or controlled traffic of hauling, spreading, or shaping equipment Fill material placed at natural water content (i.e., no water content control) Usually placed in thicker lifts than compacted method	Used where: a. Steep-sloped compacted embankments are not required b. Relatively weak foundations exist that cannot support steep-sloped compacted embankments c. Underseepage requirements are such as to require a wider embankment base than is necessary for compacted embankments d. Water content of fill material or amount of rainfall during construction season is such as to not justify compacted embankments, but low enough to support equipment
Uncompacted	Hauled (dumped in place), cast, or pumped hydraulically Little or no spreading or compaction Usually shaped to final lines and grade No lift thickness control Fill material placed at natural water content (i.e., no water content control)	Used where: a. Nearby materials are inadequate for compacted or semicompacted construction b. It is the most economical method of placement c. Dike heights are low for cast or dumped-in-place methods d. Relatively weak foundations exist e. Embankments with wide bases are required for stability (for pumped methods)

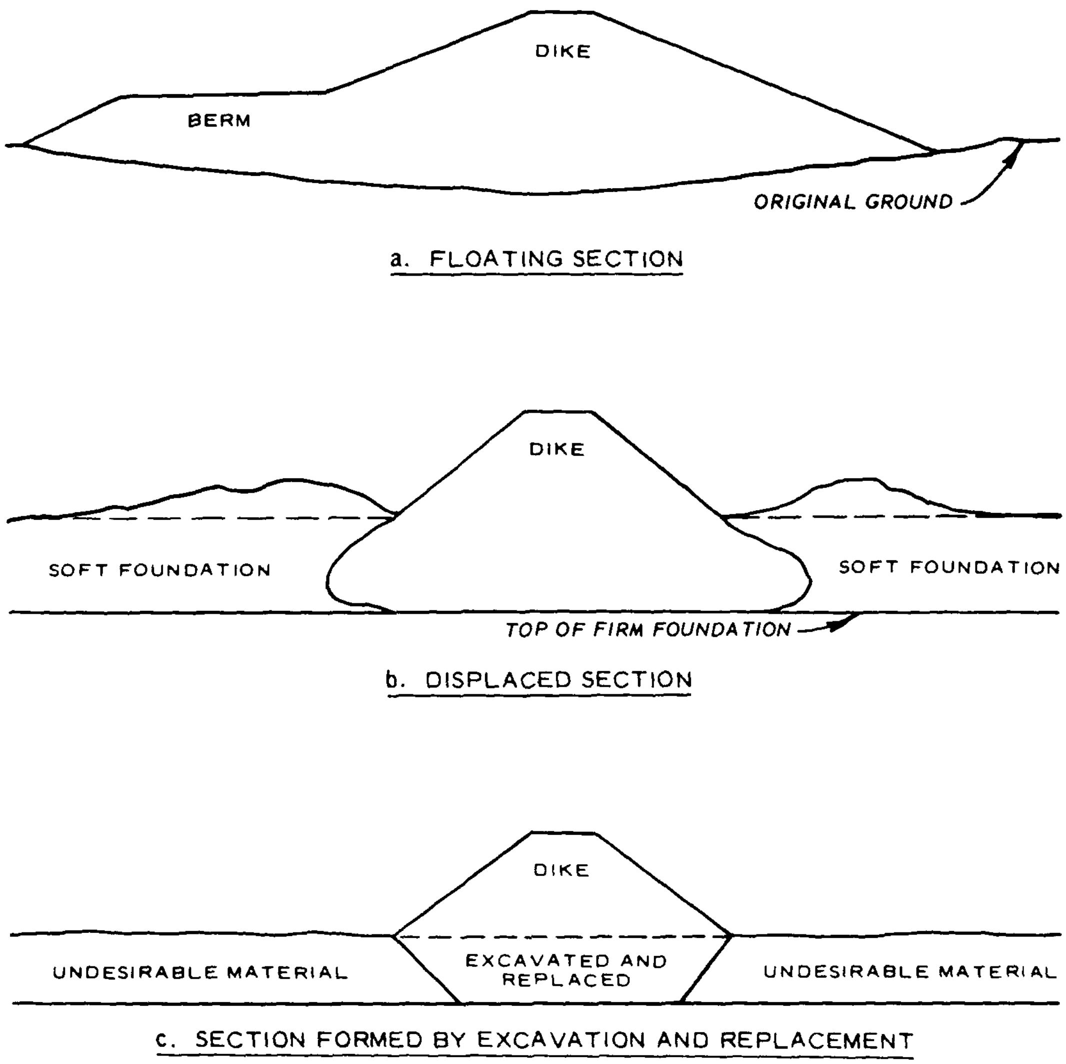


Figure 14. Basic methods of forming dike sections for stability

displacement, and excavation and replacement. There are many variations of these basic concepts, especially of the section built by floating, which can be used on any type of foundation. The displacement and the excavation and replacement sections are applicable, respectively, to very soft foundations and to foundations containing soft, organic, or otherwise undesirable material to a reasonably shallow depth. These basic concepts along with combinations and variations are discussed in detail in Parts VII and VIII. The determination of which method to use is based on available embankment materials and foundation conditions.

Floating method

105. The floating section gets its name from use on soft foundations but is applicable to stronger foundations as well. The concept involved with this type of section is to spread the embankment load sufficiently by the use of flat slopes and berms so that the foundation is not overstressed. This is usually an economical method of design but becomes more uneconomical as foundations become weaker, due to the increase in material required. Geometry of the section is determined primarily by stability analyses.

Displacement method

106. Dike construction by the displacement method is just the opposite of the floating technique in that it purposely overstresses the soft foundation material until it fails and is displaced by stronger fill material. This method requires the existence of very soft foundation materials (undrained strengths less than about 150 psf) that will readily fail and displace. It is desirable to have a stronger material underlying the soft material, but the method can be used in deep normally consolidated materials.

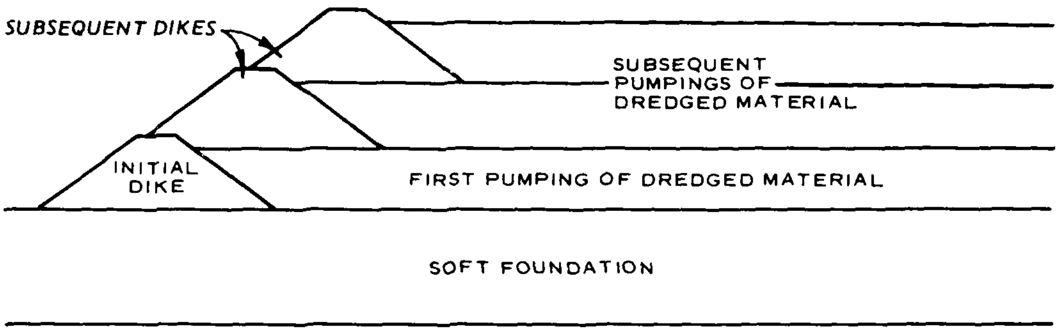
Excavation and replacement method

107. Specifying a dike section to be constructed by excavation and replacement techniques is a positive means of ensuring stability. This method involves excavating soft or undesirable material and replacing it with more desirable material. It is, however, limited by the depth of undesirable material and location of the water table, as it becomes more uneconomical as the thickness of material to be removed and replaced increases and, if dewatering is required, the higher the groundwater table. Generally, 20 ft is about the limit of excavation in the use of this technique. This method requires the existence of a firm base (stronger material) under the undesirable material.

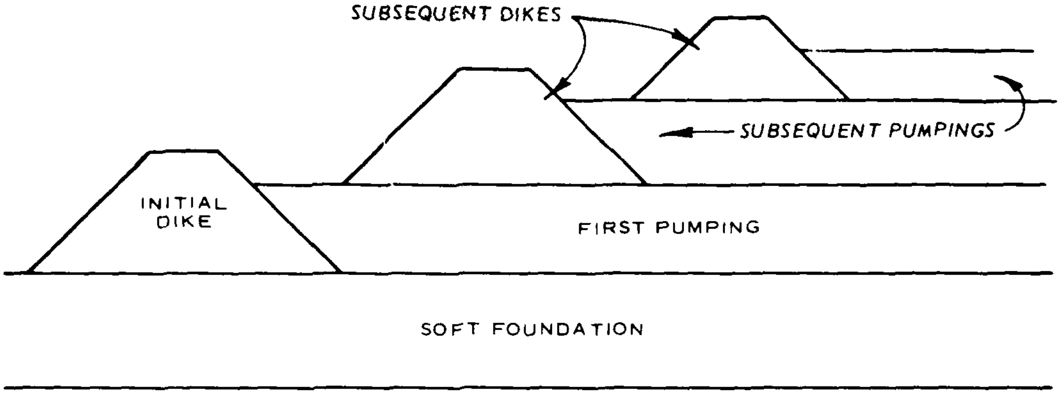
Raising of Existing Dikes

108. Due to the weakness of many dike foundations, the height to

which a dike can be built in one stage is often limited. This obviously limits the capacity of the containment area. Often, when the capacity of an area is reached, the existing dikes are raised because of the decreasing number of acceptable sites. Raising existing dikes to higher elevations than were possible initially is made possible by consolidation (and consequent strength gain) of foundation materials over a period of time due to the imposed load of the initial fill. If a dike is initially planned to be built in increments or stages, it is usually termed "stage construction." If, however, the dike is raised at some date after the disposal area is filled and was not planned initially, it is usually termed a "dike raising." In either event, the process is essentially the same. Construction of dikes in increments is usually accomplished by incorporating the initial dike into the subsequent dike as shown in Figure 15a, although in some cases interior dikes are constructed at some distance from the inside toe of the existing dike as shown in Figure 15b. Philadelphia District experience indicates that construction as indicated by Figure 15a is subject to increasing risk as dike height is increased when dikes consist of uncompacted fill. Stage construction (or dike raising) is discussed in detail in Parts VII and VIII.



a. INCORPORATION OF INCREMENTAL DIKES



b. SEPARATE INCREMENTAL DIKES

Figure 15. Incremental or stage construction of dike

PART VII: DIKE STABILITY

109. This part describes common causes of instability in dikes and presents recommended methods and procedures for analyzing dike stability with respect to inadequate foundation and/or embankment shear strength, seepage, settlement, and external erosion. The analyses described and referenced herein contain procedures that have proven satisfactory from past use, and most are currently employed by the CE. It should be recognized that any theoretical analysis is only as good as the input parameters required for the analysis. Stated in another way, a theoretical procedure may be entirely rigorous in itself, but unless the actual field conditions are duplicated as closely as possible, the results of the overall analysis will be inaccurate. In order to closely duplicate field conditions, it is necessary that soil properties and loading conditions be estimated as accurately as possible. Estimating these values is often the most difficult part of an analysis. Consequently, the determination of material properties and field loading conditions is also discussed along with the methods of analyses themselves.

Causes of Dike Instability

Inadequate shear strength

110. Shear failures in retaining dikes are the result of overstressing the embankment and/or foundation materials. Low shear strengths in the dike and/or foundation (often coupled with seepage effects) are the cause of most dike failures. Failures from this cause are usually the most catastrophic and damaging of all since they usually occur quickly and can result in the loss of an entire section of dike along with the contained dredged material. The photographs in Figure 16 show a dike failure initiated by inadequate shear strength and the resulting damage to a sewage treatment plant caused by escape of the previously confined dredged material.

111. Dike failures from inadequate shear strength have occurred that involve the dike alone and that involve both the dike and the

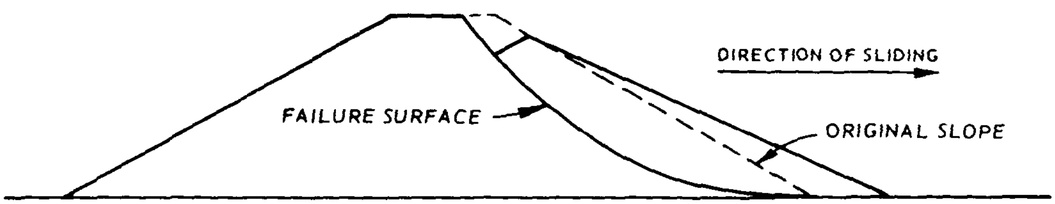
a. A 150-ft-wide break in a 20-ft-high dike

b. Flooded sewage treatment plant

Figure 16. Retaining dike failure that resulted in inundation of a nearby sewage treatment plant

foundation. Failures within a dike alone result when the dike material possesses insufficient shear strength. Failures of this type generally take the form of a rotational slide involving the dike slope as shown in Figure 17. However, if a weak plane or layer should exist at the contact between the dike fill and the foundation due to naturally existing weak surface material, inadequate foundation preparation, underseepage effects, or construction techniques that allow soft material to be placed or trapped in the lower part of the fill, the failure could take the form of a wedge-type configuration characterized by horizontal sliding or translation near the base of the fill (see Figure 18). Rotational type slides as shown in Figure 19 also occur that involve the foundation as well as the embankment. This type of failure generally develops where the foundation is relatively homogeneous with insufficient

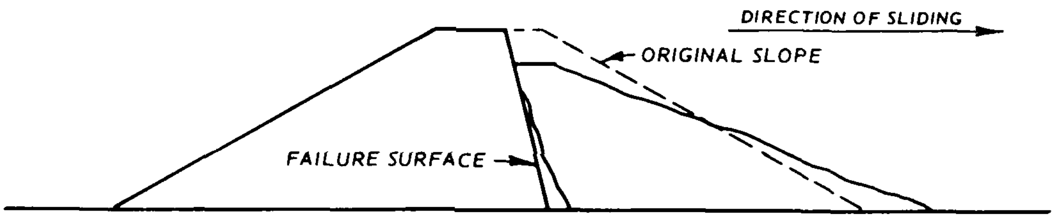
a. PHOTO OF FAILURE



b. CROSS SECTION OF FAILURE

Figure 17. Rotational failure in dike

a. PHOTO OF FAILURE WHERE SLIDING TOOK PLACE AT EMBANKMENT/FOUNDATION CONTACT



b. CROSS SECTION OF FAILURE

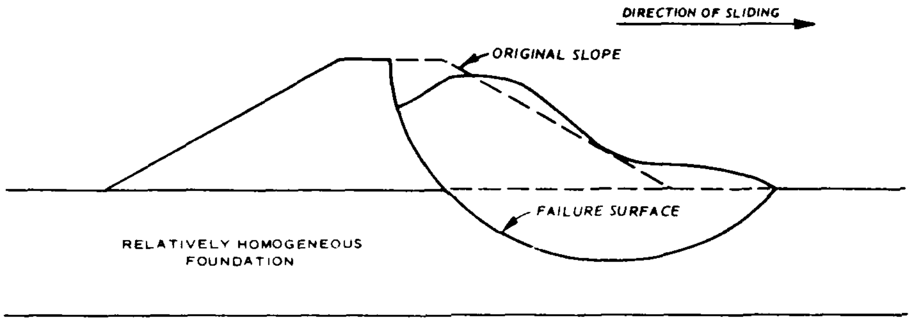
Figure 18. Translatory failure in dike

foundation shear strength being the usual cause of failure. A translatory or wedge-type failure can also occur in the foundation where the foundation consists of stratified strata of various soil types (see Figure 20). Horizontal sliding generally occurs in one of the weaker strata in the foundation.

Seepage

112. Uncontrolled seepage will occur through earth dikes and foundations consisting of pervious or semipervious material unless prevented by positive means such as impervious linings, blankets, or cutoffs. Seepage effects can create instability through internal erosion (piping) of dike or foundation materials or may lead to a shear failure by causing a reduction in the available shear strength of the dike and/or foundation

a. ROTATION OF MATERIAL BEYOND DIKE TOE



b. CROSS SECTION OF FAILURE

Figure 19. Rotational failure involving both dike and foundation

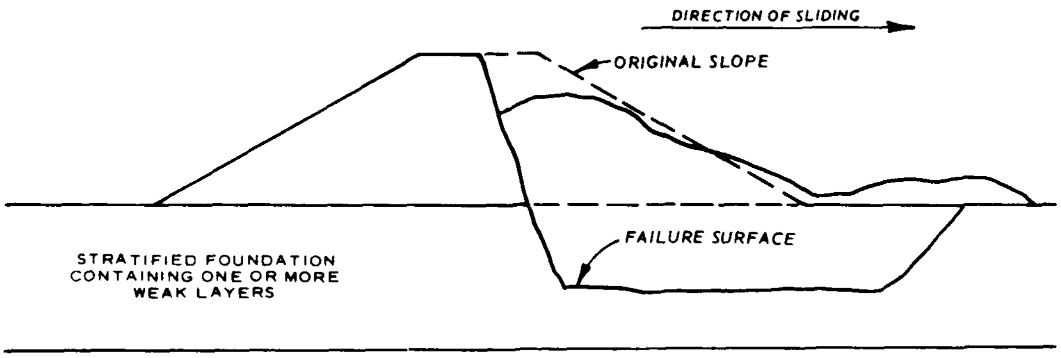


Figure 20. Translatory failure in dike and foundation

through increased pore pressure or by the introduction of seepage forces. A dike failure caused by uncontrolled seepage is shown in Figure 21.

113. The following conditions may create or contribute to seepage problems in retention dikes:

a. Washout at sluice structure

b. Debris on tidal flats downstream of failed sluice structure

Figure 21. Dike failure caused by uncontrolled seepage

a. <u>Dikes with steep slopes composed of coarse-grained pervious materials or fine-grained silt.</u> In this case the seepage line through the embankment may exit on the outer slope above the dike toe resulting in raveling of the slope. If the dike contains alternating layers of pervious and impervious materials, the seepage surface may even approach a horizontal line at the ponding surface elevation, thus creating an even more severe stability problem (Figure 22).

b. <u>Dikes built on pervious foundation materials or where pervious materials are near the surface or exposed as a result of nearby excavation (Figure 23).</u> This is a common condition where dikes are constructed by dragline using an adjacent borrow ditch. In this case surface or near-surface peat and other fibrous materials are included as pervious foundation materials. This condition may lead to the development of large uplift pressures beneath and at the outer toe of the dike causing overall

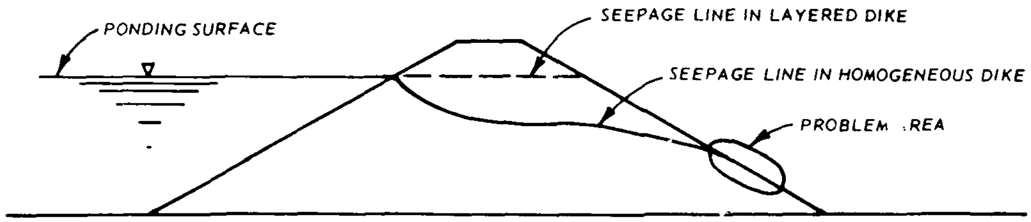


Figure 22. Seepage lines through dike

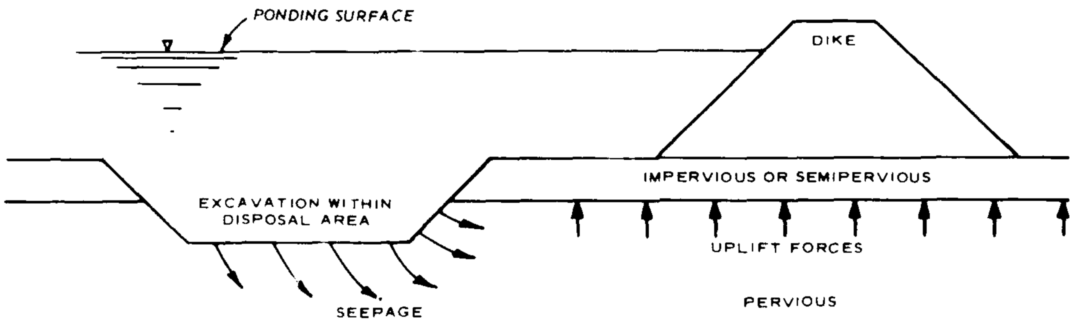


Figure 23. Seepage entrance through area excavated within disposal area

instability from inadequate shear strength or may result in piping near the embankment base.

c. Dikes constructed by casting methods with little or no compaction. This method of construction may leave voids within the dike through which water can freely flow, resulting in piping of dike material.

d. The existence of seepage paths along the plane between the foundation and the dike. Reasons for the occurrence of this condition have previously been given in paragraph 111. Seepage occurring at this point can result in piping of the embankment material along the base of the dike or the development of high uplift pressures, either of which can eventually cause failure of the embankment.

e. The existence of seepage paths along the contact between structures within the dike and the dike. This condition can be caused by inadequate compaction of dike materials against structures, shrinkage of material adjacent to structures, or differential settlement. As in previous cases, piping of the dike material usually results and normally leads to breaching of the dike.

Settlement

114. Settlement of dikes can result from consolidation of embankment and/or foundation materials, shrinkage of embankment materials, or lateral deformation of foundation materials. Like uncontrolled seepage, settlement of dikes can result in failure of the dike, but more likely will serve to precipitate failure by another mode such as seepage or shear failure. Distress from settlement usually takes some time to develop as consolidation, shrinkage, and lateral deformation are time dependent, directly related to the soil permeability and loading. Some lateral deformation can occur quickly, however, such as during construction (particularly in relation to the displacement method of construction). Settlement problems in dikes are almost always related to fine-grained soil because settlement of coarse-grained permeable soil is generally much less, occurs relatively quickly, and is compensated for during construction.

115. Specific forms of settlement that commonly cause problems with dikes include: (a) excessive uniform settlement, (b) differential settlement, (c) shrinkage of noncompacted embankment materials, and

(d) settlement resulting from lateral deformation (sometimes referred to as creep) of soft foundation soils. Excessive uniform settlement can cause a loss in containment area capacity due to loss of dike height (Figure 24). Differential settlement can result in cracking of the

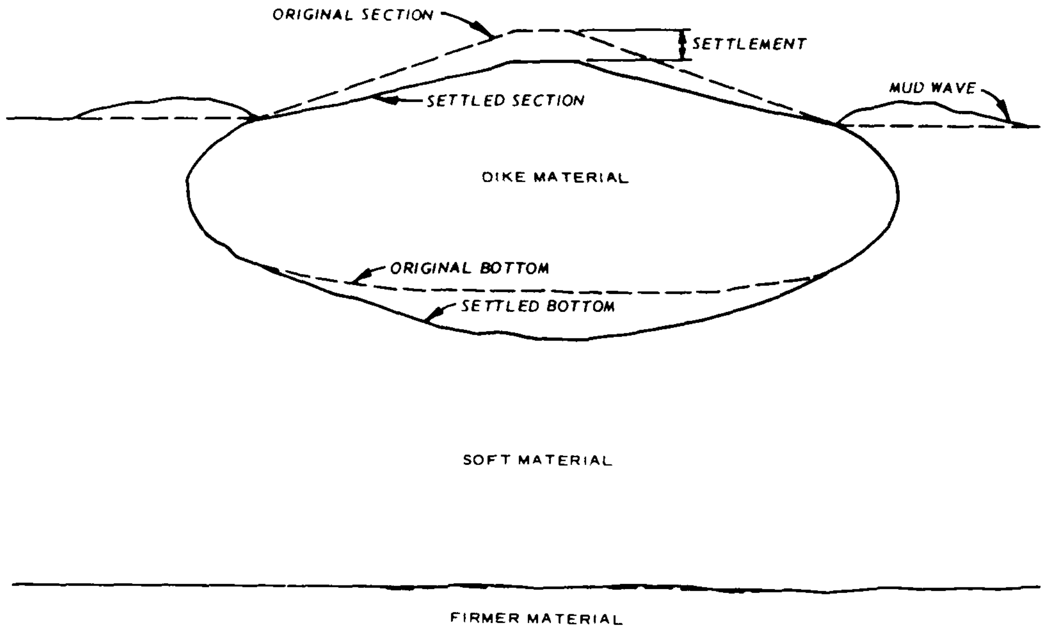
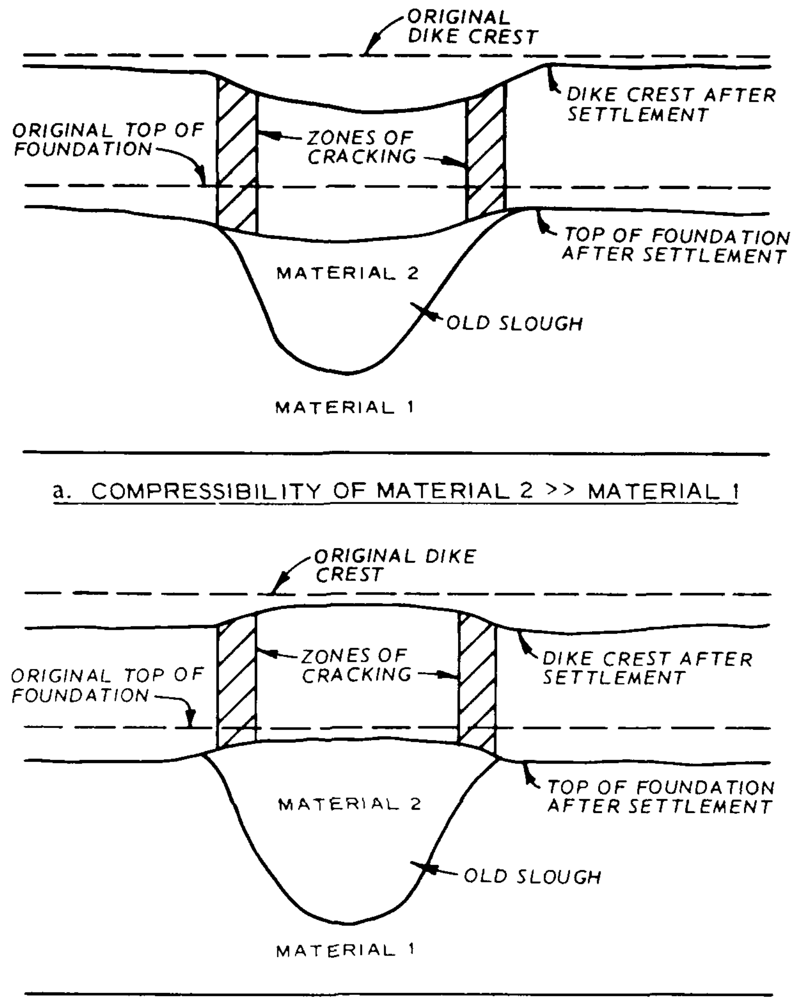


Figure 24. Example of excessive uniform settlement

dike, which can lead to a shear or piping failure. This is an especially acute problem at junctions between dikes and structures in dikes. Differential settlement is caused by the foundation being subjected to varying loads over a relatively short distance (as in the case of a structure within a dike), or by a foundation consisting of materials of different compressibility, usually of varying thicknesses (as in the case of a foundation containing an old slough filled with soft compressible material or noncompressible material). Examples of differential settlement resulting from these different causes are shown in Figures 25 and 26. Both excessive uniform and differential settlement can cause distortion and/or rupture of weir discharge pipes located under or through dikes and can cause distortion of the weir box itself. Embankment shrinkage in dikes built with fine-grained cohesive material



a. COMPRESSIBILITY OF MATERIAL 2 >> MATERIAL 1

b. COMPRESSIBILITY OF MATERIAL 2 << MATERIAL 1

Figure 25. Differential settlement from foundation containing materials of different compressibility

by hydraulic or cast methods can result in volume reductions as high as 35 percent.* Shrinkage of loosely placed cohesive materials is differentiated from consolidation in that it occurs from evaporation of water in the soil rather than a squeezing out of water, as occurs with consolidation, although both result in a loss of volume.

* Unpublished report by New Orleans District.

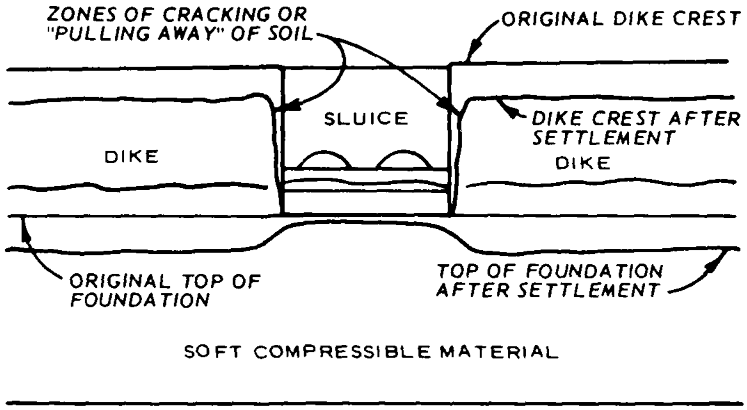


Figure 26. Cracking at dike-structure junction caused by differential settlement due to dike load on foundation being much greater than sluice load

Erosion

116. Retaining dike failures can be initiated by the effects of wind, rain, waves, and currents that can cause deterioration of interior and exterior dike slopes. The exterior slopes of dikes subject to constant or intermittent wave and/or current action of tidal or flood waters are generally exposed to the most severe erosion. However, interior dike slopes may also be subjected to this type of erosion, particularly in large confinement areas during periods of high discharges from disposal operations. Dikes adjacent to navigable rivers and harbors are also subject to erosion from wake waves of passing vessels.

117. Weathering. Erosion of dike slopes due to the effects of wind, rain, and ice is a continuing process. While these forces are not as immediately damaging as wave and currect action, they can gradually cause extensive damage to the dike section, particularly dikes composed of coarse-grained cohesionless materials.

118. Disposal operations. Normal disposal operations can cause erosion of interior dike slopes from pipeline discharge and to exterior slopes at outlet structures. Improper and/or poorly supervised operations of this type can cause dike failure. The pipeline discharge of dredged material is a powerful eroding agent, particularly if the flow

is not dispersed. When straight discharge is employed, a depression as shown in Figure 27 is formed at the point of impact, which, as it enlarges, can undermine the pipe foundation and, if too close to the dike, deteriorate the section. Discharge from weir and spillway outlets can damage exterior dike slopes if the discharge is located too close to the dike (Figure 28). Likewise, location of weir inlets too close to the

Figure 27. Depression at discharge point formed by impact of pumped material

Figure 28. Erosion from outfall discharge. (Note loss of one section of pipe)

dike can cause erosion of interior dike slopes. Also, disposal areas are occasionally negligently overfilled to the point of overtopping the dike. When this occurs, severe damage to the dike can result from erosion of the crest and exterior slopes. Figure 29 shows damage to dike crest caused by overtopping.

Figure 29. Damage to dike caused by overtopping

Slope Stability

119. The stability of dike slopes is dependent on forces acting on the dike and on shear strengths of embankment and foundation materials. Forces that the slope must resist include those from embankment weight, unbalanced water pressure, seeping water, and external loads such as equipment, water, etc. As previously discussed, there are many other factors that can affect dike stability with respect to a shear failure. The purpose of this section, however, is to present methods of slope stability analyses along with discussions of various possible loading conditions, determination of design shear strengths, and recommended minimum factors of safety. Also discussed are methods

of improving dike stability against shear failure. The methods and procedures described are applicable to all types of dikes such as main, cross, spur, and toe dikes, as well as dikes built by different methods of construction (dragline, hydraulic, compacted, etc.).

Methods of analysis

120. The principal methods used to analyze dike embankments for stability with respect to shear failure are conventional limit equilibrium analyses that assume either a sliding surface having the shape of a circular arc or a composite failure surface composed of a long horizontal plane connecting with diagonal plane surfaces up through the embankment and foundation. These analyses simulate the types of shear failures shown in Figures 17 through 20 and are commonly referred to as the circular arc and wedge methods. Various computer programs are available to perform these analyses; therefore, the effort of making such analyses is greatly reduced and primary attention can be devoted to defining shear strengths, unit weights, geometry, and loading conditions. It is recommended that results of all computer analyses yielding minimum factors of safety be checked manually.

121. Circular arc. There are several methods of analyses currently available that utilize a circular arc failure surface. A summary of these is given by Johnson.[19] For dike design the ordinary Swedish Method, presented in many textbooks, and the Modified Swedish Method, presented in EM 1110-2-1902, 1 April 1970,[14] are considered adequate. Analyses utilizing a circular arc failure surface are primarily applicable to homogeneous foundation and embankment materials and can be applied within the embankment only or through both the embankment and foundation. Examples of circular arc analyses are given in Appendix A.

122. Wedge method. The wedge method of analysis is appropriate for foundations containing one or more weak strata or for a condition that assumes a weak layer at the dike-foundation contact. All dikes placed on stratified foundations or foundations having known planes of weakness should be analyzed using the wedge method. Procedures for performing a wedge analysis are given in EM 1110-2-1902.[14] Also,

another simpler and more expedient wedge type of analysis considered appropriate for use in dike design is referred to as the LMVD Method of Planes and is described in Appendix A. Examples of the use of the wedge method are also given in Appendix A.

123. Minimum factor of safety. In performing stability analyses by the circular arc or wedge method, it must be ensured that the minimum factor of safety is found. In a circular arc analysis this ususally involves varying the center and radius of the circle until a minimum is found. For an analysis by the wedge method, the location of the active and passive wedges must be varied along with the depth of the failure plane.

124. Infinite slope method. For dikes composed of cohesionless soil without seepage, the factor of safety FS with respect to sliding is independent of slope height and is given by:

$$FS = \frac{\tan \phi}{\tan \beta} \tag{1}$$

where

ϕ = angle of internal friction of the soil

β = slope angle

For a dike composed of cohesionless material subjected to a condition of steady seepage with the phreatic surface coincident with the outer slope, the factor of safety can be approximated by:

$$FS = \frac{\tan \phi/2}{\tan \beta} \tag{2}$$

Examples of analyses using the above equations are given in Appendix A.

125. Slope stability charts. Graphical solutions to certain slope stability problems are presented by Taylor. Although these solutions are applicable only to simple homogeneous embankments with finite slopes, they may also be used for rough approximations and preliminary solutions to more complex cases.

126. Bearing capacity. A quick assessment of the stability of dikes on soft clay without the use of more sophisticated stability

analyses can be made by employing bearing capacity equations and an influence chart. Although approximate, this analysis can provide answers suitable for preliminary estimates of embankment heights.

127. The bearing capacity analysis assumes a general shear failure from the weight of the dike only and utilizes equations from theory of plasticity for the bearing capacity of $\phi = 0$ materials in undrained conditions. In order to apply these equations to dike embankments, the embankments must be assumed to be shallow continuous footings of infinite extent. The ultimate undrained bearing capacity of a clay q_d loaded as previously described is given by:

$$q_d = 5.14c \text{ for a smooth base} \tag{3}$$

and

$$q_d = 5.7c \text{ for a rough base} \tag{4}$$

where c = soil cohesion. Since, in reality, the base of an embankment is neither entirely smooth nor rough, but is probably nearer rough than smooth, the following equation is recommended:

$$q_d = 5.5c \tag{5}$$

128. The soil pressure at the embankment base or unit load q is given by:

$$q = \gamma H \tag{6}$$

where

γ = unit weight of embankment material

H = embankment height

The soil pressure q_z at some depth z below the embankment base resulting from a unit load q can be obtained from an influence chart such as the one shown in Appendix A (Plate A12). The stability of an embankment of height H can then be checked by comparing the ultimate bearing capacity q_d and the corresponding soil pressure q_z : if $q_d > q_z$, embankment is likely to be stable; and if $q_d < q_z$, embankment stability is questionable.

129. To compute the maximum height H to which an embankment can be built, merely solve Equation 6 for H and substitute the ultimate bearing capacity q_d for the soil pressure q at the embankment base:

$$H = \frac{q}{\gamma} = \frac{q_d}{\gamma} = 5.5 \frac{c}{\gamma} \qquad (7)$$

Note that by use of ultimate bearing capacity, a factor of safety of one is assumed. Also, the use of Equation 7 assumes failure at the embankment base. Its application to deeper strata may therefore be conservative since in doing so the assumption would be made that the full embankment load is transmitted to the deeper strata. Examples of the above analysis are given in Appendix A.

Conditions of analysis

130. There are three primary conditions for which dikes can be analyzed with respect to slope stability: end of construction, steady seepage, and sudden drawdown. End of construction and steady seepage are the most commonly analyzed conditions with sudden drawdown being applicable to a lesser degree. It should be emphasized that the conditions for which any dike is analyzed must be those expected to occur under operating conditions, recognizing there may very well be variations from the aforementioned conditions that may be most applicable. In any case, it is imperative that the conditions analyzed be those that most nearly match actual field conditions. In other words, considerable judgment must be exercised in determining the most applicable conditions of loading to which a given dike will be subjected. The following paragraphs contain a discussion of each of the conditions mentioned. Appropriate shear strengths and recommended minimum factors of safety for each condition are discussed in subsequent paragraphs in this part.

131. End of construction. For most dikes constructed on foundations of soft weak materials or on foundations containing a weak stratum

in an otherwise strong foundation, the most critical period involving failure due to inadequate shear strength is at the end of construction. This is because at this time the material is usually in its weakest state, not having had time to consolidate and gain strength under the imposed loading conditions. Consequently, all dikes should be checked for stability during the end of construction condition.

132. Analysis for the end of construction conditions is applicable to both interior and exterior slopes. The effects of underseepage and resulting hydrostatic uplift pressure acting in pervious foundation strata must be considered. An example of an end of construction analysis is contained in Appendix A.

133. Steady seepage. A condition of steady seepage through the dike resulting from the maximum anticipated storage level in the containment area may be critical for stability of exterior dike slopes. A sketch depicting this condition is shown in Figure 30 and an example analysis is contained in Appendix A.

