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SUSCEPTIBILITY OF DISPERSIVE CLAY AT GRENADA DAM, MISSISSIPPI, --ETC(U)

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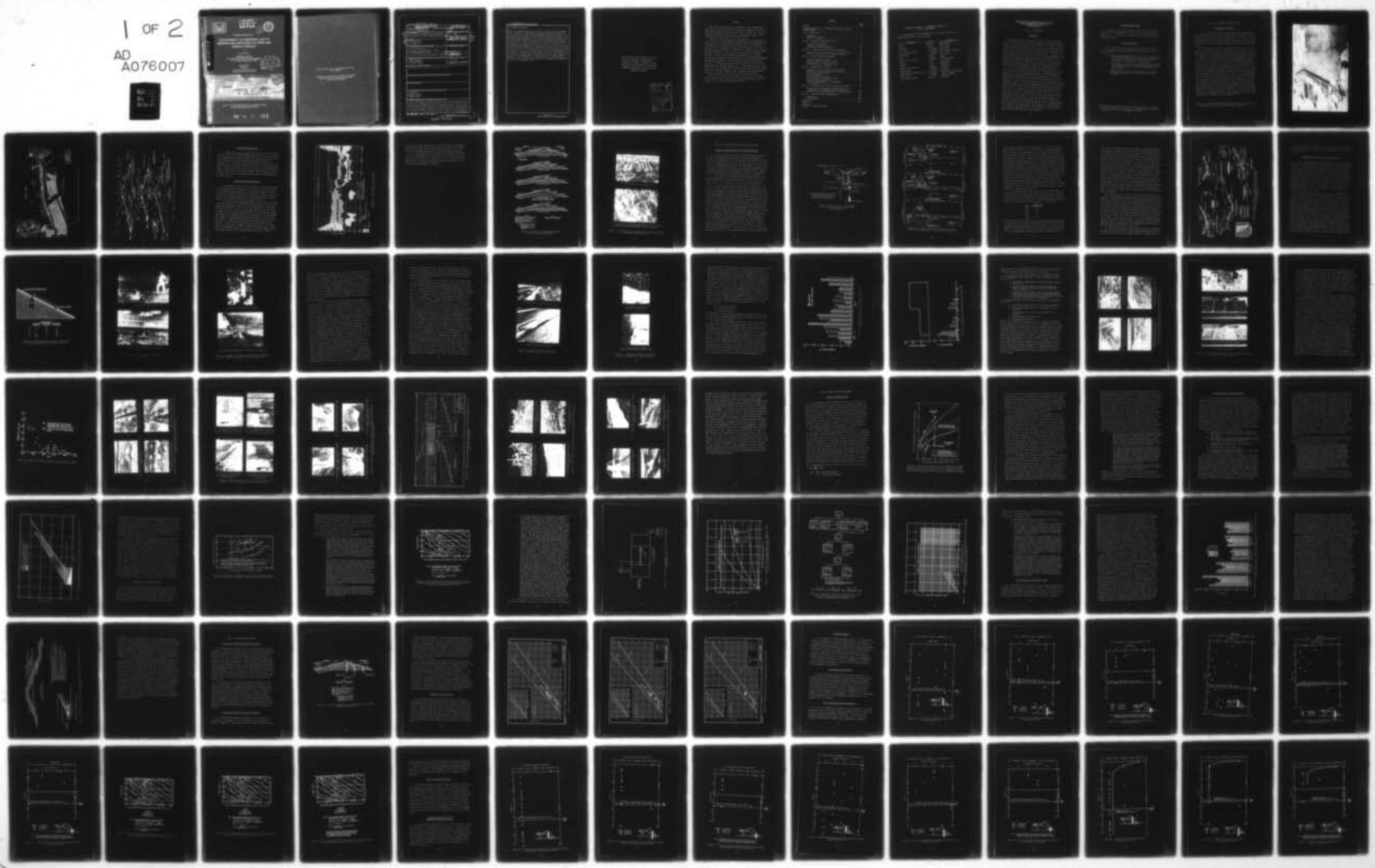

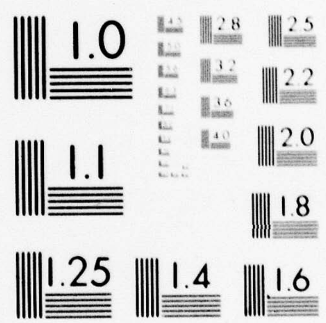

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TECHNICAL REPORT GL-79-14

SUSCEPTIBILITY OF DISPERSIVE CLAY AT GRENADA DAM, MISSISSIPPI, TO PIPING AND RAINFALL EROSION

by

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Past performance of Grenada Dam indicates piping of embankment and foundation soils through joints of the collector pipe for the toe drainage system (prior to its replacement with an open paved ditch in 1961) occurred. Also, rainfall erosion tunnels developed on the downstream slope of the dam, primarily in the valley section between sta 105+00 and 145+00, soon after completion of the main embankment in 1949 and continued to develop at a decreasing rate to the present. Based upon laboratory tests conducted on undisturbed soil C.W.

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20. ABSTRACT (Continued).

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samples and reservoir water samples obtained in 1973 and 1976, the embankment soil is nondispersive at the surface and dispersive below a depth of about 6 ft. Limited data obtained below the embankment indicate the foundation soil is dispersive at the surface and nondispersive to dispersive with depth. The Australian method of analysis, using the exchangeable sodium percentage of the soil, total ionic concentration of the reservoir (eroding) water, and predominate clay mineral of the soil, indicates both the embankment and foundation soils would be potentially susceptible to dispersive clay piping if Grenada Dam contained cracks or sandy lenses traversing the width of the dam where the reservoir water would have a path of rapid access across the dam. Since piping failure through the embankment or foundation has not occurred, either the dam is free from cracks or sandy lenses traversing the width of the dam, or if cracks are present, the soil is able to swell and seal the flow channels. Based upon the history of occurrence of rainfall erosion tunnels on the downstream slope of the dam, it is apparent that the embankment soil at the time of construction was dispersive from the surface (below the 12-in. layer of topsoil) throughout its depth. The downstream slope of Grenada Dam has low susceptibility to rainfall erosion at present due to an amelioration process that has taken place with time that has changed the upper portion of the embankment from a dispersive soil to a nondispersive soil.

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PREFACE

This study on susceptibility of dispersive clay at Grenada Dam, Mississippi, to piping and rainfall erosion was conducted for the U. S. Army Engineer Lower Mississippi Valley Division (LMVD). The study was authorized by LMVED-G letter dated 12 September 1975, subject: WES Investigational Work Program, FY76, FY7T, and FY77.

During the period January 1976 - September 1977, the study was conducted at the U. S. Army Engineer Waterways Experiment Station (WES) by Dr. Edward B. Perry under the general supervision of Mr. Clifford L. McAnear, Chief, Soil Mechanics Division, and Mr. James P. Sale, Chief, Geotechnical Laboratory (GL). Undisturbed soil samples from the embankment and foundation of Grenada Dam were furnished by the Vicksburg District. Reservoir water samples from Grenada Lake were obtained through the cooperation of Mr. Floyd Methvin, Jr., Assistant Park Manager, Grenada Lake Field Office. Pinhole erosion tests were conducted by Mr. Levi R. Coffing, Jr., under the direct supervision of Mr. Gene P. Hale, Chief, Soil Testing Facility, GL. Soil and reservoir water chemistry tests were conducted by Mr. Dennis L. Bean under the general supervision of Mr. Tony B. Husbands, Chief, Chemistry and Plastics Branch, Structures Laboratory. This report was prepared by Dr. Perry.

COL G. H. Hilt, CE, COL J. L. Cannon, CE, and COL N. P. Conover, CE, were Directors of the WES during the study and preparation of this report. Mr. F. R. Brown was Technical Director.

CONTENTS

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
acres	4046.856	square metres
acre-feet	1233.482	cubic metres
cubic feet	0.02831685	cubic metres
cubic feet per second	0.02831685	cubic metres per second
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
gallons (U. S. liquid)	3.785412	cubic decimetres
inches	2.54	centimetres
microns	0.001	millimetres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square miles	2.589988	square kilometres
tons (2000 lb, mass)	907.1847	kilograms

SUSCEPTIBILITY OF DISPERSIVE CLAY AT GRENADA DAM, MISSISSIPPI, TO PIPING AND RAINFALL EROSION

PART I: INTRODUCTION

Background

1. Dispersive clays are a particular type of soil in which the clay fraction erodes in the presence of water by a process of deflocculation. This occurs when the interparticle forces of repulsion exceed those of attraction so that the clay particles go into suspension and, if the water is flowing such as in a crack in an earth embankment, the detached particles are carried away and piping occurs. In addition to the possibility of piping failure, the slopes of earth embankments constructed with dispersive clays are susceptible to rainfall erosion. In natural soil deposits of dispersive clay, erosion due to wave action may occur along shorelines of reservoirs, and erosion due to current action may occur along channels constructed in dispersive clay.[1]

2. One of the largest known areas of dispersive clays in the United States is north-central Misssissippi.[2] Grenada Dam, which is located in this area, has had numerous subsurface erosion channels and subsequent cave-ins develop at relatively shallow depth on the downstream slope of the dam since construction of the main portion of the embankment in 1949.[3] Following reservoir storage in 1954, piping of embankment foundation soils through nonwatertight joints in the toe drainage system collector pipe necessitated many repairs and continuous inspections until the collector pipe was grouted and replaced with an open paved ditch in 1961.[3,4] Although the occurrence of cave-ins on the downstream slope are definitely related to dispersive clays, the piping of foundation soils into the collector pipe was probably due to the omission of watertight connecting bands at the joints. Piping of soil into the collector pipe is considered important to this study because of the possibility that underground erosion channels were being formed in the embankment foundation (shown to be dispersive by limited data obtained in this study) leading to the reservoir.[4]

Objective of the Study

3. The objective of the study was to determine the dispersion characteristics of embankment and foundation soils at Grenada Dam and susceptibility of these soils to rainfall erosion and dispersive clay piping, respectively.

Scope of the Study

4. This study is limited to laboratory tests conducted at the U. S. Army Engineer Waterways Experiment Station (WES) from 6 April to 24 June 1976 on:

a. Undisturbed soil samples from borings 1-73, 2-73, and 3-73 taken from the downstream slope at sta 136+10, offset 16, 94, and 186 ft* right of center line, respectively, of Grenada Dam from 30 January to 7 February 1973.

b. Undisturbed soil samples taken 10 May 1976 from the surface of the downstream slope of Grenada Dam (designated borings 1-76, 2-76, and 3-76) in the vicinity of borings 1-73, 2-73, and 3-73.

c. Reservoir water taken on 10 May 1976 from the surface of Grenada Lake opposite sta 136+10 about 100 ft from the shoreline.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

PART II. BACKGROUND INFORMATION ON DAM

Description of the Dam

5. The Grenada Lake project is located on Yalobusha River approximately 3 miles northeast of Grenada, Mississippi, and is part of a comprehensive flood-control plan for the Yazoo River.[3-11] Table 1 presents the selected data of the Grenada Lake project features. Figures 1 and 2 show an aerial view of the completed project and a general plan, respectively.

6. The main features of the project, as given in Table 1, are a lake, a rolled earthfill dam, a controlled outlet structure with a 17-ft-diam conduit, and a 200-ft-wide uncontrolled concrete chute-type spillway in a saddle in the south abutment ridge. The dam embankment is 13,900 ft in length with a crown width of 40 ft, and the maximum height above the valley floor is 80 ft at crest el 256.0.* Figure 3 shows the typical cross sections of the dam. The upstream slope of the embankment is protected by a 2.5-ft thickness of dumped riprap overlying a 1-ft-thick layer of gravel bedding. The downstream slope was covered with a 12-in. layer of topsoil and spot-sodded with Bermudagrass. During the design phase of the dam, at the second meeting of the Board of Consulting Engineers in January 1946, BG Hans Kramer suggested riprap for downstream slope protection.[6] Although this suggestion was not adopted, in retrospect it would prove to have some merit. A sand drainage blanket was placed beneath the downstream portion of the embankment to control any seepage which might occur through the embankment. A relief well system consisting of 140 wells (replaced by new wells in 1977) along the downstream toe of the embankment controls underseepage in the pervious strata underlying the floodplain of the Yalobusha River.[11,12]

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* All elevations are given in feet above mean sea level.

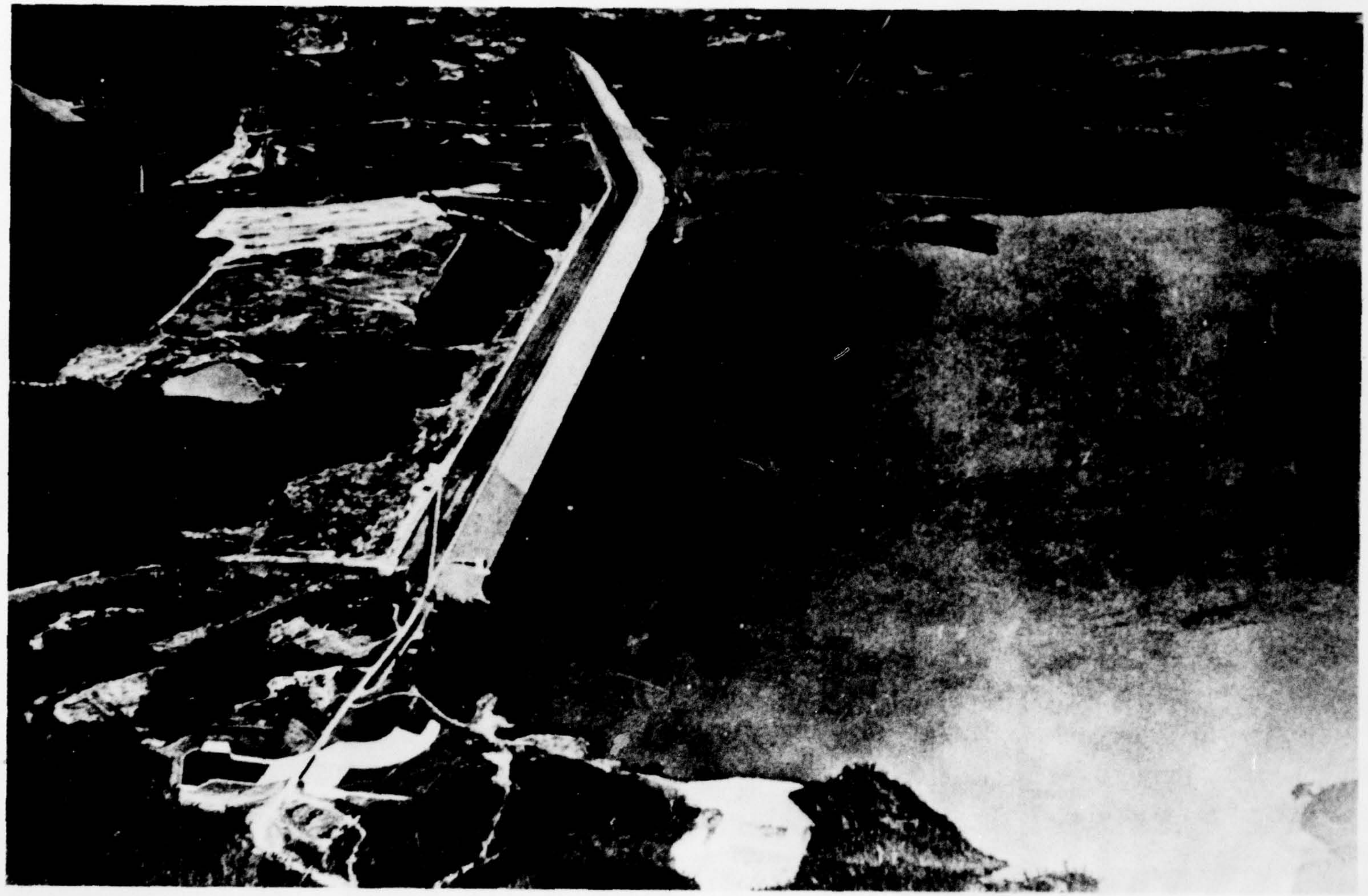


Figure 1. Aerial view of Grenada Lake Project (from Reference 3)

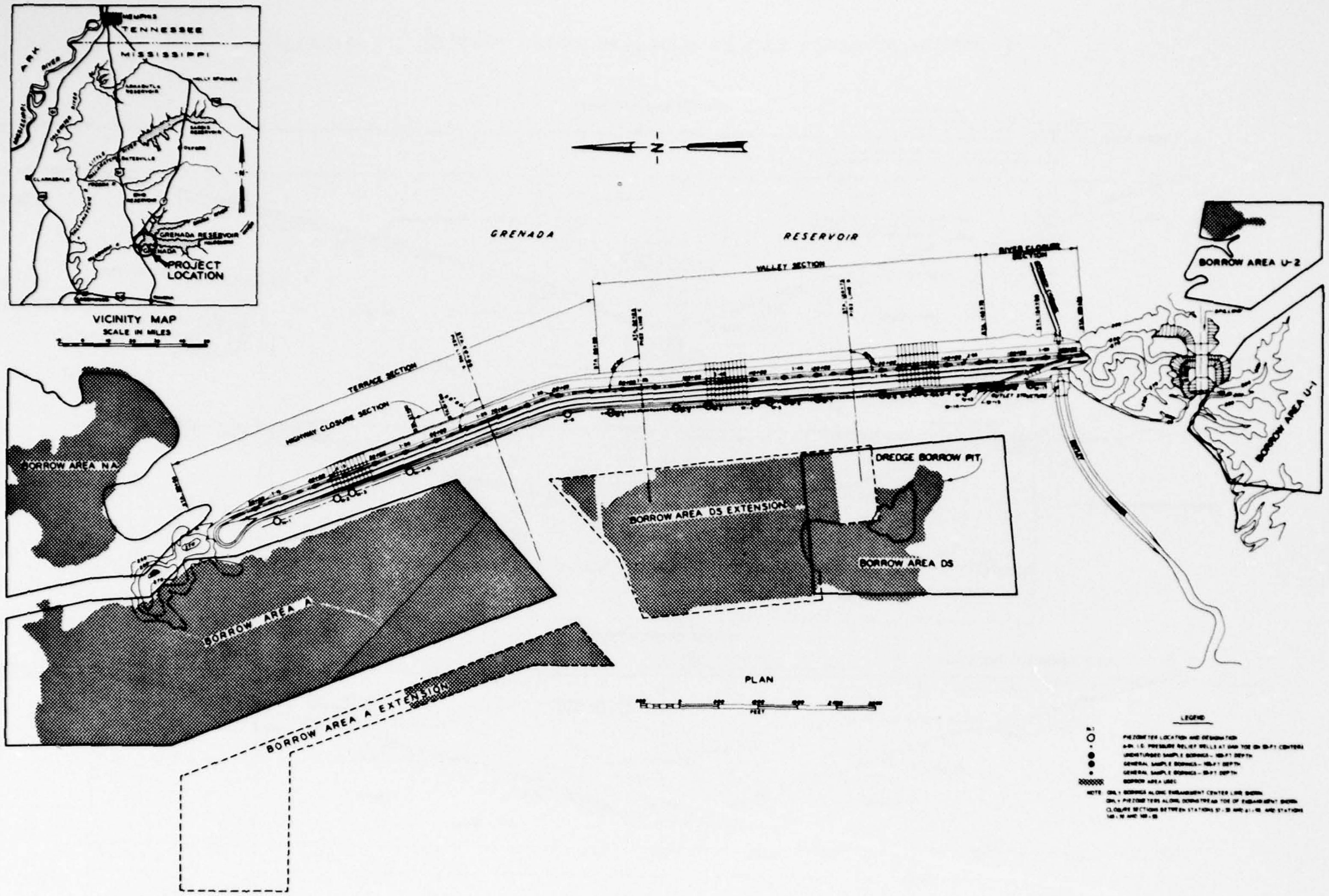


Figure 2. General plan of dam and borrow areas (from Reference 3)

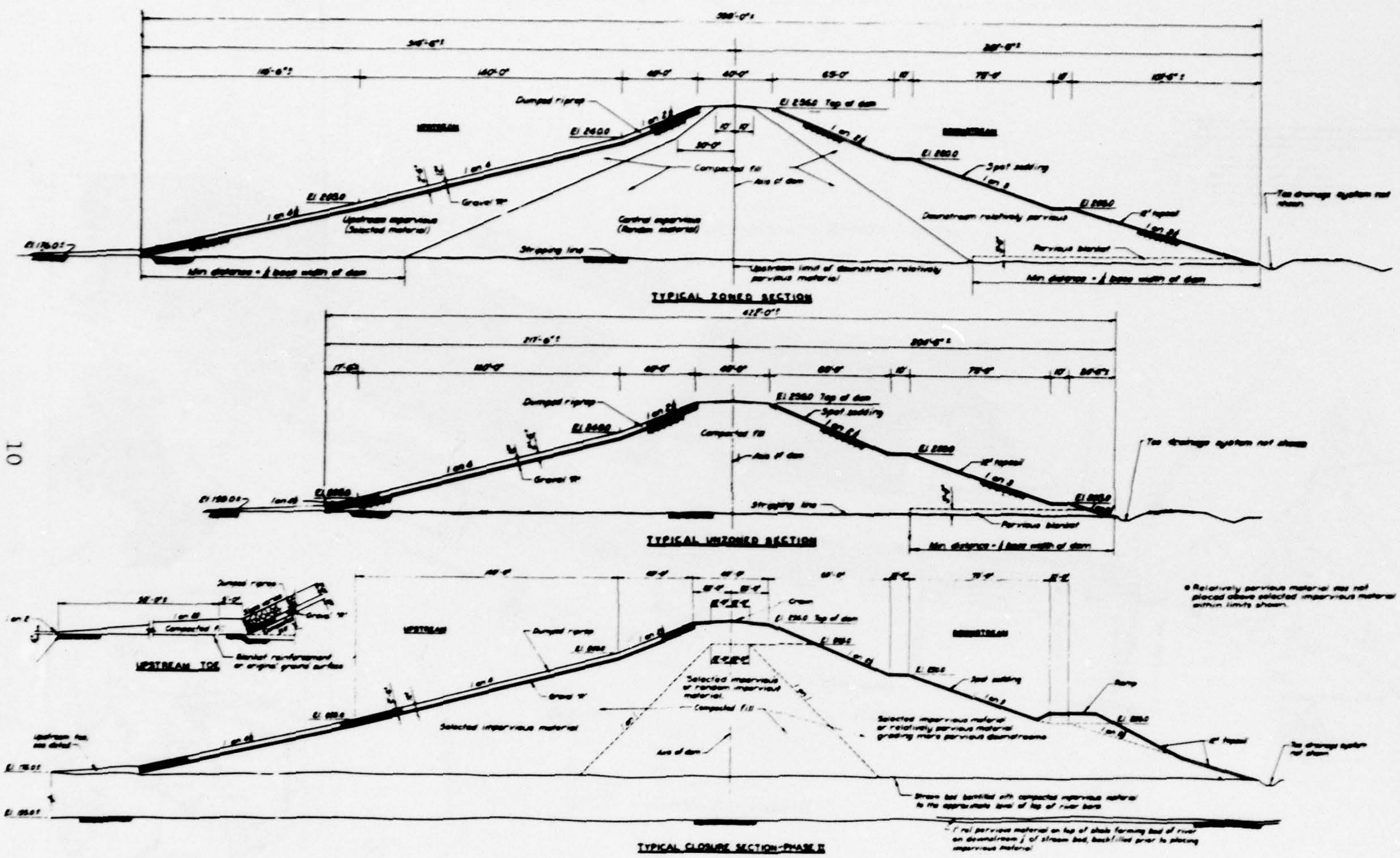


Figure 3. Typical cross sections of dam (from Reference 3)

Schedule of Construction

7. Grenada Dam and appurtenances were constructed under two major contracts. The first contract covered construction of the main portion of the embankment, spillway, and most of the relief wells. Work on the first contract was initiated in February 1947 and completed in September 1949. The second contract, delayed due to lack of funds, was awarded in April 1951 and provided for construction of the outlet structures, river closure section (sta 145+70 to 160+50, south abutment), and highway closure section (sta 57+30 to 61+90) for the highway, railroad, and power line. All work was completed and reservoir impounding began in January 1954.[3,9]

Foundation and Borrow Materials

8. The foundation soils, with the exception of the Recent alluvium in the Yalobusha River floodplain, are of Tertiary origin, dating back to the Claiborne (middle Eocene) age. The Claiborne sediments consist of Basic City deposits of gray silts and clays with layers of shale and older underlying Meridian gray sand with occasional and discontinuous layers of silty sand and silty clay as shown in Figure 4.[3]

9. The most acute problem arising during construction was obtaining sufficient suitable borrow materials.[9] This problem was caused by the widespread distribution of shale in the Tertiary deposits. The extent of laminated strata of shale in borrow areas A and NA (locations shown in Figure 2) had not been suspected because, in the exploration program, auger sampling had remolded and mixed the shale lenses to such a degree that samples of sand-shale mixtures were classified as clay sands and silty sands. In many places the contractor excavated only 2 to 3 ft to separate the shale from other materials. The south abutment borrow areas, U-1 and U-2 shown in Figure 2, were used only to a very limited extent for embankment construction because of the large amounts of shale and organic material and the rugged terrain in this area. Before

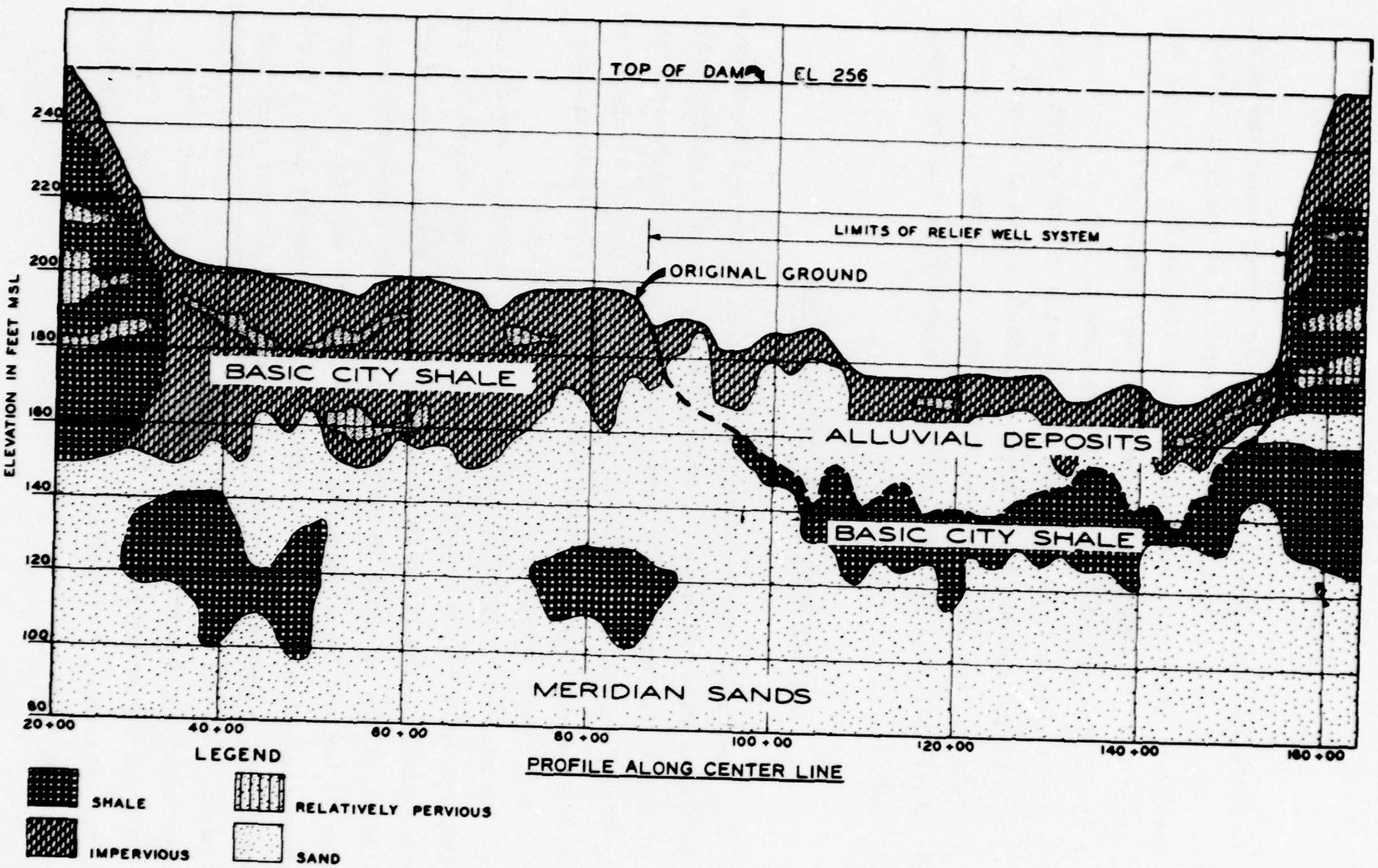


Figure 4. Generalized geologic profile along center line of dam (from Reference 3)

construction of the closure sections, it was decided to extend borrow areas A and DS rather than to use borrow areas U-1 and U-2.[7] Figure 5 shows the utilization of materials in typical sections of the dam.

10. In May 1960, about 11 years after the completion of the main portion of the embankment, erosion patterns were documented in the excavated slope of borrow area DS (Figure 6). This erosion pattern has been reported subsequently in excavated slopes of dispersive clay in Mississippi (see Reference 14, Figure 7).

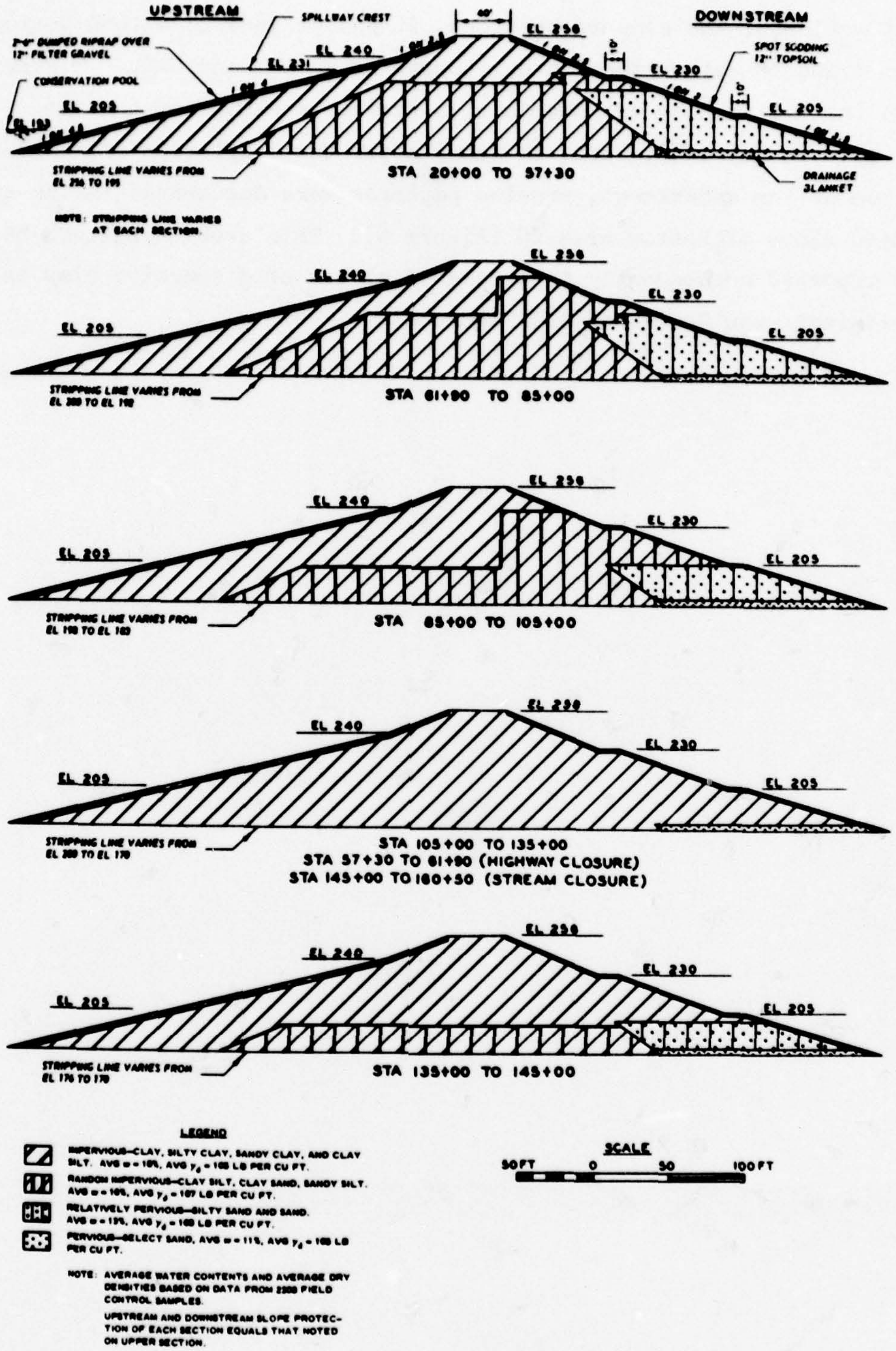
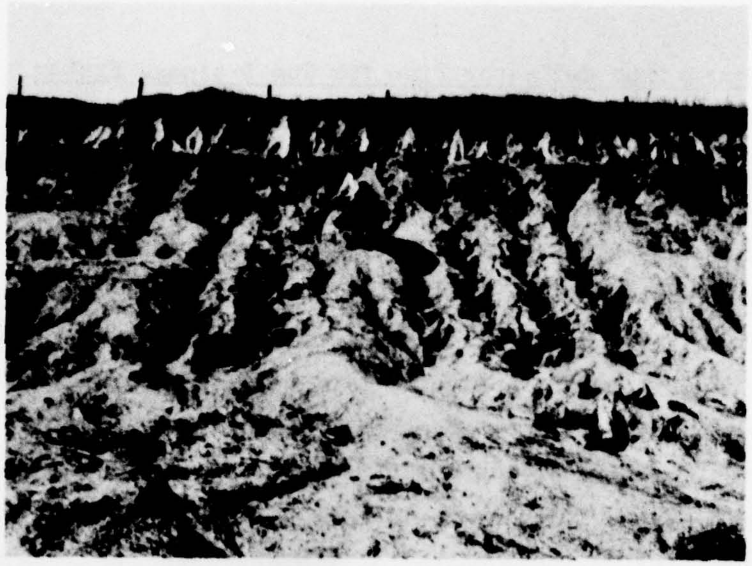


Figure 5. Utilization of materials in typical sections of dam (from Reference 3)



a. General view of slope showing rills

b. Close-up view of slope showing vertical erosion tunnels and surface cracking resembling a karst topography

Figure 6. Erosion pattern of excavated slope of borrow area DS, 24 May 1960 (from Reference 13)

PART III: PIPING AND RAINFALL EROSION PERFORMANCE

Piping into Collector Pipe for Toe Drainage System

11. The collector pipe for the toe drainage system, as installed, consisted of corrugated metal pipe 48 in. in diameter beginning at sta 86+00, then changing to 54 in. in diameter at sta 114+00 and finally to 60 in. in diameter at sta 136+00. The pipe sections were joined with standard (not watertight) 24-in.-wide connecting bands. In addition to relief well discharge, the collector pipe carried surface drainage from the downstream slope entering at manholes on 400-ft centers. Figure 7 shows a typical section of the downstream toe drain at a manhole.[3,4]

12. In February 1954, approximately a month after reservoir impounding began, fairly large cave-ins appeared over the collector pipe at sta 96+94, 98+75, and 144+86, with smaller cave-ins appearing in other locations. The reservoir was at conservation pool stage of 193.0 ft. Piezometer observations taken in April 1955 when the reservoir was at el 223, shown in Figure 8, indicate no seepage had occurred through the embankment. Inspection of the collector pipe showed piping of surrounding material through pipe joints in the vicinity of the sinks. Considerable quantities of sand and silt had been deposited in the pipe, probably as a result of the piping as well as from inwash from the downstream slope and toe road slope through the manholes. It was believed that piping through the joints of the collector pipe had been occurring for a considerable time since installation because of high groundwater level and was not the result of initial storage in the reservoir, although a rise of pool level might have accelerated the piping. Repairs were started immediately on the largest sink at sta 144+86 with excavation of the compacted fill over the pipe by open cut to approximately the top of the pipe where water was encountered. Wood sheet piling was driven in an area about 15 ft square around the pipe joint. When the excavation approached the elevation of the pipe invert, two boils appeared on the downstream side of the pipe near the joint. A boring was made and carried to el 151 on the upstream side of the pipe opposite the

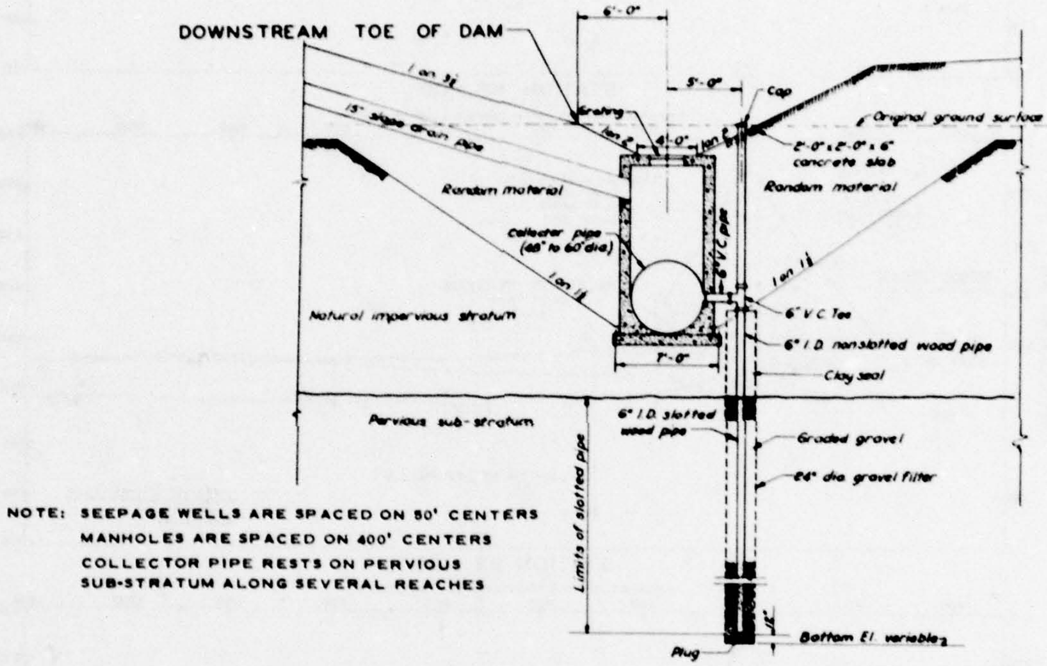


Figure 7. Typical section of downstream toe drain at manhole showing collector pipe (from Reference 3)

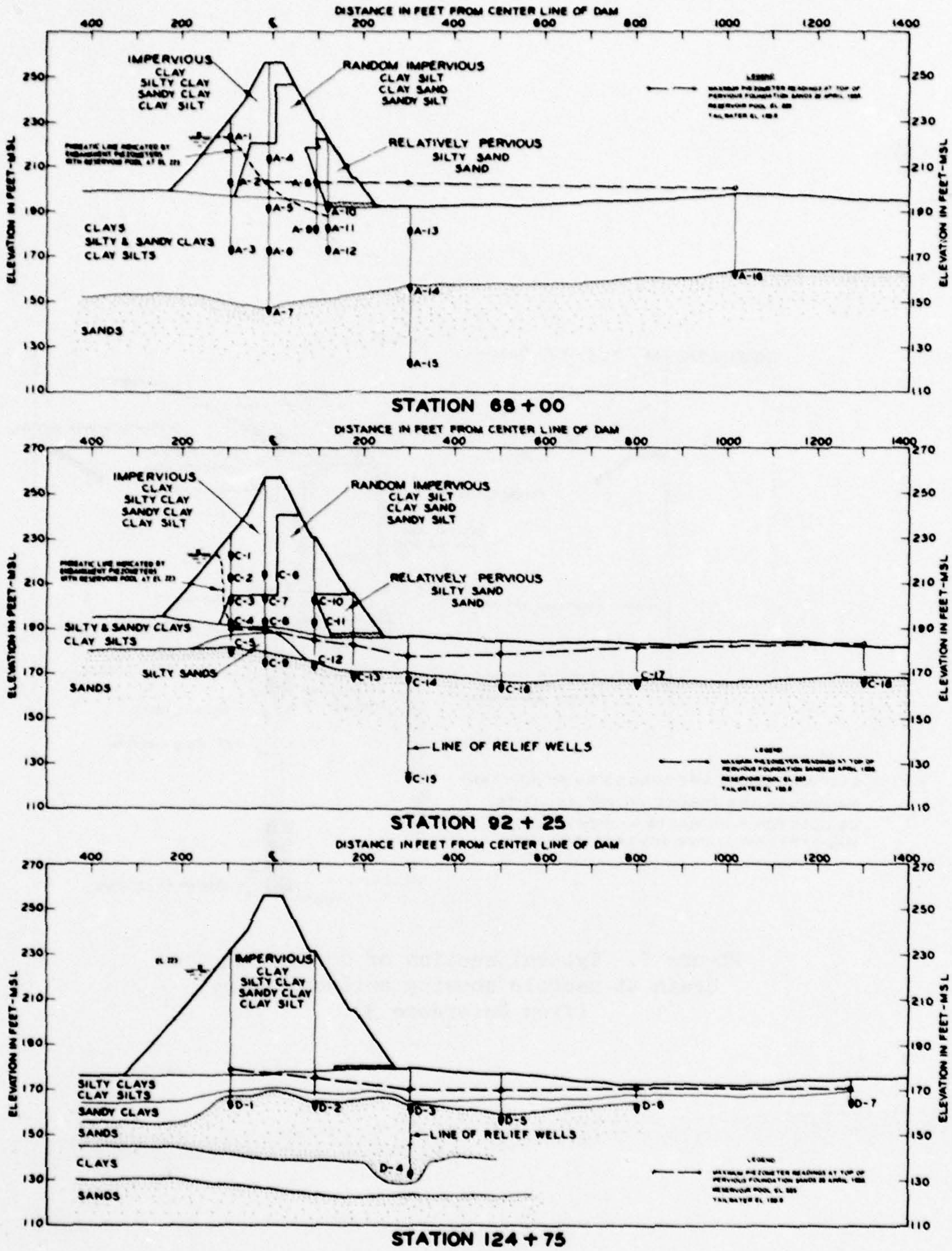


Figure 8. Piezometer observations taken 26 April 1955 during second storage season of reservoir (from Reference 3)

boil locations. The water in the casing rose to el 175.0, which was about 8 ft above the top of the collector pipe. A dewatering system of 20 well points was installed; after a day of pumping, the water was lowered about 11 ft below the pipe invert, and the excavation was completed without difficulty. Inspection of the joint revealed that piping had occurred through a gap about 1.5 in. between the pipe and the connecting band at the bottom downstream end. A reinforced concrete collar, with a minimum thickness of 1 ft from the pipe and a length of 4 ft along the pipe, was constructed around the connecting band. A filter blanket, consisting of 20 in. of pea gravel surrounded by 12 in. of concrete sand on the bottom and sides and 16 in. on top, was placed at the pipe joint. This filter blanket was drained by a 6-in. slotted wood well pipe connected to the collector pipe by a 6-in. asphalt-coated helical pipe. Three small wells, which extended through the alluvial sands to the Tertiary clays near el 142, were installed about 5 ft apart on the downstream side of the pipe. At sta 96+94 and 98+75, repair work consisted primarily of recaulking the joints.[3,4]

13. Following repair of the collector pipe in February 1954, the pipe and relief well outlets were inspected daily to weekly, depending on the reservoir pool, by men walking through the 6800 ft of pipe and looking closely for any indications of foundation sand being carried into the pipe. Leaks that piped foundation sand are listed below:

Year	Number of Piping Leaks
1954	124
1955	171
1956	79
1957	34
1958	37

Profiles taken inside the collector pipe at both crown and invert points of each pipe joint at intervals (June 1954, March 1955, and July 1958) indicated that the settlement and the deformation of the pipe were continuing. In July 1958, a cave-in, about 35 by 20 ft in size and about 1.5 ft in average depth, occurred at sta 148+44. Readings of two piezometers, one located about 80 ft south and the other about 120 ft north, did not indicate any unusual conditions or any substratum pressures different from those previously recorded. The collector pipe was flooded to stop the piping, and a dewatering system of 40 well points was installed to lower the water level to about 2.5 ft below the pipe invert. A steel sheet pile inclosure, 15 ft wide by 52 ft long, was driven inside the well-point system. An inspection at sta 148+44 after the pipe was uncovered showed that five bolts holding the connecting band around the pipe either had pulled out of the angle braces or had failed in tension. Probings with a 3/4-in. reinforcing rod to a depth of 3 to 4 ft indicated that no voids or cavities were present in the bottom of the excavation (approximately el 161, about 1 ft below the pipe invert). Two new sections of collector pipe with watertight connecting bands were installed, and the excavated material was used as backfill with the more clayey material being placed under and around the pipe. The emergency repairs, which were started on 8 July 1958, were made continuously on a three-shift, 7-day-week basis and were completed on 24 July 1958.[3,4]