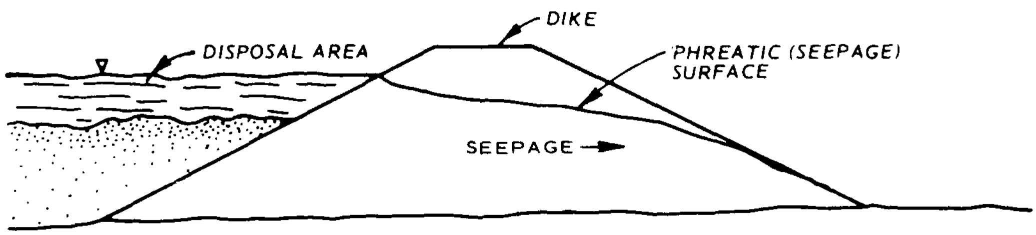
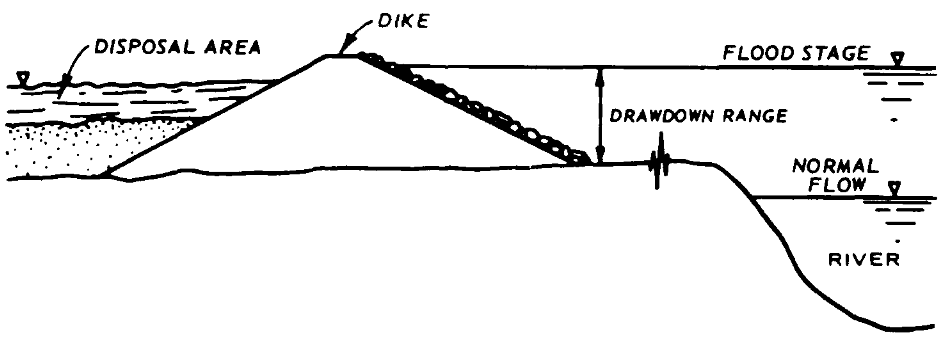


Figure 30. Dike subjected to steady-state seepage condition

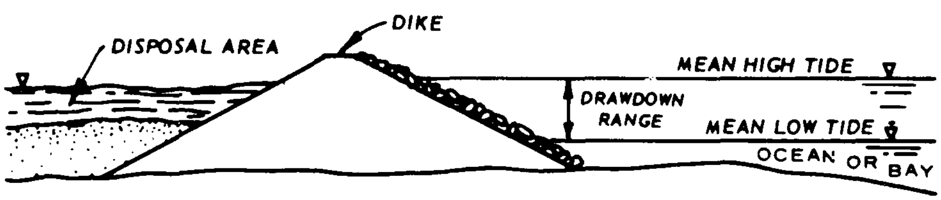
134. All dikes should be analyzed for this condition if it is anticipated that saturation of the embankment will occur and a condition of steady seepage will develop within the dike and/or foundation. This condition is especially applicable to dikes composed of semipervious and pervious materials but should also be considered for dikes composed of any material. This is because it is very important that the dike be stable against failure resulting from steady seepage conditions since failure from this cause generally occurs with a considerable depth of dredged material in the disposal area and could, therefore, result in considerable damage due to the loss of a high volume of dredged material.

135. Sudden drawdown. Exterior dike slopes may become saturated during high water levels from adjacent streams or from high tides. If the water level then falls faster than the material can drain, excess pore water pressures and unbalanced seepage forces result. In performing an analysis for this condition, it is generally assumed that the water level drops instantaneously so that no pore pressure dissipation occurs. An example of this type of analysis is given in Appendix A.

136. The sudden drawdown condition is applicable to those dikes situated near large bodies of water or streams whose level may reach near the dike crest, remain there long enough to saturate the dike, and then fall fairly rapidly. It may also be applicable to dikes subject to the effects of substantial tidal fluctuations (Figure 31). Failure from sudden drawdown will usually be in the form of relatively shallow sloughing of the affected slope and thus is not considered as critical as failure from the end of construction or steady seepage conditions



a. DIKE SUBJECTED TO FLOODING FROM ADJACENT RIVER



b. DIKE SUBJECTED TO TIDAL FLUCTUATIONS

Figure 31. Situations conducive to a sudden drawdown condition

where an entire dike section may be lost. Loss of slope protection and a weakening of the dike is the usual consequence of failure from sudden drawdown. There are no recorded dike failures from sudden drawdown, but large dikes, especially those with substantial slope protection, subjected to the conditions previously described should be analyzed for the effects of sudden drawdown.

Section for analysis

137. Generally speaking, selected dike sections for analysis should be typical of as long a reach of dike as possible. It is often possible to group reaches of dikes according to like foundation conditions and then analyze the highest dike section in the reach for stability and consider its design as typical for the entire reach. However, if stability is very sensitive with respect to dike height, these reaches may have to be subdivided into reaches with smaller variations in height. This may also be necessary if there exists substantial variation in dike height along a reach of similar foundation conditions.

138. It must be emphasized, even at the risk of sounding obvious, that geometric factors as well as soil characteristics must be considered in making dike stability analyses. For example, if dikes are fairly close to streambanks, channels, canals, old sloughs, borrow excavations, etc., the most critical potential sliding surface may very well be on or at the slopes of such features rather than near the dike toe (Figure 32). Such sections should certainly be checked for stability. Also, old in-filled sloughs or streams crossing the dike alignment form critical areas with respect to slope stability and should be analyzed for such. The actual location of the dike with respect to natural or man-made depressions previously mentioned should be determined by stability analyses as should the distance of the dike from any planned excavation (Figure 32). The consequence of constructing a dike too near a stream crossing is shown in Figure 33. Figure 33a shows the end dike after construction and dredged material being placed. Figure 33b shows the end dike section after failure into the stream.

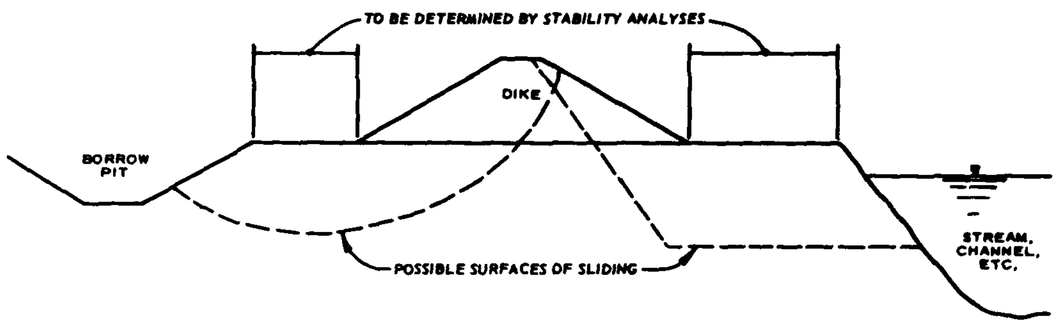


Figure 32. Analysis of dike near depressions

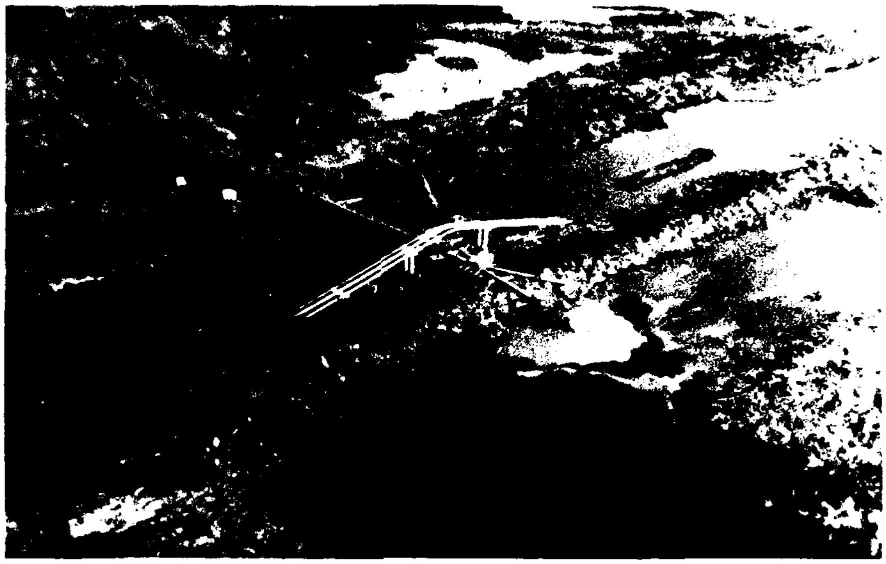
Recommended minimum
factors of safety

139. Recommended minimum factors of safety for slope stability analyses of retaining dikes are given in Table 12. These values are to be used where reliable subsurface data from exploration and testing are available for input into the stability analysis. The factors of safety given in Table 12 are applicable to dikes less than 30 ft in height where the consequences of failure are not extremely severe. For dikes greater than 30 ft in height and where the consequences of failure are severe, the criteria given in Table 1 of EM 1110-2-1902[14] should be used.

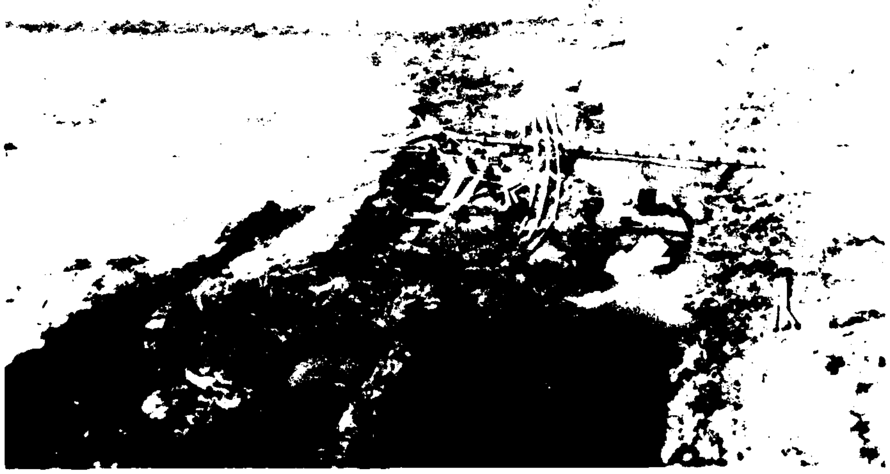
Selection of
design shear strengths

140. In the past, soil strengths for dike design have largely been assumed. However, as the need for more sophisticated analyses and design increases, it is imperative that shear strengths be determined from reliable test data whenever possible. This by no means rules out the use of experience. Experience with respect to shear strengths should continue to play a vital role in dike design, but as a supplementary rather than a primary means of shear strength determination.

141. Appropriate laboratory and field tests for the determination of shear strengths have previously been discussed as well as the importance of the selection of the most representative strength values for



a. Dike constructed and dredged material being placed in disposal area (right foreground)



b. Failure of dike at left into channel (view from opposite direction of photo above). Note deformation of utility crossings

Figure 33. Consequences of constructing a dike too close to strecmbank

Table 12

Applicable Shear Strengths and Recommended

Minimum Factors of Safety*

Condition	Shear Strength: Impervious Soils**	Shear Strength: Free-Draining Soils	Slope Analyzed	Minimum Factor of Safety†: Main Dikes	Minimum Factor of Safety†: Appurtenant Dikes
End of construction	Q	S	Exterior and interior	1.3‡	1.3
Steady seepage	Q, R††	S	Exterior	1.3	1.2
Sudden drawdown	Q, R††	S	Exterior	1.0	NA

* Criteria not applicable to dikes greater than 30 ft in height or where the consequences of failure are very severe. For such dikes use criteria given in Table 1 of EM 1110-2-1902.[14]

** For low plasticity silt where consolidation is expected to occur rather quickly, the R strength may be used in lieu of the Q strength; see paragraphs 70 and 71.

† To be applied where reliable subsurface data from exploration and testing are available; where assumed values are used, recommended minimum factors of safety should be increased by a minimum of 0.1.

†† Use Q strength where it is anticipated loading condition will occur prior to any significant consolidation taking place; otherwise use R strength.

‡ Use 1.5 where considerable lateral deformation of foundation is expected to occur (usually where foundations consist of soft, high plasticity clay).

use in stability analyses (see Part IV). The appropriate strengths applicable for each condition of analysis are given in Table 12. The following paragraphs contain a general discussion of the selection of strengths for embankment and foundation materials.

142. Coarse-grained cohesionless soil. Most coarse-grained cohesionless soils are considered to be free-draining (i.e., pore pressures are dissipated as fast as loading occurs so that no excess pore pressures develop during shear). Strengths from the S-test are therefore most appropriate for these soils for all conditions of loading. Generally, it is conservative to use a shear strength of ϕ equals 30 deg and c equals 0 for these materials whether in the embankment or foundation. However, higher strengths should be based on triaxial or direct shear S-tests.

143. Fine-grained soil.

a. Embankment materials. The strengths of dike materials will have little effect on stability analyses for dikes founded on relatively deep deposits of soft clay where the critical depth of failure will also be correspondingly deep. However, as the critical depth of failure becomes less, the shear strength of the dike material will have a greater effect on the calculated factor of safety.

(1) Compacted and semicompacted fill. Representative samples of proposed fine-grained material intended for use in a compacted fill should be compacted (usually by the 25-blow standard Proctor test) within a range of expected water contents, and the appropriate strength tests performed on specimens trimmed therefrom. Samples for semicompacted fill should be subjected to the 15-blow compaction test. These samples should be tested at the anticipated natural water content as water content control is rarely exercised for semicompacted fill. Appropriate strength tests, as given in Table 12, should then be performed on these compacted samples.

(2) Uncompacted fill (other than hydraulic). The determination of shear strengths for fine-grained fill cast or dumped without regard to water content and receiving little or no compaction is very difficult as it is virtually impossible to produce a representative laboratory sample for testing. Design strengths for fine-grained materials placed in this

manner should be based on back-figured strengths from existing fills placed in a similar manner or from strength tests on samples from similar fills. When testing samples from existing fills, it must be kept in mind that some gain in strength (depending on how long the fill has been in place) has occurred and the strengths obtained reduced appropriately, especially for Q-strengths to apply to the end of construction condition.

(3) Hydraulic fill. Clay and silt deposited as slurries will have very low initial shear strengths. Unless substantiating data are available, shear strengths of about ϕ equals 0 and c equals 50 to 100 psf should be used for these materials. The strength of clay deposited as clay balls will vary greatly depending on in situ strength, size of the clay balls, type of material in the interstices, and time. An analysis of data from the New Orleans District indicates the initial strength of hydraulic fill deposited as clay balls from the excavation of Recent soft to medium consistency clay averaged about 25 to 30 percent of the in situ strength. The shear strength of similar clay ball fills derived from stiffer Pleistocene clay may vary from 40 to 50 percent of the in situ strength.

(4) Dredged material. The strength of fine-grained dredged material behind a dike should be assumed to be negligible (i.e., zero) unless test data can substantiate a definite strength.

b. Foundation. The shear strength of the dike foundation is generally the most important factor in dike stability, especially where dikes are built on soft foundations, which is the rule rather than the exception for most projects. It is, therefore, essential that the condition of the foundation be defined as accurately as possible so that appropriate foundation strengths can be selected for the stability analysis. Details of foundation exploration and laboratory testing have previously been discussed in Parts III and IV.

For fine-grained cohesive materials, the undrained-unconsolidated or Q-strenth of foundation soils is appropriate for most analyses (except for some silt - see paragraphs 70 and 71). For clay deposits, a plot of the unconsolidated-undrained shear strength versus depth as shown in Figure 34 should be developed to aid in the selection of design strengths and to help locate critical depths of failure. For most coastal area soil and other soft clay deposits, a desiccated zone exists near the

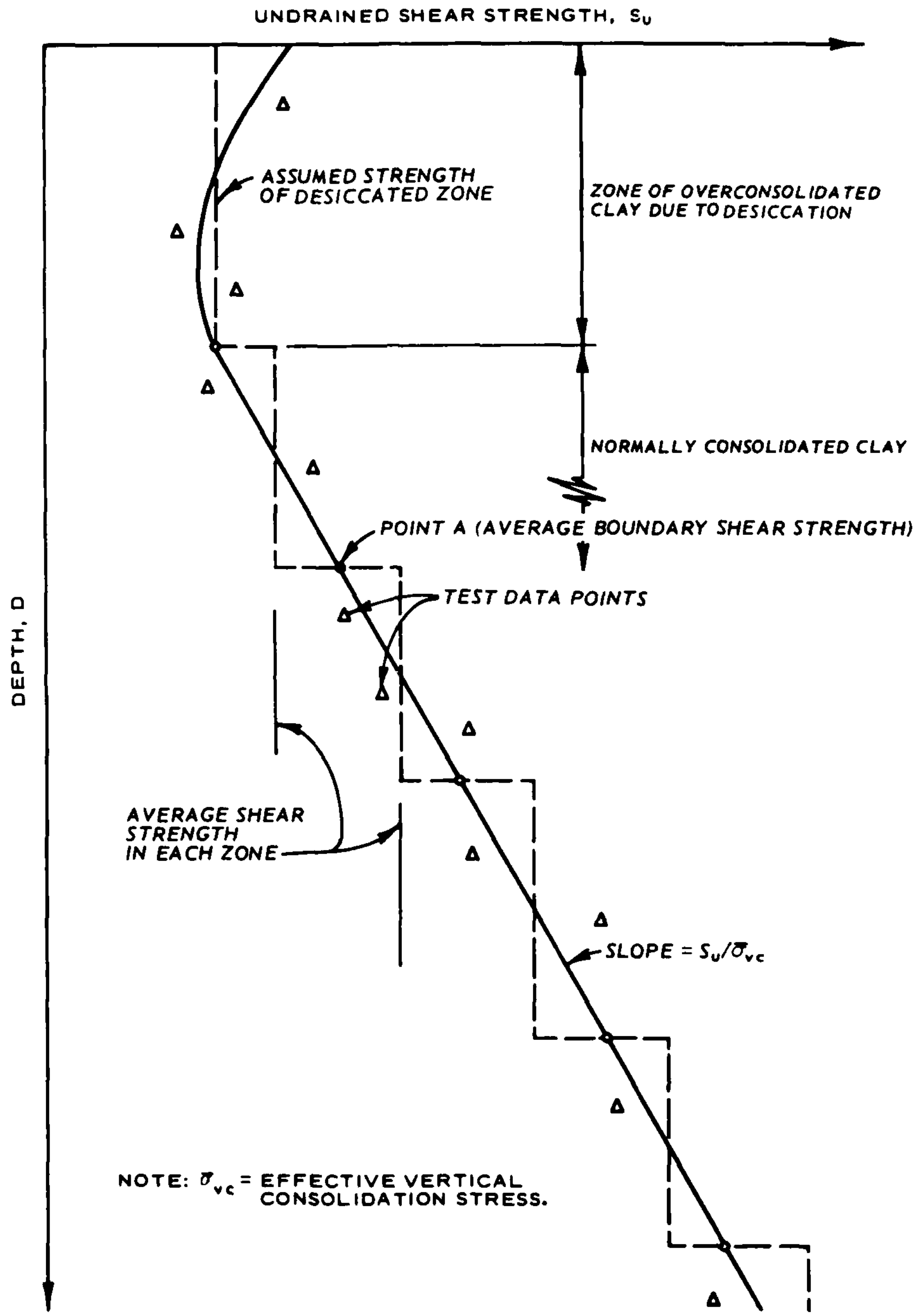


Figure 34. Example of plot of unconsolidated-undrained shear strength versus depth

surface. This zone will vary in depth and will generally exhibit higher strengths and lower water contents than the underlying normally consolidated clay.

(1) Strength increase with depth. The strength of normally consolidated clay increases with depth, usually at some constant rate. This rate of strength increase with effective overburden stress (depth) $\bar{p}$ in the normally consolidated range is referred to as the $S_u/\bar{\sigma}_{vc}$ or $c/\bar{p}$ ratio and for various normally consolidated clay deposits has been found to vary from about 0.18 to 0.29. An average value of 0.24 has been found to correlate well with laboratory test data in the New Orleans District (Figure 35).

When applicable, the $S_u/\bar{\sigma}_{vc}$ ratio should be used to help establish the shear strength with depth profile in the foundation as indicated in Figures 34 and 35. The use of this technique will assist in interpreting the validity and accuracy of laboratory and/or field test data and will help increase one's confidence when interpolating data for similar soil conditions between borings. As a general rule of thumb, the increase in strength with depth for normally consolidated clays below the water table can be assumed to be 10 psf/ft of depth.

To facilitate the design stability analysis, it is helpful to simulate the linear variation in shear strength with depth by the stepped profile as shown by the dashed line in Figure 34. The bottom of each zone is chosen to correspond to various depths of failure to be investigated in establishing the minimum factor of safety. In using this type of strength profile, the wedge method is the most applicable method of analysis. The average values of shear strength along the central block failure surface, for depths of failure at the bottom of each zone, should be the average of the overlying and underlying zones as shown by Point A in Figure 34. When the failure plane is assumed at the base of the desiccated upper zone, the shear strength of the desiccated zone should be used along the central block base. The average strength of each zone should be used along all inclined active and passive failure surfaces cutting through the respective zone.

(2) Estimating strength increase with time. When loaded, foundation clay gains shear strength with time. As previously discussed, this fact is extremely important when an existing dike is to be raised or stage

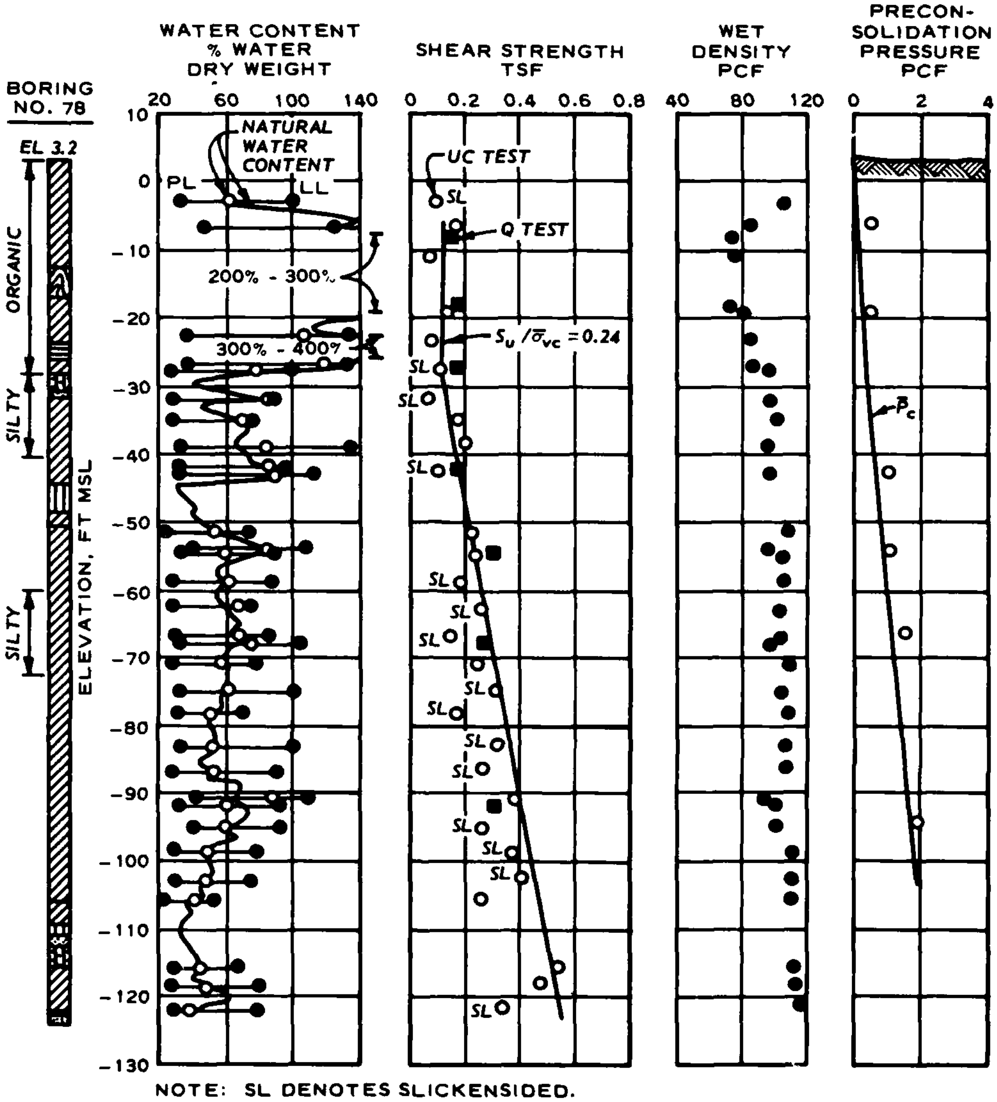


Figure 35. Typical soil test data plotted versus depth

construction is to be employed for new dike construction. Normalized shear strength parameters as set forth by Ladd and Foott[16] can be used to estimate the increase in undrained shear strength with increased effective vertical stress. This procedure involves a knowledge of the initial Q shear strength, the value of $S_u/\bar{\sigma}_{vc}$ $(c/\bar{p})$, and the degree of consolidation. The increase in strength of the clay ΔS_u is given in Equation 8.

$$\Delta S_u = (S_u/\bar{\sigma}_{vc})\Delta\bar{\sigma}_{vc} \quad (8)$$

where $\Delta\bar{\sigma}_{vc}$ = increased vertical consolidation stress

The resulting strength increase should then be added to the initial strength at that depth to obtain the estimated total undrained shear strength. Since this gain in strength is time dependent, the variation in consolidation that has occurred throughout the depth of the layer must be considered because the layer will not consolidate uniformly to the same degree throughout its entire thickness. For instance, suppose the average degree of consolidation of a clay layer is calculated to be 20 percent. In reality, the majority of the consolidation will likely have taken place at the top and bottom of the layer (assuming double drainage), with essentially no consolidation at the center. For this case there would be no increase in effective stress and hence no strength gain at this depth.

Strength studies on clay-ball hydraulic fill* as shown in Figure 36 indicate that considerable strength gain can occur with time. The data shown in Figure 36 are average undrained strength values over a 6- to 8-ft depth of clay balls hydraulically dredged from soft to medium consistency Recent clays.

It should be emphasized that the preceding procedures and data are only for estimating the increase in strength with time and should only be used for preliminary design. Final designs relying on increased shear strengths due to prior loadings should be based on additional borings and laboratory tests made prior to adding the second stage of fill.

Methods of improving foundation stability

144. The condition of a dike foundation can be the decisive factor in determining the feasibility of constructing a retaining dike. Since suitable areas for disposal of dredged material are usually limited, retaining dikes must be so aligned as to make optimum use of the disposal area, often without regard to foundation conditions. Thus, dike foundations must often be improved in order that the dike may be built.

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* Unpublished report by New Orleans District.

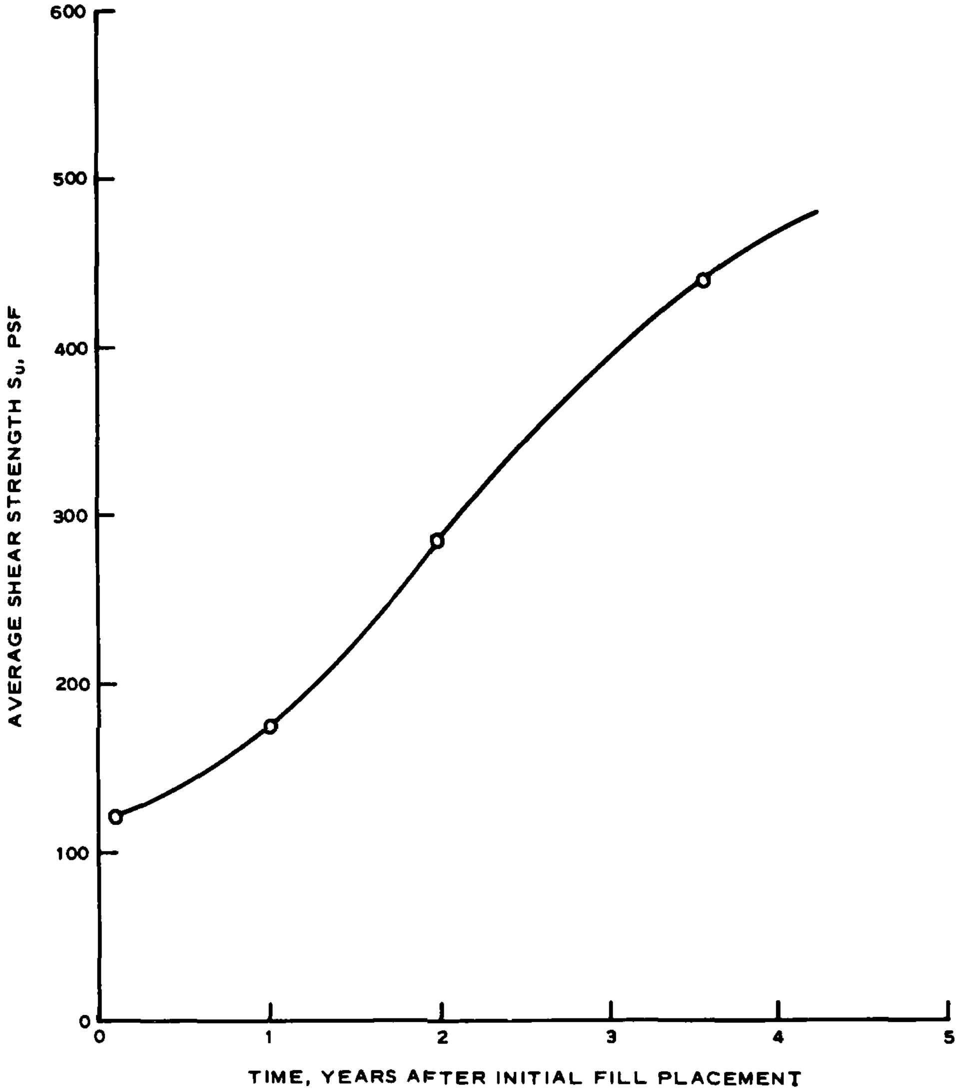


Figure 36. Shear strength increase with time for hydraulically placed clay fill

Economically feasible methods of improving dike foundations are limited, but it should be recognized that the economic justification of a given method is not an absolute value but is directly related to the particular project.

145. Soils that require treatment cannot be identified solely on the basis of their physical characteristics since the need for treatment

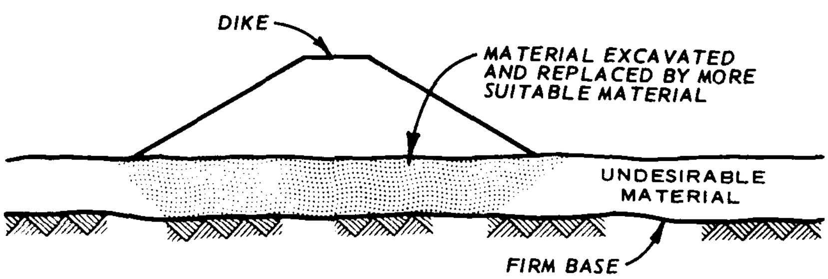
depends largely on imposed loading conditions, i.e. the same foundation may be perfectly stable under one loading but inadequate under another. However, foundation deposits that are prone to cause problems may be broadly classified as follows: (a) very soft clay, (b) sensitive clay, (c) loose sand, (d) natural organic deposits, and (e) man-made organic deposits.

146. Very soft clay is susceptible to shear faulure and excessive settlement. Sensitive clay is brittle and, even though possessing considerable strength in the undisturbed state, is subject to partial or complete loss of strength upon disturbance. Fortunately, extremely sensitive clay is rare in the United States. Loose sand is also sensitive to disturbance and may liquefy and flow when subjected to shock or even shear strains caused by erosion at the toe of slopes. Most organic soils are very compressible and exhibit low shear strength. The physical characteristics of natural organic deposits such as peat can sometimes be predicted with some degree of accuracy. Highly fibrous organic soils with water contents of 500 percent or more generally consolidate and gain strength rapidly. The behavior of organic debris deposited by man, such as industrial and urban refuse, is so varied in character that its physical behavior is difficult, if not impossible, to predict.

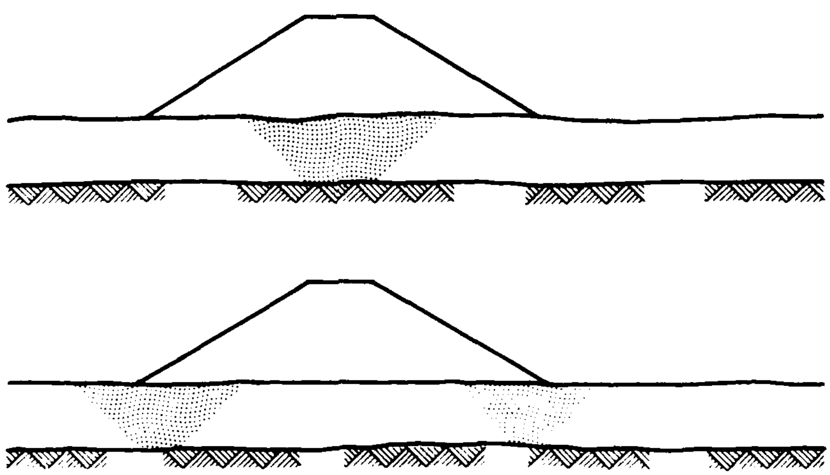
147. The following paragraphs discuss methods of dealing with foundations that are inadequate from the standpoint of available shear strength for construction of proposed dikes. These methods are excavating and replacing poor materials, displacing undesirable material by end-dumping fill material, constructing the dike in stages to permit consolidation of the foundation, densifying loose sand, flattening embankment slopes, and constructing stability berms.

148. Excavation and replacement. The most positive method of dealing with excessively weak and/or compressible foundation soils is to remove them and backfill the excavation with more suitable material. This procedure is usually feasible only where deposits of unsuitable material are not excessively deep (i.e. up to about 20 ft in thickness), where suitable backfill material is available, and where a firm base exists upon which to found the backfill. The excavation and

replacement can be accomplished by any practical means, but for most dikes in areas of high water tables (i.e. marshes, tidal flats, etc.) excavation is best accomplished with dredges, matted draglines, and barge-mounted draglines. Where backfilling is to be accomplished in the wet, only coarse-grained material should be considered for use as backfill. The amount of excavation need not always be under the entire section or to full depth of soft material, but can be partial if determined by stability analyses to be appropriate. Some sections successfully used in the past to prevent horizontal sliding of the embankment are shown in Figure 37. Excavation and replacement should be considered wherever possible.



a. COMPLETE EXCAVATION AND REPLACEMENT



b. PARTIAL EXCAVATION AND REPLACEMENT

Figure 37. Typical use of excavation and replacement method to improve stability

149. <u>Displacement of undesirable material by end-dumping fill.</u> Dikes must frequently be built over areas consisting of very soft materials. Although the depths of these deposits may not be great, the cost of their removal may not be justified and a dike having adequate stability can be constructed by end-dumping fill and utilizing it's weight to displace the undesirable material.

150. It is desirable to use this method where a firm bottom exists at a reasonably shallow depth; it has, however, been successfully employed in areas where no definite firm bottom existed, but the displaced material merely increased in strength with depth, in which case the depth of displacement is considered to be that necessary to stabilize the embankment at the desired height (Figure 38). However, use of the

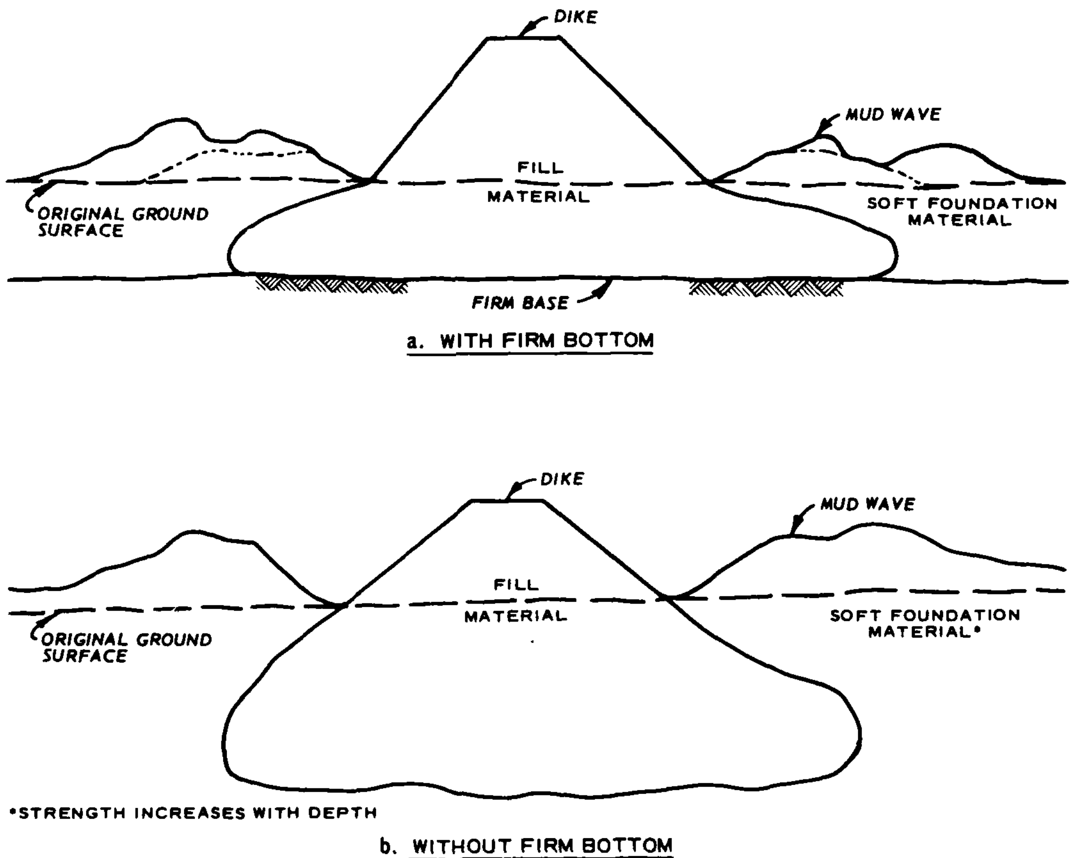


Figure 38. Final dike sections after displacement of soft foundation material

displacement method in the latter case does increase the likelihood of post-construction settlement.

151. Due to the construction techniques required to successfully use this method, it is highly desirable to place fill by end-dumping methods rather than by hydraulic means. It is also desirable that the material to be displaced exhibit some sensitivity and have average in situ shear strength of less than about 150 to 200 psf. The greater the sensitivity of the material and the lower its in situ strength, the easier it is to displace.

152. Basically, the displacement technique consists of advancing the fill along the desired alignment by end-dumping and pushing fill over onto the soft material with dozers, thus continually building up the fill until its weight displaces the foundation soils to the sides and in front of the fill (Figure 39). By continuing this operation, the dike can finally be brought to grade. Since this method involves the encouragement of foundation displacement, the section should be as steep sloped as possible and built as high as possible as it advances across the foundation. The fill should be advanced with a V-shaped leading edge so that the center of the fill is always the most advanced, thereby displacing the soft material to both sides (Figure 40). This will greatly lessen the chances of trapping soft material beneath the fill. A wave of displaced material will develop (usually visible as is evidenced by the photograph in Figure 41) along the sides of the fill. These mud waves have been known to be as high as the top of fill; however, they should not be removed.