14. Piping continued in spite of all efforts at repair, and the severity of piping tended to increase with an increase in reservoir pool elevation. Apparently, there was no indication that maintenance costs would decrease with time, and it was believed that major failures, such as occurred in February 1954 and July 1958, would continue in the future. A major concern was the possibility that underground erosion channels were being formed in the embankment foundation leading to the reservoir, and that these channels were being enlarged and lengthened with continued piping. In 1961, the collector pipe was grouted and replaced with an open paved ditch as shown in Figure 9. To relieve potential uplift pressures beneath the collector ditch, 12-in.-diam vertical sand drains were installed. Weep holes were located alternately along each side of the ditch paving at 5- and 10-ft intervals.[3,4]

15. The identical collector pipe system installed at Grenada Dam was used at Enid Dam, which was constructed between February 1947 and July 1951, about 25 miles to the north (Figure 2). Problems with the

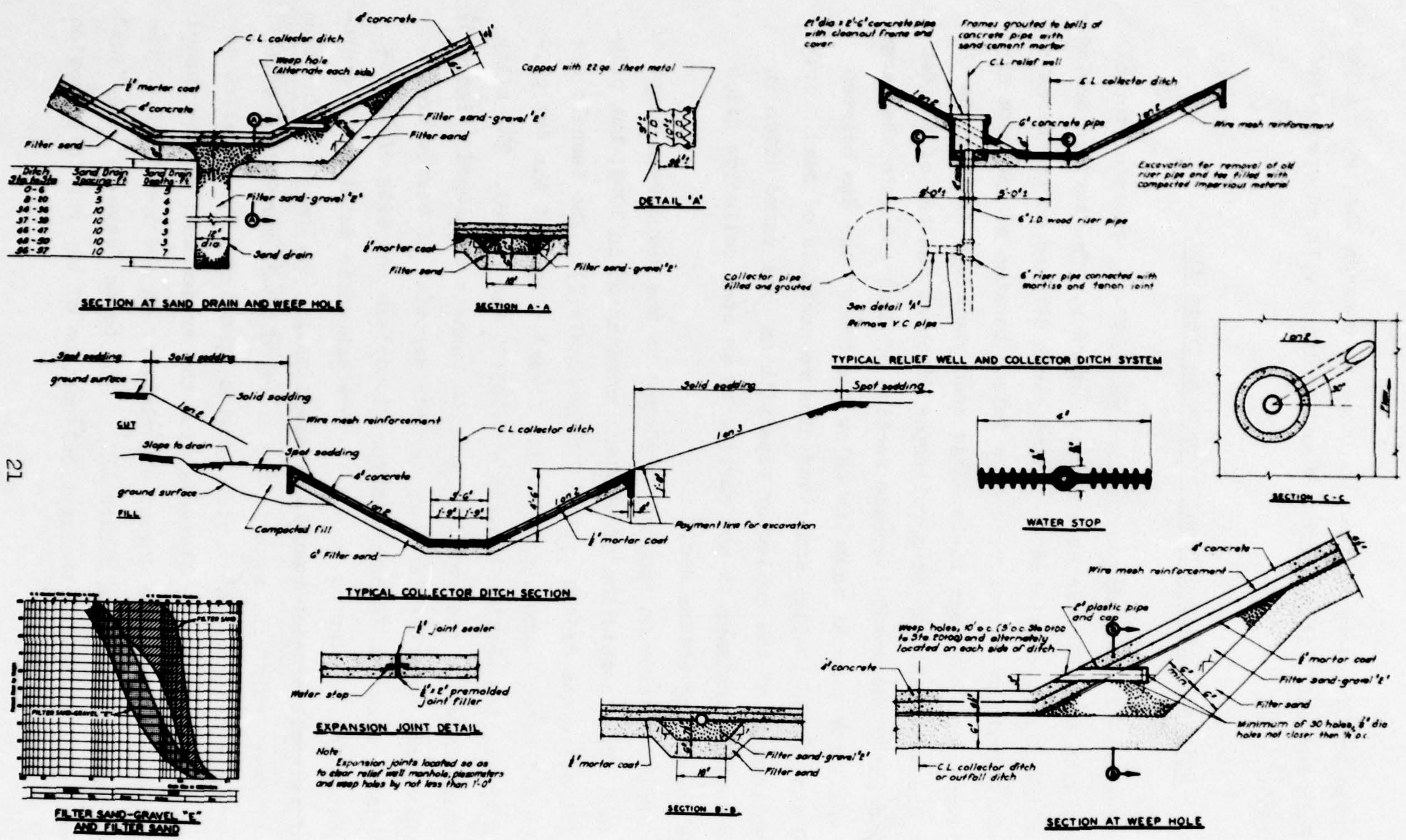


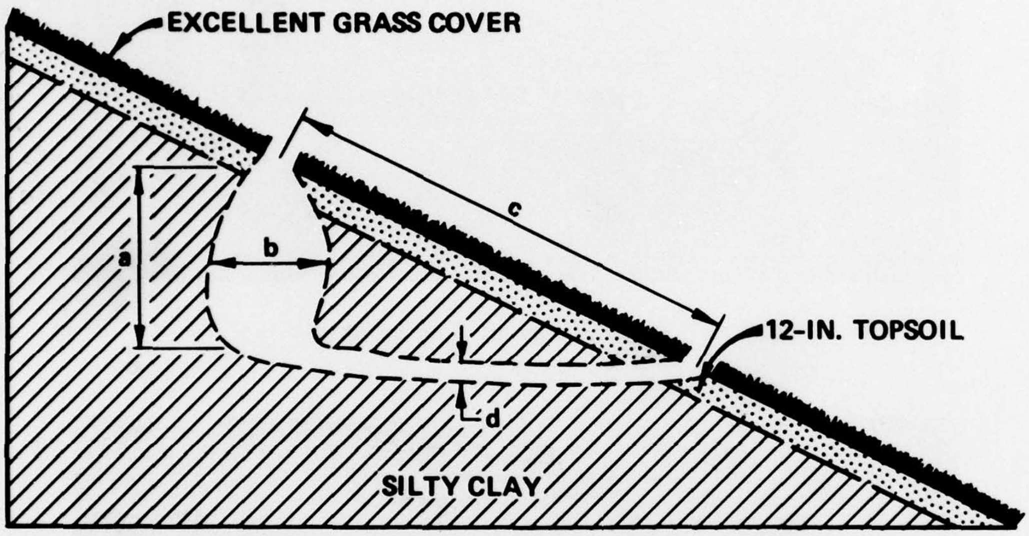
Figure 9. Typical section of collector ditch replacement for collector pipe in downstream toe drainage system (from Reference 3)

collector pipe system, very similar to those at Grenada Dam, were experienced at Enid Dam until it was grouted and replaced with an open paved ditch in 1965.[15]

Rainfall Erosion Tunnels on Downstream Slope

16. The downstream slope protection was a 12-in. layer of topsoi spot-sodded with Bermudagrass as shown in Figure 5. The surface drainage system, as installed, consisted of sodded earth ditches on the berms to carry surface drainage to concrete drop inlets located on the berm on 400-ft centers, and eighteen 15-in.-diam concrete slope drain pipes to discharge the water into an open collector ditch at the toe of the dam between sta 25+00 and 86+00 (terrace section of dam) and into the large diameter collector pipe at manholes below the toe of the dam between sta 86+00 and 160+50 (valley and river closure sections of dam). Following replacement of the collector pipe with an open paved ditch in 1961, the surface drainage is now carried by an open collector ditch along the toe of the entire dam.[3]

17. Rainfall erosion tunnels developed in the downstream slope of the dam soon after completion of the main embankment in 1949, but principally following the drought year of 1952 (Table 2). The tunnels developed on all three (upper, middle, and lower) slopes of the dam primarily in the valley section between sta 105+00 and 145+00. The silty clay material in which the rainfall erosion tunnels developed (Figure 5) was obtained from both the Tertiary terrace deposits of borrow area A and the Recent alluvium Yalobusha River floodplain deposits of borrow area DS extension.[6] Figures 10 and 11 show schematic details and photographs of rainfall erosion tunnels, respectively. During short duration periods of heavy rainfall, such as the storm of 20-21 March 1955 with 9.3 in. of rainfall, the horizontal erosion tunnel may run parallel to the ground surface for some distance with the eroded soil at the tunnel exit being deposited on the downstream slope or in the toe ditch in the form of a mound as shown in Figure 12.[17] The terms "cave-in" and "jug" have been used to describe the vertical portion of the rainfall erosion



	DIMENSIONS, FT		
	MINIMUM	AVERAGE	MAXIMUM
a.	1	4	7
b.	1	3	5
c.	2	6	25
d.	0.5	1	3

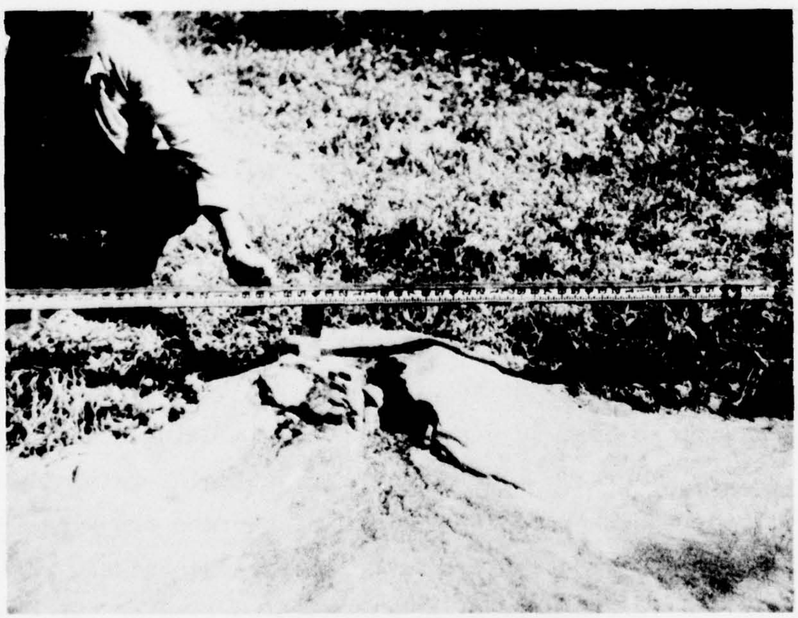
Figure 10. Schematic details of rainfall erosion tunnel in downstream slope (after References 13 and 18)

a. View upstream toward crest

b. View downstream toward collector ditch

Figure 11. Photograph of rainfall erosion tunnel, 30 January 1976

a. View downstream showing tunnel entrance



b. View upstream showing mound of eroded soil at tunnel exit

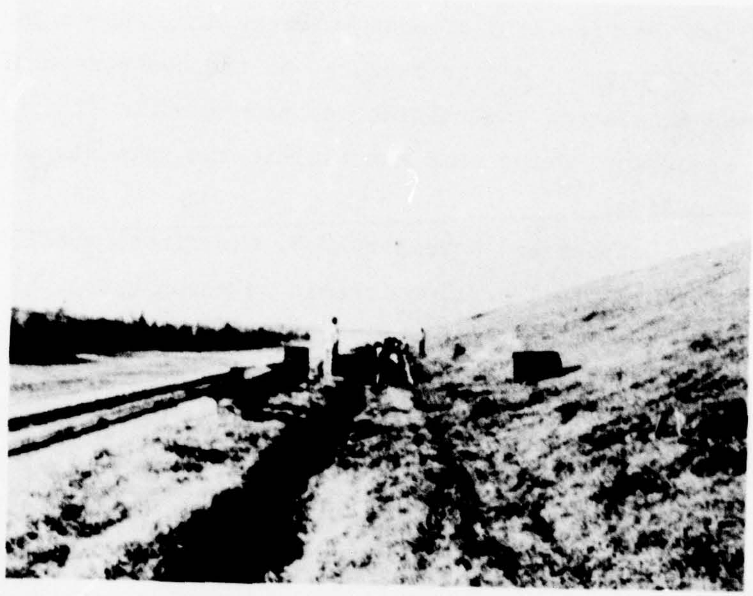
Figure 12. Photograph of rainfall erosion tunnel following the 9.3-in. rainfall, 20-21 March 1955 (from Reference 17)

tunnel. Although Figures 10 and 11 show only one horizontal erosion outlet tunnel, there were often several horizontal erosion outlet tunnels for each cave-in. For example on 14 March 1956, there were 133 cave-ins and 459 horizontal outlet tunnels found on the downstream slope.[13] The largest cave-in reported to date at Grenada Dam was located on the 205 el berm at sta 122+50 and was about 5 ft in diameter and 5 ft deep with a subsurface channel about 3 ft in diameter heading downslope.[18] The largest cave-in reported by the Soil Conservation Service (SCS) occurred in the crown of a 25-ft-high dam in Oklahoma and was 4 ft in diameter and 7 ft deep.[19] Both of these cave-ins occurred at locations where the contributing watershed areas were small.

18. The rainfall erosion tunnels are initiated when shrinkage cracks, as much as 1 in. wide at the ground surface and tapering to zero at a maximum depth of 7 ft, are formed during periods of low rainfall and high evaporation. At Grenada, Mississippi, from May through October the mean precipitation is about 19 in. while the mean pan evaporation is about 39 in.[20] The type and amount of vegetation influence the amount of water lost through evapotranspiration, size and number of shrinkage cracks, and rainfall infiltration. Surface water from rainfall runoff enters the cracks and erodes the soil (carries away the dispersed fines) before the cracks can swell and close. The cracks generally run together on the surface of the slope in a dendritic pattern, like the branches of a tree, to form progressively larger cracks as the surface water moves down the slope. Eventually the vertical portion of the rainfall erosion tunnel becomes so large that the overlying soil collapses into the cavity. Although the development of the rainfall erosion tunnels with time was not investigated, it has been suggested that the cave-ins develop over a period of several years.[21] During the excavation of rainfall erosion tunnels in 1954, a rather large and unusual surface crack pattern was found. A similar erosion pattern was discovered about six years later (about 11 years after completion of the main portion of the embankment) in the excavated slope of borrow area DS (Figure 6). Although the dam was not damaged structurally by the rainfall erosion tunnels, which were limited to a depth of about 7 ft, the cave-ins were unsightly, some manholes along the collector pipe (prior to its replacement in 1961) were

undermined, and the stability of some concrete drop inlets located on the berm was threatened. Repair measures to the downstream slopes consisted of hand excavating the rainfall erosion tunnels (later bulldozers were used), recompacting the same material in the channels with power tampers, and sodding.[3,18]

19. During October and November 1954, the first experimental methods of preventing rainfall erosion tunnels were conducted. Two 400-ft sections of the downstream slope were modified by concrete paving of portions of the berm ditches between sta 132+00 and 136+00 and by grouting with a 10 percent bentonite and 90 percent silt mixture between sta 136+00 and 140+00. As shown in Figure 13, the paved ditch, 4 ft wide and 4 in. thick, at the junction of the slope toe and berm sloping gently to the drop inlets was intended to stop water from ponding on the berm and feeding the cracks to form rainfall erosion tunnels. Initially, grouting was tried in horizontal erosion tunnels that were exposed in the sidewalls of a 1.5-ft-wide by 3.5-ft-deep machine-dug ditch. The erosion tunnel was covered with a steel grout cover plate and held in place with jacks bearing on the opposite wall of the ditch, as shown in Figure 14. Grout was pumped into the erosion tunnel using a mud-jacking machine, ordinarily employed in lifting concrete pavements and foundations, which was capable of pumping a stiffer grout mixture than conventional cement grouting equipment. Very low grouting pressures were used because the soil overlying the horizontal erosion tunnels contained shrinkage cracks which prevented the buildup of grout pressure and made it relatively easy to lift the soil overlying the tunnel. This method proved to be slow and tedious and was abandoned for a method of grouting from holes. Holes for grouting, 5.5 ft deep, 3 in. in diameter, and spaced 18 in. apart, were drilled with one line at the edge of the dam crown and the other lines located immediately downstream of the two berms. Grout was introduced near the bottom of the hole through a 2-in.-diam pipe and grouting continued until grout returned in adjacent holes in an attempt to provide a continuous grout curtain to a depth of about 5 ft. Numerous shrinkage cracks in the soil allowed the grout to penetrate the 18 in. between drill holes 90 percent of the time. Good grout take occurred

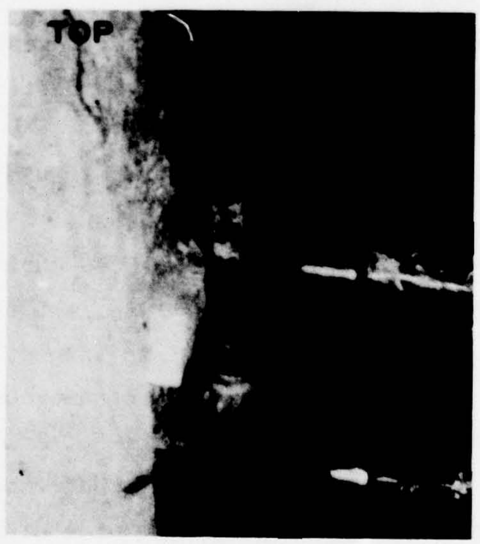


a. Excavation and forming work on berm ditch

b. Completed berm ditch paving

Figure 13. Experimental berm ditch paving, October - November 1954 (from Reference 22)

a. Horizontal erosion tunnel
exposed on side of ditch



b. Steel grout cover plate
in place over horizontal erosion tunnel

Figure 14. Experimental grouting from ditches,
October - November 1954 (from Reference 22)

when the drill holes encountered an occasional horizontal erosion tunnel.[22] It was hoped that the effectiveness of the experimental work could be determined from hosing water on the slope and ponding water in the berm ditches before and after the work was done. However, this was unsuccesful because stakes marking the locations of the original horizontal outlet tunnels were disturbed or lost. From observation of the downstream slope following the 9.3-in. rainfall of 20-21 March 1955, a reduction in the number of cave-ins in all three experimental sections when compared with the number of cave-ins in the remainder of the main embankment was noted. Test pits dug in the grouted section about one year later showed that the grouted horizontal erosion tunnels were well filled with grout and there was no shrinkage development.[23,24] After about a year's observation, noting the evidence in favor of any one method was not conclusive, the relative order of effectiveness (in terms of most to least) was given as:[25]

a. Grouting from ditches.

b. Grouting from holes.

c. Berm ditch paving.

In January 1956, it was decided to observe the downstream slope during and following another heavy rainfall prior to adopting a method of preventing rainfall erosion tunnels.[26]

20. During the winter months from October 1955 to January 1956, the rainfall was 9.76 in. below normal as shown in Figure 15. Prior to 3 February 1956, there were no cave-ins noticed during an inspection of the downstream slope. From 1-6 February 1956, a total of 5.71 in. of rainfall occurred as shown in Figure 16. An inspection of the downstream slope on 7 February revealed 55 cave-ins and numerous horizontal outlet tunnels. During the period 16-20 February 1956, a total of 2.28 in. of rainfall occurred, after which it was found the number of cave-ins had increased to 114. Cave-ins were found over nine of the eighteen 15-in.-diam concrete slope drain pipes that carried surface drainage from the concrete drop inlets on the berm ditches to the collector pipe at manholes at the toe of the dam.[27] On 14 March 1956, there were 133 cave-ins and 459 horizontal outlet tunnels found on the downstream slope.[28]

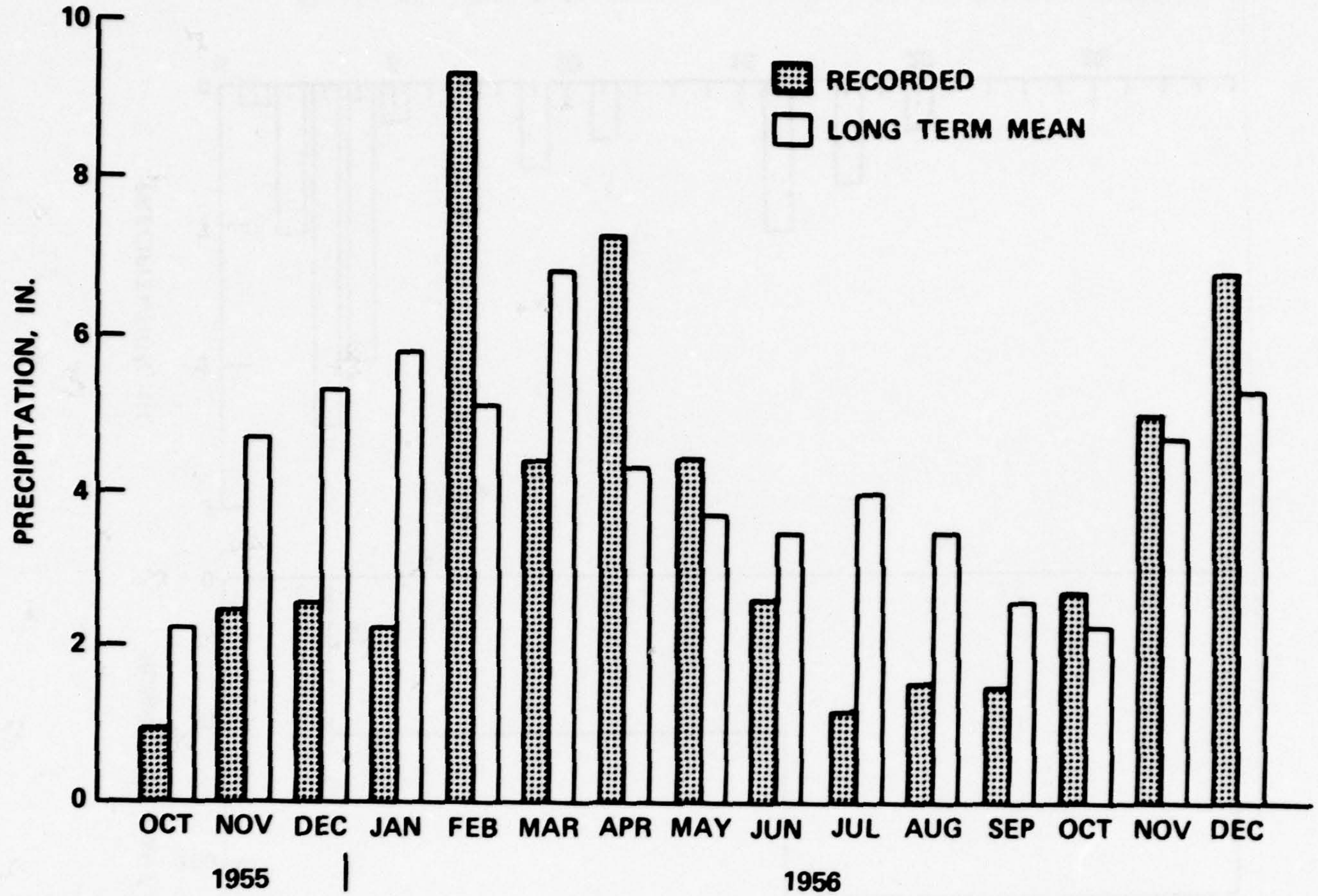


Figure 15. Precipitation from October 1955 to December 1956 for Grenada, Mississippi (from Reference 16)