153. A disadvantage of this method is that all the soft material may not be displaced, which could result in slides as the embankment is raised and/or differential settlement after construction. Another disadvantage is that final in-place quantities are difficult to determine due to an appreciable amount of fill material being below the ground surface. It is therefore recommended that quantities be based on excavated yardage or provisions be made to take borings after construction or, where the displacement is not too great, settlement plates be installed beneath the proposed alignment prior to construction. All of

a. Coarse-grained fill

b. Fine-grained fill

Figure 39. Shoving fill onto soft foundation with dozers

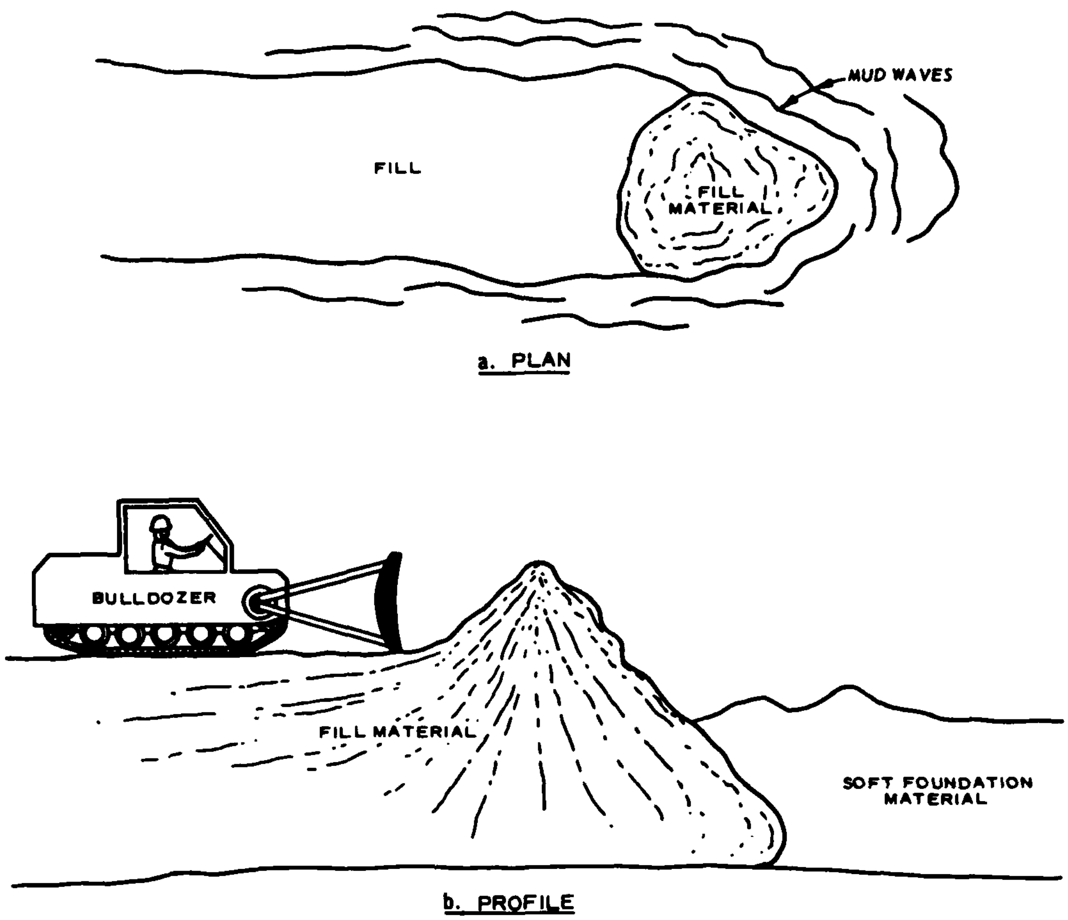


Figure 40. Advancement of fill using end-dumping and displacement technique

the above techniques for determining pay quantities have been successfully employed for displacement construction in the past.

154. If a surface root mat or a desiccated layer exists immediately over the soft material to be displaced, it should be broken up prior to fill placement. Since this type of construction produces essentially uncompacted fill, the design of the dike section must take this into account.

155. When this method of foundation treatment is being considered for long reaches of dikes over deep deposits of soft sensitive clays, the possibility of facilitating displacement by blasting methods should be evaluated (see Blasters Handbook[21] for general information on

a. View parallel to dike

b. View perpendicular to dike

Figure 41. Mud waves from displaced material

blasting used to displace soft materials). Generally, the greater the required depth of displacement, the more economical the blasting method becomes.

156. Stage construction. Stage construction refers to the building of an embankment in increments or stages of time. This method of construction is used when the strength of the foundation material is inadequate to support the entire dike if built at one time. Using stage construction, the dike is built to intermediate grades and allowed to rest for a time before placing more fill. Such rest periods permit dissipation of pore water pressures and consolidation that results in a gain in strength so that higher dikes can be supported. Obviously, this method is most appropriate for foundations that consolidate rather rapidly. This procedure works best for clay deposits interspersed with continuous seams of highly pervious silt or sand. However, lack of speed of consolidation may not be a drawback if the filling rate of the disposal area is slow enough to allow considerable time between construction of the various dike stages. In fact, stage construction appears to be a promising method of constructing retaining dikes as the intervals of construction can, in many cases, coincide with the filling of the disposal area; i.e., full dike height may not be needed until many years after initial construction.

157. In using stage construction, estimates of strength gain with time should be made as described in paragraph 143b(2). Also, it is highly desirable to have piezometers available to monitor the dissipation of pore water pressures. Disadvantages of this method include the need for separate construction contracts and uncertainties with respect to the gain in strength with time.

158. Densification of loose sand. In seismically active areas, the possibility of liquefaction of loose sand deposits in dike foundations may have to be considered. Since methods for densifying sands such as vibroflotation, blasting, etc., are costly, they are generally not considered except for dikes where the consequences of failure are very severe or at locations of important structures in the diking system. However, less costly defensive design features may be provided,

such as additional freeboard, wider dike crest, and flatter slopes.

159. Flattened embankment slopes. Flattening embankment slopes will usually increase the stability of an embankment against a shallow foundation failure or a failure that takes place entirely within the embankment. Flattening embankment slopes reduces unbalanced gravity forces that tend to cause failure and increases the length of potential failure surfaces, thus increasing resistance to sliding.

160. Stability berms. Berms provide essentially the same effect as flattening embankment slopes but are generally more effective since they concentrate additional weight where it is needed most and force a substantial increase in the potential failure surface. Thus, berms can be an effective means of stabilization, not only for preventing shallow foundation and embankment failures, but for preventing 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.

161. 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 dike and prevent further movement. The main disadvantages of berms are the increase in area occupied by the embankment and the amount of material required for berm construction.

162. Stabilization prior to and after failure. With the use of proper observational techniques, impending stability failures may be detected and measures taken to improve the stability of the section prior to failure. Lateral movement of slopes, slight sinking of the crest, or heave near the toe, as well as development of tension cracks, can give advance warning of failure. Since most failures begin slowly, early detection and immediate corrective action can often prevent complete failure. Flattening dike slopes and adding berms have often been effective as stop-gap measures for increasing stability.

163. Once failure has occurred in a soft clay foundation, the

process of rebuilding is often more difficult than initial construction because many soft clays are sensitive and their remolded strengths are often much less than their initial shear strengths. It is good practice after a failure to allow time for some consolidation and resulting gain in shear strength before attempting to rebuild. This will give the remolded clay time at least to partially overcome the effects of strength reduction due to remolding. When remedial construction is started, care should be taken not to load the foundation too quickly. Reconstruction should be done as slowly as possible with the entire area brought up together rather than building to full height in sections.

Settlement

164. Problems with dikes caused by settlement of embankment or foundation materials are almost always limited to fine-grained cohesive soil. This is because it can usually be safely assumed that most of the consolidation of pervious or semipervious materials will occur relatively quickly, usually during construction. However, the settlement of fine-grained compressible soil can occur over a period of years; thus a need exists for analyzing conditions where such soil exists and incorporating into the design measures that will minimize problems resulting from settlement. Methods of analysis, applicability of these methods, and preventive measures are discussed in the following paragraphs.

Settlement analyses

165. Where estimates of amount of time and total settlement are needed, a conventional analysis such as that contained in EM 1110-2-1904, "Settlement Analysis,"[22] or in various textbooks on soil mechanics is recommended. NAVFAC DM-7[17] is also recommended for guidance in performing settlement analyses. In order for an estimate of settlement by theoretical means to be valid, the materials analyzed must be fairly uniform and capable of being represented by a laboratory consolidation test, and the drainage conditions must be well defined. Unfortunately, the above conditions are often not satisfied with respect to dike materials or dredged material. This fact is discussed in more detail in subsequent paragraphs.

Uniform settlement

166. For most earth structures on compressible foundations, uniform settlement resulting from consolidation of the foundation can cause a loss of design grade and must be compensated for in the initial design. However, for retaining structures a unique situation exists with respect to the effects of uniform dike settlement: the containment area will also be loaded and should also undergo settlement that may compensate for the dike settlement, resulting in little or no loss in capacity of the retaining area. For dikes on compressible foundations, this fact should be verified, however. This can be done by performing settlement analyses for both the dike foundation and the containment area (using projected filling rates) and comparing the amount and rate of settlement of each. If such an analysis shows a net loss of dike height (as is often the case when a considerable period of time elapses between the time of dike construction and filling of the disposal area), it should be compensated for by overbuilding the dike or by making provisions to raise the dike back to the original design grade at a later date (i.e., use stage construction).

167. Overbuilding. Overbuilding dikes by the amount of anticipated loss of grade due to settlement often appears the easiest and cheapest solution to the problem, but is really not practical in many cases as it can significantly affect stability of the dike against shear failure (i.e., can require large dike sections), as well as cause additional settlement. This is not to say that use of overbuilding to compensate for anticipated settlement should be ruled out, but it should be closely studied before being specified as a compensating procedure.

168. Stage construction. The use of stage construction (i.e., raising dikes as necessary after settlements occur) is somewhat more troublesome and expensive than overbuilding, but is often the only practical solution, especially for dikes on highly compressible foundations where overbuilding can create more problems than it solves, as previously discussed. The use of stage construction to compensate for dike settlements has often been successful in the past on many dike projects.

Embankment consolidation and shrinkage

169. Consolidation and shrinkage of embankment materials will vary considerably, being dependent not only on material type but on method of placement. Generally, methods for theoretical settlement analyses of embankment materials are only applicable to dikes composed of compacted uniform materials (these materials will usually exhibit the least amount of consolidation and shrinkage). The amount of embankment consolidation and shrinkage usually must be estimated.

170. Semicompacted fill. As a general rule, dikes built of semicompacted fill will experience a reduction in volume on the order of 10 to 15 percent. Usually, this small amount of volume decrease can be compensated for by overbuilding.

171. Uncompacted fill. Estimating the reduction in volume of uncompacted fill (i.e., fill placed by casting) is a difficult task as it will depend greatly upon the consistency and water content of the material being placed and the construction procedures used, i.e., the amount of equipment coverage during shaping, etc. Estimates of reduction in volume of uncompacted fill should generally be based on knowledge of the previously mentioned factors and experience with fills built of similar materials and by similar construction procedures. In the absence of any supporting data, a reduction in volume of 15 to 20 percent should be applied for uncompacted fill.

172. Hydraulic fill. The compressibility of hydraulic fill containing stiff cohesive soil results primarily from deformation of the clay lumps, while the rate of consolidation is determined by the characteristics of the matrix surrounding the clay lumps. Hydraulic fills containing soft cohesive soil are highly compressible, but again the rate of consolidation is dependent on the matrix material. Consolidation of cohesive materials with a sandy matrix may be essentially complete within a few weeks, while consolidation of cohesive materials with a clay matrix may continue for years.

173. Results of volume loss from shrinkage tests performed on four samples of cohesive materials obtained from a hydraulic fill berm constructed in 1964 in the Atchafalaya Basin Floodway, Louisiana, are

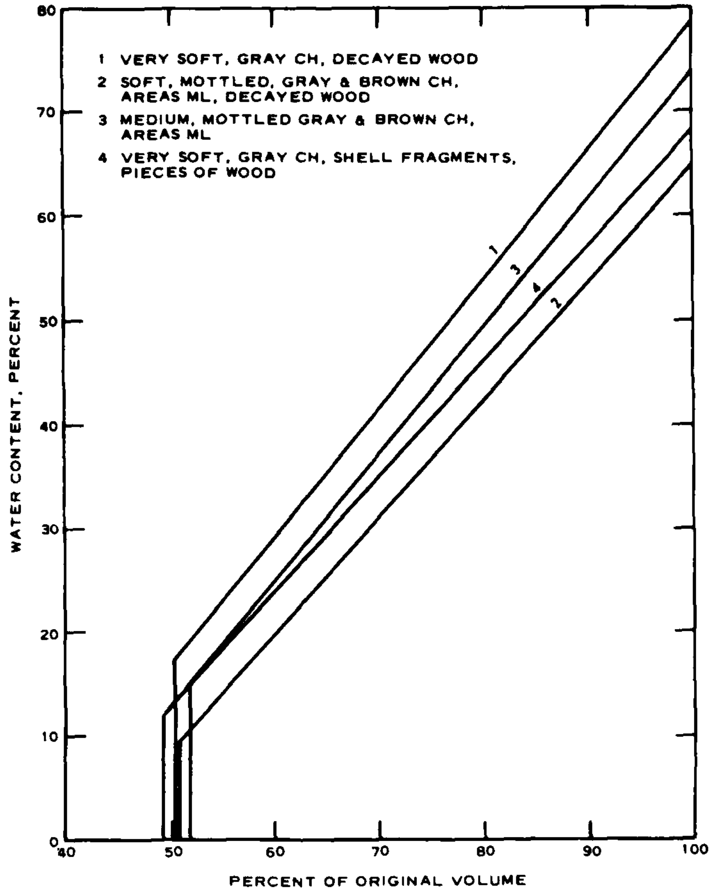


Figure 42. Results of shrinkage tests, hydraulic fill berm, Atchafalaya Basin Floodway, Louisiana

shown in Figure 42. During the first 2 years after placement, the average water contents decreased from 81 to 41 percent in materials having lower values of initial water contents and from 124 to 77 percent in the higher ranges of initial water contents. Based on information presented in Figure 42, which represents material placed within the lower range of water contents, this decrease in water content could result in a 30 to 35 percent decrease in volume. Therefore, volume decrease of hydraulically placed cohesive materials may be very substantial and should be considered in determining the design grade.

Differential settlement

174. The causes and effects of differential settlement have

previously been discussed in paragraph 115. Where the possibility of differential settlement (as shown in Figures 25 and 26) exists, an analysis should be made to determine the total differential settlement across the area under concern. Although there are no specific criteria that set forth how much differential settlement a particular soil can withstand before cracking, measures can be taken to reduce the magnitude of the differential settlement so that the chances of distortion and cracking are lessened. These measures include (a) removing all or part of the compressible material and replacing with more suitable material; (b) using flatter excavation slopes (1V on 4H minimum) where excavations (usually for structures) are involved; and (c) specifying good compaction procedures and more plastic embankment materials adjacent to structures.

Lateral spreading

175. In some cases where extremely poor foundation conditions are encountered, settlement due to lateral movement of foundation materials may also warrant consideration. Experience with instrumented test sections in the Atchafalaya Basin, Louisiana, in the New Orleans District, has shown that more than 30 percent of observed settlement induced by the addition of an 11-ft height of fill was due to lateral movement of foundation materials. This was observed in an area where the foundation consisted of peat and soft organic clay with very high water contents underlain by soft and medium clays of high plasticity and where the sections were constructed with safety factors of about 1.3 against shear failure. Other sections constructed with safety factors of about 1.1 indicated as much as 50 percent of observed settlement was due to lateral movement of foundation materials. These data are presented in Figures 43 and 44. Experience from the Atchafalaya Basin Floodway has shown that overbuilding should not be considered as a solution for lateral spreading as the additional load from overbuilding will generally tend to aggravate the problem rather than help solve it. This same experience has also shown that vertical settlement due to lateral movement will be minimized by designing a section with a higher minimum factor of safety with respect to shear failure (on the order of 1.5).

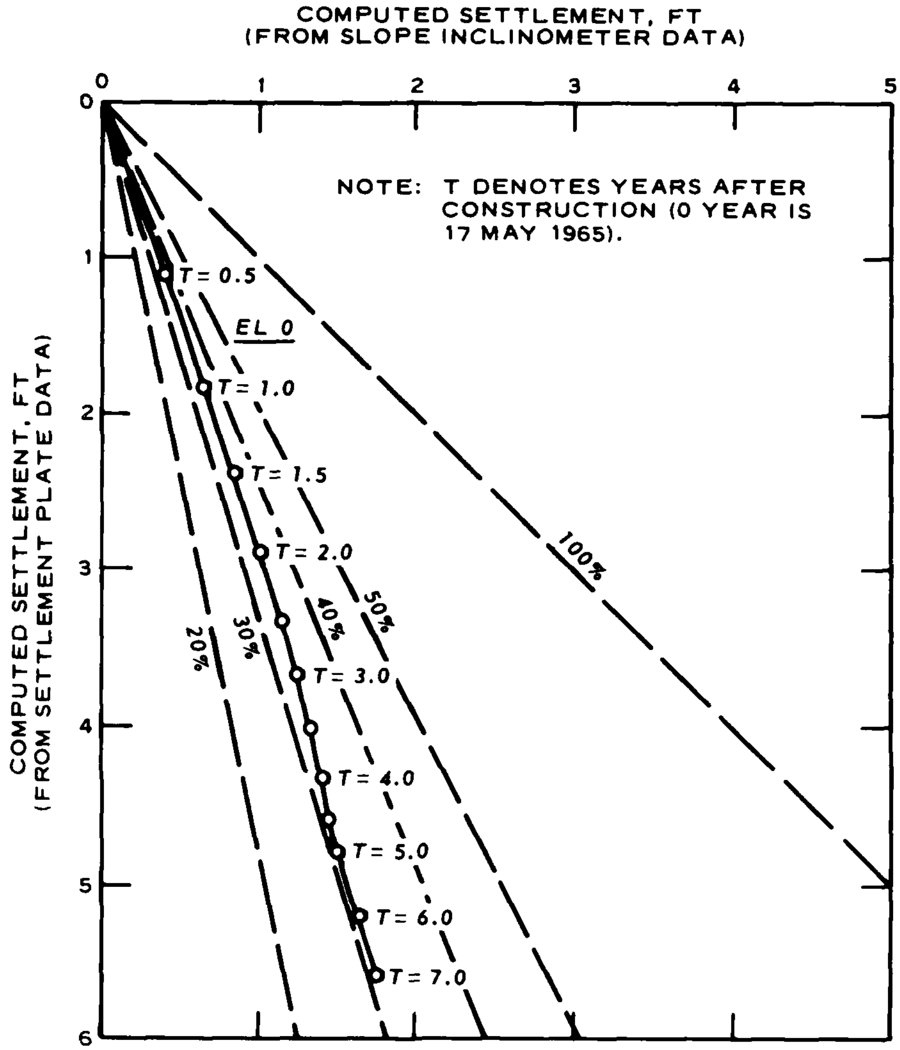


Figure 43. Total settlement versus settlement caused by lateral deformation, Atchafalaya Basin Floodway, Louisiana (Test Section III, FS = 1.3)

Seepage

176. Problems associated with uncontrolled seepage and the consequences resulting therefrom were discussed in paragraphs 112 and 113. This section deals with analyses for seepage and discusses methods of seepage control applicable to retaining dikes.

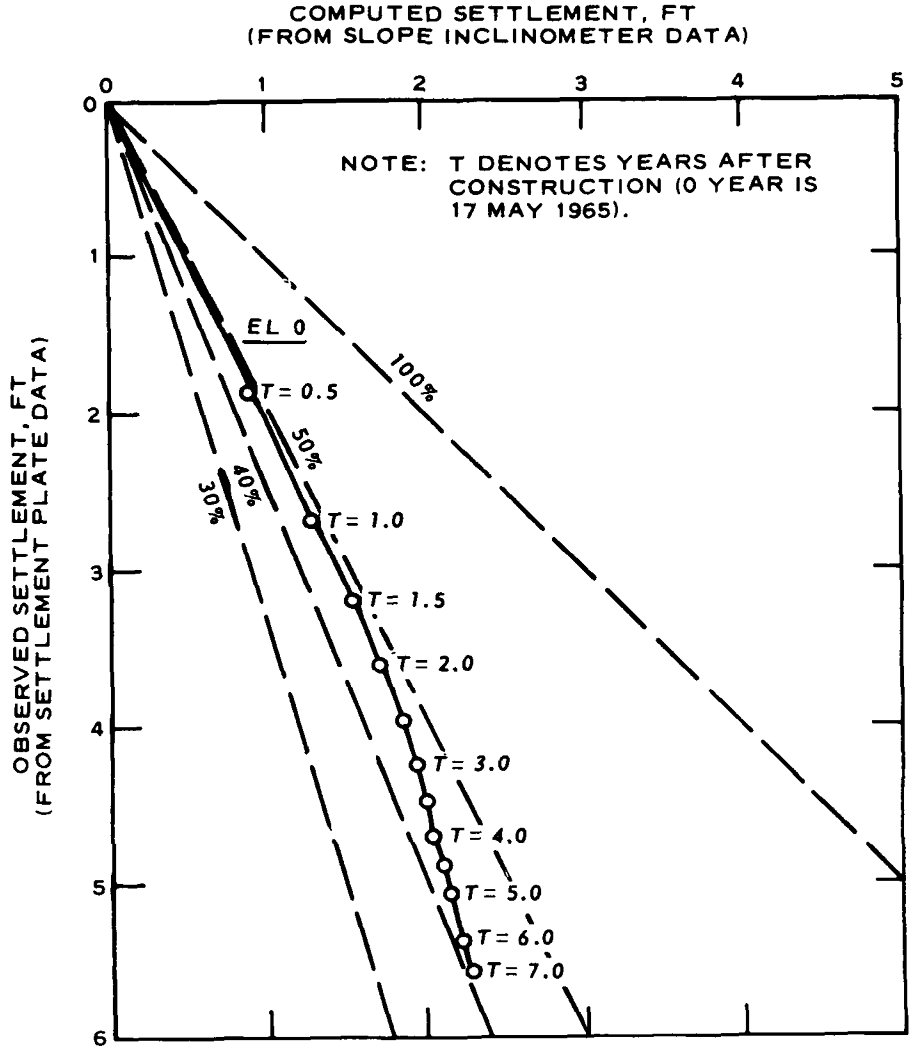


Figure 44. Total settlement versus settlement caused by lateral deformation, Atchafalaya Basin Floodway, Louisiana (Test Section II, FS = 1.1)

177. Seepage problems in retaining dikes are almost always related to coarse-grained soil such as sand and gravel and some fine-grained soil such as silt. In addition, some organic deposits such as root mats and peat are pervious and can also cause seepage problems. Some seepage problems do occur in fine-grained cohesive soil but are usually limited to dike materials and are the result of the method of placement (for instance, uncompacted clay with large voids) rather than the soil itself.

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178. One feature unique to retaining dikes is the fact that fine-grained materials in the form of a slurry are deposited behind the dikes and may eventually clog even the most pervious dikes. In other words, most retaining dikes form their own seepage barriers. However, there are many unknowns associated with this phenomenon such as the time required for the barrier to develop, the maximum elevation of water or slurry against the dike that can occur prior to clogging, the development of clogging as the containment area is filled, etc. Until some of these questions can be answered, it is recommended that the dike be analyzed rather conservatively for seepage. Considerable judgment must be exercised in making assumptions for dike seepage analyses.

Seepage analyses

179. Seepage analyses for dikes will primarily consist of determination of the position of the seepage line (or phreatic surface) within the dike itself, determination of uplift pressures resulting from foundation underseepage, and, to a lesser degree, determination of the quantity of flow. Several mathematical and graphical methods are available for these determinations. References 23 through 25 contain guidance in the analysis of seepage problems and their control. A graphical solution for estimating the position of the seepage surface developed by L. Casagrande is given on p. 184 of Reference 20. A chart for estimating the time required for the development of the seepage line of an embankment is given on page 253 of Reference 25.

180. Once the position of the seepage line is determined, it should be compared with the location of the outer slope line to determine if measures are needed to avoid the emergence of seepage on the outer slope. Uplift pressures should be applied in the stability analyses and either the design made to take such pressures into account or steps taken to reduce the uplift pressures to acceptable values. Flow quantities are needed to design and size exterior ditches to handle the

water. This is often required where the dike or parts of the dike are designed as filtration devices for the dredged material. The references previously given also contain guidance on the design of filters to avoid piping. The phenomenon of piping cannot be analyzed theoretically, but conditions conducive to it, such as high gradients, can be determined by theoretical means. Methods of seepage control are discussed in the following paragraphs.

Seepage control

181. Embankment through-seepage. Seepage through retention dikes constructed of pervious or semipervious materials may be controlled by placement of an impervious barrier on the interior dike slope to restrict flow. This barrier may consist of a layer of impervious soil or polyethylene sheeting. Impervious soil barriers should be a minimum of 3 ft in thickness and thoroughly compacted. Sheeting placed for this purpose should have a minimum overlap of 2 ft at joints, and provisions should be made to ensure that the joints are sealed. Recent developments in the area of chemical spray-on plastics have also shown possibilities in the control of through-seepage.

182. Experience in the Philadelphia District has shown that for pervious dikes in low hazard areas, a policy of compaction of the dike material plus increasing the section width by slope flattening or by increasing the top width has proven adequate against failure, although through-seepage in the dike does develop.

183. Seepage problems resulting from the presence of voids in dikes constructed by casting can best be controlled by requiring the dikes to be compacted to some degree in order to eliminate open voids. Adequate compaction for this purpose can usually be attained by extra tracking by the dozer during shaping. In performing this operation, it is necessary that the dike be cast up in lifts rather than built to grade as the section advances across the foundation.

184. Foundation underseepage. Where pervious foundation materials are encountered, the seepage path can be blocked by constructing an impervious cutoff through the pervious materials, the dike section can be increased in weight to counteract the seepage pressures, or the dike

section may be increased in length in order to reduce exit gradients to within tolerable limits.

185. Cutoffs are feasible only for relatively shallow and thin pervious deposits as they should fully cut off the pervious stratum. Partial cutoffs have been shown to be relatively ineffective. If a cutoff is considered to reduce seepage through a surface root mat or peat deposit, its effect on the overall stability of the section should be considered. In many cases these surface deposits have been shown to be beneficial from a slope stability standpoint, but they must be fairly continuous in order to be of benefit. It is therefore recommended that if such a cutoff is considered, it should be placed at or near the interior dike toe rather than under the dike center line.

186. To prevent piping of foundation materials, it is recommended that the exit gradient have a safety factor of at least 1.5 when compared with the critical exit gradient of the material through which flow is occurring. A factor of safety of about 1.5 based on net uplift forces is also recommended for failure due to uplift of semipervious or impervious top strata (Figure 23). Larger safety factors may be required where the consequences of dike failure are great. The seepage path may be lengthened by berms, impervious blankets, and/or flattening of exterior dike slopes.

187. Seepage at dike-structure contact. Seepage problems at the contact between a sluice and the dike may be avoided by ensuring that adequate compaction of the dike material is obtained at the contact. Also, it is desirable to use material on the wet side of optimum to increase its plasticity, thereby increasing its resistance to cracking and the formation of seepage paths. It is also desirable to install impervious seepage fins extending from the structure into the dike. An additional degree of security may be obtained by increasing the dike cross section at these locations. Prevention of seepage at the dike-structure contact is further discussed in Part IX.

188. Seepage at dike-foundation contact. Proper clearing and preparing of the dike foundation to receive the newly constructed dike can avoid problems caused by seepage paths between the ground surface and

dike. In areas with very soft foundations where marsh grass and root mats are to be left in place for stability, measures previously discussed should be taken to reduce or block seepage through this material. Also where these materials are to be left in place, if the dike crosses a hard spot such as an old dike or road, the hard spot should be completely denuded of all vegetative growth. The Mobile District reported a failure in a retaining dike because this material was not stripped where the new dike crossed an old dike resulting in seepage and piping of the dike material in this area.

Erosion (Slope Protection)

189. Almost all dikes will require some sort of protection against failure due to erosion of their exterior slopes and possibly their interior slopes. For dikes where the consequences of failure would be so severe as to be intolerable, slope protection must be designed to prevent failure under the worst foreseeable conditions. Where failures can be tolerated, the expense and degree of protection must be weighed against the expense and frequency of repairing failures. Generally, it will be more desirable to provide adequate protection rather than suffer the economic and environmental damages of failure.

190. There are many methods of slope protection. These methods vary from minimal, such as grassing to prevent damage from weathering, to substantial, such as massive stone or concrete revetments to prevent damage from storm waves such as that shown in Figure 45. Since the conditions affecting design of retaining dikes are widely varied, the design of slope protection for each structure must be considered on an individual basis. This section discusses some of the methods commonly used for slope protection.

Flat beaches

191. Where material quantities and real estate are available, a gently sloping beach, as shown in Figure 46, may be used to protect the dike against wave action. Gently sloping beaches are effective since wave energy is dissipated by runup on the flat slope. This type of protection is of particular interest for use as protection on exterior

Figure 45. Displacement and loss of stone protection on dike due to storm damage

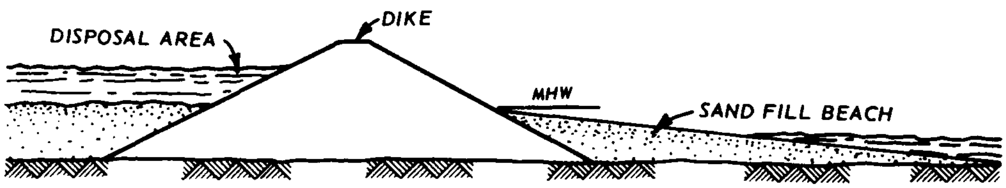


Figure 46. Use of sand beach for dike slope protection (after Reference 26)

slopes of dikes that are adjacent to large bodies of water and continuously experience wave action. Where the material and space are available, flat beaches are often far more economical than riprap, particularly if long haul distances are involved for transportation of the riprap. Another consideration in favor of flat beaches is that for dikes constructed of pumped hydraulic fill, flat slopes normally result anyway.

192. Design of flat beaches should be based on a study of nearby existing beaches with similar controlling conditions. A slope of 1V on 10H should be suitable for preliminary design. It should be recognized that partial or complete replacement by riprap or other means may be necessary in certain areas such as at structures within the embankment or areas subjected to particularly severe wave or current action.

Guidance for use in the design of flat beaches may be obtained from the Coastal Engineering Research Center (CERC) publication, "Shore Protection Manual."[27]

Riprap

193. Quarry-run riprap or graded stone riprap placed over a crushed stone bedding material (filter) or filter cloth is the most commonly used method of substantial slope protection against wave and current erosion. The widespread use of riprap is due to several reasons, some of which are (a) quarried stone is readily available in most areas; (b) common construction equipment and techniques are utilized in placement; (c) the performance history of riprap is good; and (d) riprap is usually the most economical method to achieve the protection desired. A typical riprap protected dike is shown in Figure 47.

Figure 47. Typical riprap slope protection

194. Design procedures using riprap to protect against wind-driven or ship-generated waves are presented in EM 1110-2-2300, "Earth and Rockfill Dams, General Design Considerations."[28] EM 1110-2-1601, "Hydraulic Design of Flood Control Channels"[29] contains guidance on riprap design for protection against current or flow velocities. Guidance for coastal installations is contained in CERC's "Shore Protection Manual."[27]

195. The upper limit (or maximum height) of riprap protection should provide adequate freeboard above the maximum water level (usually high tide, highest expected interior water level, or design flood stage) plus design wave height; the lower limit should provide a toe or key

below minimum water level (low tide or minimum flow). In any event, riprap protection should extend well above and below design high and low water levels. Often this will be the dike crest and a minimum of 2 to 3 ft below low water, respectively (Figure 48).

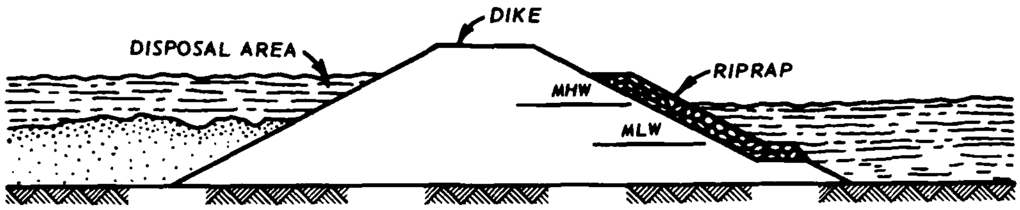


Figure 48. Cross section of dike with exterior slope protected by riprap (after Reference 26)

196. The use of filter cloth to replace bedding material should be considered since filter cloth is often considerably less expensive than crushed stone. Guidance in the use of filter cloth is contained in Guide Specification CW 02215.[30]

Control of disposal operations

197. Interior slopes. To prevent direct washout and erosion of interior dike slopes from the pipeline discharge of dredged material, the discharge pipe should extend at least 50 to 100 ft beyond the dike toe. In addition, a diffuser should be used to dissipate as much energy as possible. Also, a trench 100 to 200 ft long should extend from the discharge point toward the center of the disposal area to prevent the discharge from flowing along the dike toe in the vicinity of the discharge pipe (Figure 49). If, due to the topography of the area, channelization develops along the toe of the dike or through other undesirable areas, spur dikes or cross dikes should be constructed.

198. Exterior slopes. Outfall pipes for sluice discharges should extend at least 10 to 15 ft from the exterior dike slope. Also a ditch should be cut to allow ready escape of discharge water away from the dike toe. Where spillway outlets are used, special consideration should be given to protection of the dike in the area of discharge. Included in these considerations should be riprapping or concreting of the dike slope in the area.

Figure 49. Channelization along dike toe

199. Overtopping. Prevention of erosion due to overtopping caused by overfilling the disposal area can only be controlled by eliminating negligence on the part of personnel in charge of disposal operations. The fact that failures such as this occur indicates the need for constant inspection of disposal operations by qualified personnel.

Other methods

200. A small amount of cohesion in dike embankment materials greatly increases resistance to erosion caused by wind and rain. On the other hand, where frost heave is common, dikes of cohesionless material will be less susceptible to damage than those of cohesive materials. Cohesionless material subject only to effects of weathering may best be protected by establishing a vegetative cover. Often a layer of topsoil is necessary to establish such growth, along with a light cover of emulsified asphalt or mulch to prevent erosion until such time as the vegetation is established. The Mobile District has successfully protected sand dikes from erosion caused by rain by cupping the dike crest to catch rainwater and providing drains at certain locations along the alignment. This method of protection is shown in Figure 50.

201. Polyethylene sheeting, if properly placed and overlapped, can be effective in preventing erosion of interior dike slopes from wave and current action and heavy discharge flow. Polyethylene sheeting can also be used on exterior slopes on a short-term basis where erosive forces are not too severe. Disadvantages from the use of polyethylene sheeting

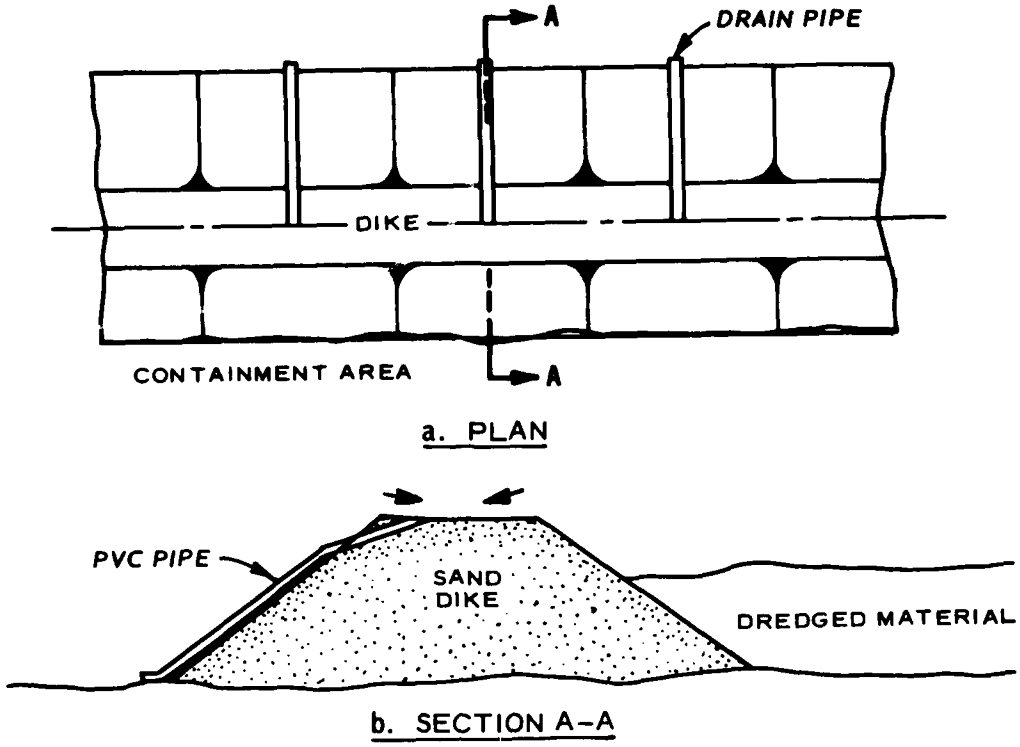


Figure 50. Protection of sand dike slopes from slopewash due to rainfall

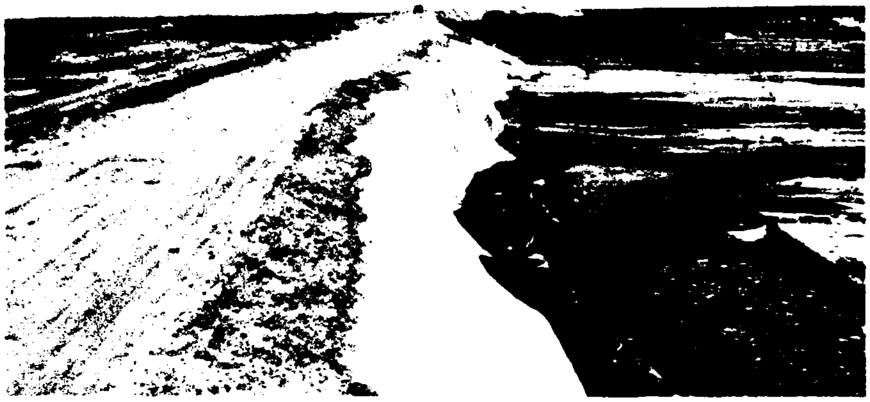
are deterioration from sunlight, damage from burrowing animals, and removal due to wind action and vandalism.

202. Although riprap is the most common method of substantial slope protection, other methods should be considered to determine which is the most feasible and economical. Factors such as site access, high transportation cost, availability of suitable stone, or other considerations peculiar to a particular site can make alternative methods of slope protection more feasible. Other available methods of slope protection include (a) grout-filled nylon revetments (FABRIFORM, VSL HYDROMAT, etc.), (b) interlocking concrete blocks (LOK-GRAD), (c) concrete paving, (d) sacked concrete, (e) stone-filled wire mesh baskets (GABIONS), (f) soil-cement, and (g) precast concrete forms (Tribars, Tetrapods, etc.). Specifications and design criteria for most newly developed slope protection systems can be obtained from manufacturer's

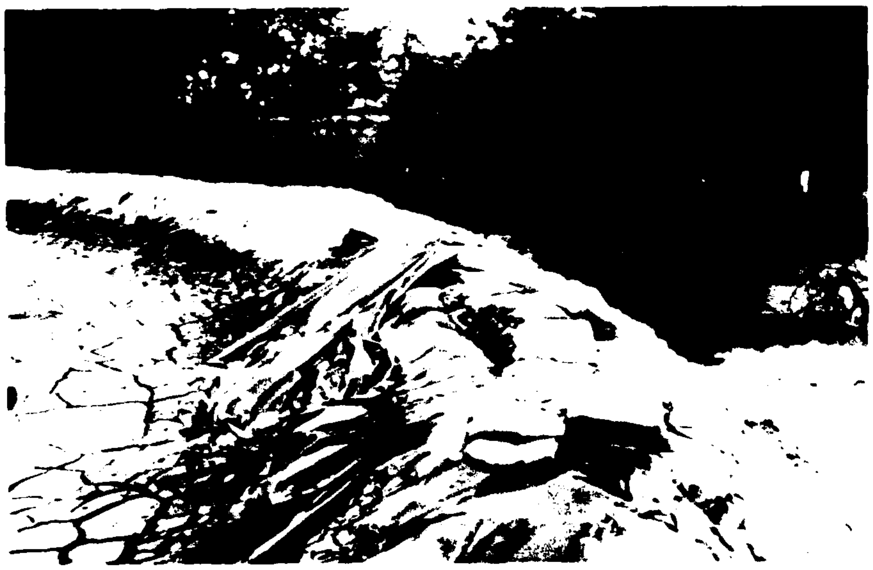
literature. In addition, a number of these methods have been tested at WES and CERC with the results of these tests available in various publications of these agencies.[31-39]

Emergency and temporary protection

203. There are times when erosion cannot be prevented, as in the case of severe storms that exceed design criteria. In such occurrences, some method of temporary protection may be needed to prevent total dike failure until such time that permanent remedial measures can be implemented. As previously stated, polyethylene sheeting can be used for temporary protection in areas of damage. Also, sandbags or stockpiled stone can be utilized to afford temporary protection to damaged areas. Photos of polyethlyene and sandbags used as temporary protection are shown in Figure 51.



a. Use of polyethylene sheeting as temporary protection against erosion of sand slopes by weathering



b. Use of sandbags and polyethylene sheeting for temporary protection of dike against overtopping and erosion

Figure 51. Temporary slope protection

PART VIII: DIKE CONSTRUCTION

204. As previously discussed, the method of dike construction is of primary importance and can have a profound effect on the final dike cross section. Generally speaking, there are three basic categories of dike construction: hauled, cast, and pumped (hydraulic fill). Of course, there are many variations and combinations of these methods that can and have been used. The purpose of this part of the report is to discuss some of the salient features of each type of construction, including advantages and disadvantages, applicability, inherent effects on the dike cross section, effect of material types, etc.

Equipment

205. Types of equipment commonly used in dike construction are listed in Table 13 according to the operation they perform. Most of the equipment listed in Table 13 is familiar to all engineers. However, because many dikes are founded on soft to very soft ground, some of the equipment is especially made for such conditions. A brief discussion of some of the more commonly used types of equipment is contained in the following paragraphs. Green and Rula[40] should be consulted for more detailed information.

Bulldozers

206. Bulldozers are often used for spreading, compacting, and shaping fill material for dike construction. They are used in construction of nearly all types of dikes including hauled, cast, and pumped. They are also extensively utilized in foundation preparation.

207. Conventional crawler tractors that exert ground pressures of about 8 psi and higher are often unable to operate on soft ground. Several equipment manufacturers now offer modified tractors with lower ground pressures made especially for soft-ground construction. These machines utilize wider tracks and exert ground pressures of 4 psi and lower. A photograph of a small bulldozer working on soft dredged material is shown in Figure 52.

Table 13

Equipment Commonly Used in Dike Construction

Operation	Equipment
Excavation	Draglines Scrapers Dredges
Transportation	Scrapers (hauled) Trucks (hauled) Draglines (cast) Dredges (pumped)
Spreading	Bulldozer
Scarification	Disk
Compaction	Sheepsfoot roller Pneumatic roller Vibratory roller Bulldozer Hauling equipment
Shaping	Bulldozer Dragline

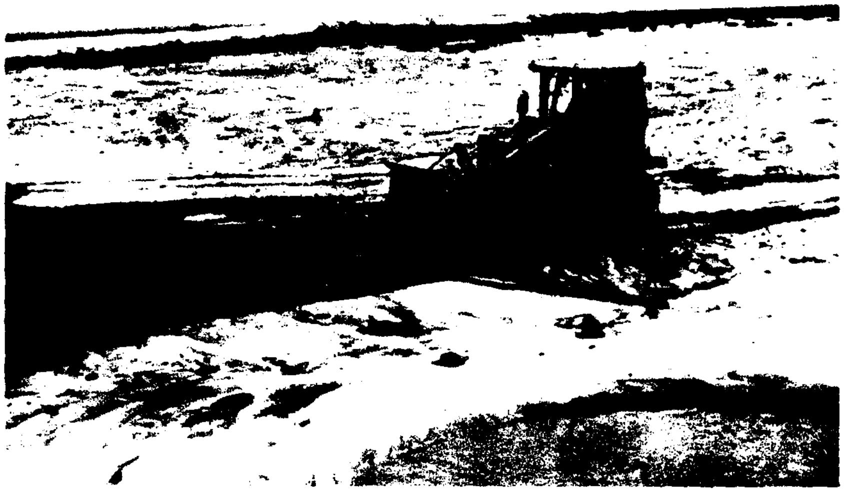


Figure 52. Low-ground-pressure machine working on dredged material

208. Bulldozers are available that utilize rubber tires rather than tracks for drive. These dozers operate at speeds four times that of tracked dozers and have been shown to be very effective in working with granular materials. The main disadvantage of a wheel dozer is its high ground pressure, which prohibits its use with soft materials.

Draglines

209. Draglines are used to construct cast dikes as shown in Figure 53. Through the use of wide track machinery and/or proper matting techniques, draglines can operate in areas so soft they are almost inaccessible to a person on foot. This often requires use of a timber matting under the dragline that can be single, double, or triple layers of timber.

210. While small draglines may exert less ground pressure and may be more maneuverable than larger machines, they are often at a disadvantage due to their short boom and small capacity bucket. Their short reach (about 40 ft) frequently necessitates rehandling material. Also, the small bucket tends to greatly disturb the material being excavated, which is a distinct disadvantage when working sensitive materials.

211. When excavating soft, weak material along the proposed dike

Figure 53. Dragline constructing cast dike

alignment, a wide shallow cut as shown in Figure 53 is the most desirable and feasible geometric shape. To successfully handle this operation, draglines with 60- to 70-ft booms and 1-1/4- to 2-cu-yd buckets have been found adequate. Their use will allow utilization of a wide, shallow borrow cut with a minimum of disturbance to the material. Also, these size machines have been found adequate for operation on soft ground.

212. Barge-mounted draglines. Barge-mounted draglines are used extensively in areas where the groundwater table is at or very near the ground surface (Figure 54). These machines excavate their own waterway ahead and cast material to the side to form the dike. This technique allows the use of very large machines. The particular machine shown in Figure 54 has a 125-ft boom and utilizes an 8-cu yd bucket (shown in Figure 55). This machine can excavate and place about 14,000 cu yd of material in a 24-hr period. Obviously, these machines will require use of deeper, narrower borrow ditches.

213. The barge upon which the dragline works can be an assembled unit as shown in Figure 54. This eliminates the need to be near open water, a requirement for normal barges. These units can be assembled

Figure 54. Barge-mounted dragline

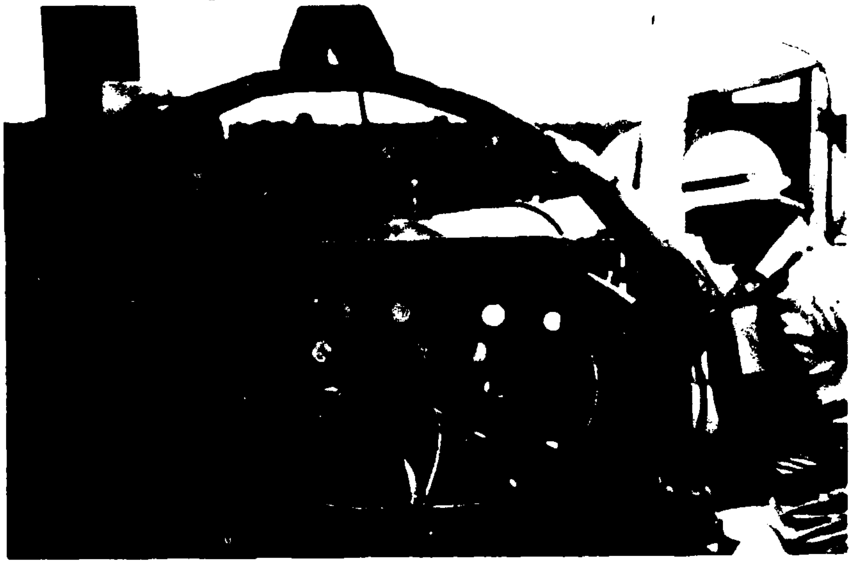
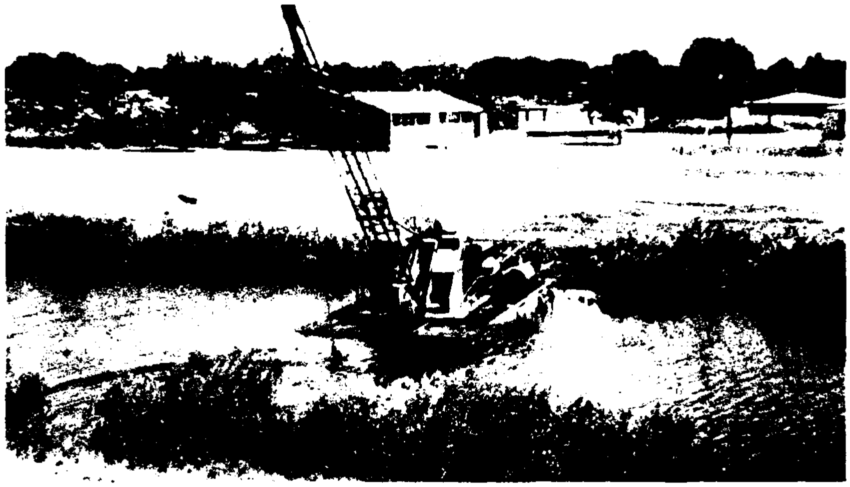


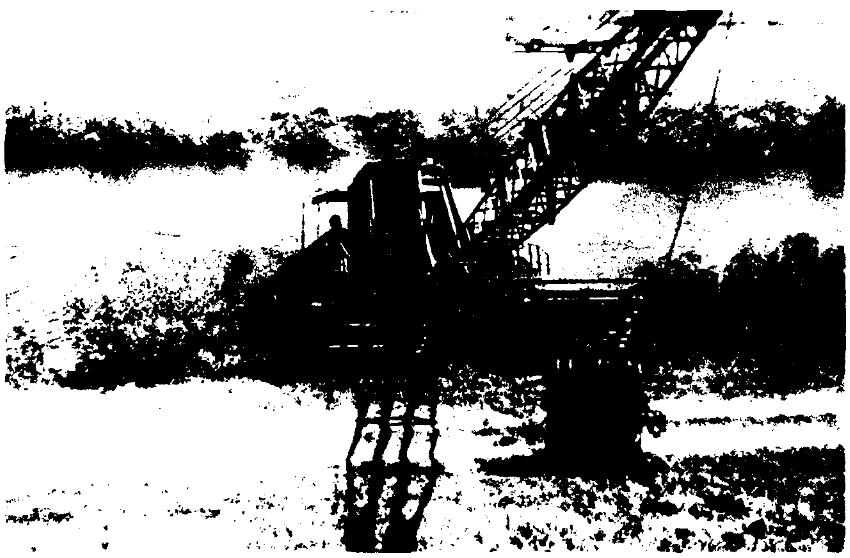
Figure 55. Large dragline bucket

at the site and, once the work is done, the dragline removed and the barge disassembled.

214. Pontoon-mounted draglines. Pontoon-mounted draglines that can actually float, such as the one shown in Figure 56, are also useful on very soft ground or in shallow inundated areas. These machines have wide tracks mounted around pontoons. The disadvantage of these machines



a. Crossing shallow stream



b. Climbing streambank

Figure 56. Pontoon-mounted dragline

is their smaller size. Pontoon-mounted draglines are often used for the construction of toe dikes used in connection with the pumping of hydraulic fill.

Dredges

215. Hydraulic cutterhead dredges as shown in Figure 57 are most often used to construct hydraulic fill dikes as they are equipped to

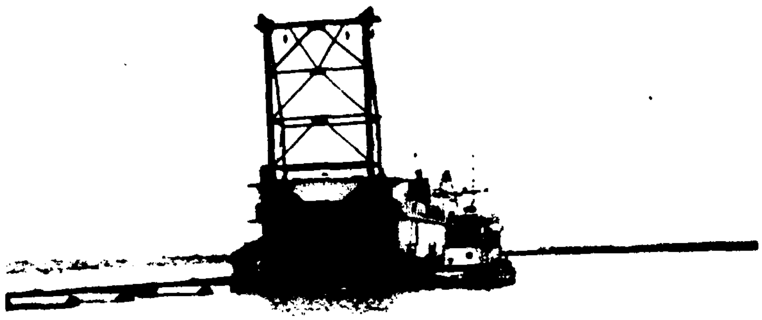


Figure 57. Hydraulic cutterhead dredge

pump the dredged material to the disposal site through a pipeline simultaneously with the dredging operation (Figure 58). Other types such as hopper and bucket dredges have the disadvantage of either having to stop dredging and transport the material to the site or load it onto scows for transportation. There are many variations and sizes of hydraulic cutterhead dredges in use today, and the type and size dredge can affect the condition of the pumped material, especially clay. For detailed information on dredges reference should be made to "Hydraulic Dredging" by John Huston.[41]

Compaction equipment

216. There are three principal types of rollers for earthwork compaction: sheepsfoot, pneumatic, and smooth-drum vibratory rollers. The sheepsfoot roller is for compaction of cohesive materials; the

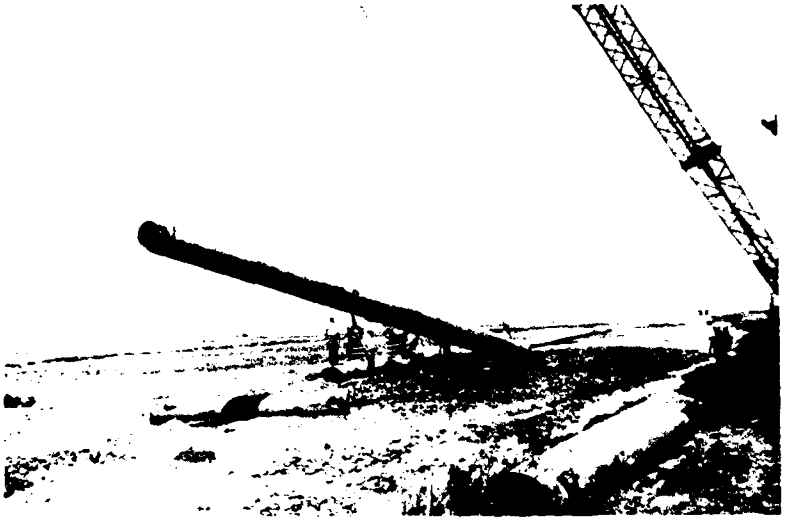


Figure 58. Crane moving pipeline for pumping hydraulic fill

smooth-drum vibratory roller for cohesionless materials; and the pneumatic roller can be used on both types of materials, but is primarily for cohesive materials. The pneumatic roller is used to a much lesser extent than the sheepsfoot and vibratory rollers. Chapter 5 of EM 1110-2-1911[42] contains detailed descriptions of these rollers including uses, features, advantages, and disadvantages of each.

Dike Materials

217. This section contains a discussion of the different types of dike materials and how they relate to the construction method used, i.e., primarily to compacted, semicompacted, and pumped dikes. For this purpose, dike materials can be categorized as fine-grained or impervious and semipervious materials and coarse-grained or pervious materials. These materials are defined according to the Unified Soil Classification System[11] as follows:

a. Impervious and semipervious materials. Impervious materials include clay (CH and CL), clayey sand or gravel (SC or GC), highly plastic silt (MH), and clay silt (CL-ML). Semipervious materials include silt (ML) and silty sand or gravel (SM or GM).

b. Pervious material. Pervious material includes free-draining cohesionless sand and/or gravel (SP, SW, GP, GW) containing less than approximately 5 percent of material that passes the No. 200 sieve.

Materials for hauled (compacted and semicompacted) construction

218. Impervious and semipervious fills. Generally, compaction curves that indicate well-defined maximum dry densities and optimum water contents as shown in Figure 59 can be developed for these materials. The more fine grained (or impervious) the material, the broader the legs of the compaction curve, the higher the optimum water content, and the lower the maximum dry density. Curve A in Figure 59 is a

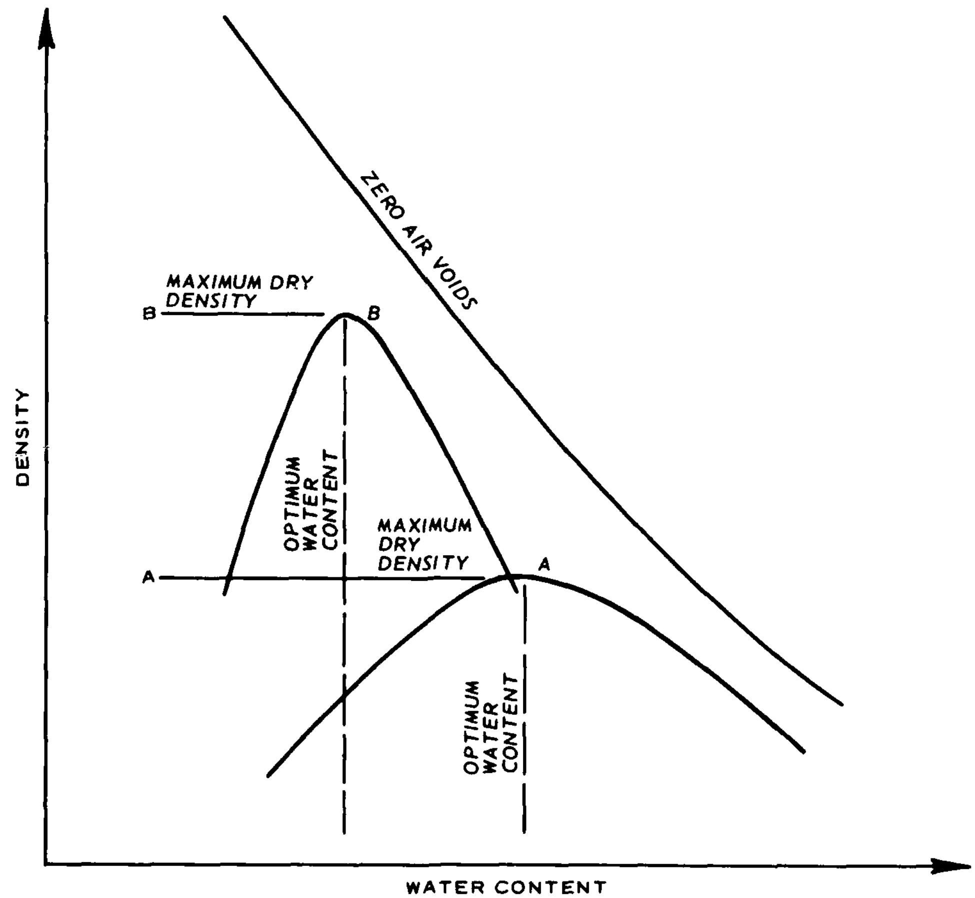


Figure 59. Typical compaction curves for fine-grained soils

typical compaction curve for a fat clay, i.e., a material containing a high percentage of clay. On the other hand, a leaner or less impervious material (semipervious) will have a compaction curve similar to curve B in Figure 59. Compaction curves for these materials typically exhibit narrow legs with lower optimum water contents and higher maximum dry densities.

219. Pervious fill. Standard impact compaction tests on clean cohesionless materials do not normally yield well-defined values of maximum dry density and optimum water content. Field densities for these materials are usually related to maximum and minimum density determinations (i.e. relative density) rather than to the maximum dry density as used for more fine-grained materials. Since these materials do not have a well-defined optimum water content, there is no field control of water content as is usually required for impervious and semipervious materials.

Materials for hydraulic fill (pumped) construction

220. Because hydraulically dredged material is deposited as a slurry containing considerably more water than solids (about 85 percent water by volume), its suitability as a construction material for dikes is primarily dependent upon the grain size and plasticity of the solids. Fine-grained materials, such as silt, clay, and silt-clay mixtures generally have poor engineering qualities when initially placed hydraulically, i.e., they are generally very compressible and have low shear strengths (except when in the form of clay balls). In addition, fine-grained materials drain slowly; consequently, improvement in their engineering properties occurs over an appreciable period of time. When such materials have a high organic content, they exhibit even poorer engineering properties. In contrast, coarse-grained pervious materials drain and consolidate rather rapidly due to their high permeability and thus stabilize into a strong, less compressible fill in a relatively short period of time after placement.

221. Impervious and semipervious fill. Fine-grained material consisting of clay and silty clay with in situ consistencies of medium

or greater are usually deposited by hydraulic means in the form of clay lumps or balls and produce a relatively good fill within a reasonable time frame. Fine-grained materials obtained from new work dredging (i.e., virgin cuts) are normally better materials for dike construction than fine-grained sediments resulting from maintenance dredging, which are often loose silt and clay slurries.

222. Clayey soils. Clayey soils from new work dredging, where the in situ consistency is medium or stiffer, separate into two portions in the dredged discharge. One portion, clay balls, is deposited in the immediate vicinity of the discharge point. The other portion, dispersed clay and silt particles, remains in suspension and is deposited in other parts of the disposal area. The latter portion is typical of some maintenance dredged material. The clay balls have the appearance of rounded gravel or cobbles and are undispersed clay at essentially their excavated in situ water content (Figure 60). The interstices are generally filled with clay and silt slurry or sand and slurry when sand is present in the discharge. The overall water content of such fill is

Figure 60. Hydraulic fill composed of clay balls

greater than that of the original in situ material, but is much less than the water content of slurry material deposited from dispersed clay and silt particles. The expected angle of repose of the clay ball portion of the discharge can vary from about a 1V on 7H to a 1V on 25H, depending on the in situ consistency of the clay. The dispersed slurry material will assume a very low or no angle of repose. Even though these clays have a very low permeability and drain slowly after hydraulic fill placement, clay balls will normally support light construction equipment soon after placement. The actual time will vary from immediately after placement to as much as several months depending on the in situ consistency of the clay borrow material. Because of the high depositional water content of dispersed clay and silty clay slurries, the drying time for these deposits is greatly increased. Without the aid of internal drainage provisions, drying times of several years are commonly required for such slurries to form a 2- to 3-ft-thick crust.

223. Silty soils. Hydraulically placed silty soils are generally totally dispersed and consequently achieve a very low angle of repose. During, and for some time after disposal, these materials behave generally like clayey soil slurries as indicated in the previous paragraph, but, because of their higher permeability and lower plasticity, they tend to gain strength and consolidate faster than clayey soil slurries. Light loads can generally be supported in 1 to 2 years, depending on the percentage of clay content and the drainage.

224. Organic clay and silt. Organic clayey silt and silty clay from both new work and maintenance operations usually have a soft to very soft in situ consistency and are completely dispersed in the discharge. Because of their high compressibility, high depositional water content, low density, and low shear strength, organic clay and silt are the most undesirable materials for dike construction.

225. Pervious fill. Hydraulically placed coarse-grained materials generally form medium dense deposits. Sand with less than 10 percent fines that is hydraulically placed in a well-controlled manner will achieve a relative density of 50 to 60 percent with no compaction.[43] These materials will normally assume an angle of repose of about 1V on

5H to 1V on 10H. Volume changes that occur after placement are generally insignificant. Because of high permeability and inherent strength, this type of material will support loads from both construction equipment and dredged effluent within a few days after placement.

Construction Control

226. If a dike is not built according to the plans and specifications so that the intended design is attained, the results will be less than satisfactory. The only way to ensure that construction is done in compliance with plans and specifications and to deal with details not adequately covered in the plans and specifications is to thoroughly inspect all operations involved in the dike construction. Past experience has shown time and time again that the importance of adequate inspection cannot be overemphasized.

227. The exact items to be closely monitored during construction will vary with the design and method of construction. However, there are some general items pertinent to all projects, regardless of their nature. These items are:

- a. Field personnel should be thoroughly familiar with the plans and specifications for the disposal area. Included should be familiarization with general aspects of the long-range plans for the area.
- b. A meeting should be held between the design engineer and field personnel in order that the designer's views may be obtained and any questions cleared up. The designer should point out any key items that should be observed and any anticipated unusual or marginal features.
- c. A document entitled "Instructions to Field Personnel" should be distributed to and thoroughly read by field personnel.
- d. Field personnel should be thoroughly familiar with the borrow sources, the stratification of each, and how each type of material will look when being placed or discharged.
- e. Field personnel must be provided access to the dike construction area at all times and should be on hand continuously during construction.

f. Good records of all observations must be maintained. This includes photographs as well as written records.

228. Details of construction control measures pertinent to a particular construction method are discussed in appropriate paragraphs given subsequently in this part.

Hauled Dikes

229. Hauled dikes are defined as dikes built by fill hauled in from borrow areas, usually by trucks or scrapers. Hauled dikes can be compacted, semicompacted, or uncompacted, depending on treatment the material receives after deposition by the hauling equipment. However, when hauling procedures are used, most dikes will be compacted or semicompacted.

Advantages and disadvantages

230. Compacted dikes. The main advantage of a compacted dike is that it results in the highest quality embankment occupying the least amount of space. It is also a product in which the designer can have the best assurance of obtaining what has been designed. Disadvantages include a relatively high cost and the fact that it requires a reasonably competent foundation, one item which, due to most prevailing dike foundation conditions, somewhat limits its applicability to dike construction.

231. Semicompacted dikes. Semicompacted dikes usually are built on weaker foundations than compacted dikes and can provide a stable dike at a lower unit price than compacted dikes. Normally, semicompacted dikes are built of materials placed at their natural water content. Semicompacted dikes are often specified because of oft-required large sections with flat slopes, which would result in an uneconomical and impracticable design if a fully compacted dike were specified. Disadvantages of semicompacted dikes include the larger section usually required and the uncertainty as to the end product with respect to uniformity of compaction.

232. Uncompacted dikes. About the only advantage of an

uncompacted hauled dike is the fact that, due to foundation conditions, it may be the only type of dike that can be built. It is also a low cost construction method. However, with uncompacted dikes there is considerable uncertainty as to the end product, and estimating required quantities with any degree of accuracy is often a hopeless task. Also, there is little or no guarantee that the design elevation will be attained due to uncertainty as to the amount of settlement of the embankment. Uncompacted hauled dikes should only be considered if construction of other types of dikes appears impossible.

Construction procedures

233. Hauling, spreading, and blending. Where borrow conditions permit and where space on the fill is sufficient for turning, scrapers are the most economical means of moving fill. Where borrow areas are too wet to allow direct excavation and trafficking, transportation can be by trucks loaded by clam shell, dragline, or other excavating equipment (Figure 61). After dumping, the material is spread to the proper loose lift thickness by a dozer as shown in Figure 62. For compacted fills, the material should be thoroughly worked with a disk (capable of cutting through the entire loose lift) after spreading and prior to compaction. This will help eliminate lumps, aid in a more uniform distribution of moisture, and, in general, ensure a more homogeneous fill material. When moisture control is specified and where the water content of fill material is too high, disking should continue until the water content is reduced to an acceptable level; where the water content is too low, water should be added and the material disked until a uniform distribution of moisture is attained at an acceptable water content.

234. Compaction (compacted fill). Compaction for a fully compacted fill is usually carried out by one of the rollers listed in Table 13. Sheepsfoot rollers are the most often utilized equipment for compacting impervious and semipervious fill, with rubber-tired rollers being used to a lesser extent. Loose lift thicknesses for the sheepsfoot and rubber-tired rollers are normally on the order of 8 in. and 10 to 12 in., respectively. Scarification by disking of lift

Figure 61. Trucks dumping fill material

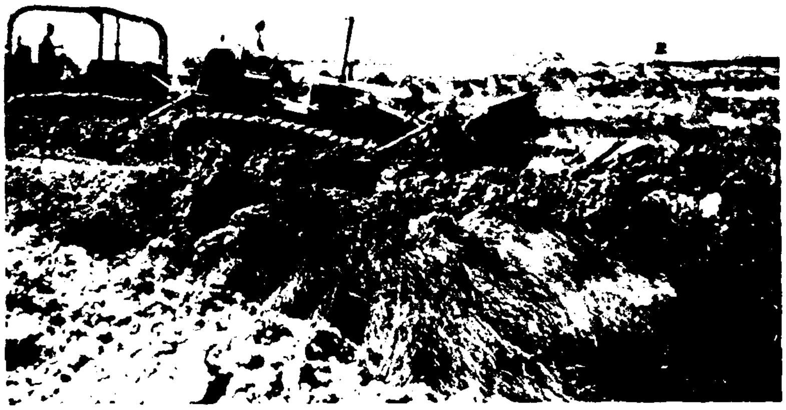
Figure 62. Bulldozer spreading fill material

surfaces after compaction to ensure good bonding between lifts is always a good procedure no matter what type of compaction equipment is used, but is a necessity when a rubber-tired roller is used because of the smooth surface left by the roller.

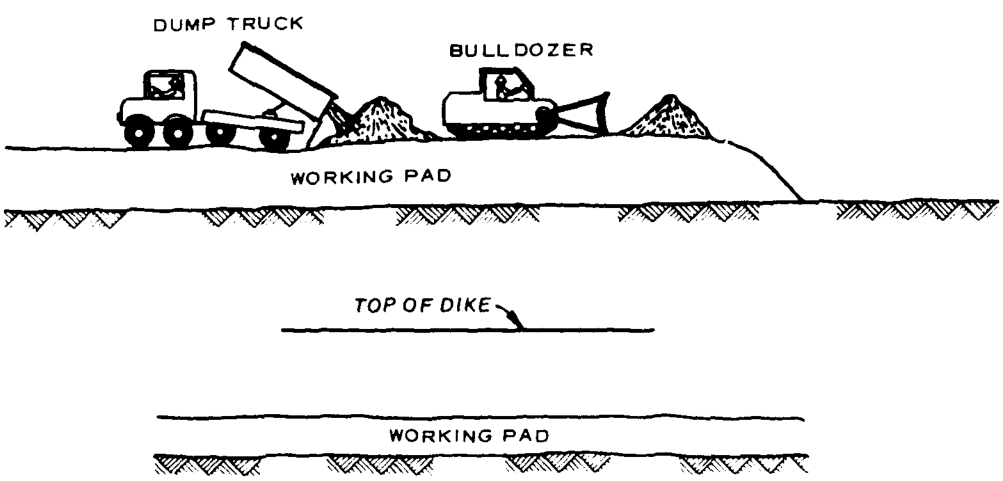
235. A vibratory roller is the best means of compacting pervious fill, although crawler tractors have often been successfully employed for this purpose. Saturation for the pervious fill during compaction will aid in the compaction process but is generally not a necessity unless very high densities are required. Merely sprinkling the material prior to compaction has little if any benefit due to bulking effects that result from the addition of only a minor amount of water.

236. Compaction (semicompacted fill). Compaction for semicompacted fill is usually accomplished through utilization of trafficking of hauling and spreading equipment on the fill, although in some instances a few passes of a light sheepsfoot roller or a dozer is specified as the compaction procedure. When utilizing traffic compaction, it is important that the equipment not be allowed to "track" (i.e. follow in the same set of tracks) but be made to operate in such a fashion that as much of the fill surface as possible is covered. Tracking not only results in an appreciable portion of the fill obtaining little compaction, but also often results in rutting and pumping of the material in the tracks.

237. Special procedures for soft foundations. Due to the difficulty of operating equipment on very soft foundations, it may be necessary when building compacted or semicompacted fill to first construct a working platform over the dike base area upon which equipment can operate. This is basically an uncompacted layer 2 to 4 ft thick (only as thick as necessary to support the equipment) formed by dumping and shoving ahead with dozers (Figure 63) until the platform covers the entire dike alignment or necessary portion. Coarse-grained soils are the best materials of which to construct working platforms, but fine-grained materials dry enough to support equipment have also been successfully employed. If coarse-grained materials are used, some sort of seepage barrier may be required in order to prevent seepage through the



a. BULLDOZER SPREADING FILL



b. DIKE SECTION WITH WORKING PAD

Figure 63. Construction of working pad

platform. Material forming the platform should not be stockpiled on the platform or a shear failure may occur in the foundation. Only small dozers should be used to spread and shove ahead. When required, compaction of the platform should be accomplished by using more passes of lighter equipment (such as rubber-tired hauling or loading equipment) or tracked equipment (such as dozers, end-loaders, etc.). Where the foundation is extremely weak, it may be necessary to place the material by casting it over the area with a small dragline or clamshell. After this base has been established, controlled placement and compaction procedures may commence.

238. Uncompacted fill. Placement of uncompacted fill by hauling refers to fill placed by end-dumping and shoving ahead, resulting in a dike section formed by the displacement technique as previously discussed in Part VII. The fill above original ground does get some compaction from hauling equipment and dozers, but such traffic is usually uncontrolled and results in essentially an uncompacted section. In using this method of construction, the item of greatest concern is ensuring that no soft material is trapped in the fill. Techniques for accomplishing this were previously discussed in pargraph 152 of Part VII.

Construction control

239. The control of construction operations is an extremely important facet of dike operations. Some of the more pertinent items to be checked during construction of hauled dikes are given in Table 14. For specific instructions as to how earthwork operations should be controlled during construction, reference should be made to EM 1110-2-1911[42] and "Earth Manual."[44]

Cast Dikes

240. Dikes built by casting material up with draglines are termed cast dikes. This procedure involves use of a borrow ditch parallel to the dike (as previously discussed in Part V), usually located inside the retention area. A berm is left between the dike and the borrow

Table 14

Operations or Items To Be Checked During Construction of Hauled Dikes

Type Construction	Items or Operation to be Checked
Compacted	Proper fill material Loose lift thickness Disking Water content Type of compaction equipment and number of passes Density
Semicompacted	Proper fill material Loose lift thickness Water content (if required) Number of passes (if required) Routing of hauling and spreading equipment
Uncompacted (displacement technique)	Proper fill material Dumping and shoving techniques Ensuring fill is advanced in V-shape and with slopes as steep as possible Elevation of fill surface Prevention of rutting of fill surface by hauling equipment

ditch, the purpose of which is not only for dike stability, but also to avoid future dike increments from being founded on soft dredged material that is deposited in the ditch. This berm also provides a convenient working platform for the dragline.

241. Casting dikes with draglines has been a very common method of dike construction in the past due to its low cost, but unfortunately it often does not necessarily result in an adequate embankment. This is primarily due to the fact that it results in essentially an uncompacted dike and requires relatively steep slopes because of features inherent to draglines (i.e. limits on casting distances). Cast dikes can be semicompacted if placed in lifts and shaped and compacted by a bulldozer working simultaneously with the dragline. However, this is usually not the case as it is more expensive than casting a dike up to full height as the section advances, with no compaction.

242. Cast dikes on very soft foundations are often difficult to construct due to the relatively steep slopes required that can result in considerable displacement of the soft foundation as well as frequent shear failures. Consequently, dikes constructed by casting on soft foundations sometimes must be limited to a few feet in height and must be built in increments.

Construction procedures

243. No special techniques are normally required when handling firm or pervious materials; however, soft silt and clay cannot be handled by normal methods because of the sensitivity and very low remolded strengths these materials exhibit. When these types of materials are handled, it is necessary to keep disturbance to a minimum. During excavation of soft materials, a special effort should be made to load and pick the bucket straight up rather than dragging the bucket through the material. Past practice has shown this procedure to create the least amount of disturbance. During unloading it is desirable to place the material in its desired location and dump it without dropping the material from any appreciable height (i.e., lay it in place). If soft material is dropped from a height greater than about 1 to 2 ft, the material will tend to liquefy and flow thus creating no buildup of fill.

These procedures are slower than usual procedures but are often the only means of obtaining a satisfactory section. For purely cast dikes (i.e., no compaction specified) of firm or pervious materials, some compaction can be attained by dropping the bucket on the fill; however, this procedure should not be used on soft materials due to reasons previously discussed.

244. After the desired height of dike is attained, the dike should be shaped to final lines and grades with a bulldozer. On very soft materials subject to remolding, shaping may have to be done after the dike has cured for awhile and the surface material dried to some extent. As a final measure after shaping, the dike slopes should be trackwalked. This will greatly aid in erosion control until a vegetative cover is established.