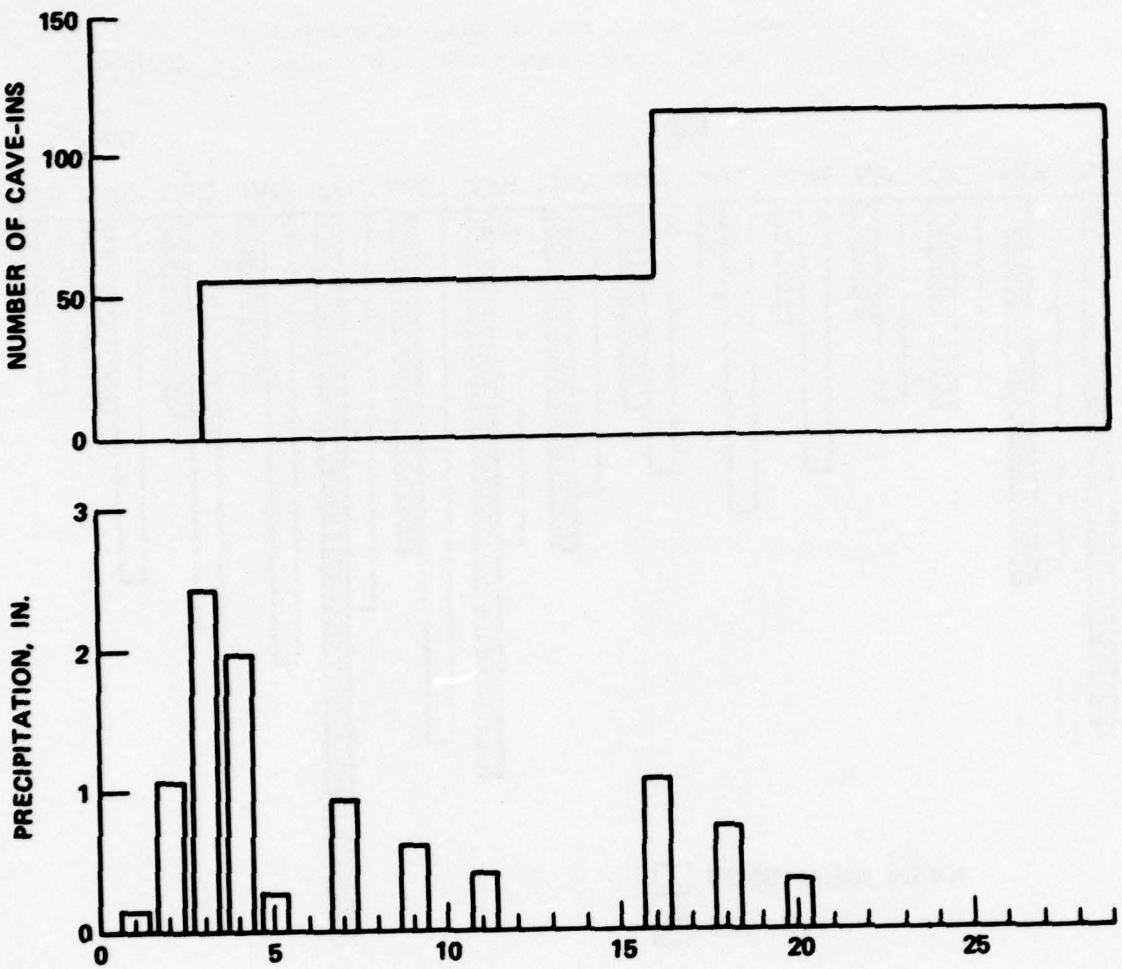


Figure 16. Precipitation and cave-ins during February 1956

Figure 17 shows the repair measures that were carried out from March to June 1956 to ready the dam for dedication on 25 July 1956.

21. On 30 April 1956, the Vicksburg District requested approval from the Mississippi River Commission to conduct a subsurface erosion study to:[28-32]

a. Determine the causes and/or factors that produce subsurface erosion.

b. Develop a practical laboratory test for determining whether a prospective borrow material has subsurface erosion tendencies.

c. Determine the most feasible means for repairing embankment slopes where subsurface erosion exists and for preventing subsurface erosion when it is indicated by laboratory test on prospective borrow material.

Soil samples were taken from the following locations in the Vicksburg District:[33]

a. North bank of Arkansas River Levee, Gillette, Arkansas.

b. Cypress Bend Levee, west bank of Mississippi River, Arkansas.[34]

c. Richland Bend Levee, south bank of Arkansas River, Arkansas.[35]

d. Panola-Quitman Levee, Mississippi.

e. Grenada Dam, Mississippi.

Samples were obtained from areas where cave-ins had occured and from adjacent areas where no erosion was evident. A comparison of the grain-size distribution, Atterberg limits, specific gravity, compaction, linear shrinkage, slaking test, capillary rise test, and clay mineral composition did not show any trend that would lead to identification of the material as being susceptible to rainfall erosion tunnels.[33] Unfortunately, physico-chemical tests on the soil, which were recommended in the proposed study, were not carried out.[28-30] A laboratory test was devised where soil specimens compacted in shallow pans were allowed to air-dry, mounted on a slope, and subjected to water sprinkled on the upper end as shown in Figure 18. However, the laboratory test was not conclusive in identifying material as being susceptible to rainfall erosion tunnels.

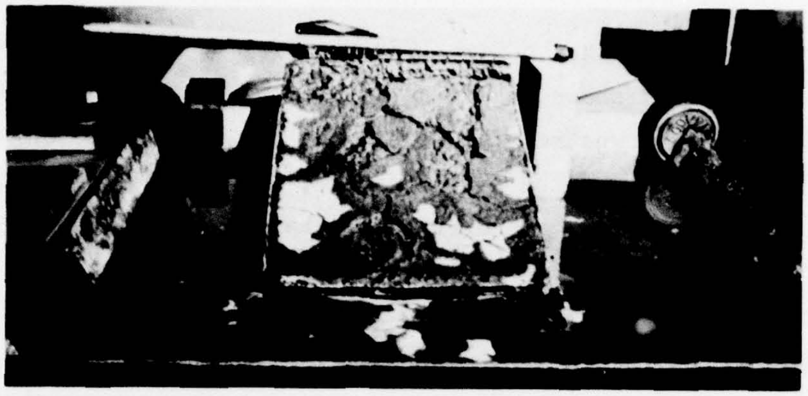
a. Excavation around concrete slope drain pipe and drop inlet, 28 March 1956

b. Backfilling around slope drain pipe sta 106+00, 19 March 1956

c. Backfilling over slope drain pipe sta 106+00, 20 March 1956

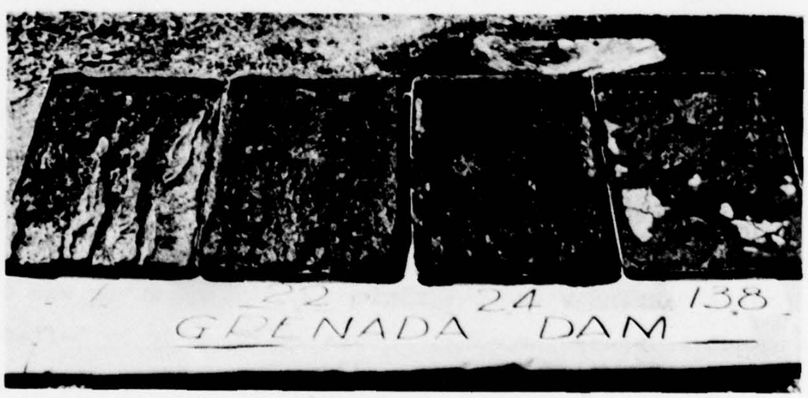
d. Excavating rainfall erosion tunnels sta 107+00, 20 March 1956

Figure 17. Repairing rainfall erosion tunnels, 19-28 March 1956 (from File 1514-02 Subsurface Erosion, Vicksburg District)



a. Rainfall simulator

b. Before erosion test



c. After erosion test

Figure 18. Rainfall simulation test employed by Vicksburg District, 16 April 1956 (from Reference 32)

22. Beginning with the year 1956 records are available, as indicated in Table 3 and shown in Figure 19, for the number and locations of cave-ins on the downstream slopes of the dam. The cave-ins were about equally distributed over the upper, middle, and lower downstream slopes with the greatest intensity occurring in the valley section between sta 105+00 and 145+00. Over 100 cave-ins per year are recorded for the period 1956 to 1960. Figures 20 and 21 show repair of rainfall erosion tunnels in the vicinity of concrete slope drain pipes and drop inlets in June 1958 and March 1960, respectively. Figure 22 shows excavated rainfall erosion tunnels prior to backfilling.

23. During May and June 1960, the second experimental method of preventing rainfall erosion tunnels was conducted. Following a suggestion made by the Greenwood Area Office in February 1956, it was decided to place barriers to the flow of water in the horizontal erosion tunnels.[27] One 400-ft section of the downstream slope was modified by curtain wall grouting between sta 120+00 and 124+00 and one 800-ft section by clay-filled trenches between sta 124+00 and 132+00 with locations shown in Figure 23. Curtain wall grouting was conducted by drilling 13 parallel lines about 20 ft apart of 2-in.-diam by 6-ft-deep holes on 18-in. centers placed longitudinally along the downstream slope of the dam between the crown and the toe (Figure 24). A grout mixture of silt with 7.5 percent granular bentonite by volume was used with the end of the grout pipe near the bottom of the hole and top of the hole plugged for about 1 ft by an enlarged section of the pipe. Granular bentonite was used in the grout mixture for its ability to swell later when surface drainage seeped into the horizontal erosion tunnels. The silt and bentonite were premixed in a small concrete mixer before being placed in the hopper of the mud-jacking machine where the minimum amount of water was added to produce a grout that was pumpable. Seven 19-in.-wide by 5-ft-deep trenches were dug with a machine approximately 35 ft apart longitudinally along the downstream slope of the dam between the crown and the toe (Figure 25). The trenches were backfilled with a lean clay (CL) and compacted in about 4-in.-thick compaction lifts using pneumatic tampers.[13] Prior to the curtain wall grouting and clay-filled trenching

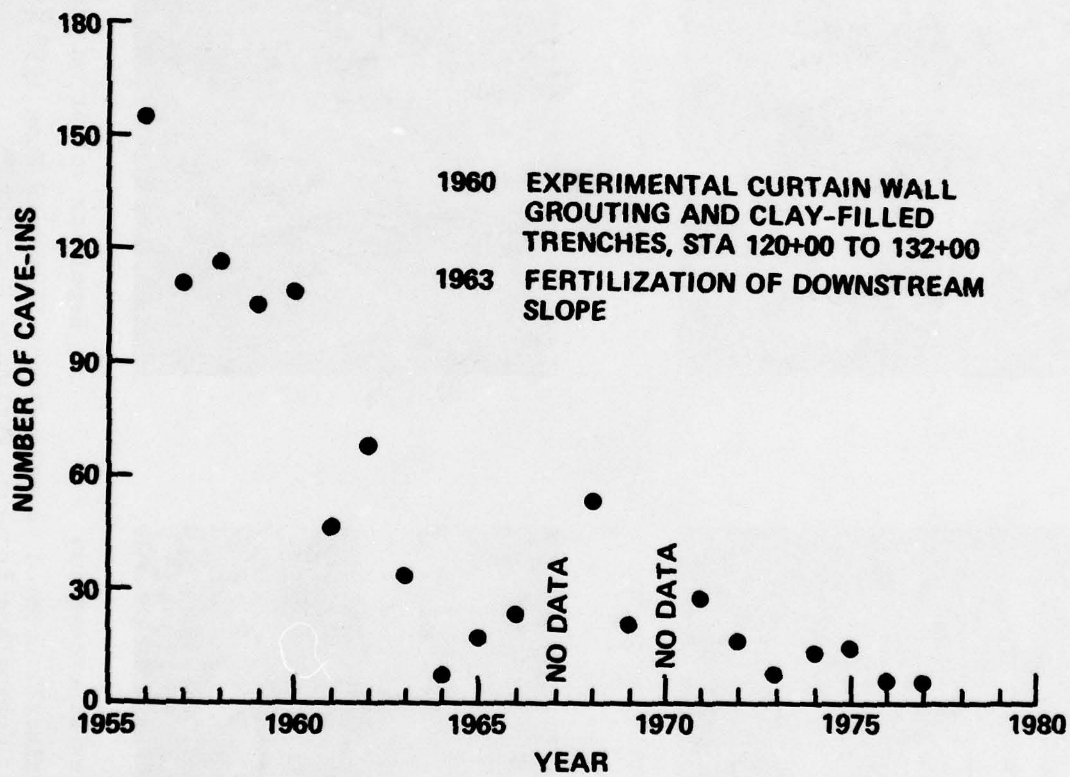
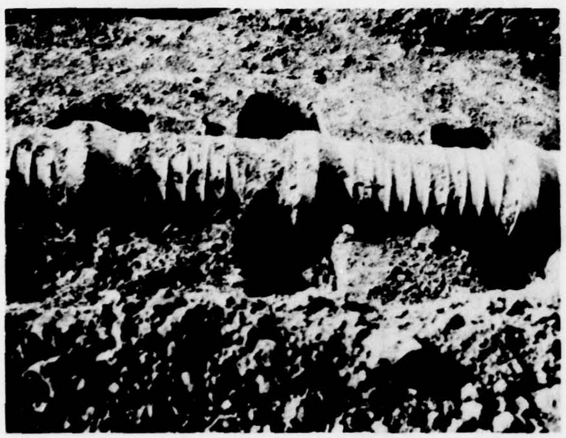


Figure 19. Number of cave-ins, 1956-1977 (from References 13, 36-44)

a. Excavation around concrete slope drain pipe and drop inlet

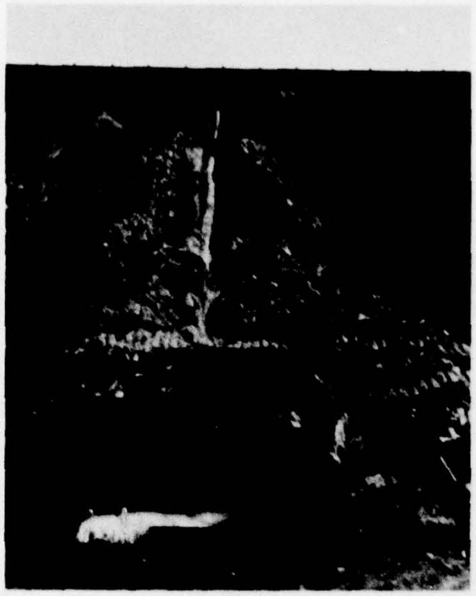
b. Close-up top view of excavation



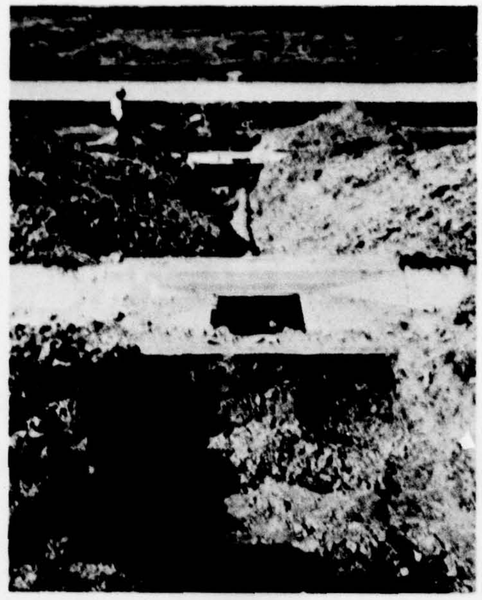
c. Close-up side view of excavation

d. Repairing collar on slope drain pipe

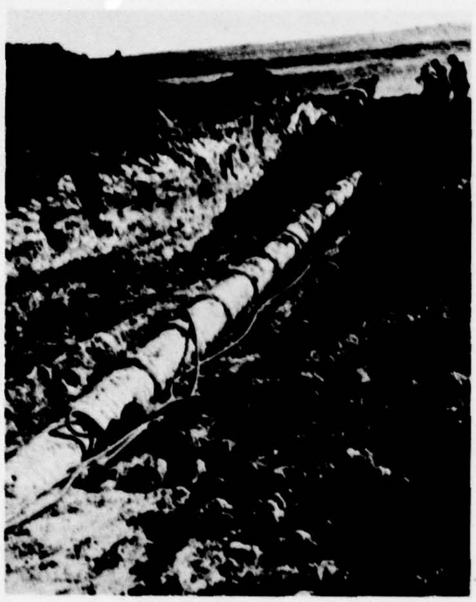
Figure 20. Repairing rainfall erosion tunnels at sta 118+00, 9 June 1958 (from File 1514-02 Subsurface Erosion, Vicksburg District)



a. View upstream of excavation around concrete slope drain pipe and drop inlet, 28 March 1960



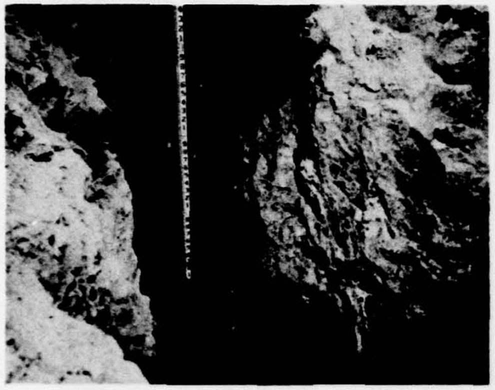
b. View downstream of excavation around concrete slope drain pipe and drop inlet, 26 March 1960



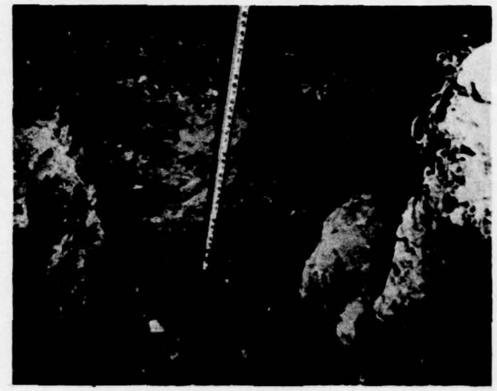
c. Backfilling around concrete slope drain pipe sta 106+00, 19 March 1960

d. View upstream following repair of rainfall erosion tunnels, 23 March 1960

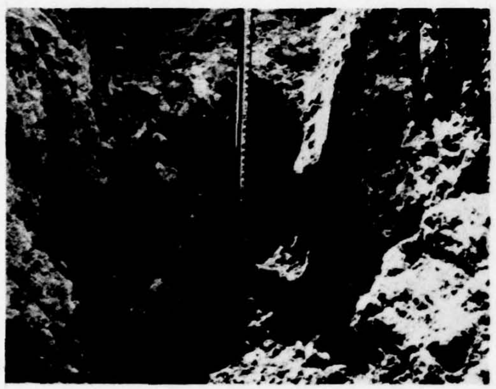
Figure 21. Repairing rainfall erosion tunnels, 19-28 March 1960 (from File 1514-02 Subsurface Erosion, Vicksburg District)



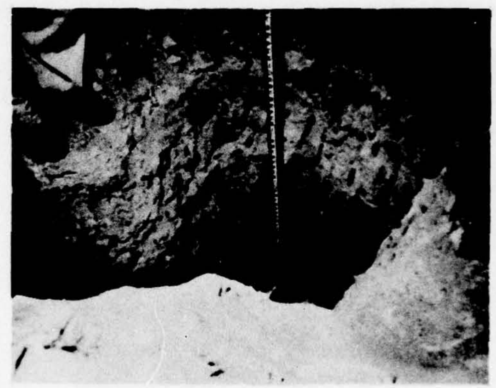
a. Rainfall erosion tunnel sta 138+00 (middle slope), 26 March 1960



b. Rainfall erosion tunnel, 26 March 1960



c. Rainfall erosion tunnel, 4 April 1956



d. Rainfall erosion tunnel sta 144+00 (lower slope), 26 March 1956

Figure 22. Rainfall erosion tunnels, 26 March - 4 April 1956 (from File 1514-02 Subsurface Erosion, Vicksburg District)

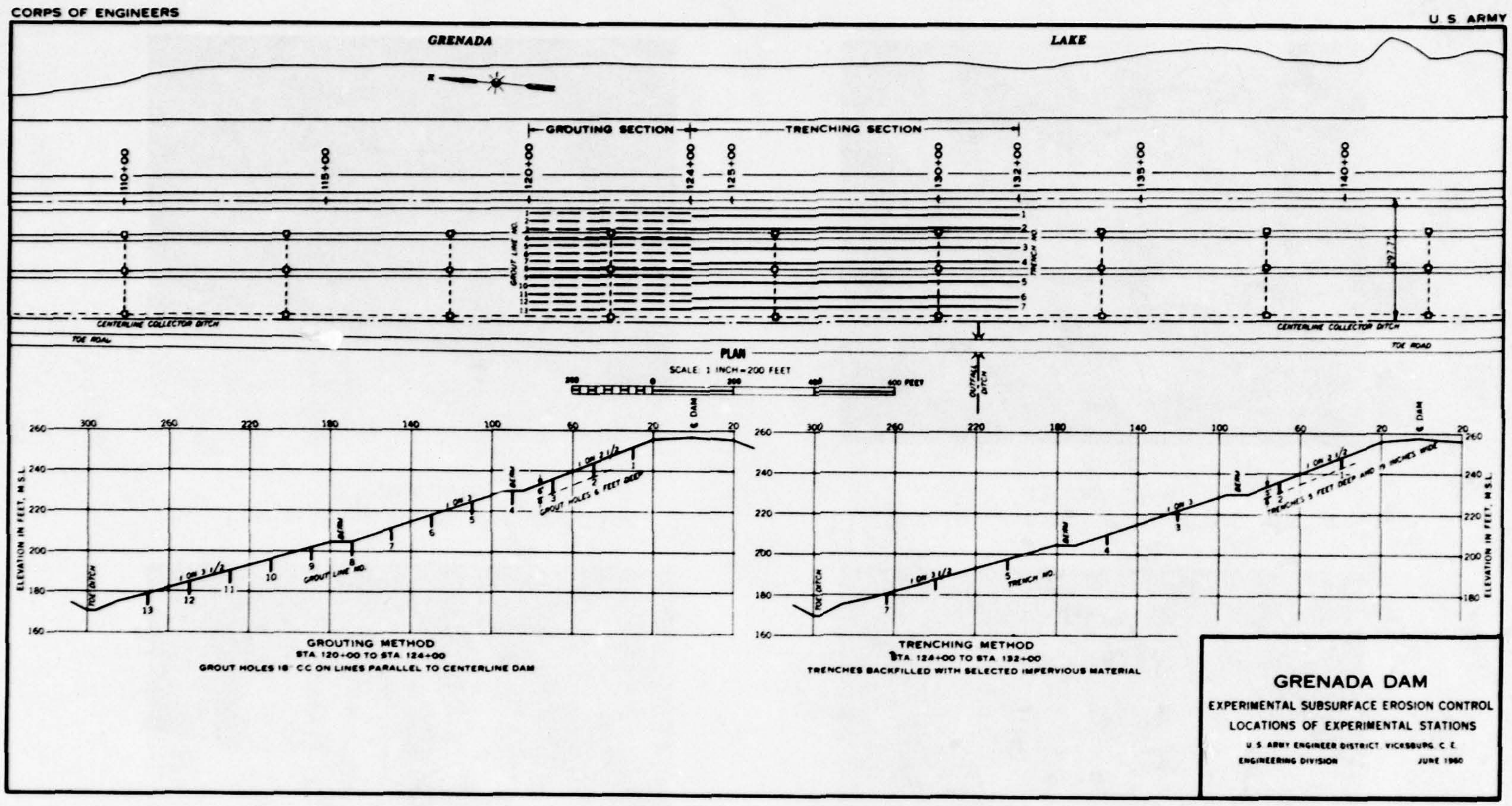
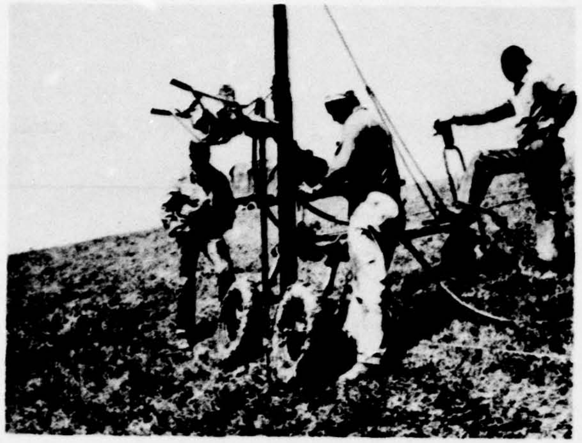