Construction control

245. Since there is no density or water content control for cast dikes, construction control (other than ensuring that the embankment is being constructed to the proper lines and grades) consists primarily of determining that construction procedures are in compliance with specification requirements and are proper with respect to providing the desired end product. For cast dikes placed in lifts and semicompacted, inspection should consist of ensuring placement of material in the proper lift thickness and proper coverage by the compaction equipment specified. For uncompacted cast dikes, inspection should be carried out to ensure that the dike material is being placed by procedures necessary to obtain the highest quality embankment obtainable. Several of these procedures (i.e. proper bucket control, placement procedures, etc.) have been previously discussed. For any type of construction involving side casting techniques, it is very important to ensure that the proper width of berm between the dike toe and excavation ditch is obtained. The importance of this berm has previously been stressed. It is also very important on jobs where construction procedures are very critical (such as cast dikes on soft foundations) that experienced personnel be assigned to construction control. In doing this, many problems can be avoided and those that do occur can be more easily solved

by working closely with the contractor, who may or may not be experienced in the area.

Hydraulic Fill Dikes

246. The hydraulic fill method of dike construction consists of excavating material with a dredge and pumping the resulting mixture of soil and water through a pipeline to the desired area. The term hydraulic fill as used herein is defined as material obtained in this manner. When dike material is obtained from the area to be dredged, the hydraulic method is usually the most economical means of construction because it combines both excavation and transportation of excavated material in one operation.

Advantages

247. The hydraulic fill method is an economical means of excavating and transporting large volumes of material over long distances and, as such, offers a practical and economical means of establishing a wide large-volume dike section that is often required for dikes located on soft, weak materials or for dikes requiring seepage control. The use of the hydraulic fill method in areas where near-surface materials consist of soft organic clay, peat, and wood can provide a practical and economical means of obtaining higher quality materials that may exist either below near-surface materials or in areas other than adjacent to the dike alignment. The higher quality material obtained in this manner may be either stronger clays occurring at depth that will discharge as clay balls or sandy materials from nearby lakes or waterways. A dike constructed of such hydraulic fill will, in most cases, be more desirable from the standpoint of stability and through seepage than will one built by casting methods using poor near-surface materials.

248. The use of suitable hydraulically dredged material for initial construction of or raising retaining dikes can result in a more efficient and effective use of a given disposal area, as the entire available disposal area is usable for placement of the dredged material. It may also eliminate the need for performing excavation adjacent to the

dike as is normally required in order to construct the dike by casting methods. As previously discussed, such excavations can contribute to the instability of the dike by providing a more ready access for seepage beneath the dike through relatively pervious surface layers of highly organic, peaty marsh deposits or through substratum sand layers that may be exposed in the excavation.

Disadvantages

249. Water is the transporting agent in hydraulic fill and is, therefore, introduced in great volume into the fill material. This, coupled with the fact that dredged material is often of poor engineering quality, can cause (a) the initial height of the dike to be limited to a relatively small value, (b) possible long time lapses between the hydraulic filling for the dike and the use of the disposal area, (c) the dike to be wider due to flatter slopes to achieve stability, thereby utilizing both more fill material and real estate, and (d) the dike to be a poor foundation for a future dike enlargement.

250. The water used to transport the fill must meet applicable water-quality standards when released to natural waters. In an attempt to satisfy this requirement, the effluent is normally held in the disposal area for some period of time to allow most of the suspended material to settle out before being discharged over weirs. Achieving an effluent suitable for release can be both time consuming and costly. Operational difficulties, such as channelization from the point of discharge to the sluice and insufficient ponding area, have resulted in excessive amounts of solids being discharged. This in turn has caused delays in pumping while the material is allowed to settle out. Also, the discharge sluices invite seepage problems that may lead to ultimate dike failure.

251. The construction of a retaining dike using directly placed hydraulic fill will often require the construction of small parallel cast retention dikes usually referred to as toe dikes (subsequently discussed in paragraph 255). This procedure requires additional types of equipment and hence may be more expensive.

252. In instances where the in situ foundation material along a

proposed dike alignment is of high quality from both a foundation and borrow standpoint, the appropriateness of using a hydraulic fill retaining dike is diminished, particularly if the material to be dredged is of poor quality. In such cases, engineering, economic, and environmental factors may favor cast or hauled fill construction.

Methods of forming dike sections

253. Hydraulically placed material can be incorporated into retention dikes by several methods: (a) discharging material directly in the location of the desired dike with no shaping, (b) discharging material directly in the location of the desired dike and shaping the material to the desired section either immediately, if coarse-grained material, or at some later date after the material has undergone some drying and strengthening, if fine-grained material, (c) moving material previously deposited by hydraulic means by conventional means and building the dike as a cast or hauled fill, and (d) some combination of the above methods. Schematic diagrams of dikes constructed by these methods are shown in Figure 64. The method selected will depend on the long-range plan for the disposal area, the type and engineering properties of both the foundation and hydraulic fill, and economics.

Use of toe, transverse, and end dikes

254. The construction of retaining dikes with hydraulic fill often requires the construction of toe dikes (as shown in Figure 65) containing sluices parallel to and along the outer edges of the main dike to confine the fill within the desired area and retain the discharge water until it can be released to natural waters as a pollutant-free effluent. Transverse dikes, also shown in Figure 65, are usually provided across the main dike alignment to separate the long, relatively narrow fill area into smaller fill areas. This is done to provide sufficient ponding or retention time within each area for optimum soil retention, to control channelization, and to help confine the hydraulic fill to desired slopes and grades. End dikes, also shown in Figure 65, are temporary retaining dikes constructed at canals, streams, or other crossings and are sometimes required to retain the fill until closure of the

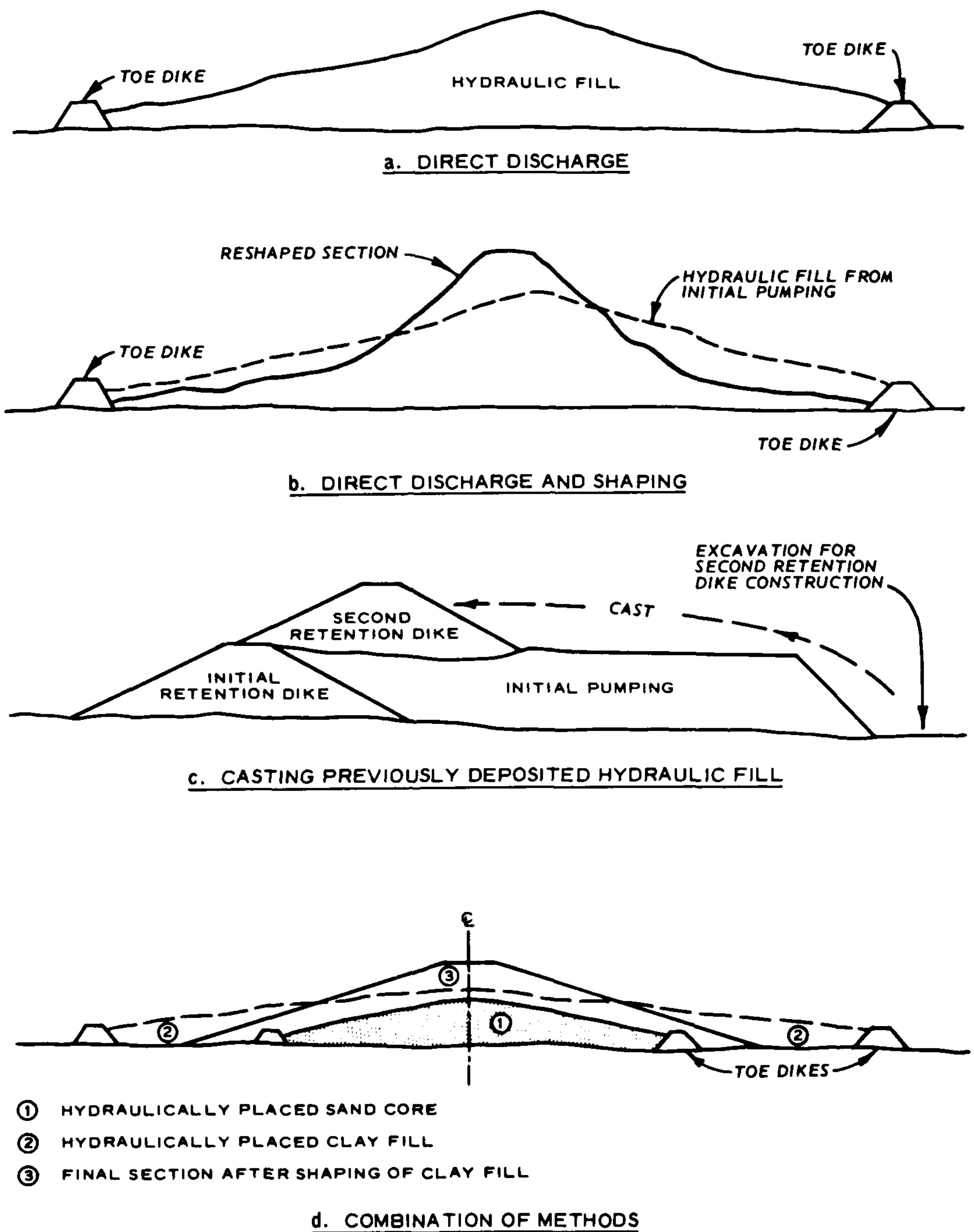


Figure 64. Dikes formed by hydraulic fill methods

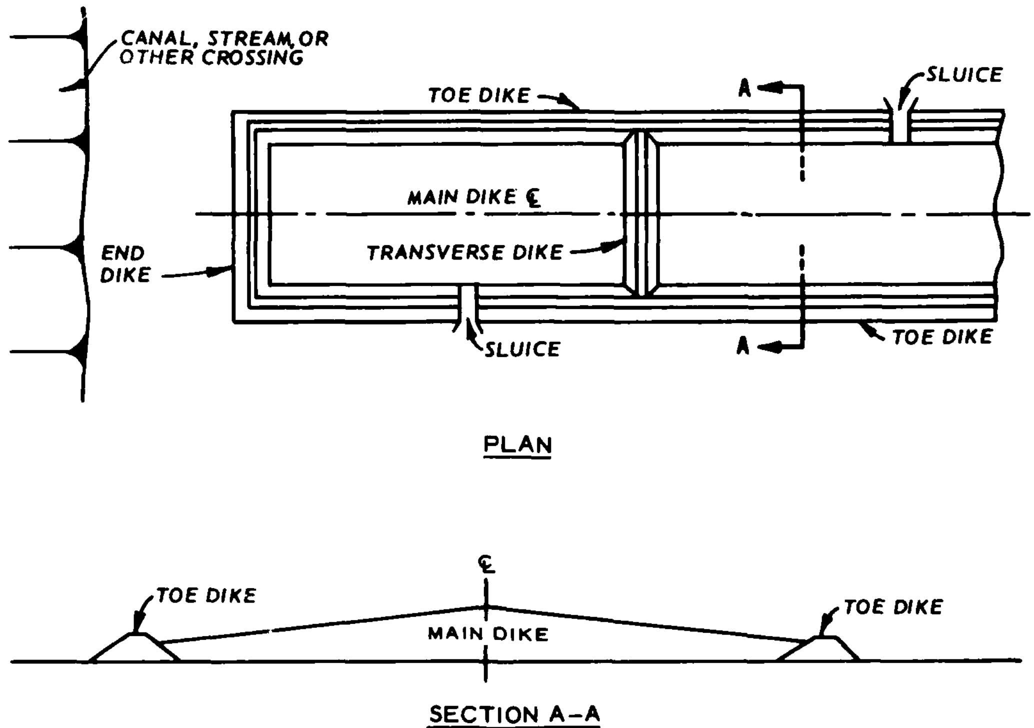


Figure 65. Toe, transverse, and end dikes

crossing can be made. Such crossings often require changes in construction techniques and/or material.

255. In some instances it may be feasible to construct the main hydraulic fill dike section without the aid of toe dikes on one or both sides. The feasibility of doing this will depend on the type of material being pumped and its angle of repose, adjacent land use and topography, and the possibility of adverse environmental effects of the unretained effluent on adjacent lands and water bodies.

Deposition of hydraulic fill

256. Hydraulic fill materials are placed directly in a retaining dike by the direct discharge method and in some cases by the bleeder pipe method. These methods are discussed briefly in the following two paragraphs. A more detailed discussion can be found in Huston.[41]

257. Direct discharge. The direct discharge method is the most commonly used procedure and involves release of the dredged material at the end of the discharge pipe as shown in Figure 66. Frequent moving

Figure 66. Release of hydraulic fill at discharge pipe

of the discharge pipe and/or adding lengths to the pipe are necessary when this method is utilized. By strategically locating the discharge pipe, the best materials can be located in the desired section of the dike. This is because the coarser or better materials settle out near the discharge while the finer particles remain in suspension longer and are carried further out.

258. Bleeder pipes. A bleeder pipe is a discharge pipe with holes on the underside varying in size from 2 × 2 in. to 6 × 6 in. The discharge line is place along the center line of the proposed dike and is supported on cribbing or piling. During pumping the heavier materials drop out as they come to the holes, but the finer particles that are in solution flow on past and out the line to a ponding or disposal area. This method is used primarily in the placement of sand since clay in the form of clay balls will tend to plug the bleeder holes. This procedure is sometimes used around utility crossings on soft foundations where the fill height must be brought up uniformly on each side of the crossing to prevent shear failure and/or lateral displacement of the utility.

Construction control

259. As with other methods of construction, the importance of construction control cannot be overemphasized. The following paragraphs list some of the main duties of an inspector during a dredging and hydraulic filling operation for dike construction.

260. Before initiating dredging, field personnel should:

a. Understand fully the method of operation to be used by the contractor.

b. Understand fully the methods of communications to be used between the dredge and discharge area.

c. Verify that the discharge facilities are constructed in accordance with the plans and specifications.

d. Verify that foundation preparation is adequate.

e. Verify that alignments and elevations are properly established.

f. Verify that toe dikes are constructed as required by the plans and specifications.

261. After dredging is commenced, field personnel should continuously:

a. Inspect toe dikes to ensure that they are being properly maintained.

b. Check toe dikes to see that they are not being overtopped and that design freeboard is being maintained.

c. Monitor the quality of the dredged material to see that it is as specified and that the dike section is being constructed as designed.

d. Observe the overall operation to ensure that no potential hazard is being created.

e. Monitor the quality of the effluent to see that it meets the specification requirements.

f. Check the discharge facilities (spill boxes) as this is probably the weakest point in the toe dike system. Included also should be the control of effluent on the outside of the toe dikes.

Foundation Preparation

262. Included in foundation preparation are clearing, grubbing, stripping, and final foundation preparation. A particular dike project

may include one or all of the above items, depending on site conditions and method of construction. In the past many retaining dikes have received no foundation preparation at all. However, it is considered that some degree of foundation preparation is desirable and necessary to help ensure the integrity of the structure. Clearing and grubbing is considered minimum foundation preparation and should be accomplished, where necessary, for all dike projects. In marshy areas where a surface mat of marsh grass and roots exists over underlying soft clays, experience has shown it is often more beneficial from a stability standpoint to leave it in place than to remove it. However, it should be remembered that such a mat is essentially pervious and may not be beneficial from a seepage standpoint. Measures to deal with this were discussed in Part VII.

Clearing

263. Clearing consists of the complete removal of all objectional and obstructive matter above the natural ground surface. This includes trees, fallen timber, brush, vegetation, abandoned structures, and similar debris. The dike foundation area should be cleared well ahead of any subsequent construction operations. Clearing should be required for all dikes except as previously noted.

Grubbing

264. Grubbing consists of the removal of stumps, roots, buried logs, and other objectional matter. All holes and/or depressions caused by grubbing operations should have their sides flattened and be backfilled in lifts up to the foundation grade with compacted fill. This will avoid soft spots under the dike and maintain continuity of the natural foundation blanket. Grubbing should be required for all compacted dikes and dikes on fairly firm foundations. It is often impractical to grub on very soft foundations.

Stripping

265. After clearing and grubbing operations have been completed, the dike area is stripped to remove low-growing vegetation and organic topsoil. The depth of stripping is determined by local conditions and usually ranges from 6 to 12 in. Stripping is normally limited to the

dike foundation proper and is not necessary beneath stability berms. All stripped material suitable for use as topsoil should be stockpiled for later use on dike slopes. Stripping is not normally required for dikes on soft, wet foundations or for dikes built by methods other than compacted.

Disposal of debris

266. Debris from clearing, grubbing, and stripping operations can be disposed of by burning in areas where permitted. Where burning is prohibited, disposal is usually accomplished by burial in suitable areas such as old sloughs, ditches, and depressions outside the embankment limits. Debris should never be placed in locations where it may be carried away by streamflow or where it may block drainage of an area. Material buried within the containment area must be such that no debris may escape and damage or block the outlet structure. All buried debris should be covered by a minimum of 3 ft of earth.

Final foundation preparation

267. Final foundation preparation consists of thoroughly breaking up the foundation surface in order to provide a good bond between the embankment and foundation. This treatment is only required for compacted dikes on firm foundations. Scarification of foundation surfaces that are adversely affected by remolding (soft or sensitive foundations for instance) should not be accomplished. Scarification should take place just prior to fill placement in order to avoid saturation by rainfall. No fill should be placed on frozen surfaces.

Construction control

268. Since the particular foundation preparation techniques vary considerably with project site conditions, design, and construction method, it is not practical to include a detailed checklist. It should suffice to reiterate the importance of proper foundation preparation on the integrity of the structure. The base of a dike is often its weakest point from the standpoint of shear strength and seepage; therefore, it is imperative that procedures in the plans and specifications be followed as closely as possible. This can only be accomplished by close, continual inspection. If specified foundation preparation

procedures seem to be inadequate or for some other reason do not appear to be in the best interests of the project, the designer should be immediately consulted. Changes in specified procedures and requirements should not be made without concurrence of the designer.

PART IX: MISCELLANEOUS FEATURES

Discharge Facilities

269. Discharge facilities, sometimes called outlet structures, sluices, or spill boxes, are provided in retaining dikes for the purpose of controlling the release of excess water from the disposal area. This control is necessary to increase detention time which, in turn, facilitates efficient retention of solid particles and release of an effluent containing as few solids as possible back into natural waters. Control of effluent is normally regulated by allowing water to flow over a variable height weir constructed within the retention dike.

270. There are several types of discharge facilities in common use today. The following paragraphs contain a brief discussion of each type and pertinent items that should be considered in the design and construction of these structures. The purpose of the following discussion is not to treat the design of the discharge structure itself, but to study the effect of the structure on the diking system in order that associated problems may be avoided.

Types of discharge facilities

271. Outfall pipe. The simplest discharge facility is termed an outfall pipe (or pipes) placed horizontally within the dike, usually near the crest (Figure 67). As the level of slurry in the retention area rises, the upper portion runs off through the pipe. This type of facility provides no variable discharge level control, thus no control is possible over detention time and effluent quality. Also, it is quite easy for the pipe to become clogged and thus totally ineffective. This method of discharge is therefore not recommended for use as a primary means of discharge and should be limited to use as a temporary measure (in toe dikes, for instance) or to provide supplementary drainage through cross dikes within large disposal areas.

272. Drop-inlet sluice. A drop-inlet sluice such as that shown in Figure 68 is the most commonly used type of discharge facility. It basically consists of a vertical inlet connected to a discharge pipe

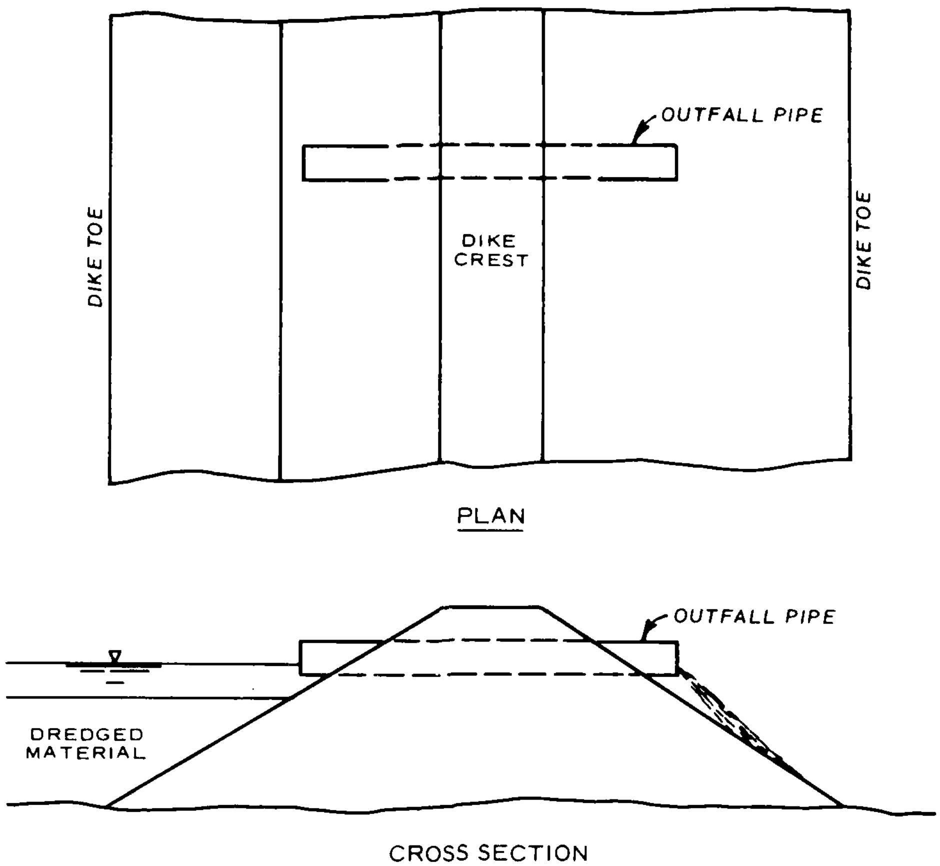


Figure 67. Outfall pipe

that leads from the base of the riser through the dike to the exterior. The inlet structure consists of a rectangular wood- or metal-framed riser or of the more common half-cylindrical corrugated metal pipe riser as shown in Figure 69. Both types of risers achieve variable inlet elevation control through the use of a gate of stoplogs (also termed riser planks and flashboards) which can be added or removed as necessary to raise or lower the inlet elevation (Figure 69). Various degrees of sophistication are achieved to this basic form by the use of multiple inlets and/or multiple discharge pipes as shown in Figure 70. Drop-inlet sluices are economical and competent as long as proper design and installation techniques are employed (subsequently discussed).

273. Box sluice. The box sluice or flume-type discharge

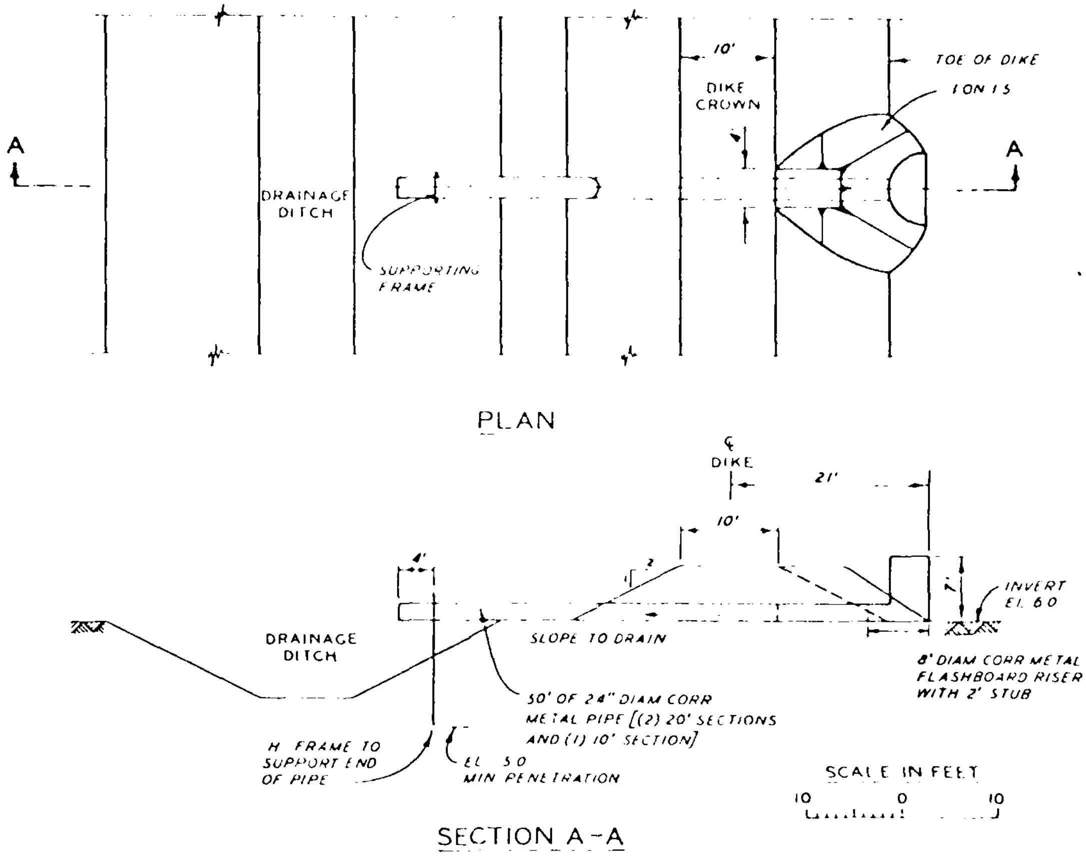
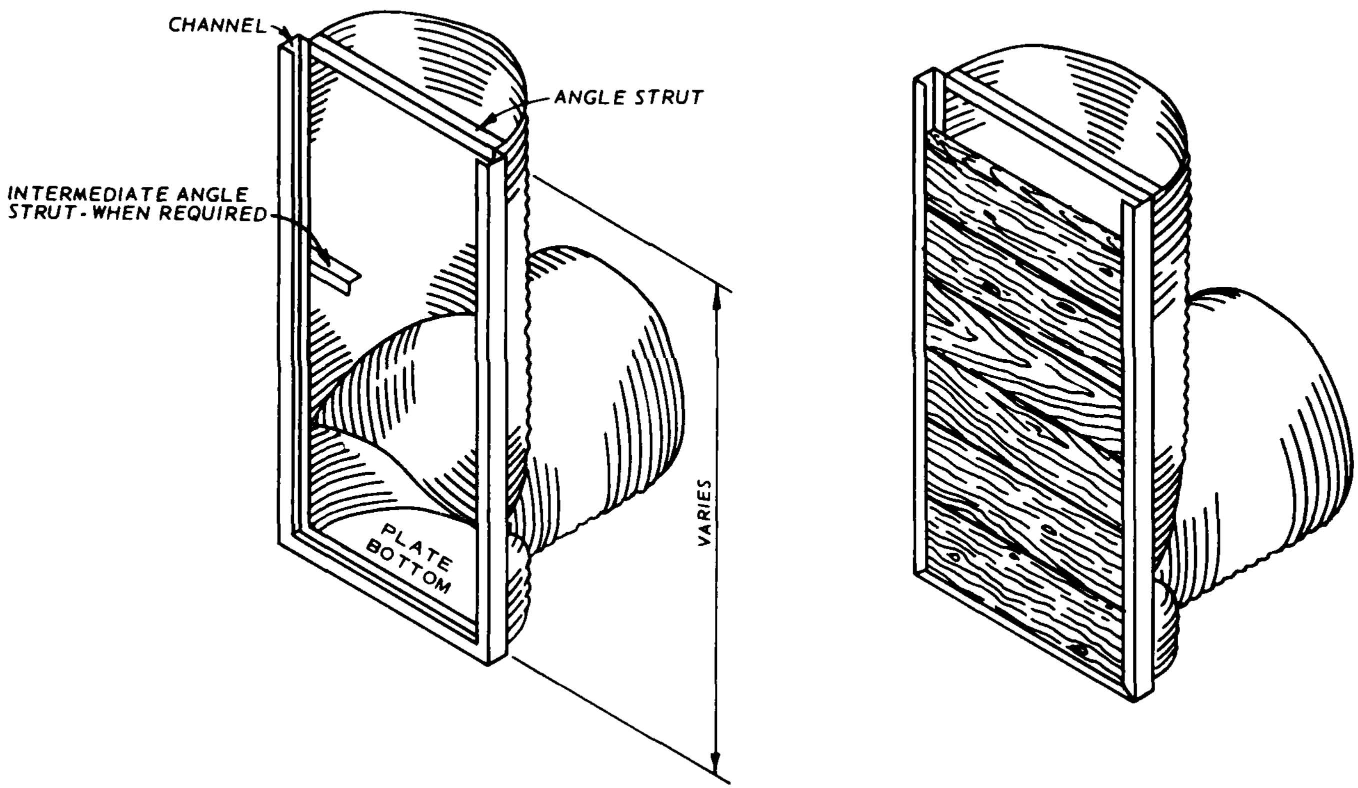


Figure 68. Typical drop-inlet sluice

facility as shown in Figure 71 consists of an open cut through the entire dike section. The cut is usually lined with timber, but could be lined with concrete or steel. Box sluices also provide for variable inlet elevations through the same means as drop-inlet structures (i.e. through the use of stoplogs). This type of structure is normally used where a large volume of discharge is required, but becomes more uneconomical as the dike section becomes wider. Timber is the most economical material for use in box sluices, but has the disadvantage of being susceptible to rot where the timber is untreated and is not inundated. At least one failure of a timber box sluice has been attributed to rotting of the timber. Box sluices have another disadvantage in that there exists a large contact area between the structure and adjacent soil that is susceptible to seepage and piping. For the above reasons, box



a. WITHOUT RISER PLANKS (STOPLOGS)

b. RISER PLANKS IN PLACE

Figure 69. Typical variable elevation inlet structure (courtesy Armco Steel Corp.)

a. Inlet

b. Outlet

Figure 70. Drop-inlet discharge structure with multiple inlet/outlet

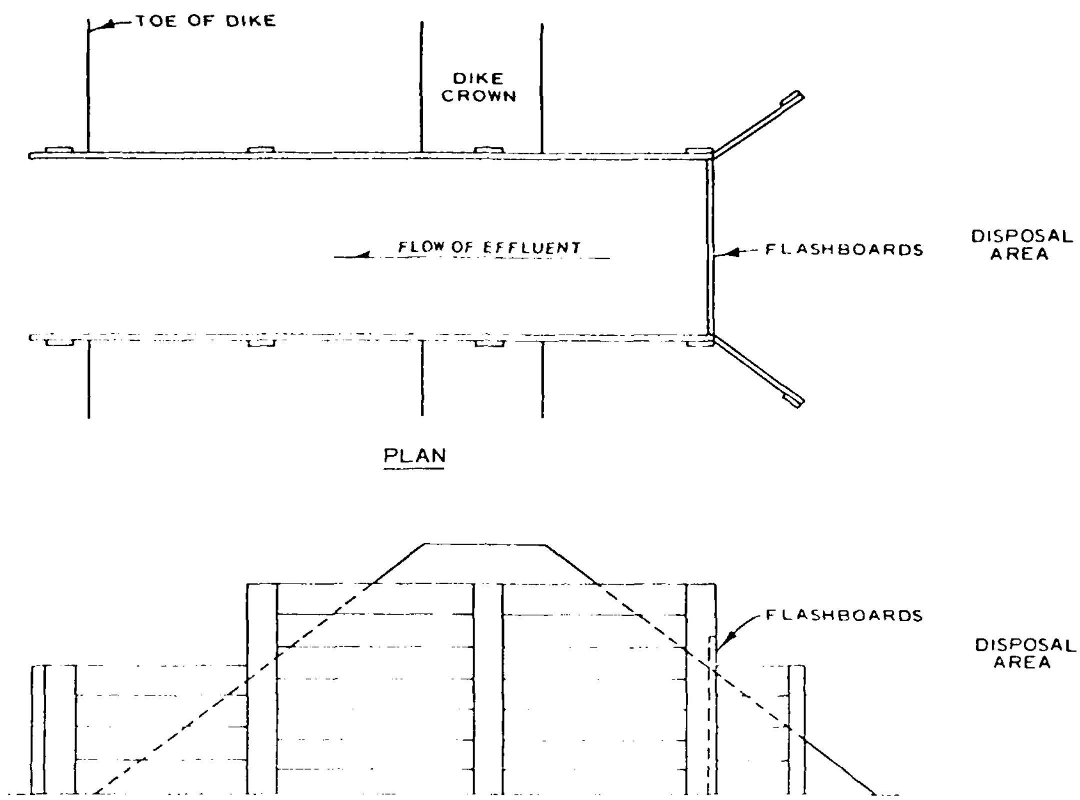


Figure 71. Box sluice or flume (reproduced, with modification, with permission from Hydraulic Dredging, by John Huston, Copyright 1970, by Cornell Maritime Press, Inc., Cambridge, Md.)

sluices are not often employed as discharge facilities in diking systems.

274. Filters. Filters composed of granular materials are another form of discharge facility sometimes used in diking systems. The filter separates out contaminant particles, while at the same time allowing release of the clean effluent. The filter may take the form of the dike material itself or may be a separate structure installed within the dike. Filters are usually employed where retention areas are of insufficient size to handle the volume of inflow of dredged material and release a clean effluent (i.e., detention time is insufficient to allow the pollutants to settle out prior to discharge). Usually some means of preventing clogging of the filter must be used such as filter cloth that can be removed and cleaned. Disadvantages of filters are

their susceptibility to clogging, required maintenance, sometimes complex design and construction (in the case of multigraded filters), and low flow capacity per unit of area. The use of filters has not been widespread to date except in the Great Lakes region, and it is suggested that if more detailed information is required, the U. S. Army Engineer District, Buffalo, be contacted. Krizek et al.,[45] which contains results of an investigation into effluent filtering systems for containment areas, should also be consulted.

General design considerations

275. Discharge facilities are generally the most vulnerable point in a diking system because of the possibility of seepage and piping of the soil at the soil-structure interface. Experience has shown that many dike failures have been initiated by seepage and piping along the sluice-dike contact with box sluices and around discharge pipes for drop-inlet structures. The uncontrolled discharge of effluent on the outside of the dike, as well as differential settlement of the structure, can also lead to failure. Consequently, special consideration should be given to both the design and construction of discharge facilities and dike sections in the area of such structures. The following paragraphs contain a brief discussion of some of the more pertinent items for consideration.

276. Materials. The material of which a discharge facility is to be composed should be selected based primarily on economy and on its resistance to deterioration relative to the project life. The corrosive nature of the effluent should be determined as many types of dredged material will be contaminated and many may well contain chemicals that will attack certain materials while being inert to others. All wood should be pressure-treated to resist rotting, and all metals should be galvanized and bituminous-coated to reduce the possibility of corrosion. Where deformation of pipes is anticipated, corrugated metal is preferred due to its greater degree of flexibility.

277. Pipes. The selection of pipes for drop-inlet structures should be based on economy, the substance to be carried, imposed loadings, and the effects of anticipated settlements and foundation creep.

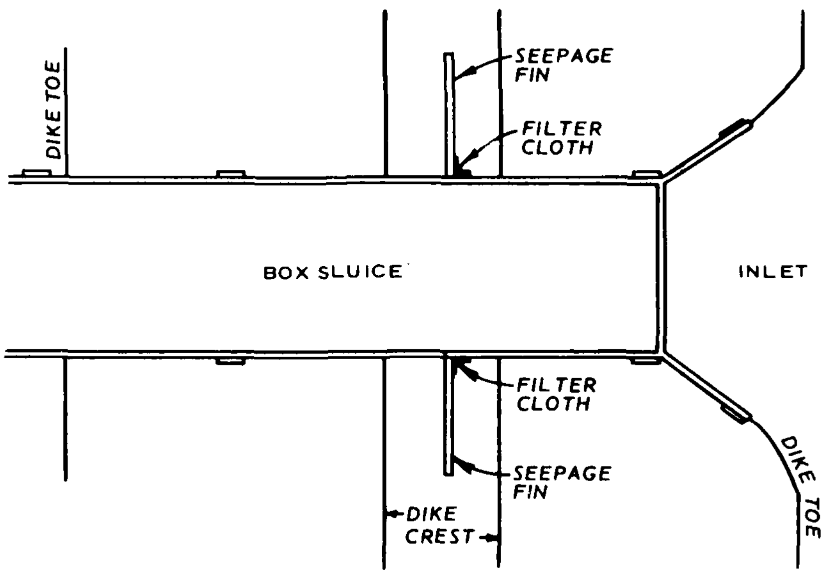
Although economy must be given weighty consideration, the overriding factor must be the prevention of failure. The earth load acting on a pipe must be determined as well as live loads imposed by the operation of equipment during construction (often overlooked) and by maintenance equipment or other traffic subsequent to construction. Pipe manufacturers' organizations have recommended procedures for determination of these loads as well as the strengths of standard commercially available pipe. These recommended procedures and strengths should be employed unless, for some other reason, more stringent requirements must be met.

278. Pipe joints. Leakage from or infiltration into any pipe passing through or under a dike must be prevented. The expected settlement and/or outward movement that could cause elongation of the pipe must be considered so that proper measures are taken to avoid pulling apart of the joints. Corrugated metal pipe sections should be joined by exterior coupling bands with a gasket to ensure watertightness. Where concrete pipe is to be used and considerable settlement or creep is anticipated, a pressure-type joint with concrete collars should be used. These collars must be designed either to resist or accommodate differential movement without losing watertight integrity. Where movements are not thought to be significant, pressure-type joints capable of accommodating minor movements are sufficient. Cast iron and steel pipe should be fitted with flexible bolted joints. Steel pipe sections may also be welded together to form a continuous conduit.

279. Seepage control. Antiseepage devices to prevent seepage and piping along the outside wall of pipes were frequently used in the past where pipes passed through or under embankments. These devices usually consisted of metal diaphragms (seepage fins) or concrete collars that extended out from the pipe into the backfill material and were often termed "seepage rings." However, many piping failures have occurred in the past where seepage fins or rings were used. Assessments of these failures have indicated that the presence of these devices often resulted in poorly compacted backfill at the soil structure interface, thereby causing more harm than good. Therefore, seepage rings for pipes

should not be relied upon as being effective in preventing seepage and/or piping adjacent to the pipe. Seepage rings should be provided only as necessary for coupling of pipe sections or to accommodate differential movement on yielding foundations. When needed for these purposes, collars with a minimum projection from the pipe surface should be used.