Figure 23. Location of experimental curtain wall grouting and clay-filled trenches, May - June 1960 (from Reference 13)



a. Predrilling holes for grouting, 13 May 1960

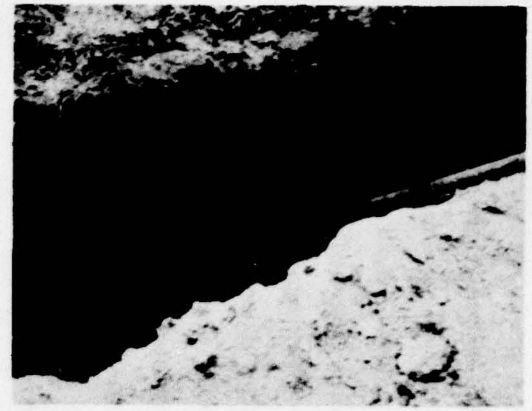
b. Obtaining samples of grout mixture, 13 May 1960

c. Grouting with mud-jacking machine, 24 May 1960

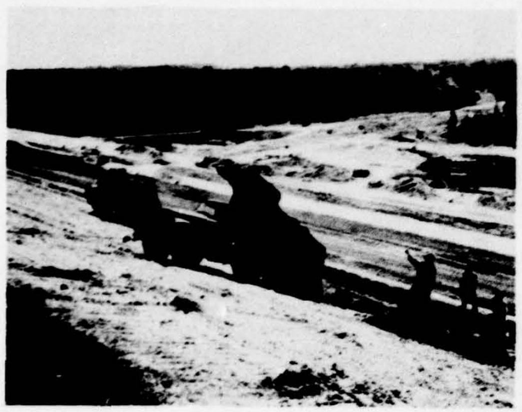
d. Close-up view of grouting, 10 May 1960

Figure 24. Experimental curtain wall grouting, 10-24 May 1956 (from Reference 13)

a. Opening trench, 10 May 1960



b. Horizontal erosion tunnel exposed on side of trench, 10 May 1960



c. Placing clay backfill in trench, 13 May 1960

d. Compacting backfill with pneumatic tampers, 13 May 1960

Figure 25. Experimental clay-filled trenches, 10-13 May 1960 (from Reference 13)

in 1960, about 110 cave-ins per year were reported with the greatest intensity occurring between sta 105+00 and 145+00. In February 1961, a total of 13.2 in. of rainfall occurred ending a prolonged drought (Table 2) and providing a good test for the experimental methods of preventing rainfall erosion tunnels installed from May to June 1960. Following the February 1961 rainfall, 46 cave-ins were found on the downstream slope (Table 3). Only five cave-ins were located in the experimental test section, two in the curtain wall grouting section, and three in the clay-filled trench section. Most of the 41 cave-ins outside the experimental test section (sta 120+00 to 132+00) occurred between sta 110+00 and 140+00.[3] In June 1963, the downstream slope of the dam was heavily fertilized to increase the density of sod cover. As shown in Table 3, the experimental test section was instrumental in reducing the number of cave-ins that occurred on the downstream slope of the dam during the period 1961 - 1969 (no data available 1967). No cave-ins were reported in the experimental test section from 1962 to 1966. In 1971, 10 years after construction of the experimental test section, the frequency of occurrence of cave-ins in the experimental test section increased. The ratio of cave-ins in the experimental test section to total number of cave-ins has averaged about 25 percent from 1971 to present (no data available for 1976). Since approximately the same number of cave-ins occurred in the curtain wall grouting section as in the clay-filled trenching section, it is concluded that the performance of the two sections is about the same.

PART IV: REVIEW OF PREVIOUS RESEARCH

Piping in Dispersive Clays

24. As stated previously, dispersive clays are a particular type of soil in which the clay fraction erodes in the presence of water by a process of deflocculation. Empirical piping criteria developed 25 years ago were based on the assumption that soil type and method of construction were the main parameters controlling the resistance of homogeneous earth dams to piping. Dispersive clays cannot be identified by conventional index tests such as Atterberg limits, particle size distribution, and compaction characteristics. Australian workers have developed a method of analysis, as shown in Figure 26, to assess the susceptibility of a homogeneous earth dam (constructed entirely or almost entirely of a single embankment material) to dispersive clay piping.[45]

25. As shown in Figure 26, prior to first filling of the reservoir, the exchangeable sodium percentage (ESP)*[46] of the soil is in equilibrium with the total ionic concentration of the pore water of the soil expressed in milliequivalents per litre (meq/ℓ). Upon first filling of the reservoir, the susceptibility to dispersive clay piping may be predicted from Figure 26. If the dam contains cracks or sandy lenses traversing the width of the dam, the reservoir water will have a path of rapid access across the earth dam. For this case, the eroding water will be the reservoir water. The total cation concentration will change rapidly from the soil pore water (initial position of dam A, Figure 26) to the reservoir water (final position of dam A, Figure 26) while the ESP of the soil remains constant because it does not have time to change. If the final position of the dam plots in the deflocculated zone, as is the case

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* $$\text{ESP} = \frac{\overline{\text{Na}}}{\text{CEC}} (100)$$

where $\overline{\text{Na}}$ = exchangeable sodium

CEC = cation exchange capacity

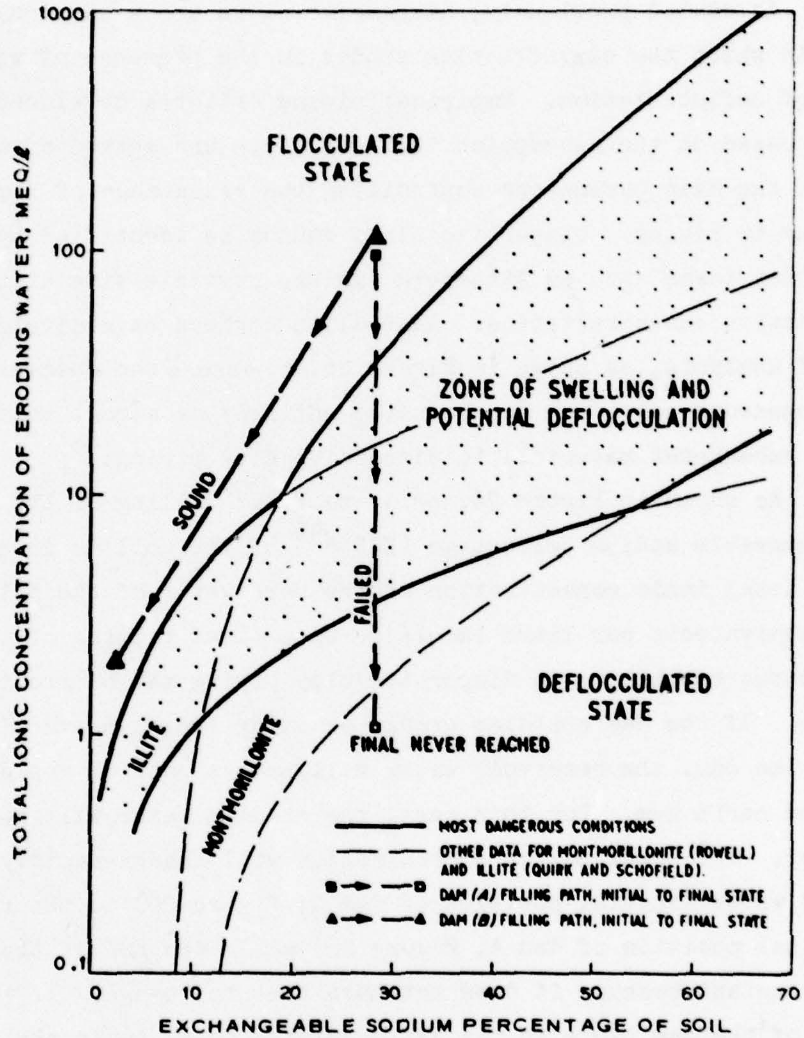


Figure 26. Prediction of piping failure in homogeneous earth dams using the total ionic concentration of the eroding water and the ESP and predominate clay minerals of the soil (adapted from Reference 45 with the permission of the Publishers Butterworth, Sydney)

for dam A, the dam may fail by dispersive clay piping upon first filling of the reservoir. The only deterrence to failure by dispersive clay piping for a homogeneous earth dam would be the ability of the embankment material to swell and seal the flow channels. The applicability of Figure 26 to predict piping failure in homogeneous earth dams upon first filling of the reservoir was demonstrated by Aitchison and coworkers in a study of 20 earth dams representative of a wide variety of climatic and soil conditions from four states in Australia.[47,48]

26. In addition to possible dispersive clay piping failure in homogeneous earth dams upon first filling of the reservoir, it is also possible that piping failure may occur at some later time if the ionic concentration of the reservoir water is substantially reduced. Ingles and Wood[49] presented a case history of a dam in an area of saline soil at Lakes Entrance in Victoria, Australia. The reservoir was originally filled with well water relatively high in ionic concentration (26 meq/ℓ) and was stable for some years although continuous seepage losses were noted. Following completion of a 20-mile pipeline to bring water relatively low in ionic concentration (1.2 meq/ℓ) from the Nicholson River, the dam failed by piping within 3 days. In the United States, the possibility of piping failure in homogeneous earth dams constructed of dispersive clay resulting from a decrease in the ionic concentration of the reservoir would probably be the greatest in the southwest where the ionic concentration of some river waters exceeds 50 meq/ℓ as compared with a range of 1-5 meq/ℓ for other regions of the country (Table 4).

27. It was suggested by Ingles and coworkers[45,51-53] that Figure 26 could be used to predict the susceptibility of a homogeneous earth dam, free from cracks or sandy lenses traversing the width of the dam, to dispersive clay piping at some future time. For this case, the eroding water will be the seepage water through the earth dam. Soil samples are taken from the earth dam at various intervals of time to determine the ESP of the soil and the total ionic concentration of the pore water of the soil. It may take several years to reach the final equilibrium state. Some variation in ionic concentration of the seepage water may result from chemical stratification with depth of the reservoir water.[54]

This procedure was used to evaluate the performance of Flagstaff Gully Dam, Tasmania, which was reconstructed after piping failure following first filling of the reservoir.[53] Over the reporting period (1964-1968), the relationship between the ESP of the soil and the total ionic concentration of the seepage water followed a path within the stable flocculated state similar to that shown for the initial portion of the curve for dam B in Figure 26. The slowness of the transient state path movement and the estimated stable final equilibrium position indicated Flagstaff Gully Dam would be safe from dispersive clay piping. Sherard and coworkers[1,55] have questioned the validity of using Figure 26 to predict the susceptibility of a homogeneous earth dam, free from cracks or sandy lenses traversing the width of the dam, to dispersive clay piping resulting from seepage water movement through the earth dam. Sherard and coworkers contend that if a homogeneous earth dam constructed of dispersive clay is retaining a reservoir and no leaks develop, then no dispersive clay piping failure will subsequently occur because:

a. If such actions could occur, it would be expected that we would have records of piping failures of homogeneous earth dams occurring after long periods of successful operation, but such records are practically nonexistent. In all records known to Sherard and coworkers, dispersive clay piping occurred either following first filling of the reservoir or, less frequently, after raising the reservoir pool to a higher level than previously existed.

b. In pinhole erosion tests (described in paragraph 33) on soil specimens giving nondispersive reactions, there is no evidence of change in the quantity of flow with time, even when the test is continued for long periods of time (hundreds of hours). If dilution of the soil pore water concentration could cause a change from the nondispersed to dispersed state, it would be expected that at least a few soils, which gave a nondispersive reaction at the beginning of the pinhole erosion test, would start to disperse toward the end of the test.

c. As the soil pore water concentration is diluted by seepage water moving through the earth dam, the percent sodium usually decreases or remains constant.

Sherard and coworkers[1] noted that under unusual conditions, such as cracking from earthquake shaking or drying cracking following a prolonged period of low reservoir, failure by dispersive clay piping could result if a leak developed.

Rainfall Erosion of Dispersive Clays

28. Soil particles are detached by raindrop impact and runoff tractive forces (force per unit area exerted by the flow of water over the soil). Runoff occurs whenever the rainfall intensity is greater than the infiltration rate of the soil. The erosion process begins when raindrops strike the earth's surface and detach soil particles by splash. The erosive potential of rainfall depends on the raindrop fall velocities, size distribution, and total mass at impact. Once the soil is detached by the raindrop impact, sheet and rill erosion begin. Sheet erosion is the removal of a fairly uniform layer of soil by the action of raindrop impact and runoff tractive forces. Once runoff starts, rill erosion soon begins and may develop into gullies (erosional features that cannot be removed by normal soil cultivation). There are five major factors which contribute to rainfall erosion:[56]

- a. The nature of the rainfall as given by its intensity, duration, drop size distribution, drop velocity, and impact energy.
- b. The properties of the soil affecting infiltration, shrink-swell potential, erodibility including dispersion potential, sediment transport, and deposition.
- c. The steepness and length of the slope and presence of shrinkage cracks in the slope.
- d. Cover provided by plants and residues.
- e. Cultural and soil-management practices that reduce runoff by modifying soil and cover conditions.

29. Table 5 gives the rainfall erosion potential of natural, cut, and fill (vegetated and nonvegetated) slopes of nondispersive and dispersive soils. There are several important differences in the rainfall erosion potential of nondispersive and dispersive soil slopes. Natural slopes in nondispersive soils, normally covered with vegetation and containing organic matter in the topsoil in humid areas, usually exhibit small erosion. Dispersive soils are usually not present in the topsoil of natural slopes due to the process of eluviation (movement of clay particles downward in the soil profile).[1,58-60] Huddleston and Lynch[58] in a study of dispersive soils in Mississippi found that although

severe rainfall erosion tunnels developed in SCS dams, no rainfall erosion damage was found in the undisturbed natural soil adjacent to the dams. Nonvegetated cut and fill slopes in nondispersive soils may exhibit small, moderate, or severe rainfall erosion depending on the index properties of the soil shown in Figure 27, steepness of the slope, and size of the contributing watershed area. Although the mechanism of the development of rainfall erosion with time for nonvegetated cut and fill slopes in dispersive clays has not been investigated, it is known that under certain climatic conditions (heavy rainfall following a drought, Figure 15) severe rill and tunnel erosion will develop in cut slopes (Figure 6). Vegetated cut and fill slopes in nondispersive soils usually exhibit small rainfall erosion. Vegetated cut slopes in dispersive clays are presently under investigation by the SCS in an excavated channel near Wynne, Arkansas.[61] As shown in Figures 12 and 22, vegetated fill slopes in dispersive clays exhibit severe tunnel erosion under certain climatic conditions (heavy rainfall following a drought, Figure 15).

30. In the study by Huddleston and Lynch[58] of homogeneous earth dams, generally in the range from 20 to 40 ft in height, constructed by the SCS in north-central Mississippi, it was stated:

> Another observation concerning dams constructed with dispersive soils is the "healing" that occurs after the first few years provided there is maintenance to prevent cracks from getting out of control shortly after construction . . . A good example of this is Piney Creek Site 21 that was completed in 1960 with no attempt to put topsoil on the outside shell. A few (jug) holes developed in the first few years that were repaired. A recent (1975) inspection of four dams in this watershed showed all embankments in excellent condition. Soil samples were collected from the slopes on Site 21 and tested . . . the same pattern of less dispersive soils in the upper one foot was found even though no topsoil was used at the time of construction. Similar experiences have occurred on some of the dams in Askalmore watershed where early maintenance was provided and no further problems have developed.

Table 6 presents the results of laboratory tests on samples obtained from the Piney Creek Sites 21 and 35 in 1975, about 15 years after construction of the dam. The test results indicate that for a homogeneous

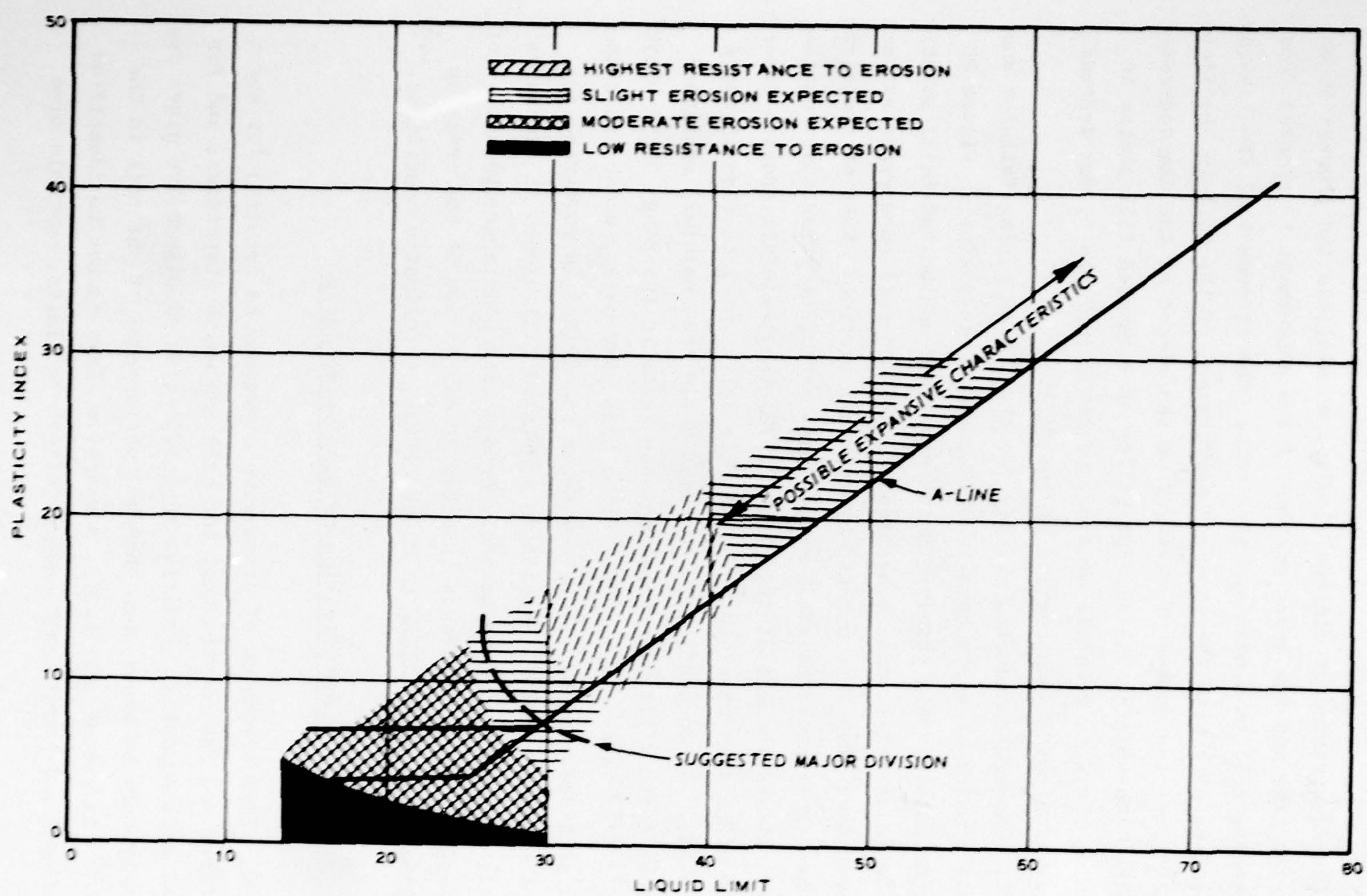


Figure 27. Relationship between plasticity and erosion characteristics for nondispersive soils from both laboratory tests and field observations (from Reference 57)

earth dam constructed of dispersive clay, an amelioration process takes place with time and the upper portion of the embankment is changed from a dispersive soil to a nondispersive soil. The agreement of the laboratory test results with the observed performance of the soil in the field indicated that the number of cave-ins on the slopes of the dam decreased with time. Therefore, the susceptibility of vegetated fill slopes in dispersive clays to rainfall erosion, as given in Table 5, may decrease with time following construction of the fill.