280. Seepage fins for box sluices such as those shown in Figure 72 should be used to aid in the prevention of seepage and piping at the sidewall-soil surface. These fins should be located under the dike crest and should extend to the full height of the structure, being placed at right angles to the structure. Their length should be a minimum of 5 ft, and, if a joint exists at the fin-structure junction, it should be covered with filter cloth on the inlet side of the structure.



NOTE: SEEPAGE FINS SHOULD EXTEND FULL HEIGHT OF BOX SLUICE SIDEWALLS AND SEEPAGE FIN-SIDEWALL JOINT SHOULD BE COVERED WITH FILTER CLOTH.

Figure 72. Timber box sluice with seepage fins

281. To aid in the prevention of piping along a pipe-soil interface, an 18-in. minimum annular thickness of drainage fill should be provided around the outlet one-third of the pipe as shown in Figure 73. This may be omitted where the outlet one-third of the pipe is located in sand. For pipe installations within the dike foundation, a drain must be provided from the drainage fill to a suitable exit.

282. Consideration should also be given to widening dike sections at discharge facility locations. This will not only lengthen the seepage path but will provide a more stable section against the possibility of a shear failure at these critical locations.

283. Settlement control. The alignment of a discharge structure must be such as to provide a continuous slope toward the outlet. Settlement of the dike and foundation can significantly alter the initial grade line of the structure, however, and can result in a swag in the structure. This is especially critical in the case of pipes under or through dikes, since it can result in sediment buildup in the swag that may eventually cause clogging of the pipe (Figure 74). The anticipated settlement of the dike should therefore be considered in establishing the initial grade line. If the settlement is of such a magnitude as to result in a significant upward gradient in the direction of flow or will not allow the desired gradient to be maintained, the pipe should either be cambered or raised as shown in Figure 75. Depending on the time required for the settlement to occur, this may result in no flow of the initial depth of effluent, but this is usually not detrimental and even may be advantageous from the standpoint of aiding in the prevention of channelization during initial pumpings.

284. The amount of camber required can usually be taken as the mirror image of the settlement curve along a line established by the final required grade. As previously mentioned, corrugated metal pipe is generally preferred where cambering is necessary due to its flexibility. Regardless of the type of pipe chosen, movements at the joints must be considered and measured and steps taken to prevent leakage (as discussed in paragraph 279).

285. Where some settlement is expected but not enough to justify

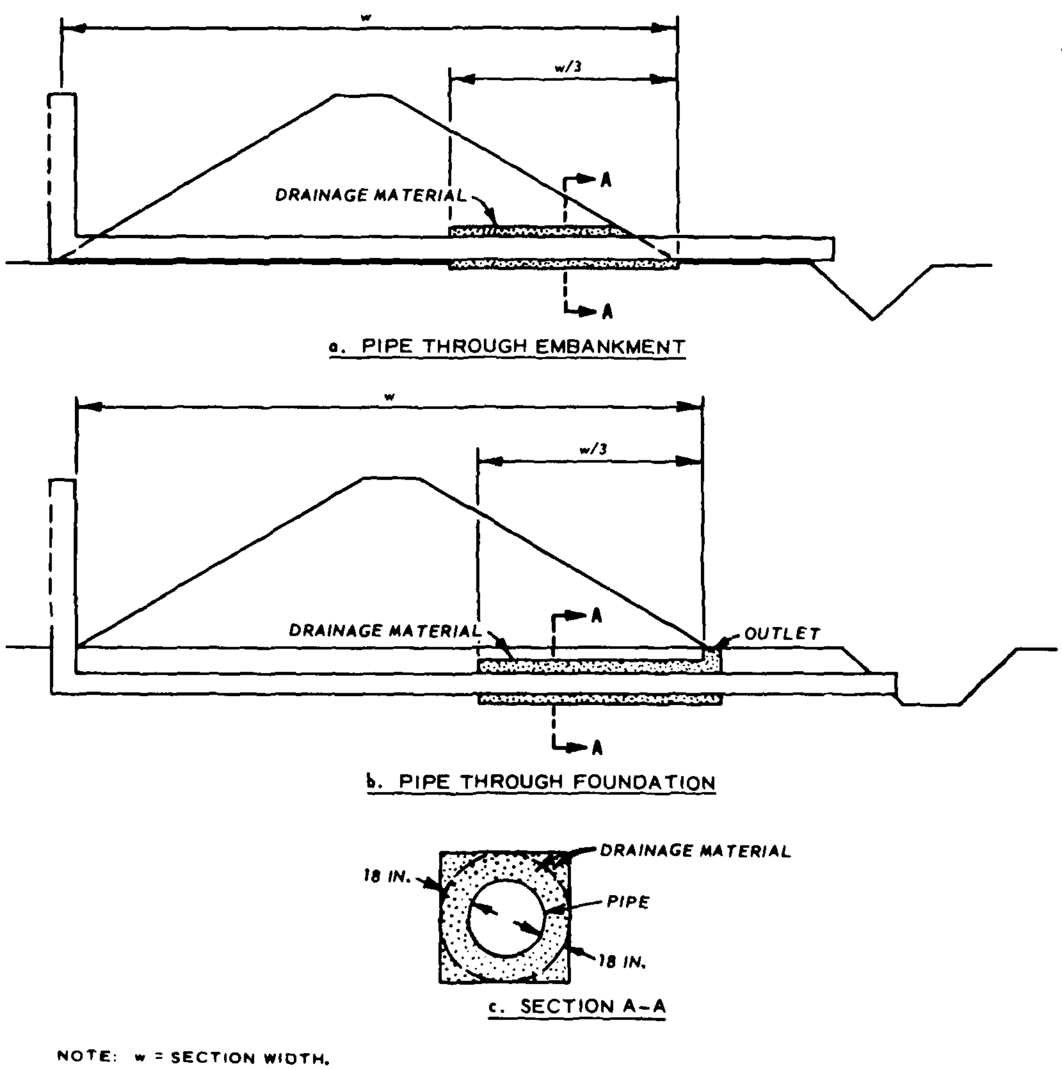


Figure 73. Annular drainage material around outlet one-third of pipe

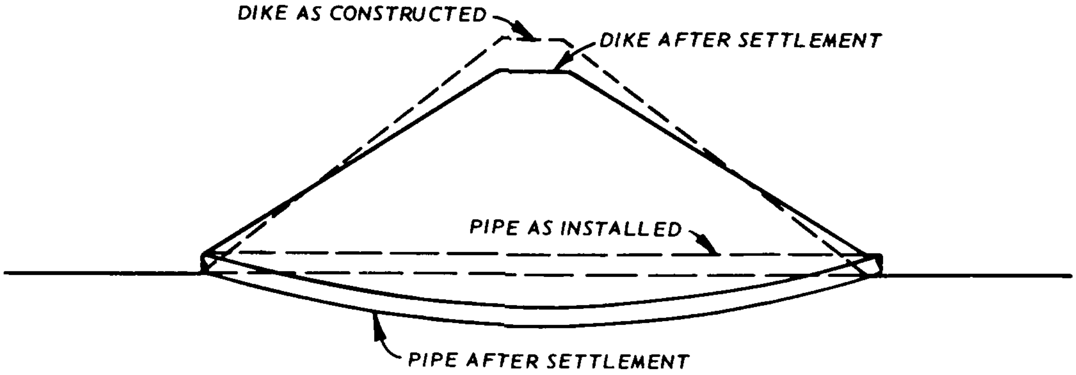
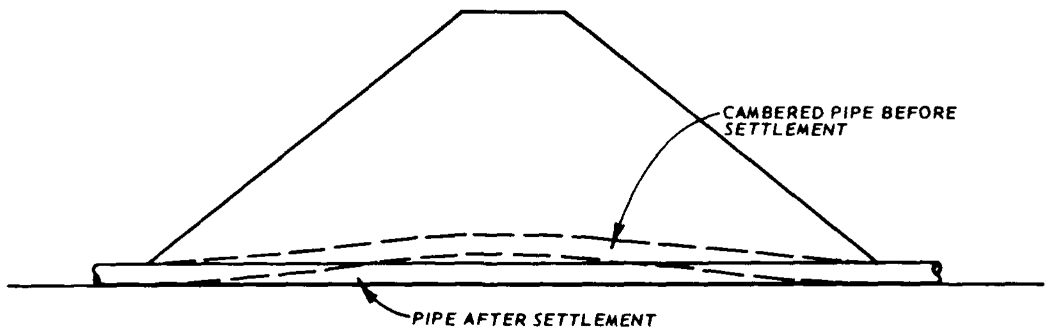


Figure 74. Swagging of pipe due to settlement of dike and foundation

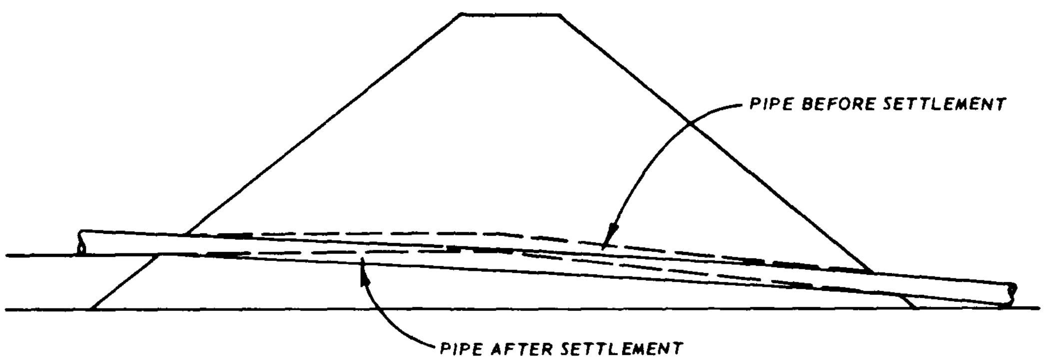
cambering, a larger section of pipe can be specified. This will allow the required flow to be maintained even if some swagging occurs and part of the pipe is filled with sediment.

286. Size of excavation. Almost all discharge facilities are installed by the open-cut method. The trench for these structures should be excavated to a depth of at least 2 ft below the bottom of the structure and to a width wide enough to allow the use of heavy compaction equipment for backfilling of the trench.

287. Sequence of construction. Preferably, the dike should be built up to a grade of at least 2 ft above the crown of the pipe or bottom of the structure prior to excavation. This allows the foundation soil to be preconsolidated somewhat before excavation and installation of the structure. After excavation, the trench should be backfilled with properly compacted material (subsequently discussed) to the structure invert elevation. After installation, backfill should be selectively placed back to the existing dike grade before beginning normal fill operations for the dike. This is especially important in the case of hydraulic fill dikes in order to protect the structure from scour that could be caused by the dredge water. Also, this will provide cover to aid in preventing damage to the structure from heavy equipment passing over the area.



a. CAMBERED PIPE



b. RAISED PIPE

Figure 75. Cambered and raised pipe beneath dike

288. Compaction. Compaction of backfill around and adjacent to the structure is of utmost importance as it probably plays the single most important part in the proper installation of the structure. Backfill should be compacted to 95 percent standard density for impervious soil and to an average relative density of 85 percent and a minimum of 80 percent for pervious soil. Heavy compaction equipment should be used as close to the structure as possible without causing damage to the structure. However, hand tamping will usually be required for soil immediately adjacent to the structure. Hand tamping should be accomplished by power-driven tampers that develop enough pressure to attain the previously given density requirements. Loose lift thickness

for hand-tamped soil should be about 4 in. It is important that the hand-tamped zone be extended away from the structure far enough to overlap with the soil being compacted by heavier equipment. There has been at least one documented failure where a piping failure along a pipe was attributed to a loose zone of backfill caused by the lack of overlap between these two zones.

289. Backfill around structures, especially pipes, should be brought up evenly on both sides in order to avoid unequal side loads that could cause distress in the structure. Also, special care must be taken in the vicinity of any joint collars or other protrusions to ensure proper compaction.

290. It is preferable that impervious backfill material used adjacent to pipes and sidewalls be placed on the wet side of optimum moisture content. This will result in a more plastic material and will allow the soil to be squeezed in against the structure as well as provide a material less susceptible to cracking.

291. Dewatering. In order to achieve the above described placement procedures, it is necessary that construction of discharge facilities be done in the dry. This will require dewatering where the structure is to be founded below the water table. Where this is necessary, the excavation should be kept dewatered until backfill is placed to at least 2 ft above the water table or until the structure is adequately covered with fill, whichever is greater.

292. Other considerations. Since many dikes are eventually raised, the effect of enlargements on the discharge facility should be considered; otherwise, the structure will have to be abandoned, raised, or relocated. This is especially important when sizing and locating inlets for discharge facilities. Here again, the need for long-range planning in the design of disposal areas is made evident.

293. Where dikes are located adjacent to waters subject to flooding, consideration should be given to providing flapgates over the structure outlet. This will prevent backflooding of the disposal area in the event of flooding on the outside of the area. The use of flapgates has one proven disadvantage--they often become inoperative due to

clogging with debris, mechanical malfunctions, or vandalism and cannot close when floodwaters rise. Therefore, when flooding seems imminent, the flapgates should be checked to be sure that they are in good working order.

294. Scour and erosion from discharge at the outlet, which can eventually work back and deteriorate the dike, have previously been discussed in paragraph 197.

295. During construction, the inlet and outlet ends of pipes passing under or through dikes should be covered or plugged to prevent filling and clogging during construction of the dike. This should be done regardless of the type construction being used for the dike but is especially critical where the dike is being built by hydraulic fill methods.

296. Construction inspection. Due to the critical nature of discharge facilities, close inspection of all facets of their installation is necessary in order to ensure proper construction. Items to which particular attention should be paid include: (a) handling and placement or forming and pouring of the structure, (b) proper grade, (c) joint installation of pipes, and (d) placement and compaction of backfill.

Utility Lines Traversing Disposal Areas

Problems

297. The term utility lines refers to pipelines or conduits usually carrying gases or fluids, sometimes under pressure. Since many disposal areas are located in or near industrial areas, utility lines frequently must be contended with in the planning and design of disposal areas. Problems associated with utility lines fall into two general categories: damage to the dike caused by the utility line and damage to the utility line caused by construction of dikes and the presence of dikes. Therefore, existing or planned utility lines traversing the disposal area should be given careful consideration during design.

298. The problems caused by pipes or conduits passing through or under dikes have previously been addressed in connection with discharge

facilities. However, discharge facilities are gravity drainage structures and, in addition to problems previously discussed, the problems that could result from pipes carrying gases or fluids under pressure are even more critical, especially in relation to leakage or rupture.

299. Damage to the utility lines themselves, resulting from dike construction or the presence of the dike, could be caused by: (a) movement and/or digging of construction equipment, (b) scour from the dredge water when hydraulic fill construction is being employed, (c) differential settlement across the lines, and (d) shear straining or creep.

Alternatives

300. The following items are possible alternatives to the problem of utility lines traversing a disposal area:

a. Leave the pipelines in place and build the disposal area over them.

b. Relocate the pipelines outside the disposal area.

c. Relocate the pipelines over the disposal area.

d. Let the pipelines remain in the disposal area, but take special measures for their protection and protection of the dike.

e. Combination of the above.

301. In order to leave a pipe or pipes in place and construct a disposal area over them, a thorough analysis to substantiate the following conditions is necessary:

a. The pipe is adequately deep or foundation characteristics are such that no damage to the pipe would be expected from construction activity.

b. The pipe can stand the stresses caused by the additional load, differential settlement, and/or foundation creep.

Only when the above conditions are satisfied can the decision be made that no relocation of the pipes is necessary.

302. Ideally, the easiest solution with respect to the disposal area design is to relocate the pipeline around or outside the area. However, this usually is the most difficult alternative to satisfy from an economic standpoint. Relocation over the area is another straightforward solution, but is also expensive. It does, however, become more attractive as the area to be bridged becomes narrower.

303. Intermediate or cross dikes through the disposal area and parallel to the pipeline as shown in Figure 76 can be employed to protect a pipeline from the stresses imposed by the weight of the dredged material. This method can be used in connection with the bridging method by bringing the pipeline up and crossing over the main dikes. In other words, intermediate dikes can be used within the disposal area and the pipeline taken over the primary dikes wherever a primary dike crossing is required. Requiring a pipe to cross over a dike as shown in Figure 77 reduces or eliminates many of the dangers to the dike and pipe inherent with pipes crossing through or under dikes. The only real remaining threat to the dike is one of scour or erosion of slopes in the event of leakage or rupture of the pipe. The threat of this occurring can be reduced by the use of special collars at pipe joints, providing slope and crest protection to the dike at the area of crossing, and providing special cutoff valves for the pipe that could be used in the event of heavy leakage or rupture. If possible, all pipes, especially pressure pipes, should be relocated over the dike.

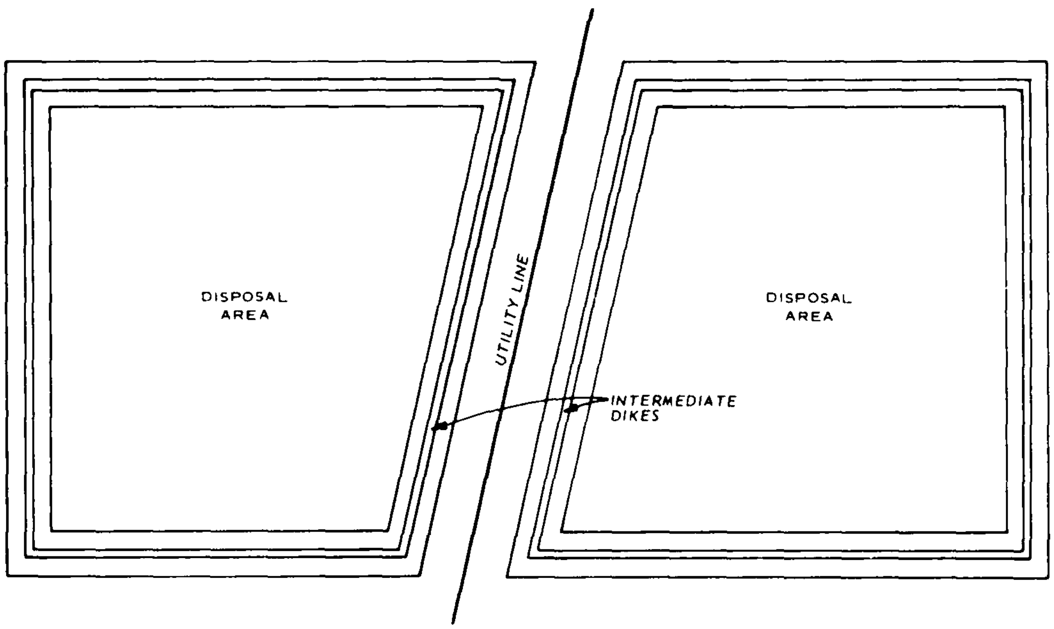


Figure 76. Use of intermediate dike to protect utility line

304. Regardless of which method of relocation is employed, all relocation of utility lines should be accomplished before dike construction.

Figure 77. Pipeline crossing dike

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APPENDIX A: SLOPE STABILITY ANALYSES

General

1. The purpose of this appendix is to present examples of slope stability analyses by the circular arc, conventional wedge, planes, infinite slope, and bearing capacity methods. The material contained herein is by no means intended to serve as a complete instructional guide to the user. Detailed descriptions of the various methods of analyses are contained in appropriate references given in the main text. Only the method of planes is described in any detail in this appendix as there exists no widely available published reference that contains a detailed description of this method.

Circular Arc

2. Plates A1 through A3 contain examples of stability analyses by the circular arc (Modified Swedish) method for the end of construction, steady seepage, and sudden drawdown conditions, respectively. Only the arc that yielded the minimum factor of safety (FS) is shown for each problem. The factors of safety for all other arcs run on the computer in searching for the minimum are plotted at their respective arc centers. Other sets of arcs that were run at different tangent (base) elevations to ensure that the minimum factor of safety was found are not shown.

Conventional Wedge

3. Plates A4 and A5 contain examples of stability analyses by the conventional wedge method for the end of construction and steady seepage conditions, respectively. Only the configuration (failure surface) that resulted in the minimum factor of safety for each complete analysis is shown for each problem. Other active and passive wedge locations tried at the same base elevation are shown by the dashed lines. Trial wedges run at other base elevations to ensure that the minimum factor of safety was found are not shown.

Method of Planes

4. In addition to the conventional wedge analysis, another wedge method known as the LMVD (Lower Mississippi Valley Division) method of planes, or just method of planes, is used extensively by the LMVD. This method is essentially the same as the conventional wedge analysis except that it is somewhat simplified. The main difference between the two methods is that a factor of safety is computed directly by the method of planes while successive iterations with trial factors of safety are required by the conventional wedge analysis. This means the method of planes is rigorous for a $\phi = 0$ analysis and generally only slightly in error for a c , ϕ , or $c = 0$ analysis.

5. The method of planes procedure is presented in Plates A6 and A7. This procedure entails dividing the failure area into three zones (an active wedge, neutral block, and passive wedge) and computing the driving and resisting forces for each zone or segment. The factor of safety is then computed by dividing the summation of the resisting forces by the summation of the driving forces. As must be done in the conventional analysis in order to ensure that the minimum factor of safety is found, the assumed depth of the failure plane and the location of the active and passive wedges must be varied. Plate A6 shows the general procedure of analysis while Plate A7 shows its application to a condition with underseepage and resulting hydrostatic uplift. An example problem using this analysis is shown in Plate A8. This example is not for a dike, but is an actual analysis of a Mississippi River levee. However, the procedure of analysis would be essentially the same for a retaining dike.

Infinite Slope

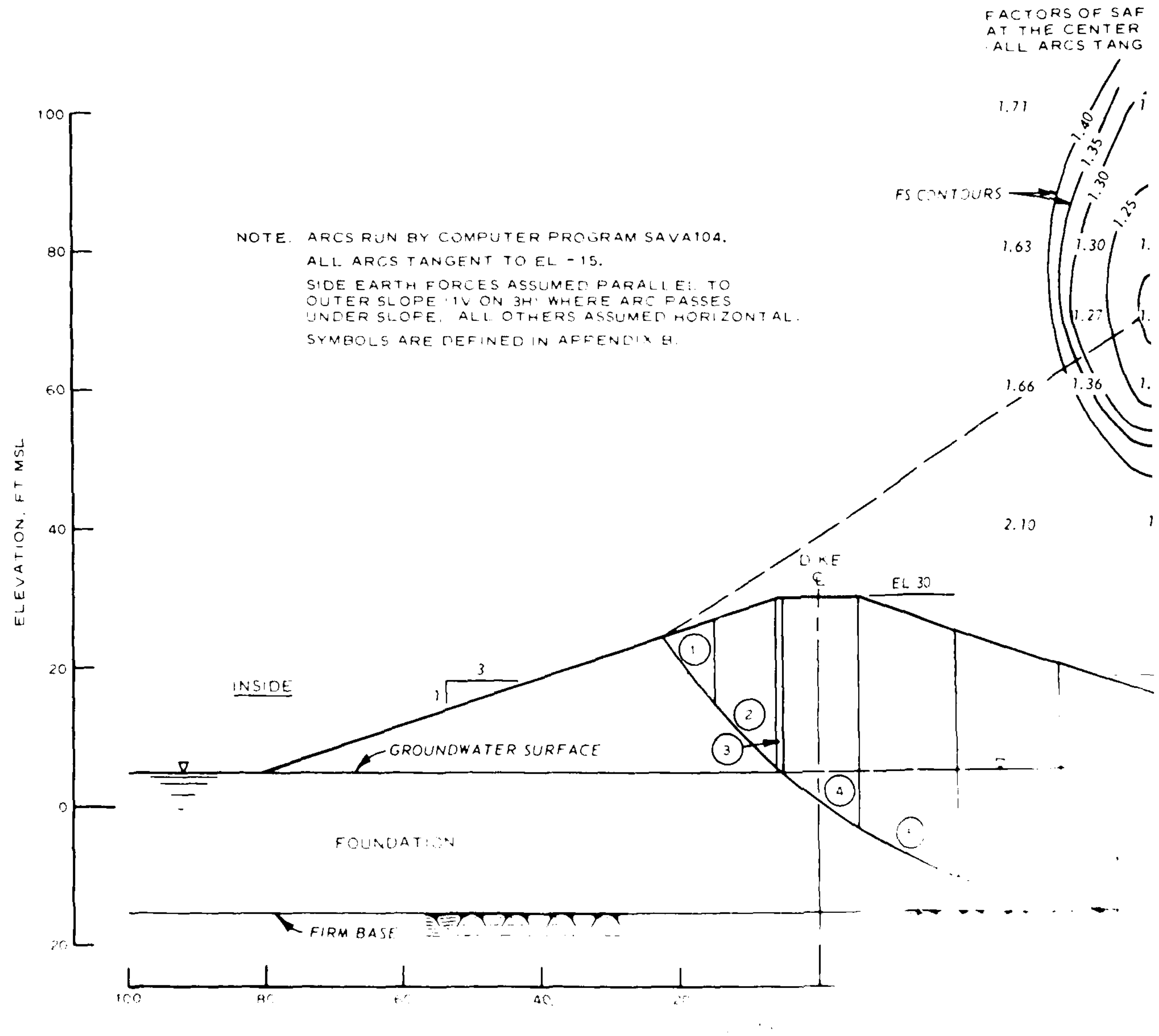
6. Plate A9 contains example problems analyzed by the infinite slope method for cohesionless soils as described in paragraph 124 of the main text. Example problem 7A (Plate A9) is an analysis without seepage while example problem 7B is an analysis of an embankment subject

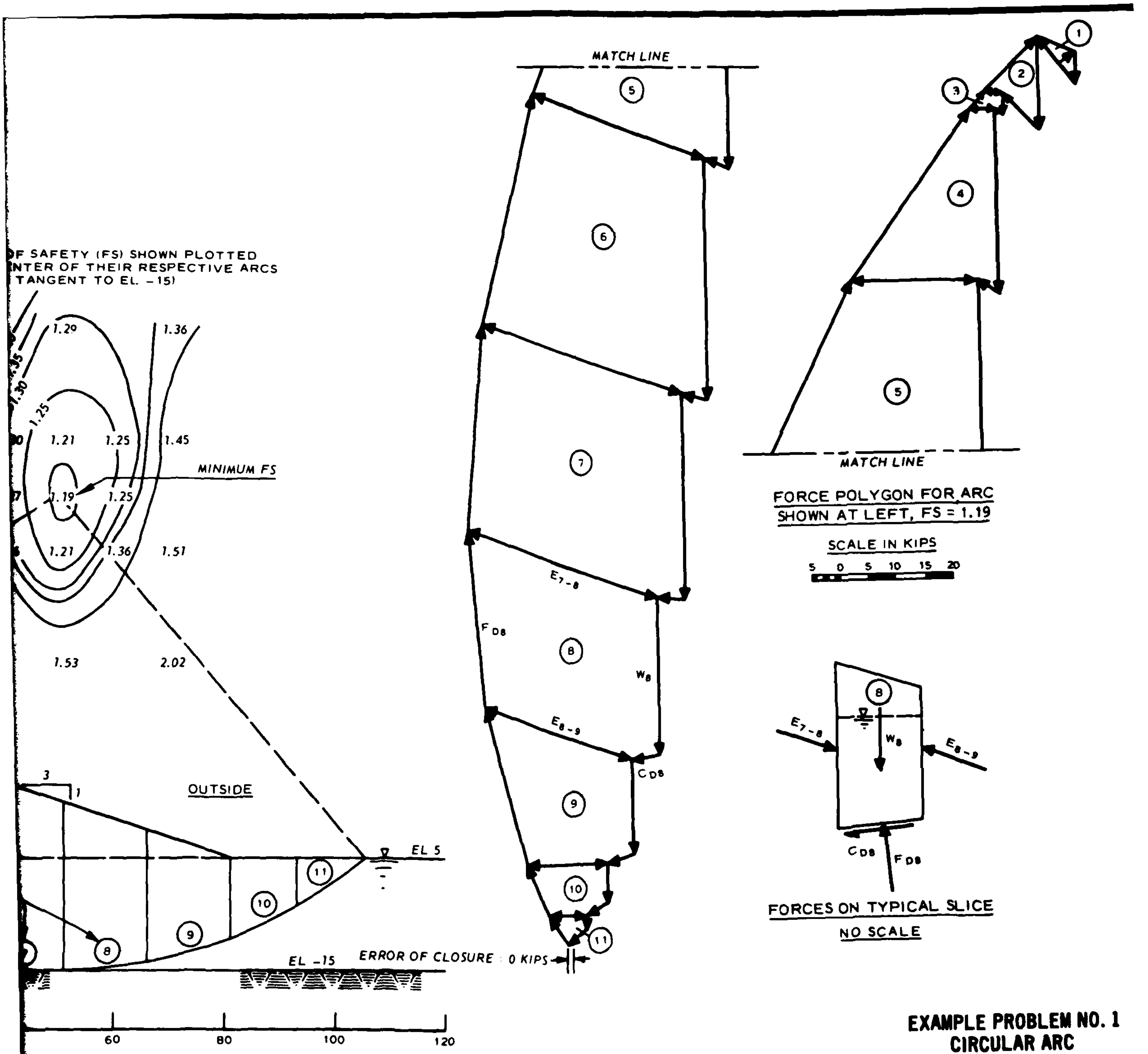
to steady state seepage assuming the phreatic surface is coincident with the outer slope, a very severe condition.

Bearing Capacity Equations

7. The equations and assumptions for this type analysis are given in paragraph 126 of the main text. Plates A10 and A11 contain analyses of two actual embankments using bearing capacity equations. Plate A12 is an influence chart for determining vertical stresses at depth due to embankment loading.

DESIGN SOIL PROPERTIES				
SOIL	UNIT WEIGHT, PCF		SHEAR STRENGTH	
	MOIST, γ_M	SATURATED, γ_S	C, KSF	ϕ, DEG
EMBANKMENT	115	125	1.0	0
FOUNDATION	—	115	0.4	0



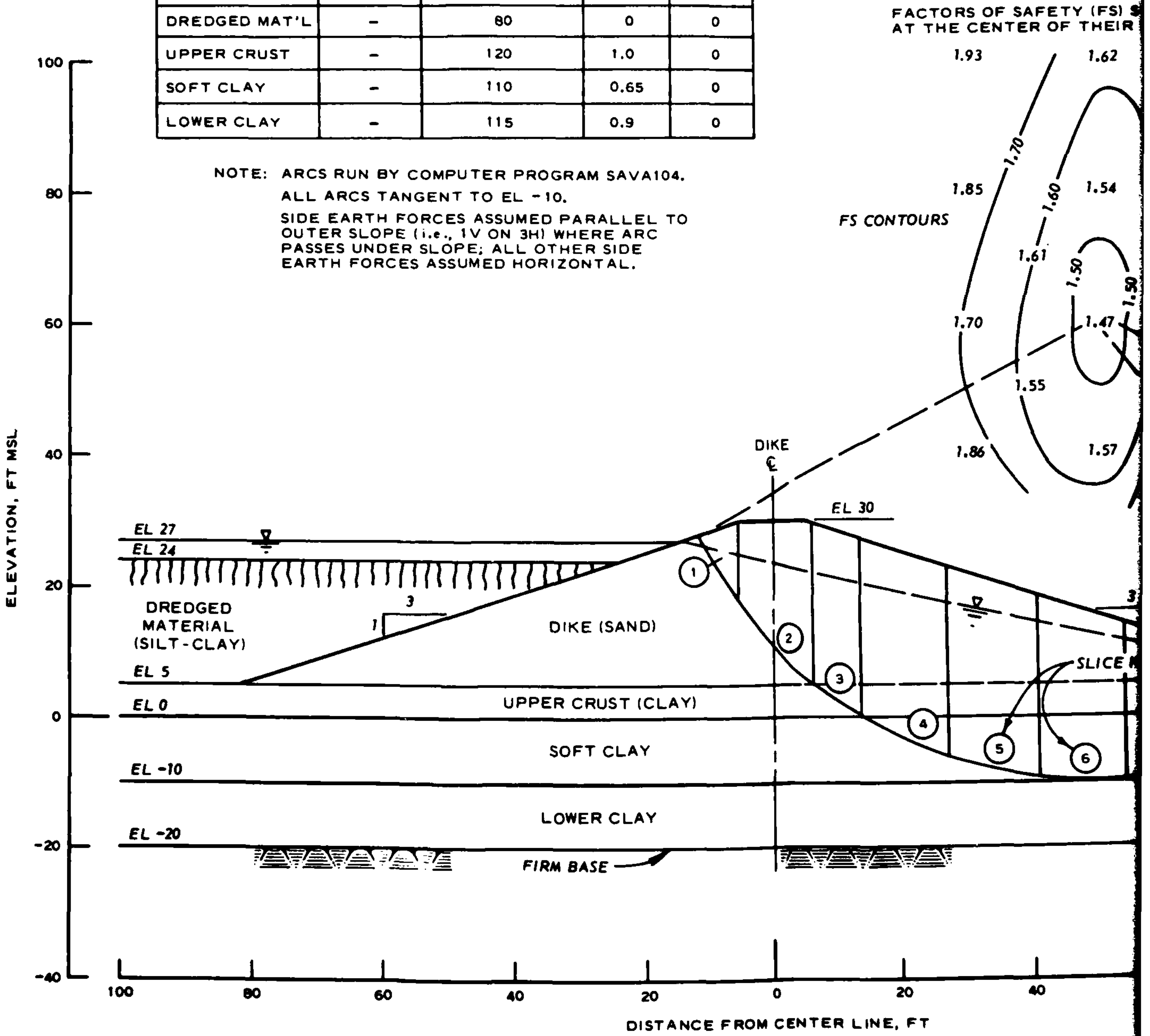


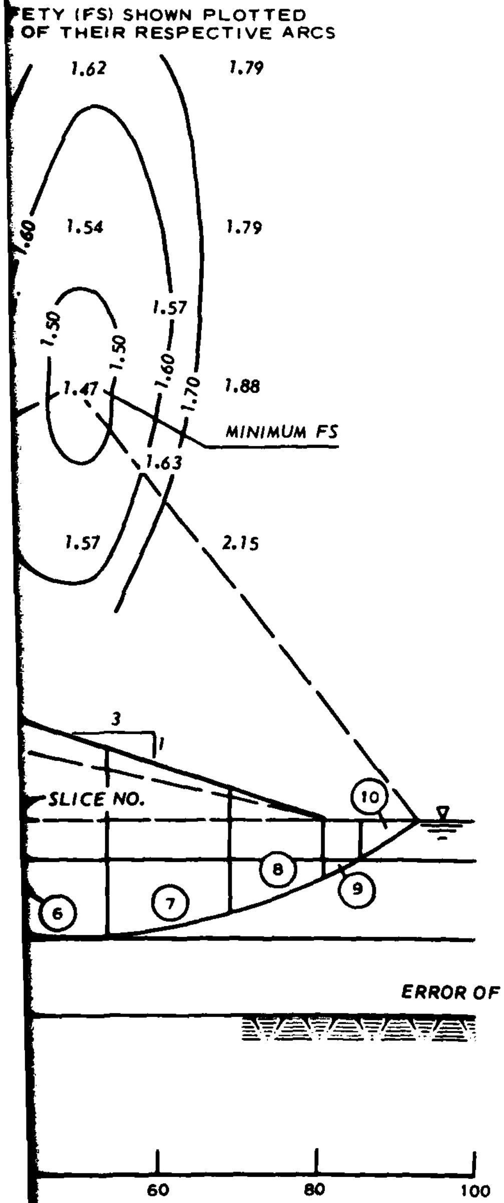
EXAMPLE PROBLEM NO. 1
CIRCULAR ARC

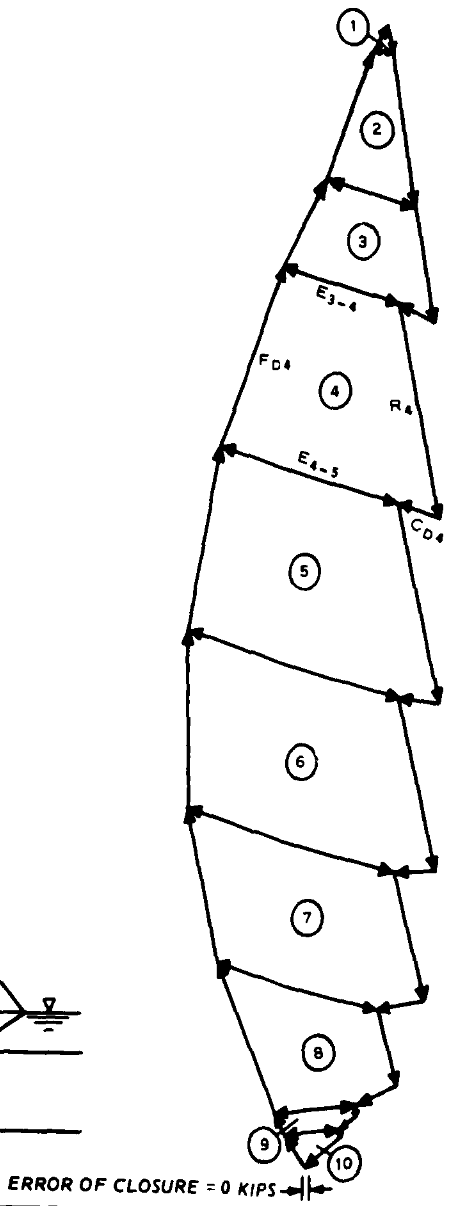
MODIFIED SWEDISH METHOD
END OF CONSTRUCTION

DESIGN SOIL PROPERTIES				
SOIL	UNIT WEIGHT, PCF		SHEAR STRENGTH	
	MOIST, γ_M	SATURATED, γ_S	C, KSF	ϕ, DEG
DIKE	120	128	0	30
DREDGED MAT'L	–	80	0	0
UPPER CRUST	–	120	1.0	0
SOFT CLAY	–	110	0.65	0
LOWER CLAY	–	115	0.9	0

NOTE: ARCS RUN BY COMPUTER PROGRAM SAVA104.
ALL ARCS TANGENT TO EL −10.
SIDE EARTH FORCES ASSUMED PARALLEL TO OUTER SLOPE (i.e., 1V ON 3H) WHERE ARC PASSES UNDER SLOPE; ALL OTHER SIDE EARTH FORCES ASSUMED HORIZONTAL.



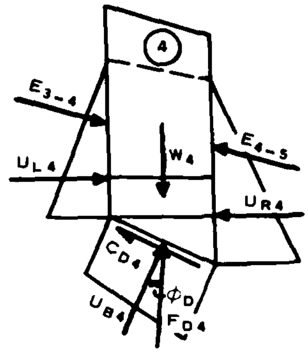




FORCE POLYGON FOR ARC SHOWN AT LEFT, FS = 1.47

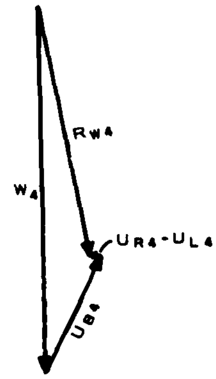
SCALE IN KIPS

5 0 5 10 15 20



FORCES ON TYPICAL SLICE

NO SCALE

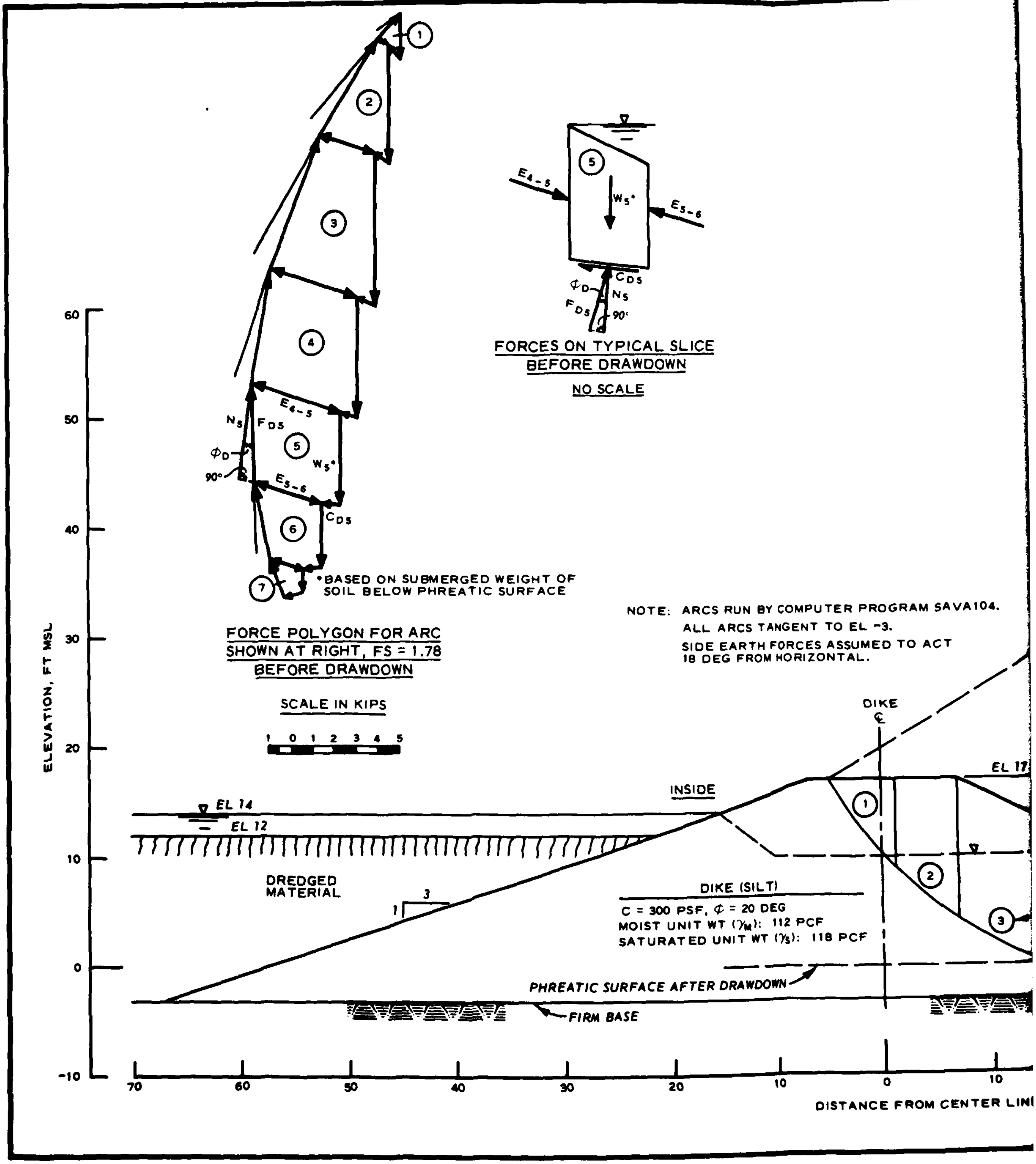


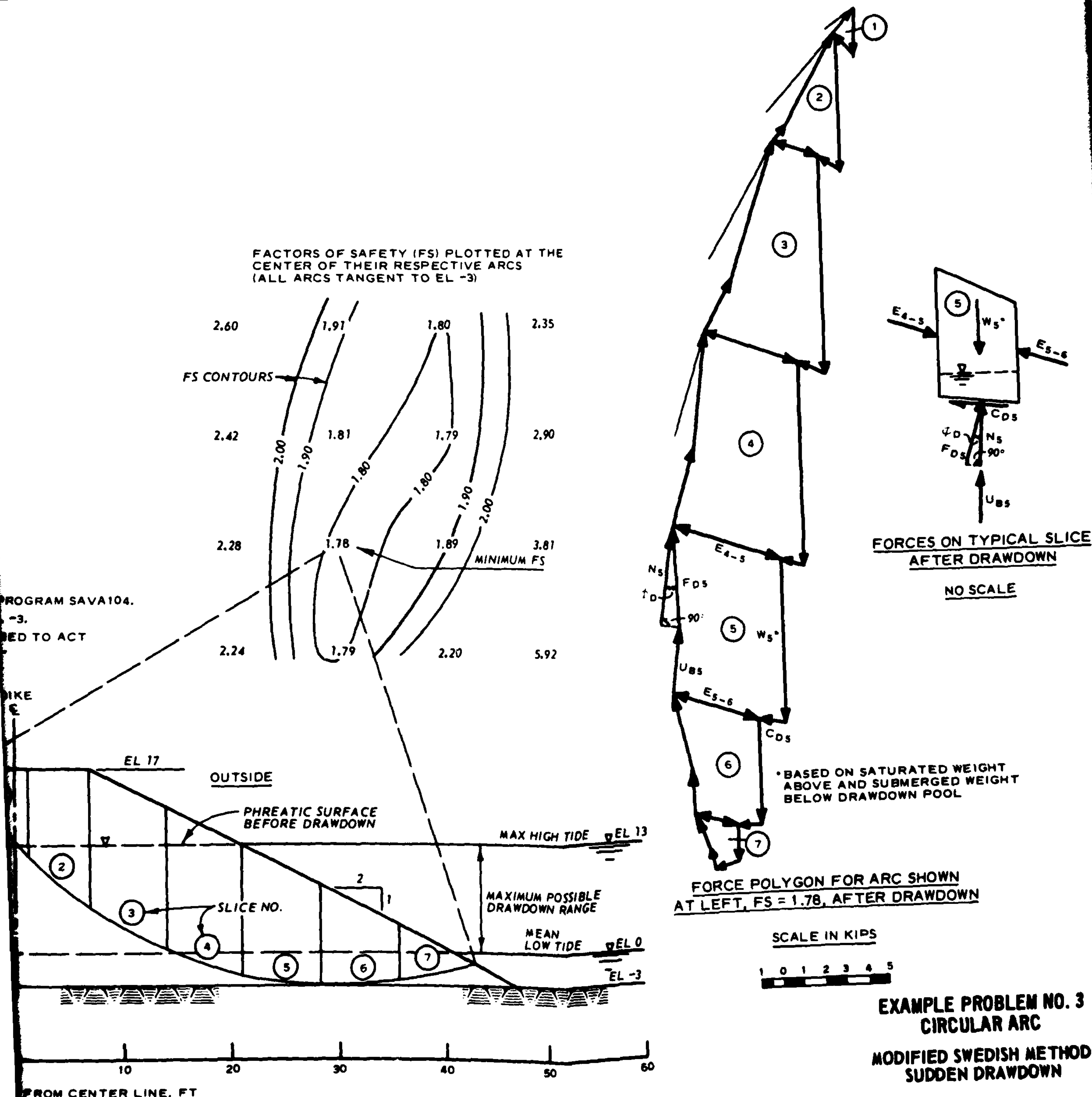
RESULTANT OF WEIGHT AND WATER FORCES

SCALE IN KIPS

5 0 5 10 15 20

EXAMPLE PROBLEM NO. 2
CIRCULAR ARC
MODIFIED SWEDISH METHOD
STEADY SEEPAGE


FORCES ON TYPICAL SLICE
BEFORE DRAWDOWN
NO SCALE
*BASED ON SUBMERGED WEIGHT OF SOIL BELOW PHREATIC SURFACE
FORCE POLYGON FOR ARC SHOWN AT RIGHT, FS = 1.78 BEFORE DRAWDOWN
SCALE IN KIPS
NOTE: ARCS RUN BY COMPUTER PROGRAM SAVA104.
ALL ARCS TANGENT TO EL -3.
SIDE EARTH FORCES ASSUMED TO ACT 18 DEG FROM HORIZONTAL.
ELEVATION, FT MSL
DIKE ℄
INSIDE
EL 14
EL 12
EL 17
DREDGED MATERIAL
DIKE (SILT)
C = 300 PSF, Φ = 20 DEG
MOIST UNIT WT (γM): 112 PCF
SATURATED UNIT WT (γS): 118 PCF
PHREATIC SURFACE AFTER DRAWDOWN
FIRM BASE
DISTANCE FROM CENTER LIN



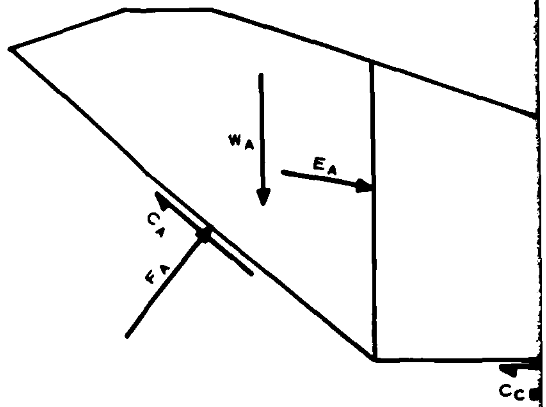
FORCE POLYGON FOR ARC SHOWN AT LEFT, FS = 1.78, AFTER DRAWDOWN

FORCES ON TYPICAL SLICE AFTER DRAWDOWN

NO SCALE

EXAMPLE PROBLEM NO. 3
CIRCULAR ARC

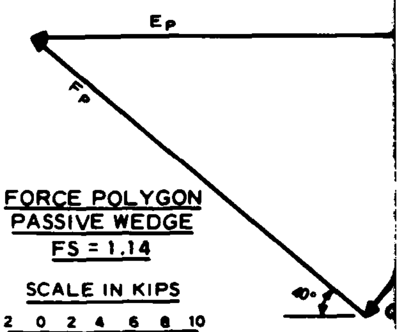
MODIFIED SWEDISH METHOD
SUDDEN DRAWDOWN



FORCES ON FAILURE WEDGES
NO SCALE

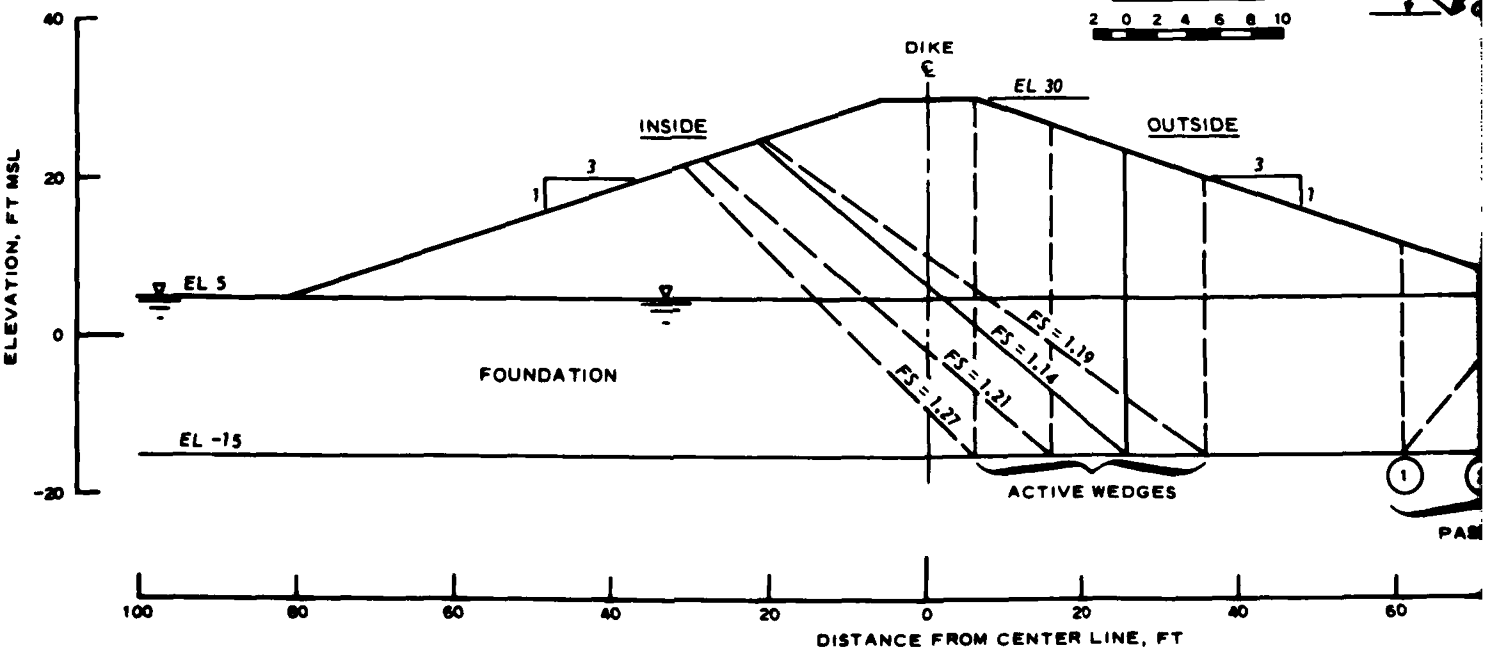
DESIGN SOIL PROPERTIES				
SOIL	UNIT WEIGHT, PCF		SHEAR STRENGTH	
	MOIST, γ_M	SATURATED, γ_S	C, KSF	ϕ, DEG
EMBANKMENT	115	125	1.0	0
FOUNDATION	–	115	0.4	0

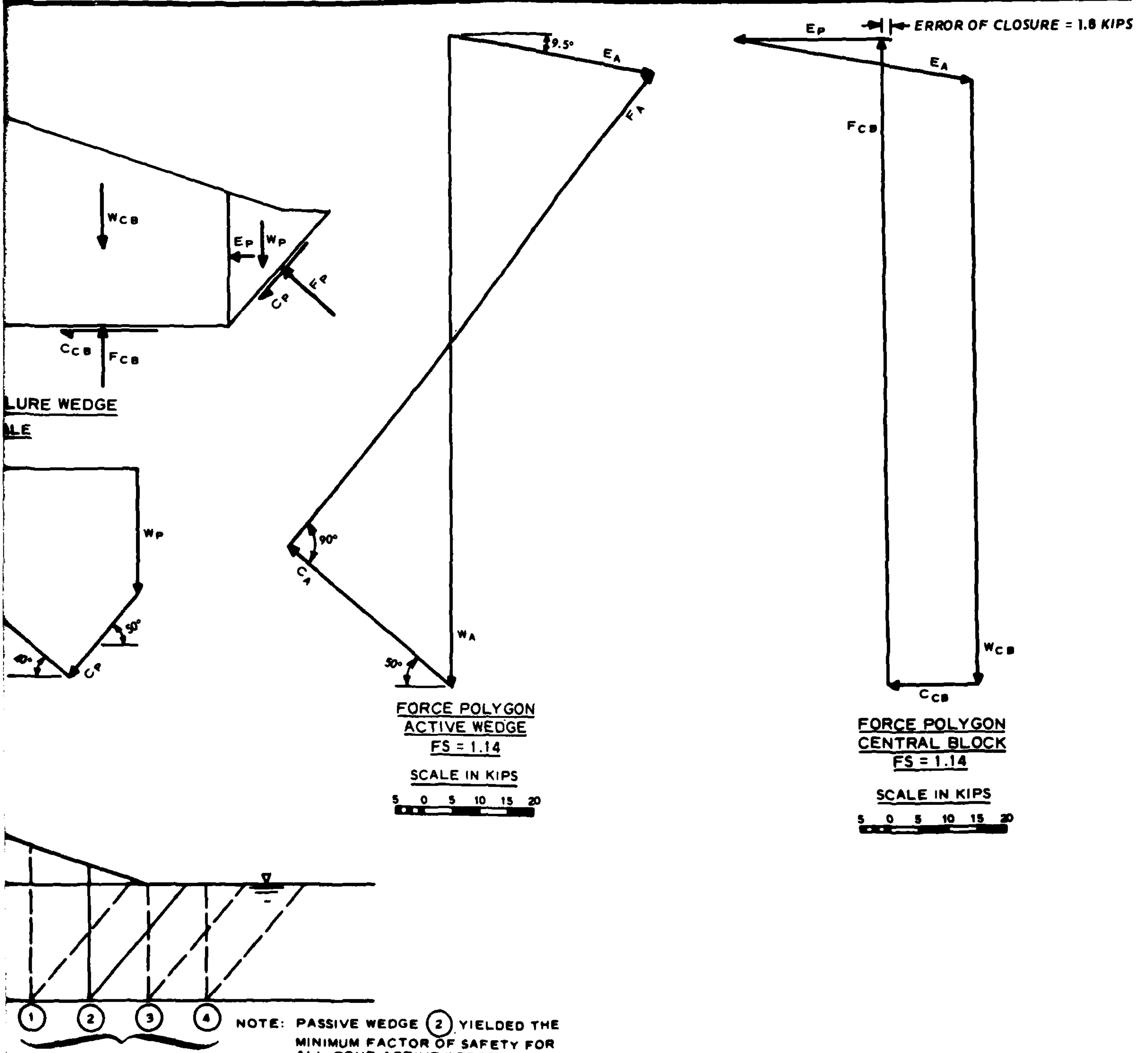
NOTE: ANALYSIS PERFORMED BY COMPUTER PROGRAM KCWEG.
SLOPE OF E_A ASSUMED TO BE 9.5 DEG.
ALL WEDGES RUN AT OTHER FAILURE ELEVATIONS (NOT SHOWN) YIELDED HIGHER FACTORS OF SAFETY.



FORCE POLYGON
PASSIVE WEDGE
FS = 1.14

SCALE IN KIPS
2 0 2 4 6 8 10

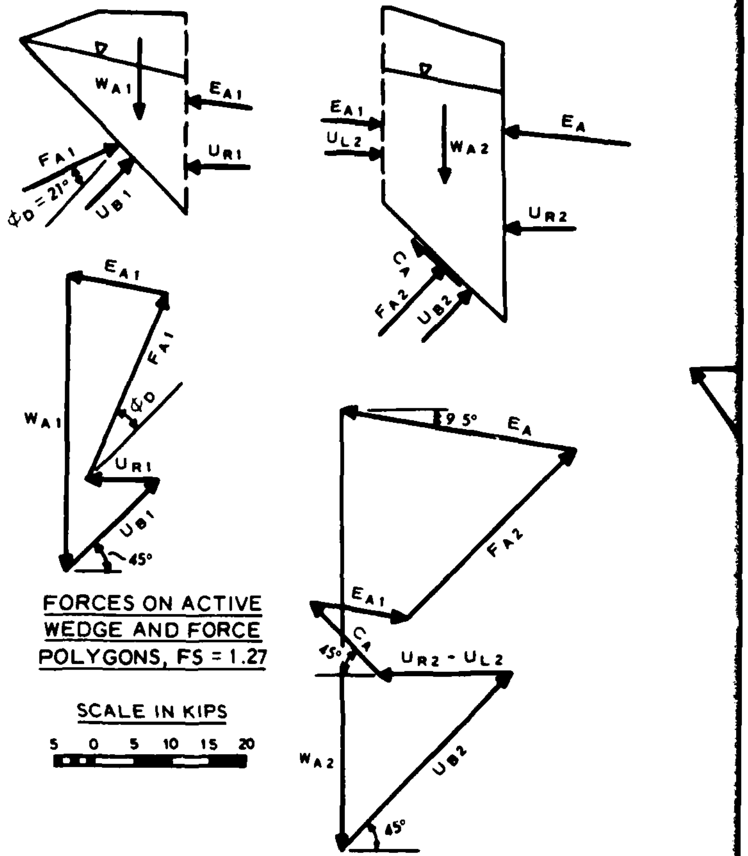




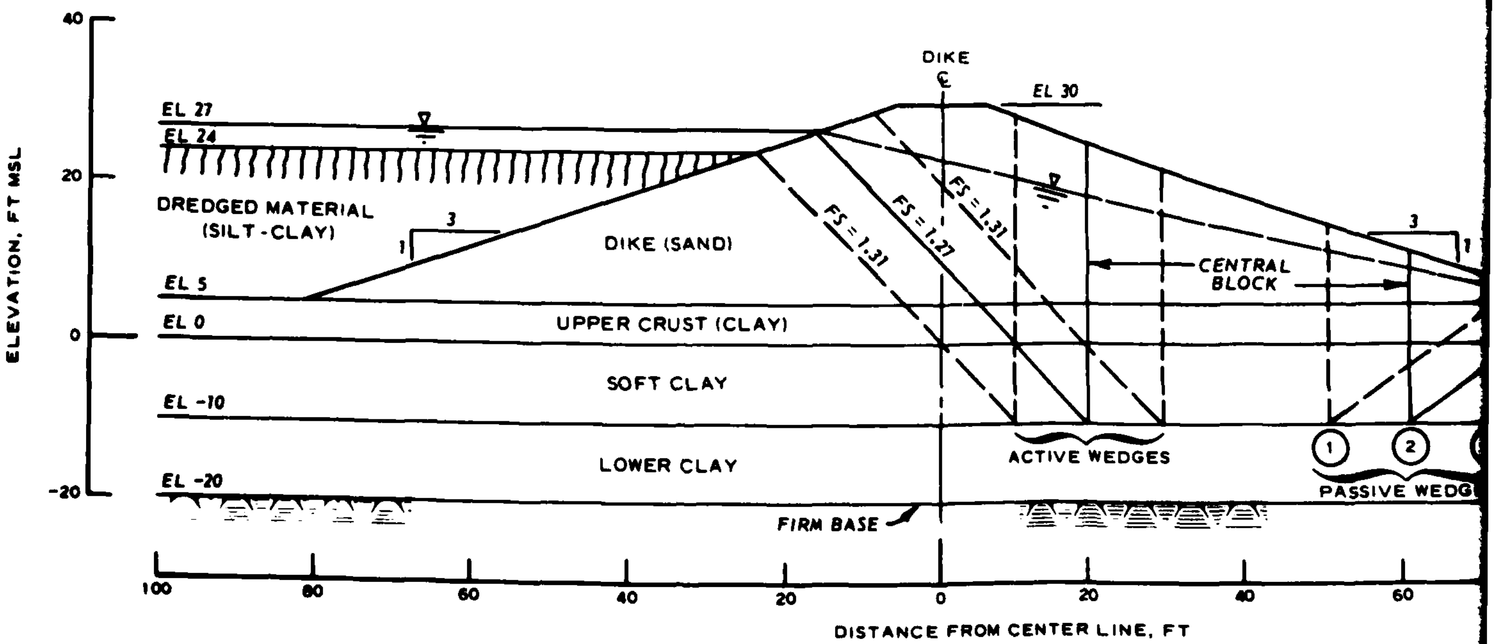
EXAMPLE PROBLEM NO. 4
WEDGE METHOD
END OF CONSTRUCTION

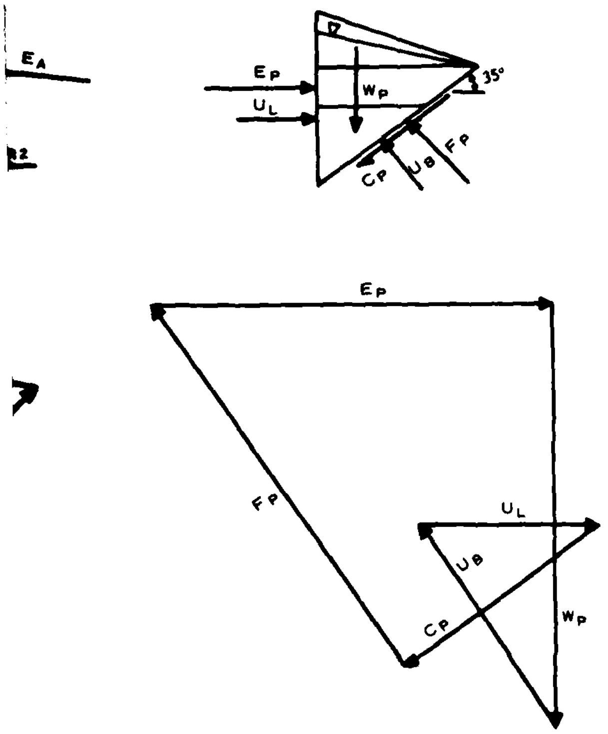
DESIGN SOIL PROPERTIES				
SOIL	UNIT WEIGHT, PCF		SHEAR STRENGTH	
	MOIST, γ_M	SATURATED, γ_S	C, KSF	ϕ, DEG
DIKE	120	128	0	26
DREDGED MAT'L	–	80	0	0
UPPER CRUST	–	120	1.0	0
SOFT CLAY	–	110	0.65	0
LOWER CLAY	–	115	0.90	0

NOTE: ANALYSIS PERFORMED BY COMPUTER PROGRAM KCWEG.
SLOPE OF E_A ASSUMED TO BE 9.5 DEG.
ALL WEDGES RUN AT OTHER FAILURE ELEVATIONS (NOT SHOWN) YIELDED HIGHER FACTORS OF SAFETY.



FORCES ON ACTIVE WEDGE AND FORCE POLYGONS, FS = 1.27

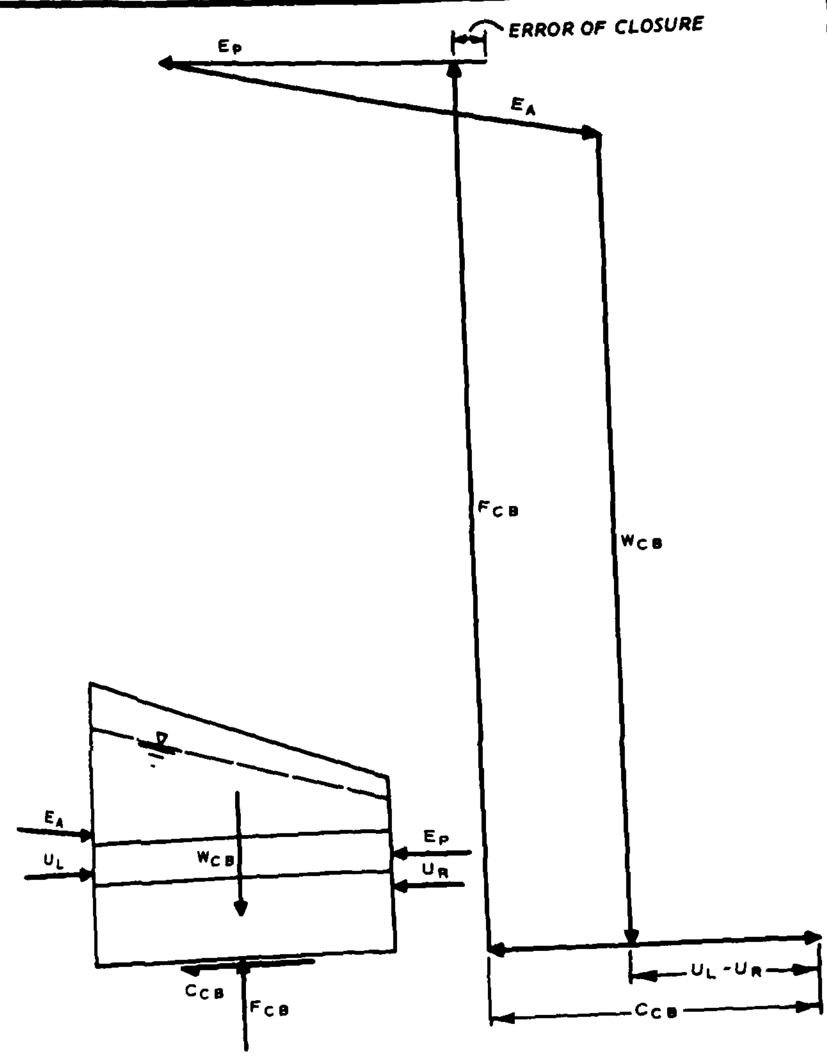




FORCES ON PASSIVE WEDGE
AND FORCE POLYGON, FS = 1.27

SCALE IN KIPS

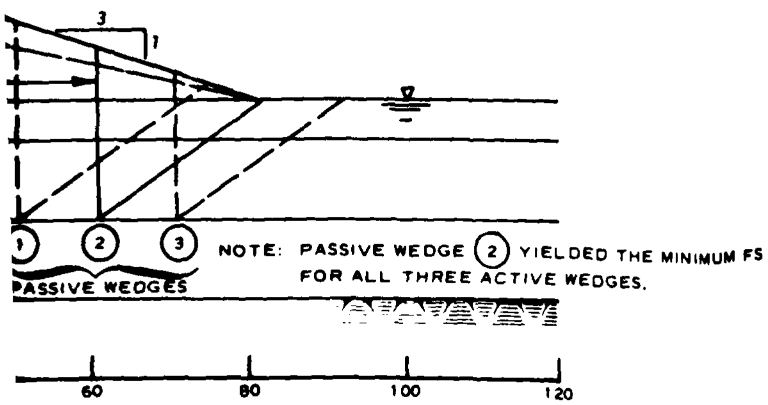
2 0 2 4 6 8 10



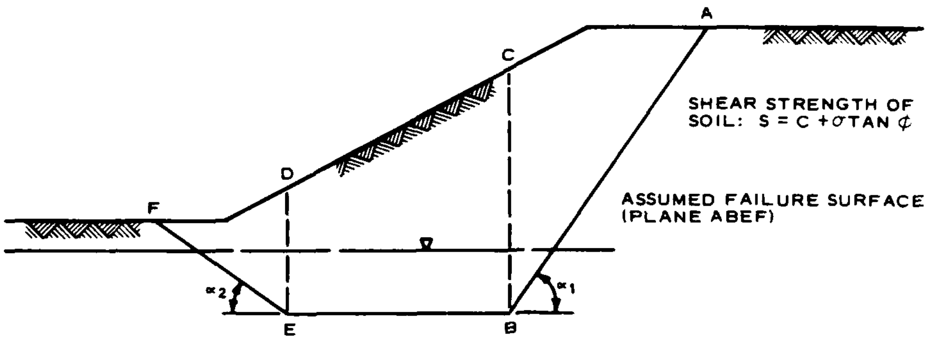
FORCES ON CENTRAL BLOCK
AND FORCE POLYGON, FS = 1.27

SCALE IN KIPS

5 0 5 10 15 20

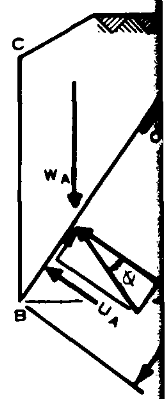


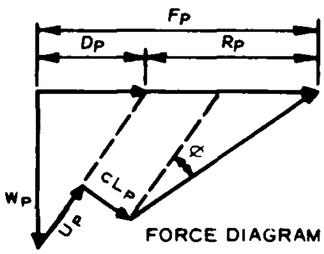
EXAMPLE PROBLEM NO. 5
WEDGE METHOD
STEADY SEEPAGE



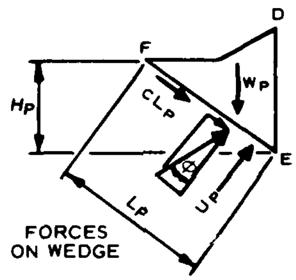
NOTE: α_1 AND α_2 USUALLY ASSUMED EQUAL TO $45° + \phi/2$ AND $45° - \phi/2$, RESPECTIVELY

SECTION THROUGH SLOPE





FORCE DIAGRAM



FORCES ON WEDGE

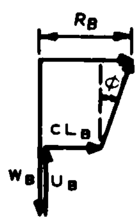
PASSIVE WEDGE

W = WEIGHT OF WATE
U = TOTAL UPLIFT F
D = HORIZONTAL DRI
SLIDING PLANE N
R = HORIZONTAL RES
SOIL BEING MOBIL
F = NET HORIZONTAL
SUBSCRIPTS A AND P
SUBSCRIPT B REFER

FACTOR OF SAFETY
TO SHEAR STRENGTH

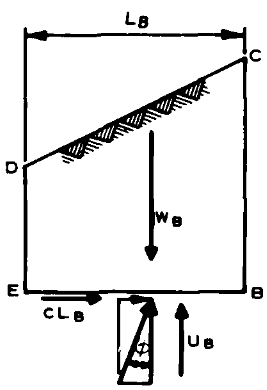
THE FOLLOWING EQU
ACTIVE AND PASSIVE
$(45° - \phi/2)$, RESPECTI

$R_A = 2\left[W_A - U_A \text{ SIN } (4\right.$
$R_P = 2\left[W_P - U_P \text{ COS } (4\right.$
$D_A = W_A \text{ TAN } (45° + \phi/2$
$D_P = W_P \text{ TAN } (45° - \phi/2$



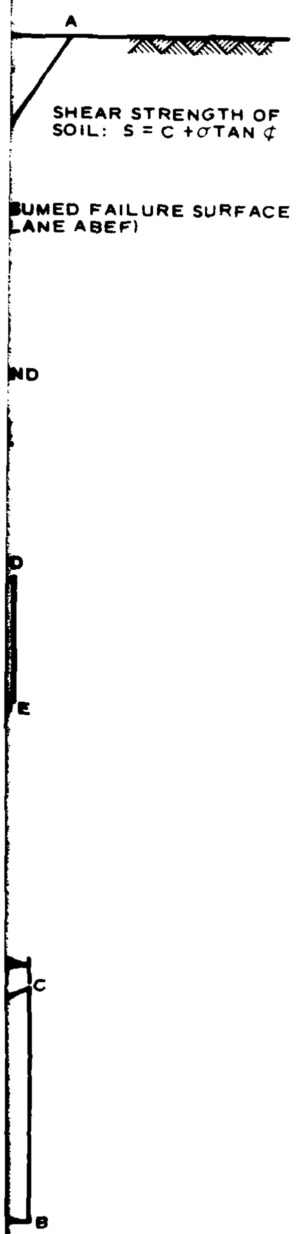
NOTE: $R_B = (W_B - U_B) \text{ TAN } \phi + CL_B$
$D_B = 0$

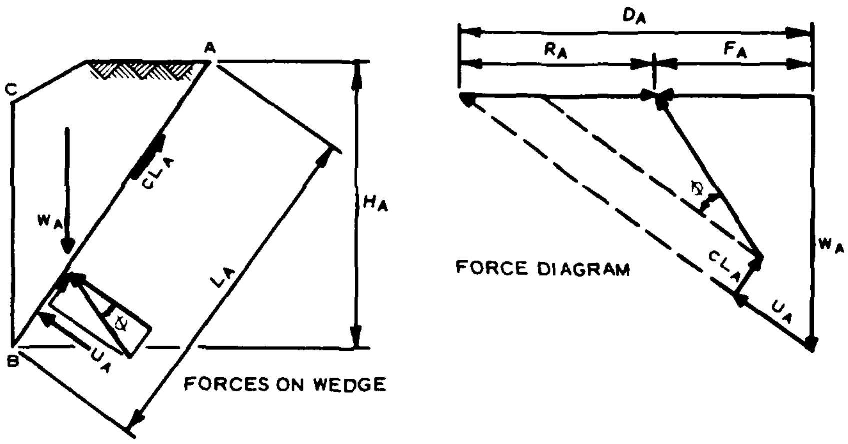
FORCE DIAGRAM



FORCES ON BLOCK

CENTRAL BLOCK





ACTIVE WEDGE

NOTATIONS AND EQUATIONS

W = WEIGHT OF WATER AND SOIL IN WEDGE

U = TOTAL UPLIFT FORCE ACTING NORMAL TO SLIDING PLANE

D = HORIZONTAL DRIVING FORCE = HORIZONTAL COMPONENT OF W AND NORMAL FORCE ON SLIDING PLANE NEGLECTING SHEAR STRENGTH OF SOIL

R = HORIZONTAL RESISTANCE FORCE = HORIZONTAL FORCE DUE TO SHEAR STRENGTH OF SOIL BEING MOBILIZED ALONG SLIDING PLANE

F = NET HORIZONTAL EARTH FORCE

SUBSCRIPTS A AND P REFER TO ACTIVE AND PASSIVE WEDGES, RESPECTIVELY

SUBSCRIPT B REFERS TO CENTRAL BLOCK

FACTOR OF SAFETY WITH RESPECT TO SHEAR STRENGTH OF SOIL $= \frac{\Sigma R}{\Sigma D} = \frac{R_A + R_B + R_P}{D_A - D_P}$

THE FOLLOWING EQUATIONS CAN BE USED IN LIEU OF DRAWING FORCE POLYGONS WHEN THE ACTIVE AND PASSIVE FAILURE PLANES ARE ASSUMED INCLINED AT ANGLES OF $(45° + \phi/2)$ AND $(45° - \phi/2)$, RESPECTIVELY, WITH RESPECT TO THE HORIZONTAL