31. Sherard[2] conducted a study for the SCS on piping failures and rainfall erosion of earth dams in Oklahoma and Mississippi. Figure 28 shows the relationship obtained between percent sodium and total soluble salts (same as total ionic concentration) in the soil pore water extract for earth dams that were damaged by rainfall erosion. Most of the earth dams that experienced rainfall erosion had excellent grass cover on the slopes. It was thought at this time (1972) that rainfall erosion occurred only on earth dams when the total soluble salts was less than 15 meq/ℓ (Figure 28). Subsequently, it has been shown that rainfall erosion occurs on earth fills with total soluble salts in the range of 50 - 150 meq/ℓ.[1,60,63] Also, it has been noted that some soils, which are classified as dispersive by laboratory tests (described in paragraph 33), may not exhibit rainfall erosion on vegetated fill slopes or excessive rainfall erosion (not more severely eroded than nondispersive soils) of cut slopes.[1] This difference in behavior may be due to the cracking potential, rate of swelling to close cracks, or climatic conditions.[1,64]

Identification of Dispersive Clays

32. Identification of dispersive clays may be required for earth structures not yet constructed, for existing earth structures, and for natural soil deposits. Positive identification of dispersive clays can presently only be based upon observed performance of the soil in the field.[65] As stated previously, dispersive clays cannot be identified by conventional index tests such as Atterberg limits, particle size

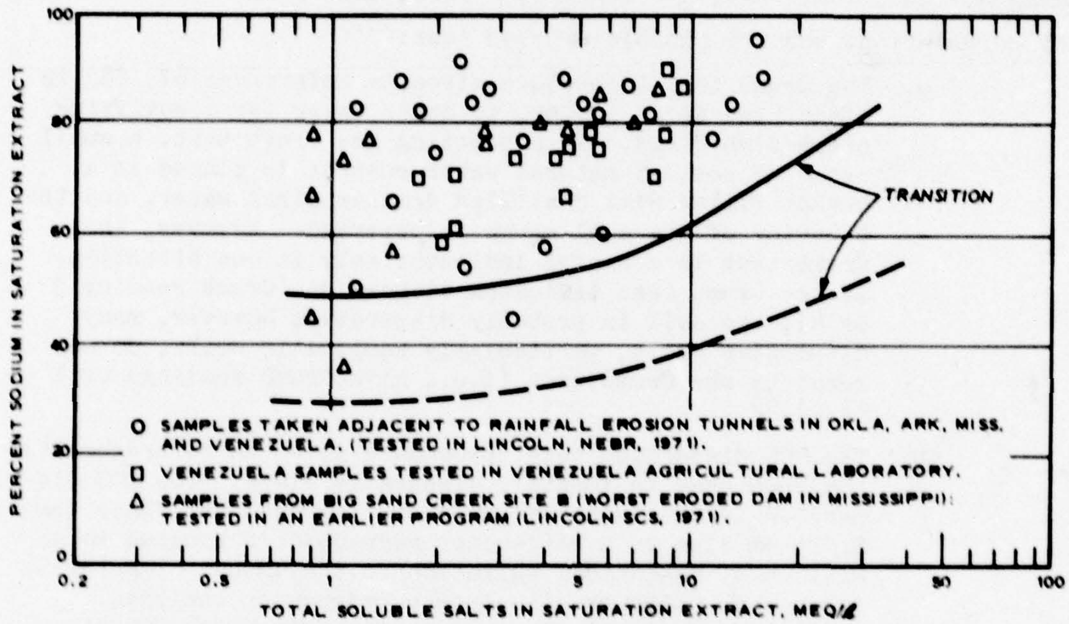


Figure 28. Results of soil chemistry tests on soil pore water extract from earth dams that were damaged by rainfall erosion (from Reference 2)

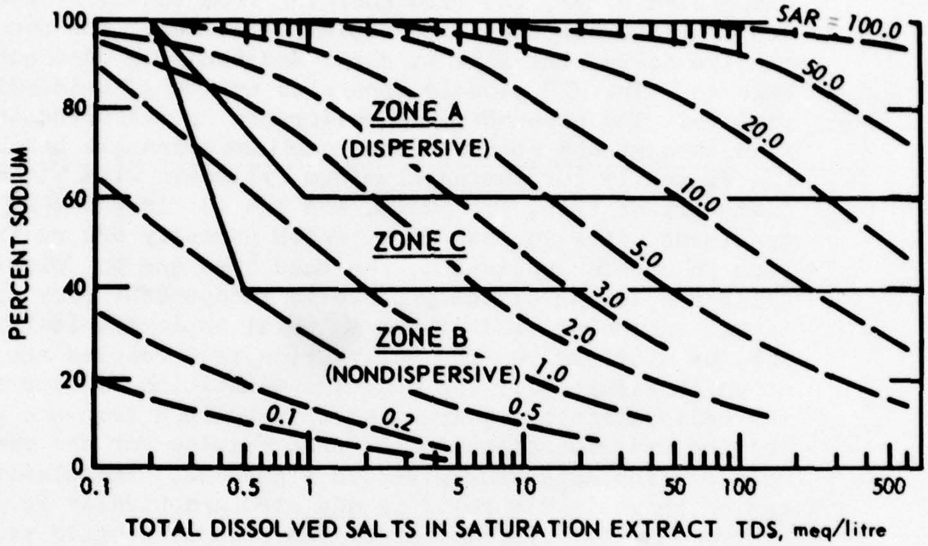
distribution, and compaction characteristics. Soils with the fraction finer than 0.005 mm $\leq$ 12 percent and with a plasticity index $\leq$ 4 generally do not contain sufficient colloids to support dispersive erosion.[19,55] However, such soils are known to have low resistance to erosion (Figure 27) and the dispersion characteristics would add little to the known field performance of the soils.[57]

33. Four laboratory tests commonly used to identify dispersive clays are the Crumb test, SCS dispersion test, soil pore water chemistry correlation, and the pinhole erosion test:[55,56]

a. The Crumb test (procedure given in References 67, 68) is often used as an adjunct to other tests for identifying dispersive clays. In conducting the Crumb test, a small crumb of soil at natural water content is placed in a beaker filled with distilled demineralized water, and the behavior of the soil crumb is observed. However, the Crumb test is a useful indicator only in one direction. If the Crumb test indicates dispersion (Crumb reading 3 or 4), the soil is probably dispersive; however, many dispersive soils, particularly kaolinitic soils, do not react to the Crumb test (i.e., give Crumb readings of 1 or 2).[69]

b. The SCS dispersion test (procedure given in Reference 14) has been used to identify dispersive clays. The SCS dispersion is the ratio of the amount of material finer than 0.005-mm size in a soil-water suspension subjected to a minimum of mechanical agitation to the amount of material finer than 0.005-mm size from a hydrometer analysis. Available results indicate that for soils with SCS dispersion $<$ 35 percent dispersive erosion will not be a problem, for soils with SCS dispersion 35-50 percent dispersive erosion may or may not occur, and for soils with SCS dispersion $>$ 50 percent dispersive erosion will be a problem.[19,55,70] The SCS dispersion test has about 85 percent reliance in predicting dispersive performance (about 85 percent of dispersive soils show more than 35 percent SCS dispersion).[70]

c. Sherard and coworkers[55] have obtained a relationship between dispersion and soil pore water chemistry based on pinhole erosion tests and observed dispersive erosion in nature as shown in Figure 29. The soil pore water correlation has about 85 percent reliance in predicting dispersive performance.[70]



NOTE: RELATIONSHIP SHOWN IS VALID ONLY WHEN ERODING WATER IS RELATIVELY PURE.

$$\text{PERCENT SODIUM} = \frac{Na\ (100)}{TDS} = \frac{Na\ (100)}{Ca + Mg + Na + K}$$

$$TDS = Ca + Mg + Na + K$$

$$SAR = \frac{Na}{\sqrt{0.5\ (Ca + Mg)}}, \text{ ALL IN meq/litre}$$

Figure 29. Relationship between dispersion and soil pore water chemistry based on pinhole erosion tests and experiences with erosion in nature (from Reference 55).

d. The pinhole erosion test (procedure given in Reference 67) is the most reliable test for identifying dispersive clays.[55,66] The design and construction of the WES pinhole erosion apparatus used in this study is given in Reference 68. A schematic drawing of the pinhole erosion test is shown in Figure 30. In conducting the test, distilled water under a low hydraulic head is caused to flow through a small-diameter hole in the soil specimen. For dispersive clays, the flow emerging from the soil specimen is cloudy and the hole rapidly enlarges. For nondispersive soils, the flow is clear and the hole does not enlarge. The WES pinhole apparatus uses a 1/16-in.-diam pinhole. The apparatus was calibrated by measuring the flow through the specimen under various hydraulic heads (2, 7, and 15 in.) using aluminum cylinders with pinhole diameters of 1/16, 1/8, 3/16, and 1/4 in. in place of soil specimens. For laminar flow, which normally occurs in the pinhole erosion apparatus, the head loss due to friction along the length of the pinhole is independent (for practical considerations) of the material employed, that is, soil or aluminum.[66] The calibration test results are shown in Figure 31. The measured quantities of flow from the calibration test were used to prepare a sequence of testing and classification of test results for the pinhole erosion apparatus given in Figure 32. The classification shown in Figure 32 is depicted graphically in Figure 33. Two limitations of the pinhole erosion test for identifying dispersive soils have been observed. Undisturbed soil samples of high sensitivity (ratio of the peak undrained strength of the soil in a natural state to the peak undrained strength after it has been remolded without change in water content) may be classified as dispersive from the pinhole erosion test, while in nature the soil may be resistant to erosion.[71] Apparently, the natural structure of the soil is destroyed by punching the pinhole in the undisturbed soil specimen and a reaction analogous to dispersion is obtained in the pinhole erosion test. Soils with high sodium (> 80 percent) and low total dissolved salts (< 0.4 meq/ℓ) in the soil pore water may show nondispersion in the pinhole erosion test while the soil may exhibit dispersive performance in the field.[65] This may occur because a decrease in the concentration gradient between the soil pore water and eroding fluid (distilled water = 0.0 meq/ℓ for pinhole erosion test) results in a decrease in the erosion rate for soils.[72] However, available data from case histories indicate very few soils with total dissolved salts < 1.0 meq/ℓ for which dispersive performance has been observed in the field.[55]

34. Four field tests that have been used to identify dispersive clays are the Crumb test (also used as a laboratory test), the ultraviolet

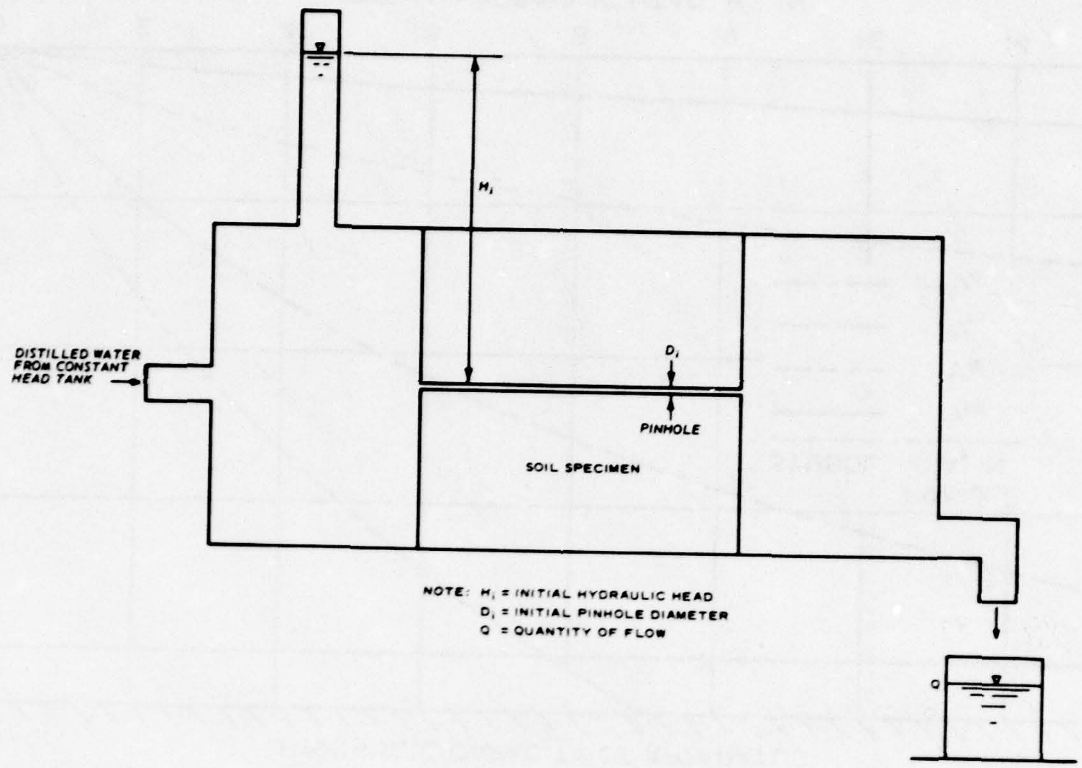


Figure 30. Schematic representation of WES pinhole erosion apparatus (from Reference 67)

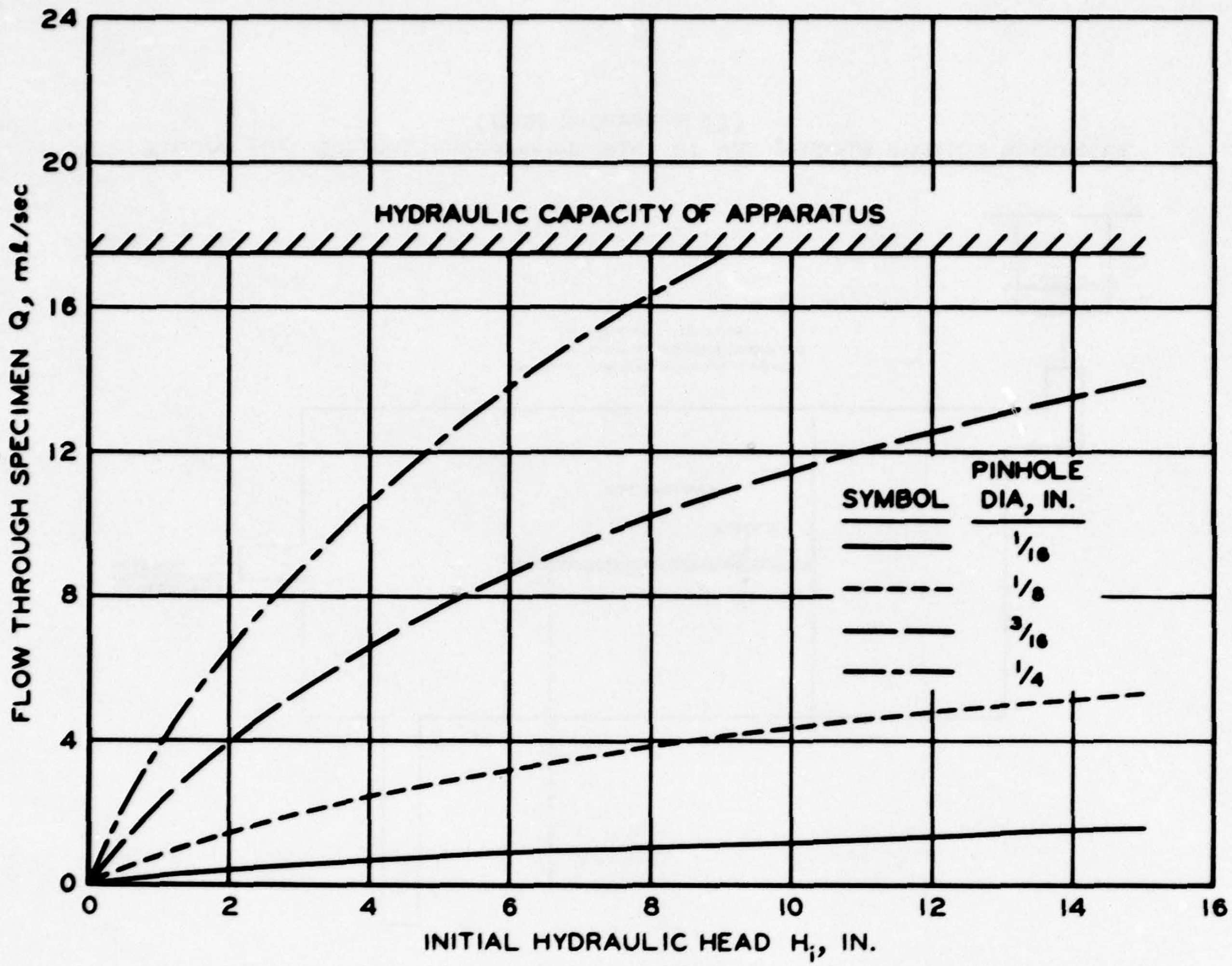


Figure 31. Calibration curve for WES pinhole erosion apparatus using 4.6-in.-long specimens (from Reference 67)

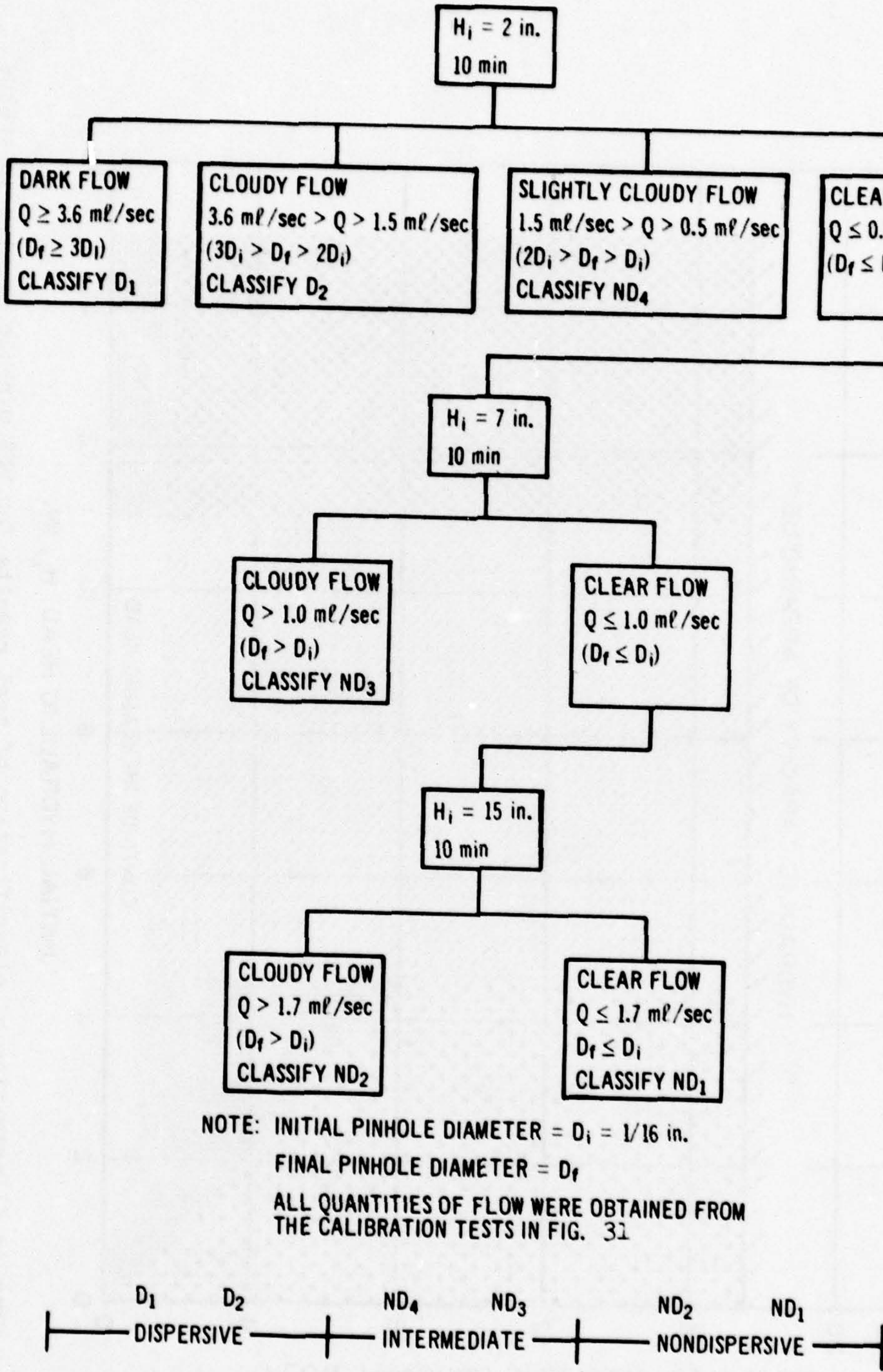


Figure 32. Sequence of testing and classification of test results for WES pinhole erosion apparatus using 4.6-in.-long specimens (from Reference 67)

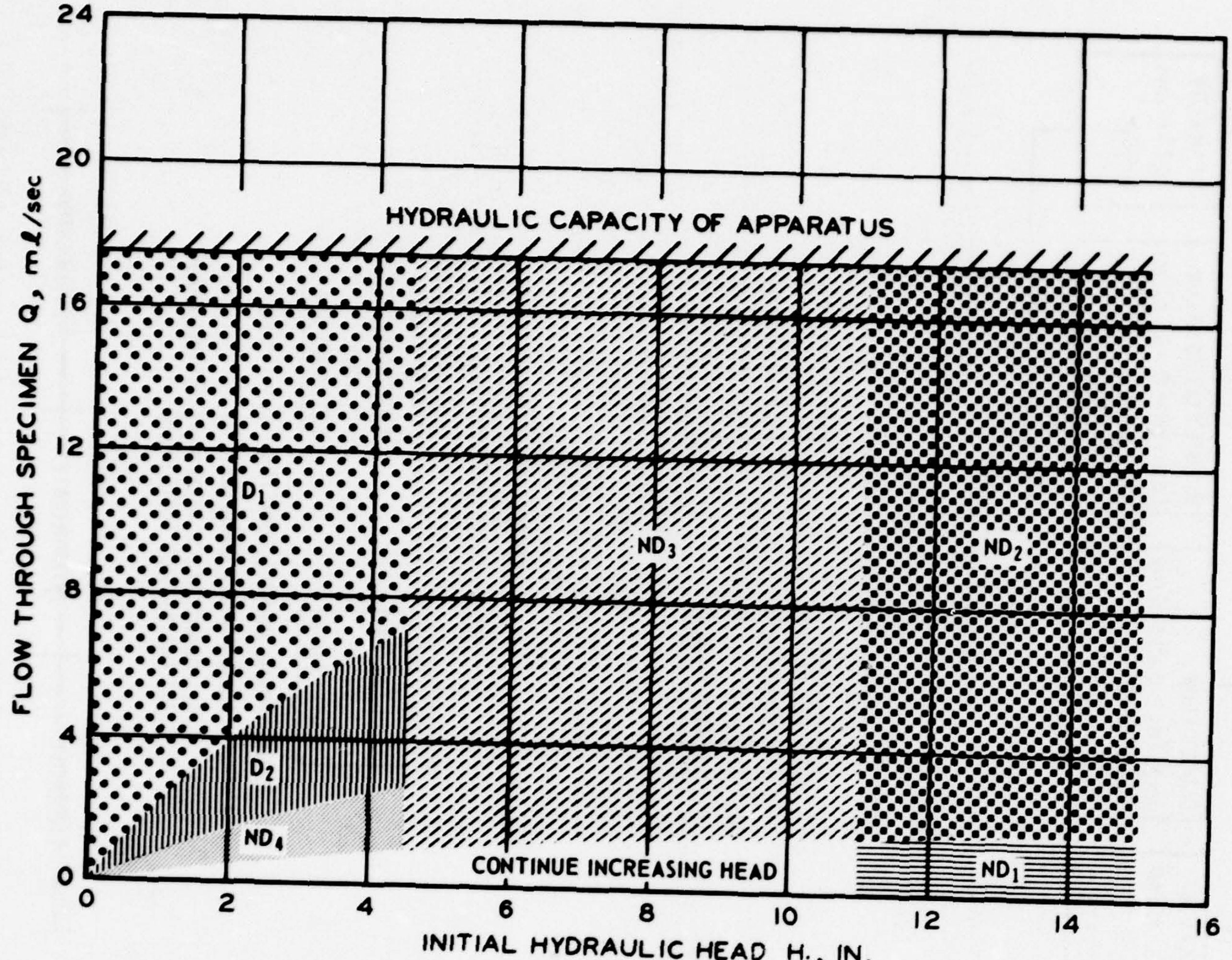


Figure 33. Graphic illustration of classification of test results for WES pinhole erosion apparatus using 4.6-in.-long specimens (from Reference 67)

light, the modified hydrometer or Dilution-Turbidity test, and determination of soil pore water chemistry by use of sodium electrode and chemical reagents or wheatstone bridge.