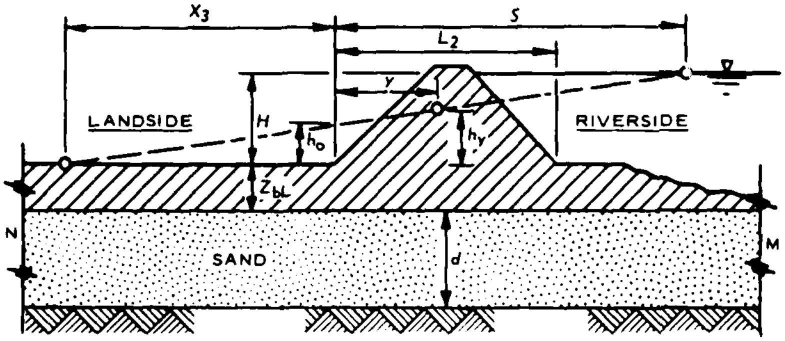
$$R_A = 2\left[W_A - U_A \sin(45° - \phi/2)\right] \tan\phi + 2\,CH_A \tan(45° - \phi/2)$$

$$R_P = 2\left[W_P - U_P \cos(45° - \phi/2)\right] \tan\phi + 2\,CH_P \tan(45° + \phi/2)$$

$$D_A = W_A \tan(45° + \phi/2) \qquad R_B = (W_B - U_B)\tan\phi + cL_B$$

$$D_P = W_P \tan(45° - \phi/2) \qquad D_B = 0$$

ANALYSIS OF SLOPE STABILITY BY METHOD OF PLANES



$$h_o = \frac{H \bullet X_3}{S + X_3}$$

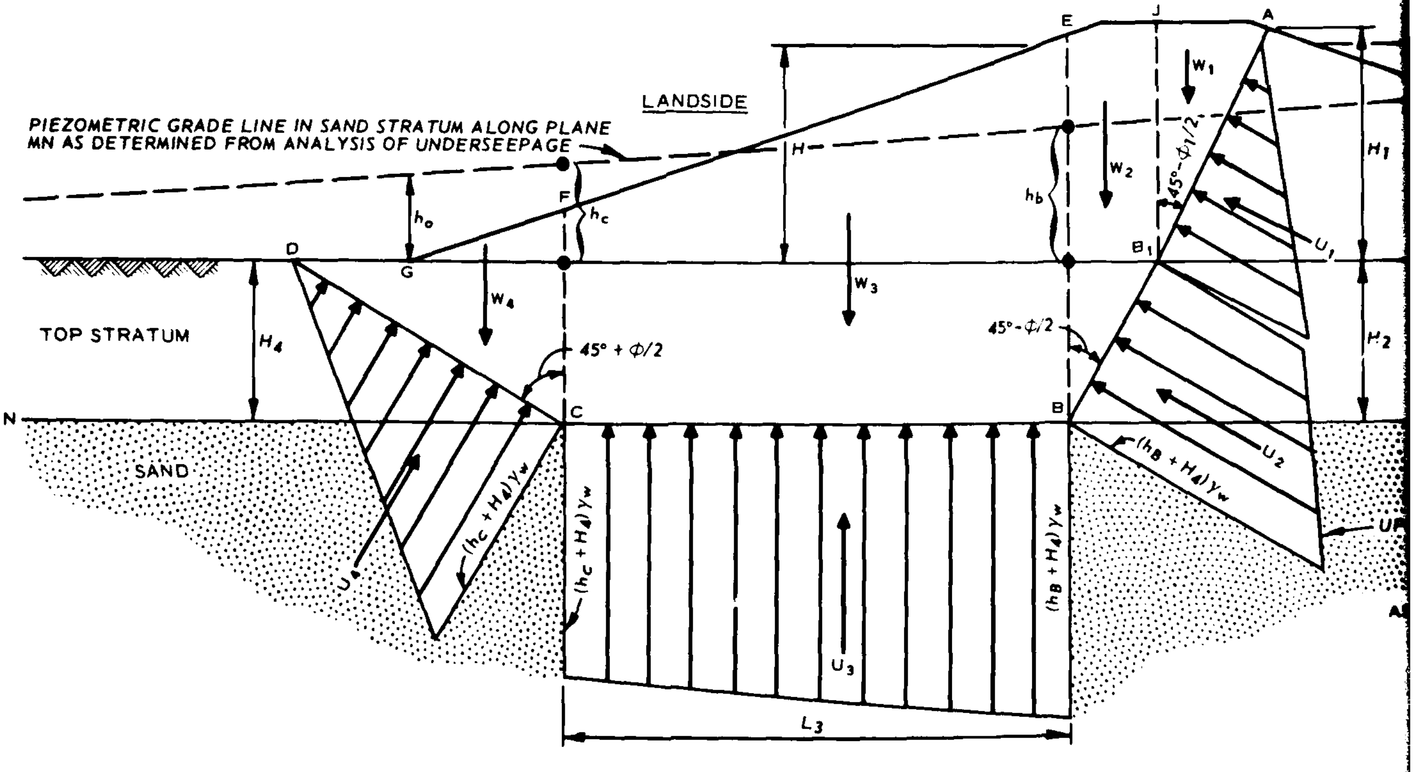
FOR $0 \leq y \leq L_2$

$$h_y = \frac{H(X_3 + y)}{S + X_3}$$

NOTE: COMPUTE S AND X_3 FROM APPLICABLE FORMULA TM NO. 3 – 424, "INVESTIGATION OF UNDERSEEPA ITS CONTROL, LOWER MISSISSIPPI RIVER LEVEES, OCT 1956.[24]

IDEALIZED SECTION FOR UNDERSEEPAGE ANALYSIS

NO SCALE



NOTATIONS AND TYPICAL SECTION FOR STABILITY ANALYSIS

NO SCALE

FORMULAS IN WES
ERSEEPAGE AND
LEVEES,"

$$FS = \frac{R_1 + R_2 + R_3 + R_4}{D_1 + D_2 + D_3 - D_4}$$

R = RESISTING FORCE COMPUTED FROM EQUATIONS AT RIGHT

D = DRIVING FORCE AS COMPUTED FROM EQUATIONS AT RIGHT

WHEREIN:

$D_1 = W_1 \text{ TAN } (45° + \phi_1/2)$

$R_1 = 2\left[W_1 - U_1 \text{ SIN } (45° - \phi_1/2)\right] \text{ TAN } \phi_1 + 2\, C_1 H_1 \text{ TAN } (45° - \phi_1/2)$

$D_2 = W_2 \text{ TAN } (45° + \phi/2)$

$R_2 = 2\left[W_2 - U_2 \text{ SIN } (45° - \phi/2)\right] \text{ TAN } \phi + 2\, CH_2 \text{ TAN } (45° - \phi/2)$

$D_3 = 0$

$R_3 = (W_3 - U_3) \text{ TAN } \phi + CL_3$

$D_4 = W_4 \text{ TAN } (45° - \phi/2)$

$R_4 = 2\left[W_4 - U_4 \text{ COS } (45° - \phi/2)\right] \text{ TAN } \phi + 2\, CH_4 \text{ TAN } (45° + \phi/2)$

W_1 = TOTAL WEIGHT OF SOIL + WATER IN WEDGE AB_1J

W_2 = TOTAL WEIGHT OF SOIL + WATER IN WEDGE JB_1BE

W_3 = TOTAL WEIGHT OF SOIL + WATER IN WEDGE EBCF

W_4 = TOTAL WEIGHT OF SOIL + WATER IN WEDGE FCDG

h_B = NET HEAD ABOVE GROUND SURFACE AT POINT B

h_C = NET HEAD ABOVE GROUND SURFACE AT POINT C

U_1 = TOTAL UPLIFT FORCE ON PLANE AB_1

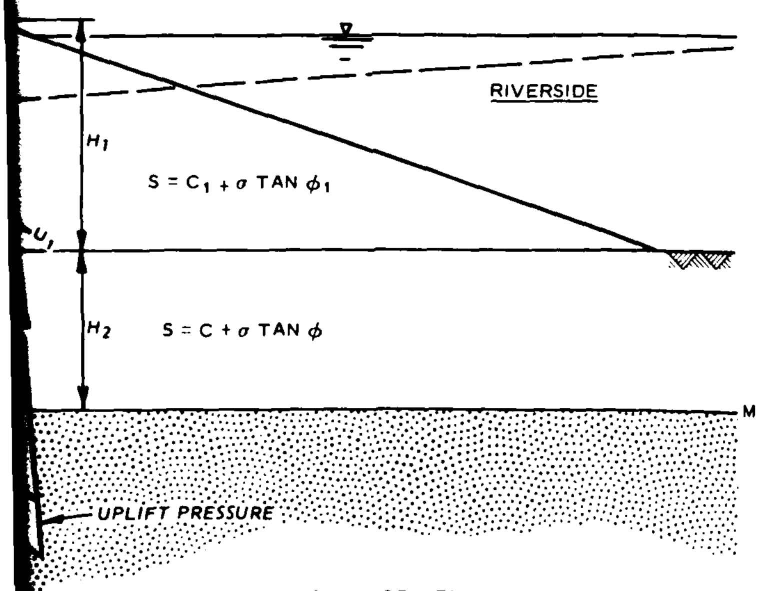
U_2 = TOTAL UPLIFT FORCE ON PLANE B_1B

U_3 = TOTAL UPLIFT FORCE ON PLANE BC

U_4 = TOTAL UPLIFT FORCE ON PLANE CD

γ_w = UNIT WEIGHT OF WATER

FS = FACTOR OF SAFETY AGAINST SLIDING WITH RESPECT TO SHEAR STRENGTH OF SOIL



ASSUMED FAILURE SURFACE = PLANE AB_1BCD

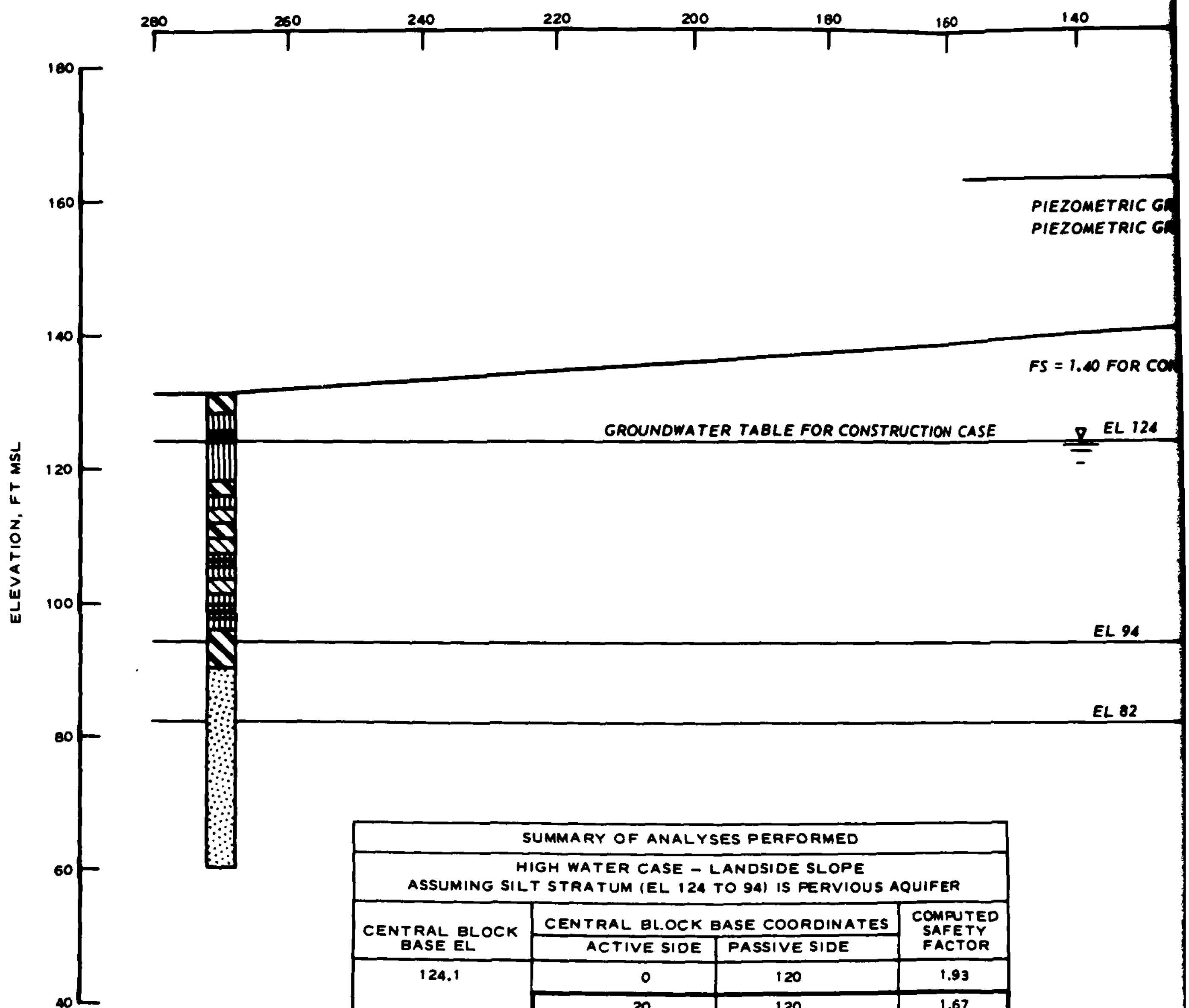
ALYSIS

ANALYSIS OF STABILITY OF LANDSIDE SLOPE OF LEVEE

BY METHOD OF PLANES CONSIDERING SEEPAGE FORCES

PLATE A7

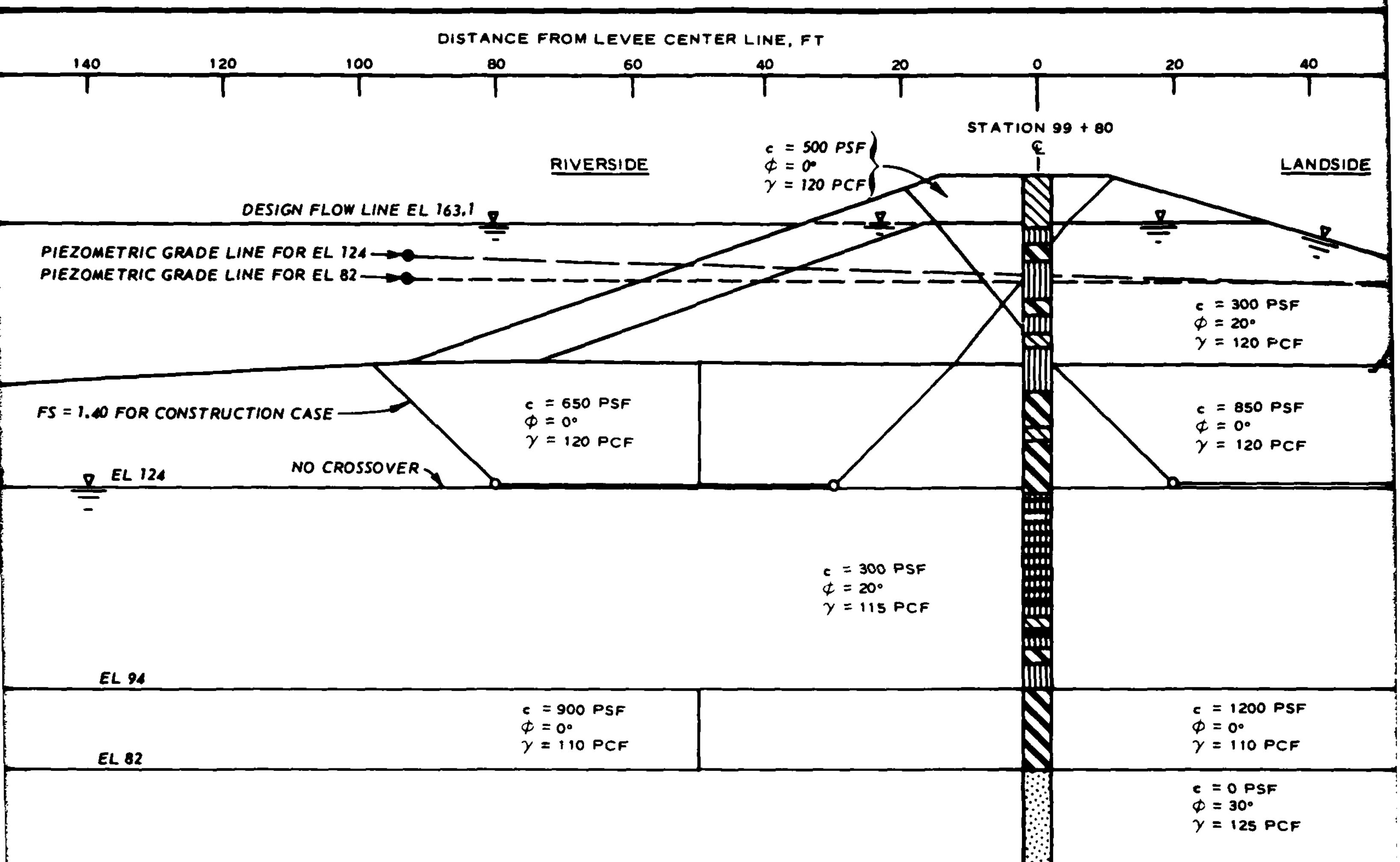
2



SUMMARY OF ANALYSES PERFORMED			
HIGH WATER CASE – LANDSIDE SLOPE ASSUMING SILT STRATUM (EL 124 TO 94) IS PERVIOUS AQUIFER			
CENTRAL BLOCK BASE EL	CENTRAL BLOCK BASE COORDINATES		COMPUTED SAFETY FACTOR
	ACTIVE SIDE	PASSIVE SIDE	
124.1	0	120	1.93
	20	120	1.67
(CLAY)	30	120	1.70
	20	100	1.71
124.1	20	140	1.77
123.9	20	140	2.05
	30	140	2.01
(SILTY SAND)	40	140	2.06
	30	120	1.98
123.9	30	100	2.09

*MINIMUM FACTORS OF SAFETY

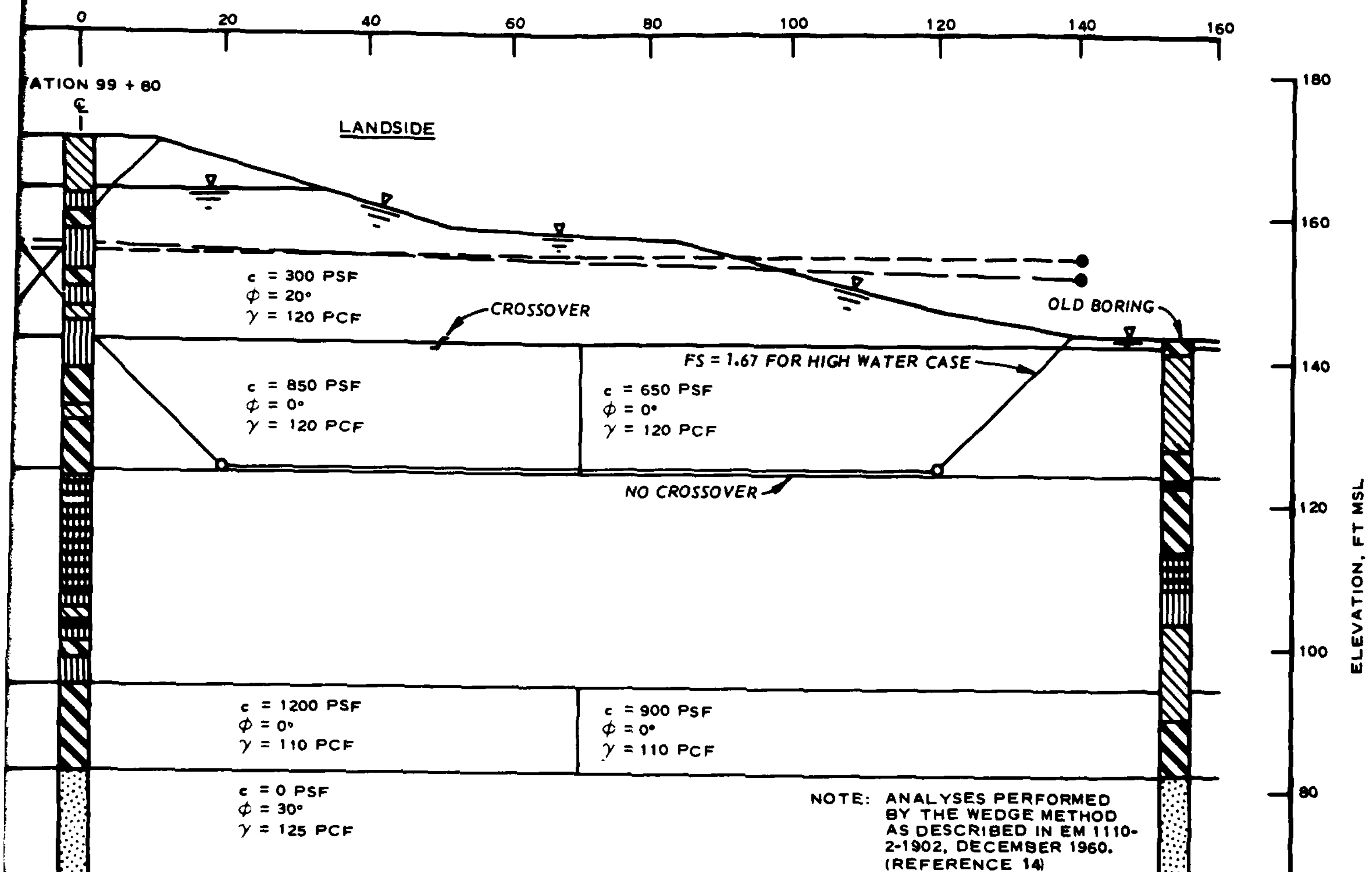
/



SUMMARY OF ANALYSES PERFORMED			
HIGH WATER CASE – LANDSIDE SLOPE ASSUMING LOWER SAND IS PERVIOUS AQUIFER			
CENTRAL BLOCK BASE EL	CENTRAL BLOCK BASE COORDINATES		COMPUTED SAFETY FACTOR
	ACTIVE SIDE	PASSIVE SIDE	
94.1	30	100	2.06
	40	100	2.00
(SILTY SAND)	50	100	2.03
	40	120	1.96
94.1	40	140	2.02
82.1	30	100	1.81
	40	100	1.75
(CLAY)	50	100	1.76
	40	121	1.70
82.1	40	140	1.73

SUMMARY OF AN	
CONSTRUCTION CA	
CENTRAL BLOCK BASE EL	CENTRAL BL
	ACTIVE S
124.1	20
	30
(CLAY)	40
	30
124.1	30
82.1	30
	40
(CLAY)	50
	40
82.1	40

* MINIMUM FACTORS OF SAFETY



NOTE: ANALYSES PERFORMED BY THE WEDGE METHOD AS DESCRIBED IN EM 1110-2-1902, DECEMBER 1960. (REFERENCE 14)

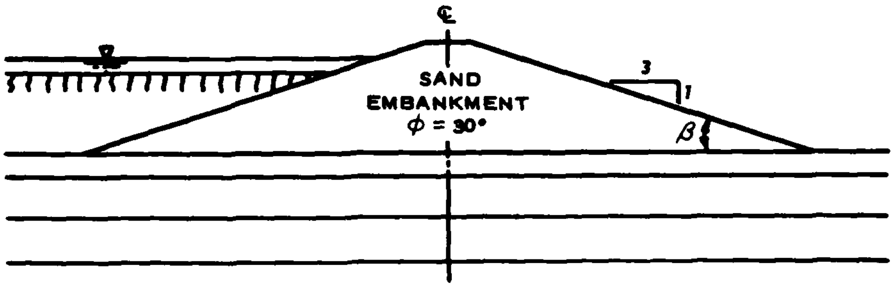
SUMMARY OF ANALYSES PERFORMED

CONSTRUCTION CASE – RIVERSIDE SLOPE

CENTRAL BLOCK BASE EL	CENTRAL BLOCK BASE COORDINATES		COMPUTED SAFETY FACTOR
	ACTIVE SIDE	PASSIVE SIDE	
124.1	20	80	1.43
	30	80	1.40
(CLAY)	40	80	1.44
	30	70	1.45
124.1	30	90	1.42
82.1	30	80	1.79
	40	80	1.76
(CLAY)	50	80	1.77
	40	93	1.75
82.1	40	100	1.76

• MINIMUM FACTORS OF SAFETY

EXAMPLE PROBLEM NO. 6
STABILITY ANALYSES
METHOD OF PLANES
MISSISSIPPI RIVER LEVEE SAFETY STUDY
WEST BANK IN ARKANSAS



DIKE SECTION

EXAMPLE PROBLEM 7A

NO SEEPAGE

$FS = \tan \phi / \tan \beta$
$FS = \tan 30° / \tan 18.4°$
$FS = 0.577/0.333$
$FS = 1.73$

EXAMPLE PROBLEM 7B

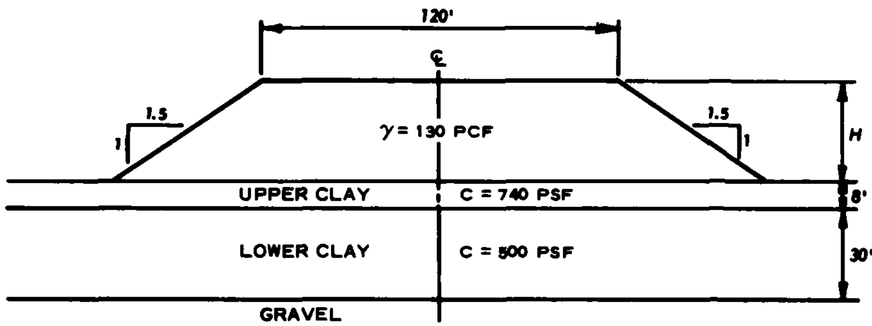
STEADY SEEPAGE*

$FS = \tan \phi/2 \; / \; \tan \beta$
$FS = \tan 30°/2 \; / \; \tan 18.4° = \tan 15° / \tan 18.4°$
$FS = 0.268/0.333$
$FS = 0.80$

*THIS ANALYSIS ASSUMES A PHREATIC SURFACE COINCIDENT WITH THE OUTER EMBANKMENT SLOPE AND SEEPAGE PARALLEL TO THE OUTER SLOPE

EXAMPLE PROBLEM NO. 7

INFINITE SLOPE ANALYSIS FOR COHESIONLESS SOILS



HIGHWAY EMBANKMENT

NOTE: ORIGINAL DESIGN HEIGHT (H) OF THIS EMBANKMENT WAS 33 FT. CHECK STABILITY AT H = 33 FT USING BEARING CAPACITY EQUATIONS (THIS EMBANKMENT ACTUALLY FAILED AT A HEIGHT OF 25 FT).

COMPUTE

ULTIMATE BEARING CAPACITY OF THE TWO CLAY STRATA

a. UPPER CLAY: $q_d = 5.5C = 5.5\,(740) = 4070$ PSF

b. LOWER CLAY: $q_d = 5.5C = 5.5\,(500) = 2750$ PSF

PRESSURE AT TOP OF UPPER CLAY (i.e. EMBANKMENT BASE)

$q = \gamma H = 130\,(33) = 4290$ PSF

PRESSURE AT TOP OF LOWER CLAY (MUST USE INFLUENCE CHART, PLATE A12)

$z = 8$ FT; $b = 120/2 = 60$; $a = 33 \times 1.5 = 49.5$

$a/z = 49.5/8 = 6.2$; $b/z = 60/8 = 7.5$

FROM INFLUENCE CHART USING $b/z = \infty$:

$I = 0.5$ & $I_q = 0.5 + 0.5 = 1.0$ (WHICH MEANS FULL EMBANKMENT LOAD IS TRANSMITTED TO TOP OF LOWER CLAY STRATUM)

$\therefore q_z = I_q q = 1.0\,(4290) = 4290$ PSF

FACTOR OF SAFETY

a. UPPER CLAY: $FS = q_d/q = 4070/4290 = 0.95$

b. LOWER CLAY: $FS = q_d/q_z = 2750/4290 = 0.64$

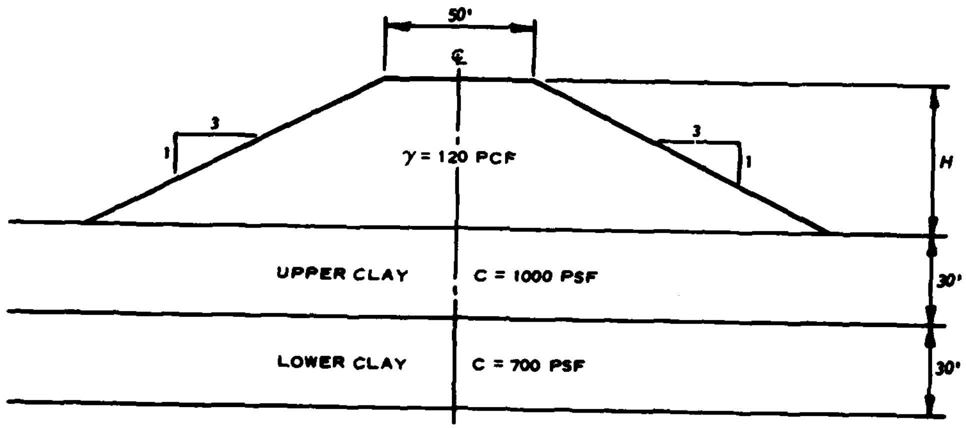
IT APPEARS VERY LIKELY THIS EMBANKMENT IS UNSTABLE AT THE DESIGN HEIGHT OF 33 FT.

PREDICT

ULTIMATE HEIGHT TO WHICH THE EMBANKMENT COULD BE SAFELY BUILT

$H = q_d/\gamma = 2750/130 = 21$ FT

EXAMPLE PROBLEM NO. 9
USE OF BEARING CAPACITY EQUATIONS



RAILROAD EMBANKMENT

NOTE: ORIGINAL DESIGN HEIGHT (H) OF THIS EMBANKMENT WAS 36 FT. CHECK STABILITY AT H = 36 FT USING BEARING CAPACITY EQUATIONS (THIS EMBANKMENT ACTUALLY FAILED AT H = 36 FT).

COMPUTE

ULTIMATE BEARING CAPACITY OF THE TWO CLAY STRATA

a. UPPER CLAY: $q_d = 5.5C = 5.5\ (1000) = 5500$ PSF

b. LOWER CLAY: $q_d = 5.5C = 5.5\ (700) = 3850$ PSF

PRESSURE AT TOP OF UPPER CLAY (EMBANKMENT BASE)

$q = \gamma H = 120\ (36) = 4320$ PSF

PRESSURE AT TOP OF LOWER CLAY (MUST USE INFLUENCE CHART, PLATE A12)

$z = 30$ FT; $b = 50/2 = 25$ FT; $a = 3 \times 36 = 108$ FT

$a/z = 108/30 = 3.6$; $b/z = 25/30 = 0.83$

FROM INFLUENCE CHART: $I = 0.48$ & $I_q = 0.48 + 0.48 = 0.96$

$\therefore q_z = I_q q = 0.96\ (4320) = 4147$ PSF

FACTOR OF SAFETY

a. UPPER CLAY: $FS = q_d/q = 5500/4320 = 1.27$

b. LOWER CLAY: $FS = q_d/q_z = 3850/4147 = 0.93$

THESE RESULTS INDICATE EMBANKMENT WOULD BE UNSTABLE AT H = 36 FT.

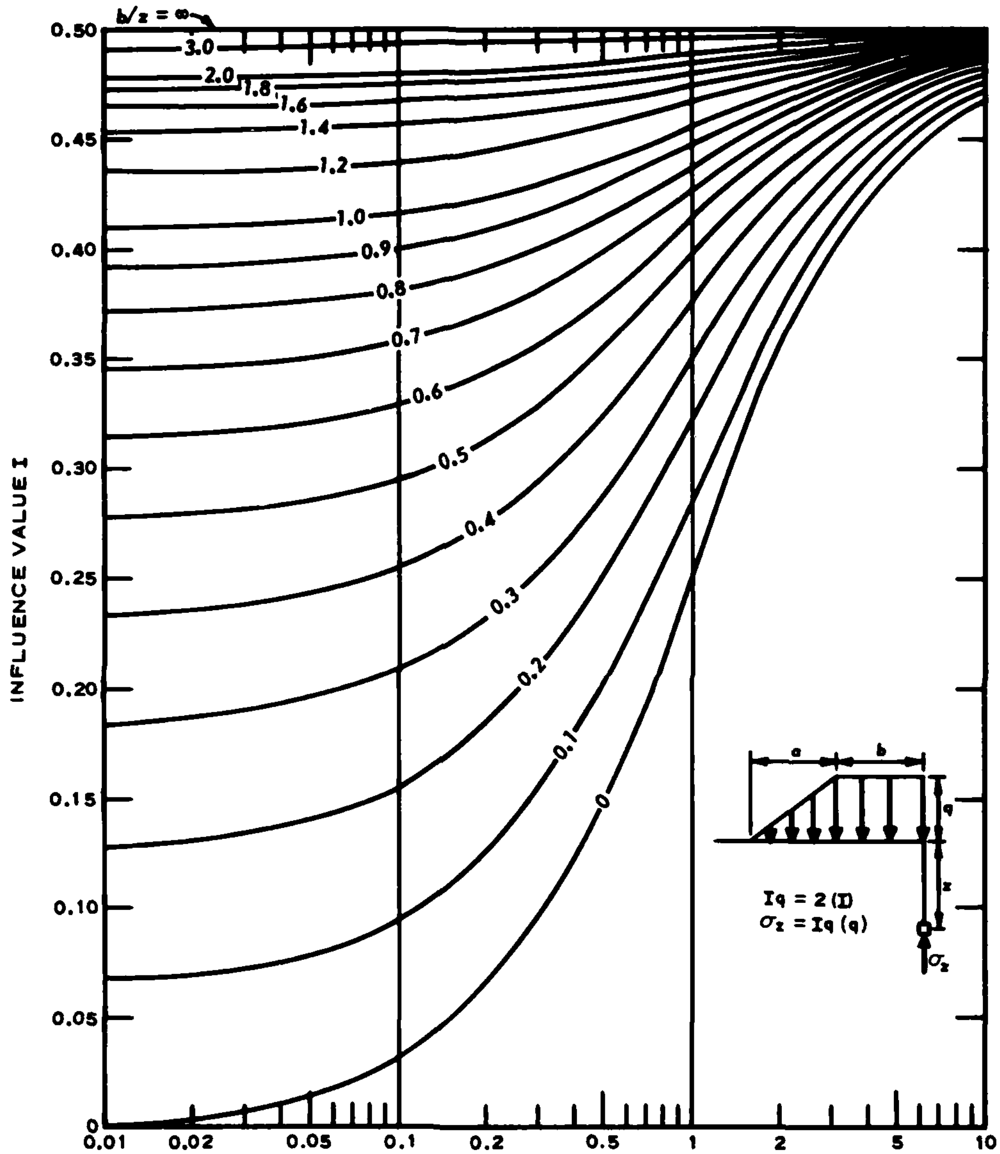
PREDICT

ULTIMATE HEIGHT TO WHICH THE EMBANKMENT CAN BE SAFELY BUILT

$H = q_d/\gamma = 3850/120 = 32$ FT

THIS IS FOR FS = 1.0, HENCE THE TERM <u>ULTIMATE</u> HEIGHT.

EXAMPLE PROBLEM NO. 8
USE OF BEARING CAPACITY EQUATIONS



After J. O. Osterberg

INFLUENCE CHART FOR VERTICAL STRESS-EMBANKMENT LOADING

INFINITE EXTENT
BOUSSINESQ CASE

APPENDIX B: NOTATION

a	Slope width
b	One-half embankment crest width
B	Berm length
c	Soil cohesion
$c/\bar{p}$, $S_u/\bar{\sigma}_{vc}$	Rate of strength increase with depth for a normally consolidated clay
C_A	Soil cohesion force acting along active wedge base
C_{CB}	Soil cohesion force acting along base of central block
C_D	Developed soil cohesion force
C_P	Soil cohesion force acting along passive wedge base
D	Depth
e	Void ratio
E	Interslice earth force
E_A	Active earth force
E_P	Passive earth force
f	Shrinkage factor
F_A	Friction force acting on active wedge base
F_{CB}	Friction force acting on central block base
F_D	Developed friction force
F_P	Friction force acting on passive wedge base
FS	Factor of safety
H	Embankment height
I	Influence value
Iq	Influence factor

LL	Liquid limit
N	Blow counts per foot from standard penetration test
N	Normal force
NSP	Normalized soil parameters
OCR	Overconsolidation ratio
p	Pressure
p_o	Overburden pressure
$\bar{p}$	Effective overburden pressure
$\bar{P}_c$, $\bar{\sigma}_{vm}$	Maximum past effective vertical stress
PI	Plasticity index
PL	Plastic limit
Q	Unconsolidated-undrained shear strength as determined in Q test
Q test	Shear test representing unconsolidated-undrained conditions
q	Unit load
q_d	Ultimate undrained bearing capacity
q_u	Unconfined compressive strength
q_z	Soil pressure at depth z
R	Consolidated-undrained shear strength
R_w	Resultant of weight and water forces
R test	Shear test representing consolidated-undrained conditions
S	Consolidated-drained shear strength
S test	Shear test representing consolidated-drained conditions
S_u	Unconsolidated-undrained shear strength
t	Time

U_B	Basal water force
U_L	Left-side water force
U_R	Right-side water force
UC test	Unconfined compression test
z	Depth below embankment base
W	Weight
W_A	Active wedge weight
W_{CB}	Central block weight
W_P	Passive wedge weight
w	Section width
β	Slope angle
γ	Soil unit weight
γ_d	Soil dry unit weight
γ_m	Soil moist unit weight
γ_s	Soil saturated unit weight
ΔS_u	Change in unconsolidated undrained strength
$\Delta\bar{\sigma}_{vc}$	Change in effective vertical consolidation stress
μ	Correction factor for effect of PI on vane shear strength
$\bar{\sigma}_{vc}$	Effective vertical consolidation stress
ϕ	Angle of internal friction
ϕ_D	Developed angle of internal friction
σ_z	Stress at depth z

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Hammer, David P

Design and construction of retaining dikes for containment of dredged material / by David P. Hammer and Edward D. Blackburn, Soils Section, U. S. Army Engineer District, Savannah, Savannah, Georgia. Vicksburg, Miss. : U. S. Waterways Experiment Station, 1977.

182, [6] p. : 12 leaves of plates : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; D-77-9)

Prepared for Office, Chief of Engineers, U. S. Army, Washington, D. C.

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References: p. 178-182.

1. Containment areas. 2. Dikes (Embankments). 3. Dredged material disposal. 4. Waste disposal sites. I. Blackburn, Edward D., joint author. II. United States. Army. Corps of Engineers. III. United States. Army. Corps of Engineers. Savannah District. IV. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; D-77-9.

TA7.W34 no.D-77-9