<u>a</u>. The Crumb test, which may be used in the laboratory or field, has been described previously.

<u>b</u>. The ultraviolet light test (procedure given in Reference 58) has been used to indicate the presence of sodium in the soil. Uranyl acid is mixed with the soil and the intensity and amount of fluorencence under ultraviolet light is observed. The ultraviolet light test has shown about 40 percent reliance in predicting dispersive performance of soils in Mississippi.[65]

<u>c</u>. The modified hydrometer or Dilution-Turbidity test (procedure given in Reference 19) has been used to identify dispersive clays. Prior to using the modified hydrometer test to identify dispersive clays in a particular area, the test results must be correlated with laboratory tests to establish a range of values. For example, test data from the Bluff Hills region of Mississippi indicate that for a turbidity ratio < 4 dispersive erosion will occur, for a turbidity ration 4-9 dispersive erosion may or may not occur, and for a turbidity ratio > 9 dispersive erosion will not be a problem.[65]

<u>d</u>. Two methods (procedure given in Reference 73) have been developed for determination of soil pore water chemistry in the field to use with the correlation shown in Figure 30 and to identify dispersive clays. The first method involves a sodium electrode and chemical reagents to determine the percent sodium and total dissolved salts (sodium, magnesium, and calcium). This method does not determine potassium which exists in small quantities (≤ 1.0 meq/ℓ) in most soils.[2] The second method uses a wheatstone bridge and a conductivity cell to determine the percent sodium and total dissolved salts (sodium, potassium, magnesium, and calcium).

Lime Modification of Dispersive Clays

35. Lime modification of dispersive clays to prevent piping failure in earth dams was reported in Australia in 1964.[74] Both laboratory and field test results showed that blending small amounts (2 percent by dry weight) of hydrated lime (calcium hydroxide) into the soil during construction would eliminate piping failure in earth dams. Subsequent

work in the United States by the SCS has shown that rainfall erosion tunnels on the embankment slopes of earth dams constructed of dispersive clays could be prevented by plating the slopes with 12-15 in. of soil (normal to the slope) mixed with 2-3 percent of hydrated lime.[14] The addition of hydrated lime to the reservoir water of an earth dam constructed of dispersive clay in an attempt to remedy an existing piping problem proved unsuccessful and a failure subsequently occurred.[75] Since 1969 the SCS in Oklahoma has lime modified seven flood control dams.[19] Three dams were treated during initial construction while four dams had been in service and were badly damaged prior to treatment. The selected lime percentage was that which raised the shrinkage limit to a value near the saturation moisture content based on the compacted density to be achieved on the dam. Subsequent research by Haliburton and coworkers[76] have provided quantitative data on the effectiveness of lime modification on reducing the erosion potential of Oklahoma dispersive clays.

36. During the spring of 1973, heavy rains (in excess of 100-year frequency) caused severe rainfall erosion damage on flood control dams constructed of dispersive clay in the Bluff Hills region of Mississippi.[77] The SCS in Mississippi initiated a flood restoration program which included lime modification of a surface layer of 20 dams.[65,78] Laboratory tests indicated no appreciable increase in the shrinkage limit with the addition of up to 4 percent lime. Pinhole erosion tests conducted on 1 percent lime-modified soil indicated the addition of lime reduced the soil from dispersive to nondispersive (D to ND, Figure 32) although it was noted that cloudy flow occurred in the lime-modified soil. There was no cloudy flow observed for 2 percent or greater lime-modified soil. Unconfined compression tests were made on 2 percent lime-modified soil to determine the effect of curing (no evaporation was permitted) in the compacted versus loose states for periods of time ranging from 1 to 5 days as shown in Figure 34. There was little difference in the strain at failure (1.0 to 2.8 percent) for all specimens tested. With 2 days of loose curing prior to compaction, the length of curing time in the compacted state did not appreciably affect the unconfined compressive strength. Based on the unconfined compression tests results and

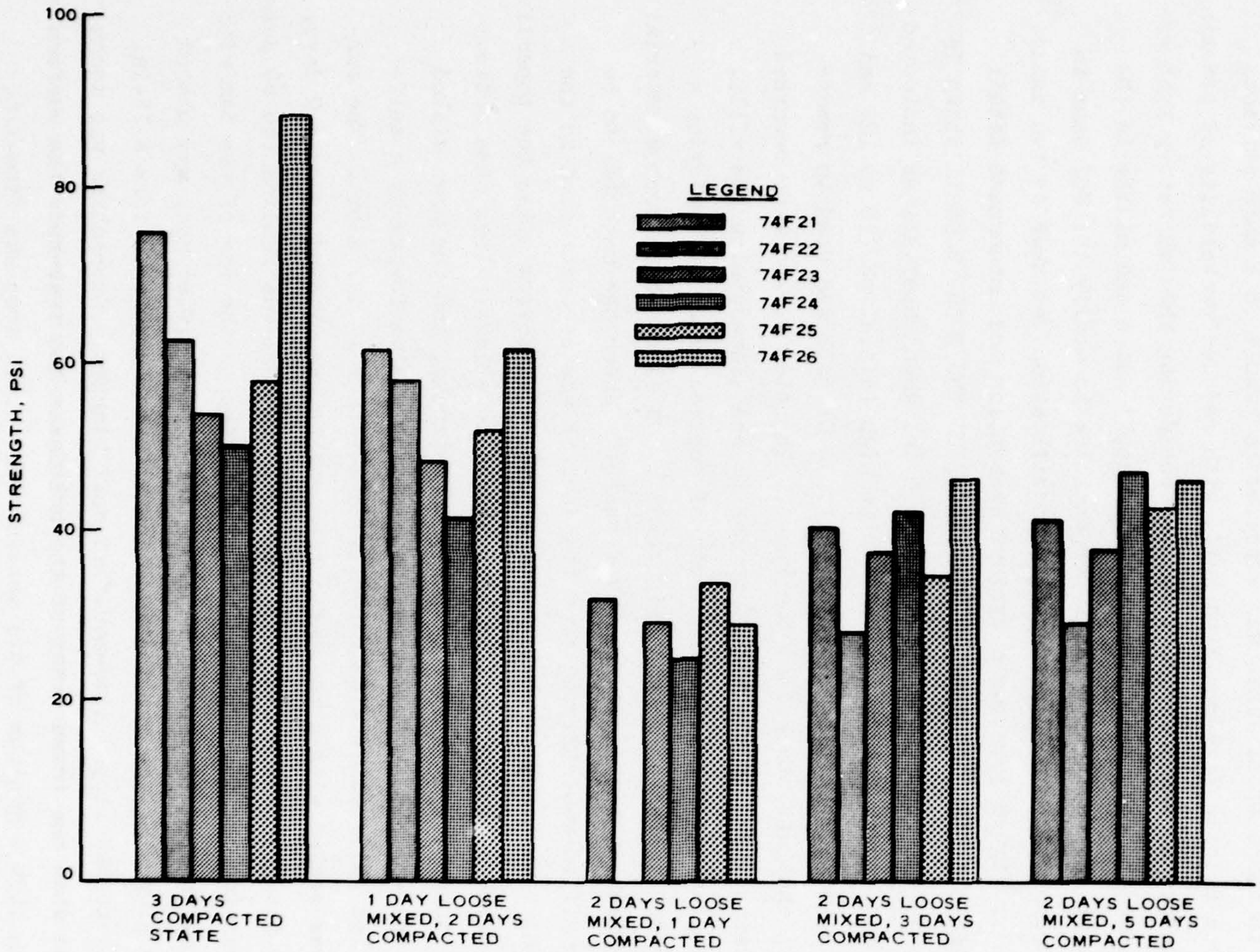
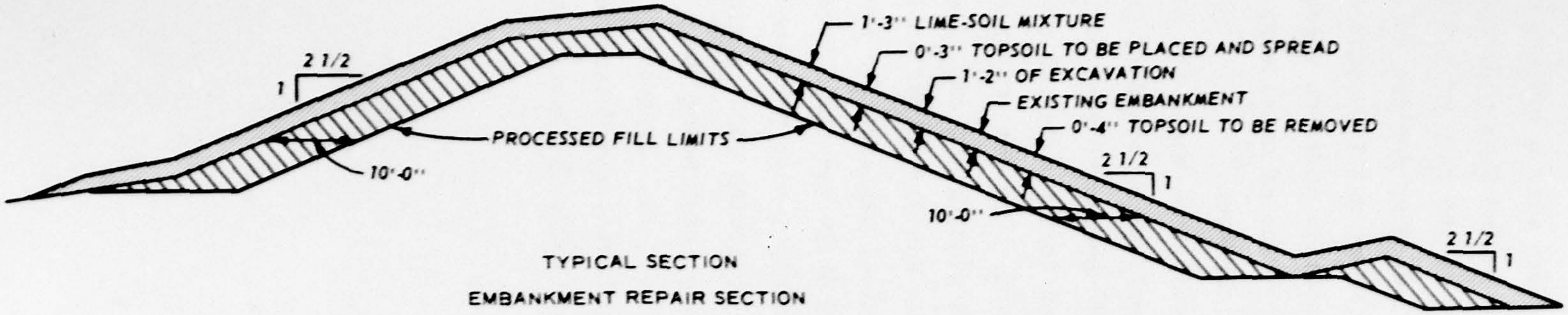


Figure 34. Effect of mixing and molding time on unconfined compressive strength for lime-modified SCS dams in Mississippi (adapted from Reference 65)

observations concerning friability of the compacted specimens, it was decided to use a minimum curing period of 2 days in the loose state prior to compaction. This was expected to result in a more pervious and less brittle lime-modified soil, with reduced probability of cracking and erosion of the underlying soil. Considering the laboratory test results and the potential variability of application and mixing in the field, it was decided to use 2 percent lime to modify the SCS dams in Mississippi (0.5 and 3 percent lime modification was used on two dams).[65,78]

37. Figure 35 shows a typical excavation and embankment detail for lime-modified SCS dams in Mississippi. The 3-in. topsoil shown in Figure 35 was subsequently changed to 6 in. when observations indicated that root growth was restricted by the high initial pH (10 to 12) and cementation of the lime-modified soil. A Gradall was used to remove topsoil that was stockpiled for reuse. The side slopes were reworked to a depth (usually about 4 ft normal to the slope) below the rills, tunnels, and deep penetrating roots of Sericea grass by employing a bulldozer stair-step method (Figure 35). On sites where borrow material was available, the dams were "over built." Where the material to be lime modified was stripped from the dam, it was reconstructed to the original elevation. When lime was applied to a borrow site, the topsoil was stripped, the area disked and watered to slightly less than optimum water content, lime spread directly from trucks, and the area disked again and watered to tack the lime. Mixing was performed by a self-propelled pulvermixer with capacity to mix to a 13 in. depth. The surface was sealed with a pneumatic tire roller and allowed to cure 2 days. Before placing on the dam, the lime-modified soil was remixed (to 60 percent passing the No. 4 sieve) and transported to the toe of the dam with self-loading paddle wheel scrapers. The lime-modified soil was placed on the slopes of the dam in two lifts with bulldozers to form a 15-in. (normal to the slope) lime-modified soil blanket. Compaction was accomplished when the tread track of the bulldozer had traversed the surface of each lift a minimum of two passes. The final step was fine-line dressing of the dam with the bulldozer, placement of the topsoil, and application of vegetative treatment. The dams were treated with



TYPICAL SECTION

EMBANKMENT REPAIR SECTION

NOTE: STRIP APPROXIMATELY 4'' OF SURFACE SOIL CONTAINING ORGANIC MATERIAL AND STOCKPILE.

EXCAVATE TO A MINIMUM DEPTH OF 14'' NORMAL TO THE SURFACE AFTER STRIPPING AND STOCKPILE FOR TREATMENT.

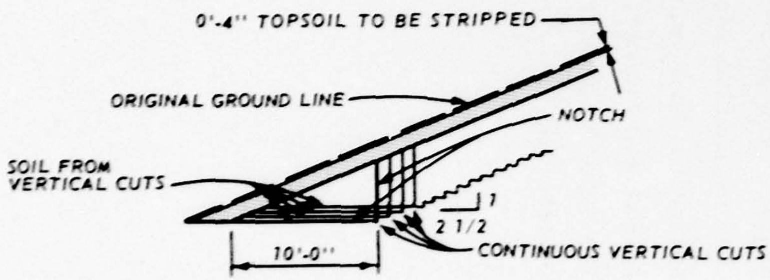
PROCESS EMBANKMENT MATERIAL IN PLACE AS SHOWN IN DETAIL A.

TREAT EXCAVATED SOIL WITH LIME AS SPECIFIED.

BACKFILL WITH TREATED SOIL TO DESIGN GRADE WITH A MINIMUM NORMAL DEPTH OF 15''.

REPLACE 3'' SURFACE SOIL AND SEED AND MULCH.

MINIMUM DESIGN ELEVATION A ℄ DAM=202.0'.



DETAIL A

STAIRSTEPPING METHOD OF EXCAVATION

Figure 35. Excavation and embankment detail for lime-modified SCS dams in Mississippi (from Reference 65)

600 pounds of 13-13-13 fertilizer per acre and seeded with a mixture of 25 pounds of Pensacola Bahiagrass and 4 pounds of hulled Common Bermudagrass per acre with asphalt-treated mulch applied at the rate of 1.5 tons per acre with 200 gallons of emulsified asphalt. Seeding for the fall season was changed to a mixture of 180 pounds of wheat and 15 pounds of unhulled Bermudagrass. The establishment of vegetative cover on these dams was not to prevent dispersive clay erosion but was of considerable importance to the landowner who grazed cattle on the dams.[65,78]

38. For seven SCS flood control dams in Mississippi where the soil was excavated from the dam, lime modified, and replaced on the dam, the average cost (based on 1974-1975 prices) was $7.78 per cubic yard of lime-modified soil in place. The 10 dams that had borrow material available for modification averaged $6.20 per cubic yard. One dam was treated in place with 3 percent sack lime. This required placing and emptying 50-pound sacks of lime by hand labor, and the cost was $13.43 per cubic yard. Lime modification was included in the original contract for two new dams under construction at an average cost of $6.04 per cubic yard.[78] The lime-modified SCS flood control dams in Mississippi have performed well with no rainfall erosion or piping reported following construction in 1974-1975. The vegetative root system is slowly penetrating the lime-modified soil with roots reaching a depth of 8-10 in. Long-term studies will include pH readings, pinhole erosion tests, and determination of soil pore water chemistry of the lime-modified soil blanket and the underlying earth fill.[65]

PART V: LABORATORY SOILS TESTING

Description of Soils Tested and Test Program

39. Undisturbed soil samples taken by the Vicksburg District from 30 January to February 1973 from borings 1-73, 2-73, and 3-73 (Figure 36) from the downstream slope of Grenada Dam at sta 136+10, offset 16, 94, and 186 ft right of the center line, respectively, were tested at the WES from 6 April to 24 June 1976. The 5-in.-ID Shelby tube samples were extruded in the field and preserved in cardboard tubes, completely sealed in wax, prior to laboratory testing. Also, 3-in.-ID Shelby tube samples taken on 10 May 1976 from the surface of the downstream slope of the dam (designated borings 1-76, 2-76, and 3-76) in the vicinity of borings 1-73, 2-73, and 3-73 were tested at the WES from 12 May to 16 June 1976. The reservoir water, used in pinhole erosion tests, was taken on 10 May 1976 from the surface of Grenada Lake opposite sta 136+10 about 100 ft from the shoreline.

40. The scope of the laboratory test program conducted for this study is shown in Table 7. As stated previously, the objective of the study was to determine the dispersion characteristics of embankment and foundation soils and the susceptibility of these soils to rainfall erosion and dispersive clay piping, respectively. As shown in Table 7, classification and index tests, soil (and reservoir water) chemistry tests, and dispersion tests were run on selected undisturbed soil samples of embankment and foundation soils from the three borings shown in Figure 36. Also, pinhole erosion tests using reservoir water as eroding fluid were run on undisturbed foundation soil samples from borings 2-73 and 3-73.

Influence of Storage Time on Soil Properties

41. Undisturbed soil samples from borings 1-73, 2-73, and 3-73 were stored in a warehouse for 38 to 41 months prior to laboratory testing. Comparison of water contents from jar samples determined within

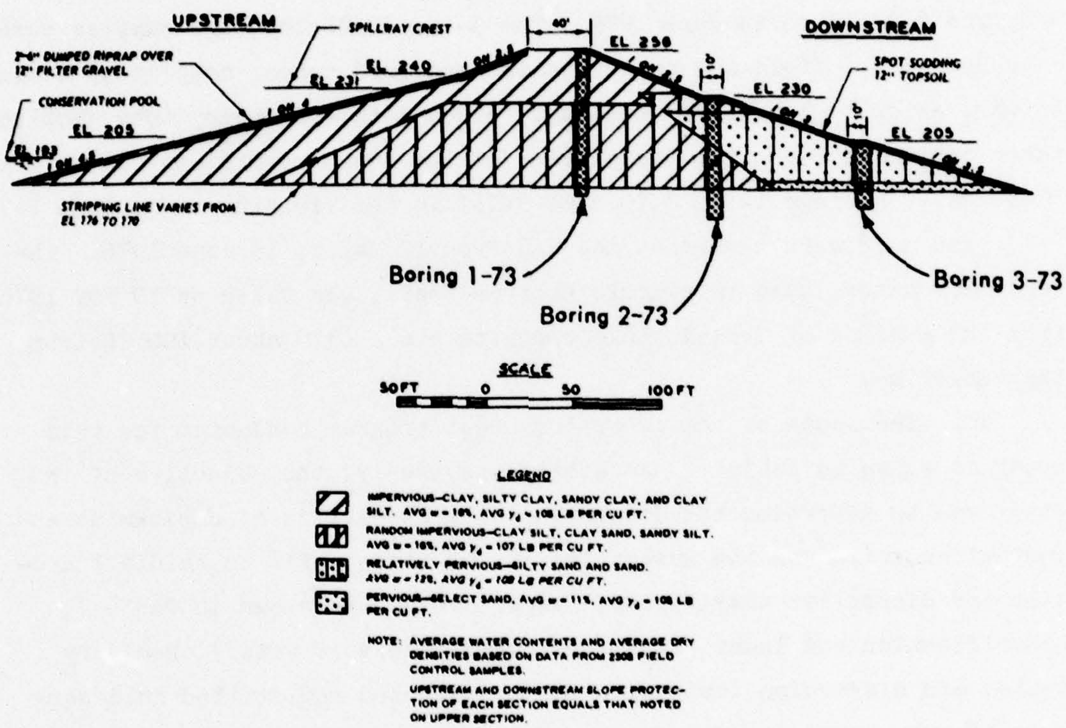


Figure 36. *Location of borings* 1-73, 2-73, and 3-73 at sta 136+10
(adapted from Reference 3)

2 weeks of sampling and water contents from undisturbed samples stored for 38 to 41 months, shown in Table 8, indicated that for 38 samples tested, 23 samples showed an average increase in water content of 3.2 percent and 12 samples an average decrease in water content of 2.9 percent (no data were obtained for 3 samples). The apparent differences in water content during storage, shown in Table 8, are probably due to spatial variations of in situ water content determined from adjacent soil samples (undisturbed samples and jar samples) and possible internal migration of water.

42. For the laboratory test program presented in Table 7, the storage of undisturbed extruded and waxed soil samples from borings 1-73, 2-73, and 3-73 for 38 to 41 months prior to testing may have affected some of the test results. The Atterberg limits, grain-size analyses, and specific gravity are not believed to have been significantly affected by storage.[79] Limited available data, shown in Table 9, indicate that changes in soil pore water chemistry can occur during storage.[80] The scarcity of data precludes the prediction of changes in the concentration of individual dissolved salts (sodium, potassium, magnesium, or calcium). To the author's knowledge, there are no available data on the effect of storage time on dispersion tests (Crumb, SCS dispersion, or pinhole erosion) which would be applicable to this study.

Classification Test Results

43. The classification test results are shown in Table 10, and the gradation curves and specific gravity test results in Appendix A. Atterberg limits for borings 1-73 and 1-76, 2-73 and 2-76, and 3-73 and 3-76 are plotted on plasticity charts in Figures 37, 38, and 39, respectively. As shown in Table 10, most of the embankment soil was classified as lean or sandy clays (CL) with a few soil samples classified as plastic or sandy clay (CH) and clayey sand (SC). The drainage blanket soil from boring 3-73 was classified as silty sand (SM). The foundation soil was classified as lean, sandy, or silty clay (CL) and clayey sand (SC).

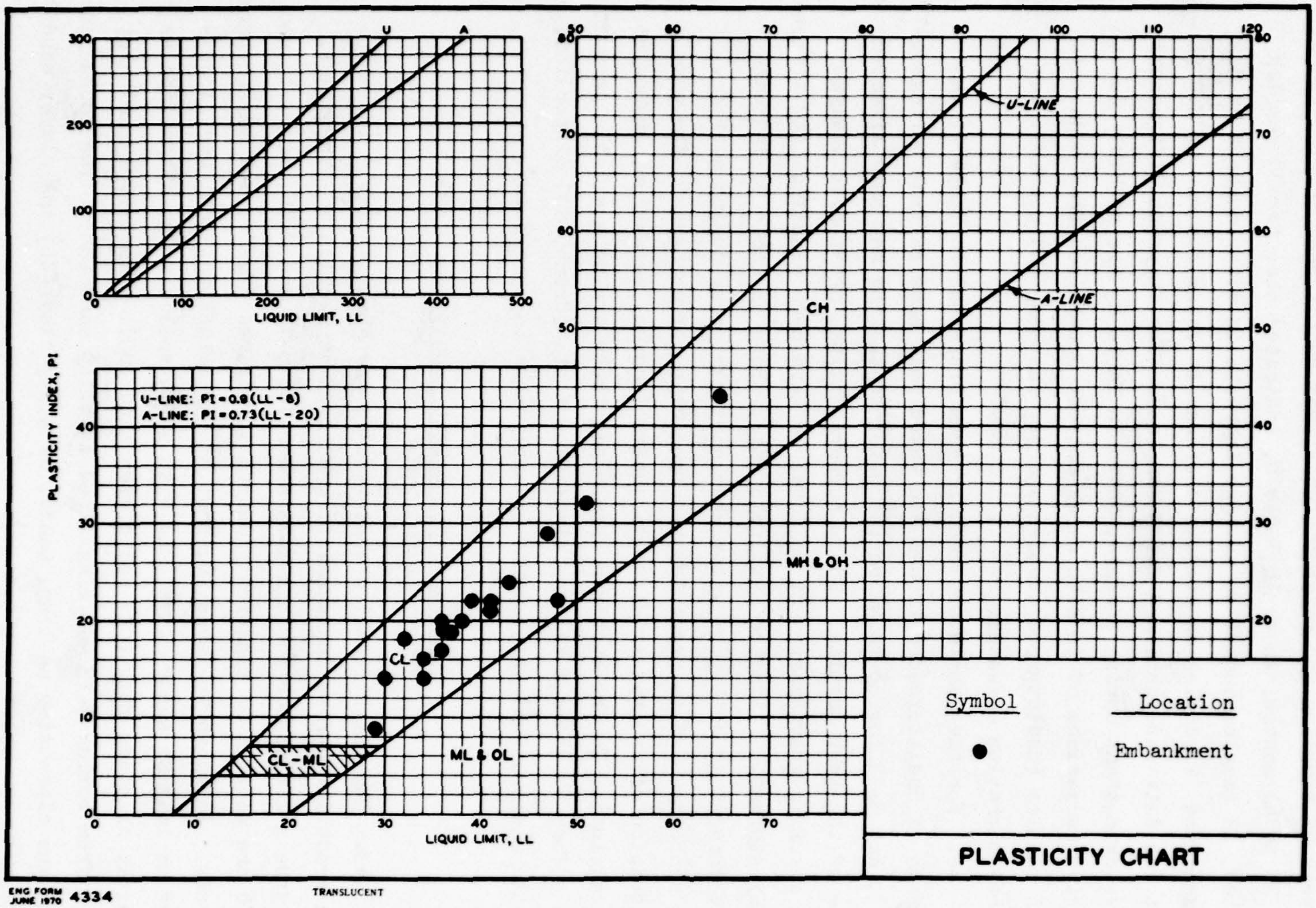


Figure 37. Atterberg limits for soil samples from Borings 1-73 and 1-76

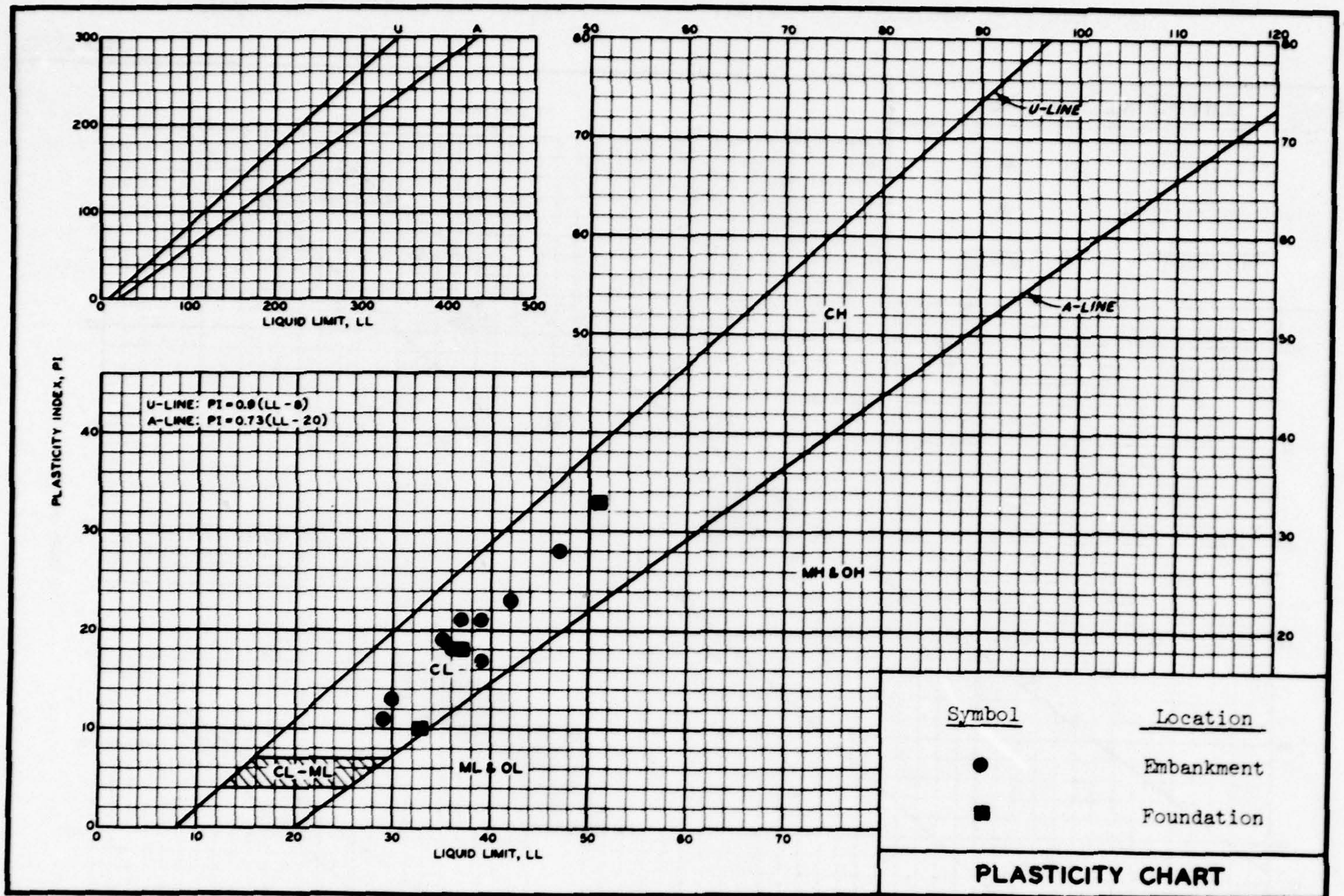


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Figure 38. Atterberg limits for soil samples from Borings 2-73 and 2-76

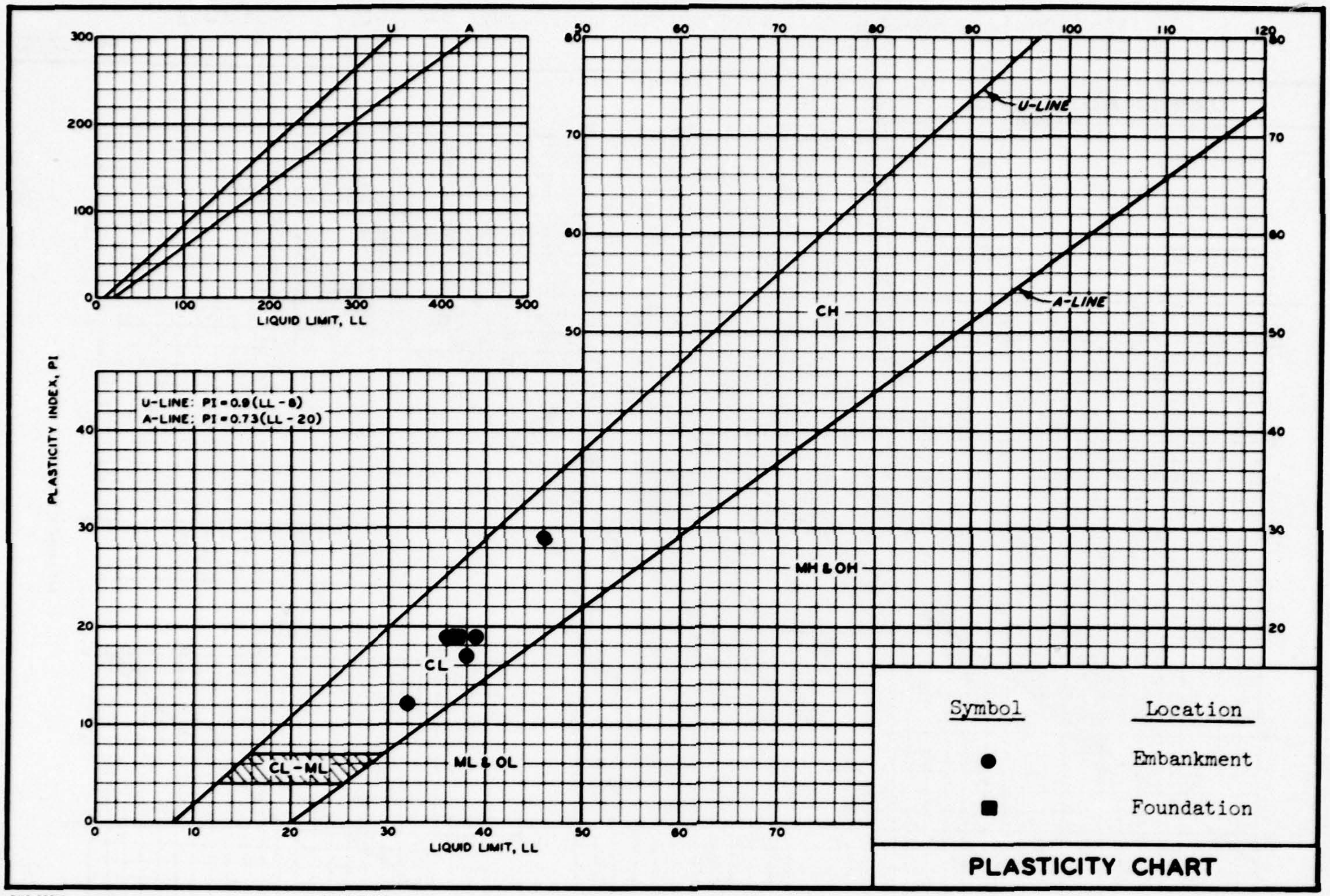

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Figure 39. Atterberg limits for soil samples from Borings 3-73 and 3-76

Crumb Test Results

44. The Crumb test results are shown in Table 11. As stated previously, the Crumb test is a useful indicator only in one direction. If the Crumb test indicates dispersion (Crumb reading 3 or 4), the soil is probably dispersive; however, many dispersive soils, particularly kaolinitic soils, do not react to the Crumb test (i.e., give Crumb readings of 1 or 2).[69] Dispersion versus depth from the Crumb test results for borings 1-73 and 1-76, 2-73 and 2-76, and 3-73 and 3-76 are shown in Figures 40-42, respectively. The Crumb test results were nondispersive (5 of 6 readings) for the upper 6 ft of the embankment (5 ft normal to the slope), and nondispersive to dispersive for the remaining portion of the embankment and foundation.

SCS Dispersion Test Results

45. The SCS dispersion test results are shown in Table 12. As stated previously, the SCS dispersion test has about 85 percent reliance in predicting dispersive performance (about 85 percent of dispersive soils show more than 35 percent SCS dispersion).[70] Dispersion versus depth from the SCS dispersion test results for borings 1-73 and 1-76, 2-73 and 2-76, and 3-73 and 3-76 are shown in Figures 43-45, respectively. The SCS dispersion test results were nondispersive (2 of 3 readings) for the surface of the embankment, dispersive (27 of 32 readings) for the remaining portion of the embankment, and dispersive (3 of 4 readings) for the foundation.

Soil Pore Water Chemistry Correlation

46. The soil chemistry test results are given in Table 13. As stated previously, the soil pore water chemistry correlation has about 85 percent reliance in predicting dispersive performance.[70] The soil pore water chemistry correlation for borings 1-73 and 1-76, 2-73 and 2-76, and 3-73 and 3-76 are shown in Figures 46-48, respectively. Dispersion

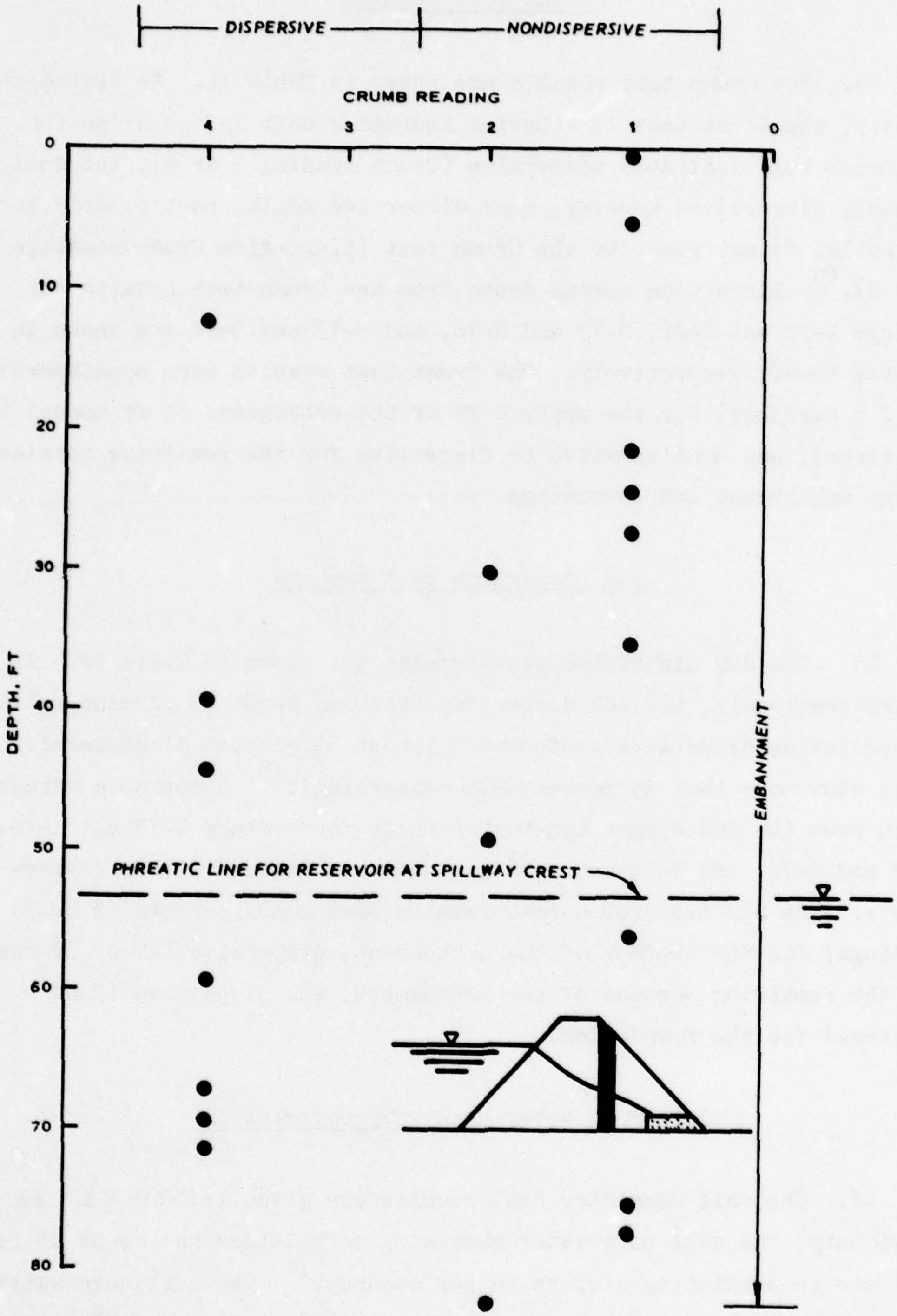


Figure 40. Dispersion versus depth from Crumb Test for borings 1-73 and 1-76

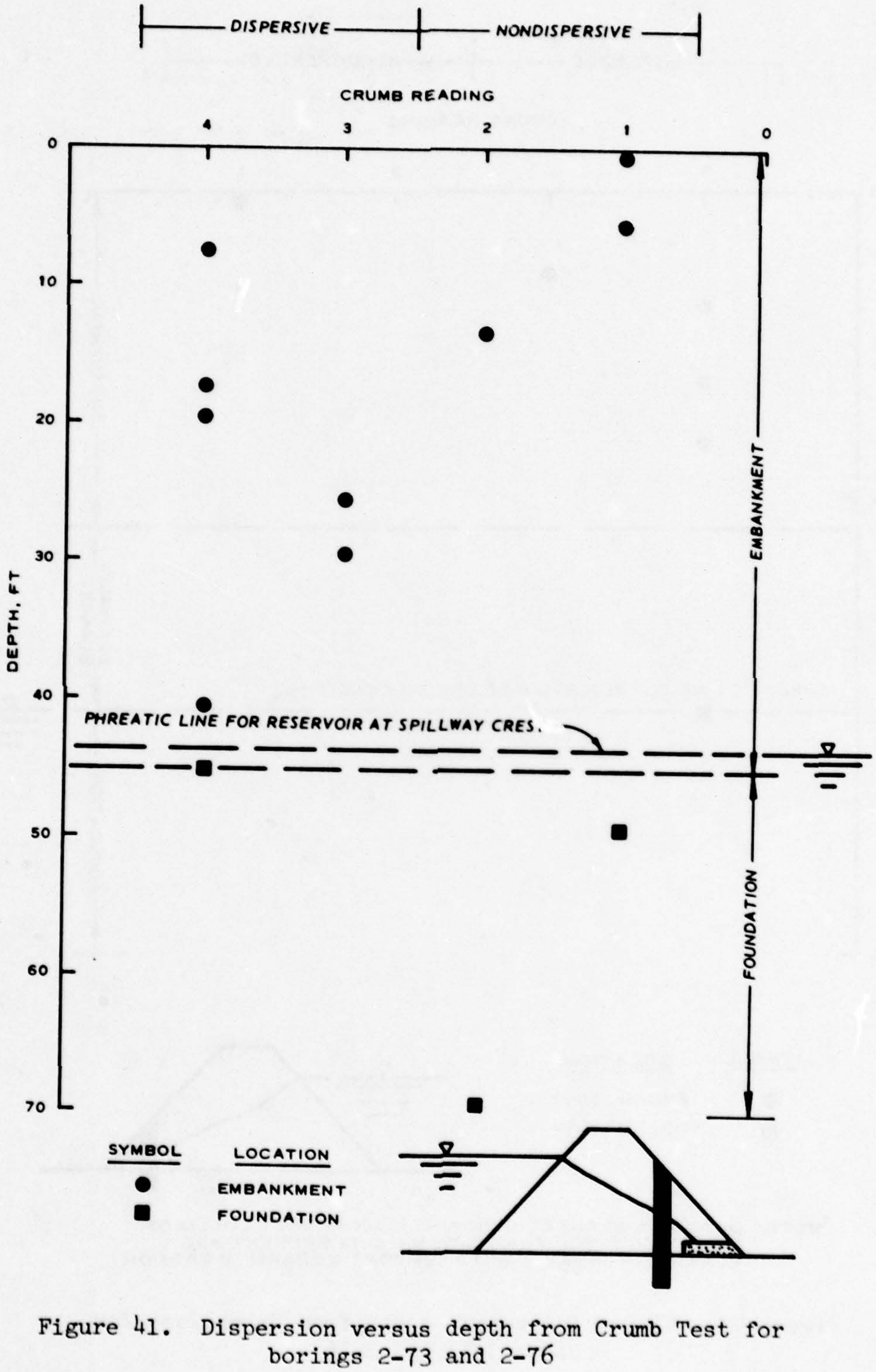
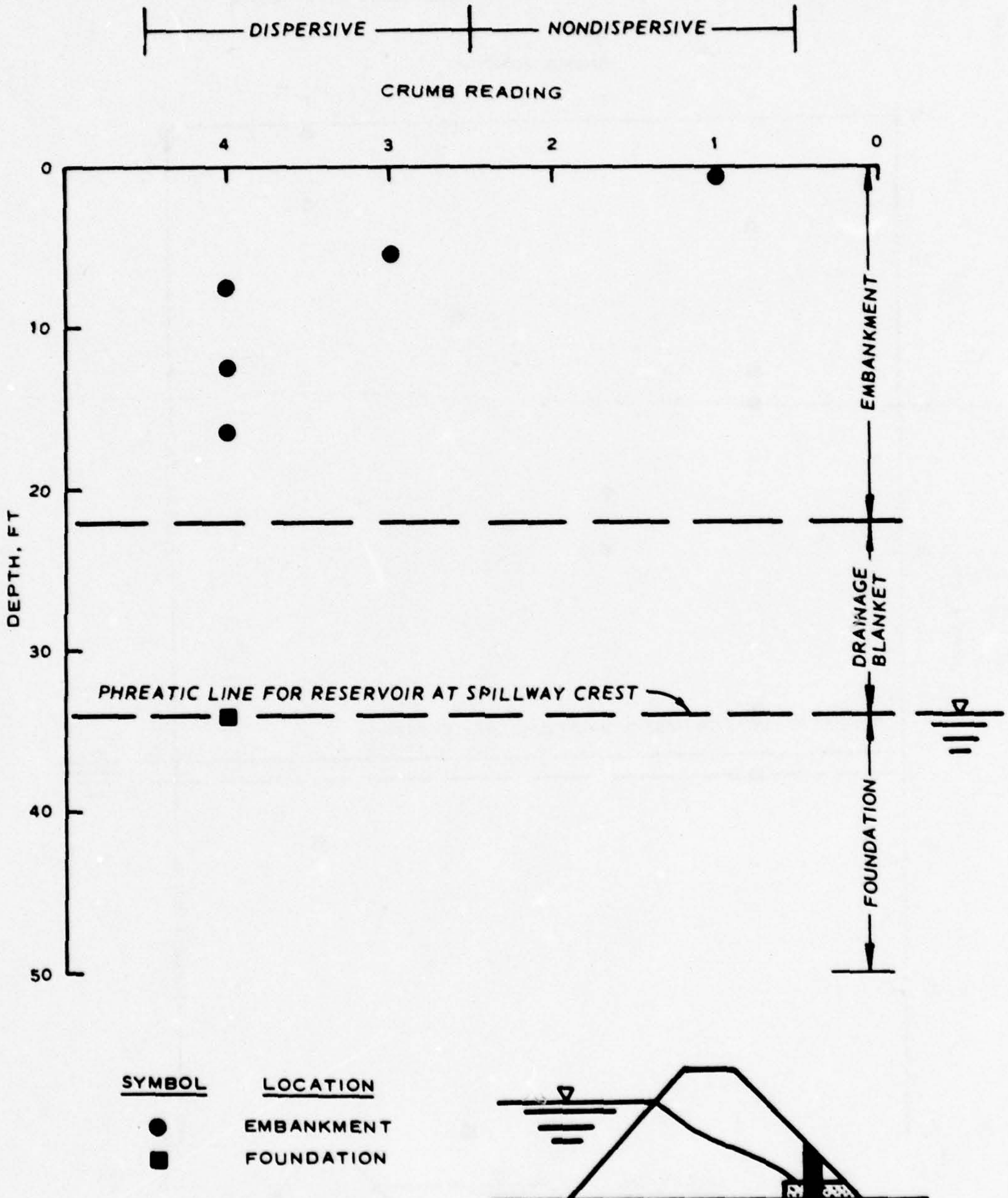


Figure 41. Dispersion versus depth from Crumb Test for borings 2-73 and 2-76



NOTE: DRAINAGE BLANKET CONTAINS INSUFFICIENT COLLOIDS (FRACTION FINER THAN 0.005 MM ≤ 12 PERCENT AND PLASTICITY INDEX ≤ 4) TO SUPPORT DISPERSIVE EROSION

Figure 42. Dispersion versus depth from Crumb Test for borings 3-73 and 3-76

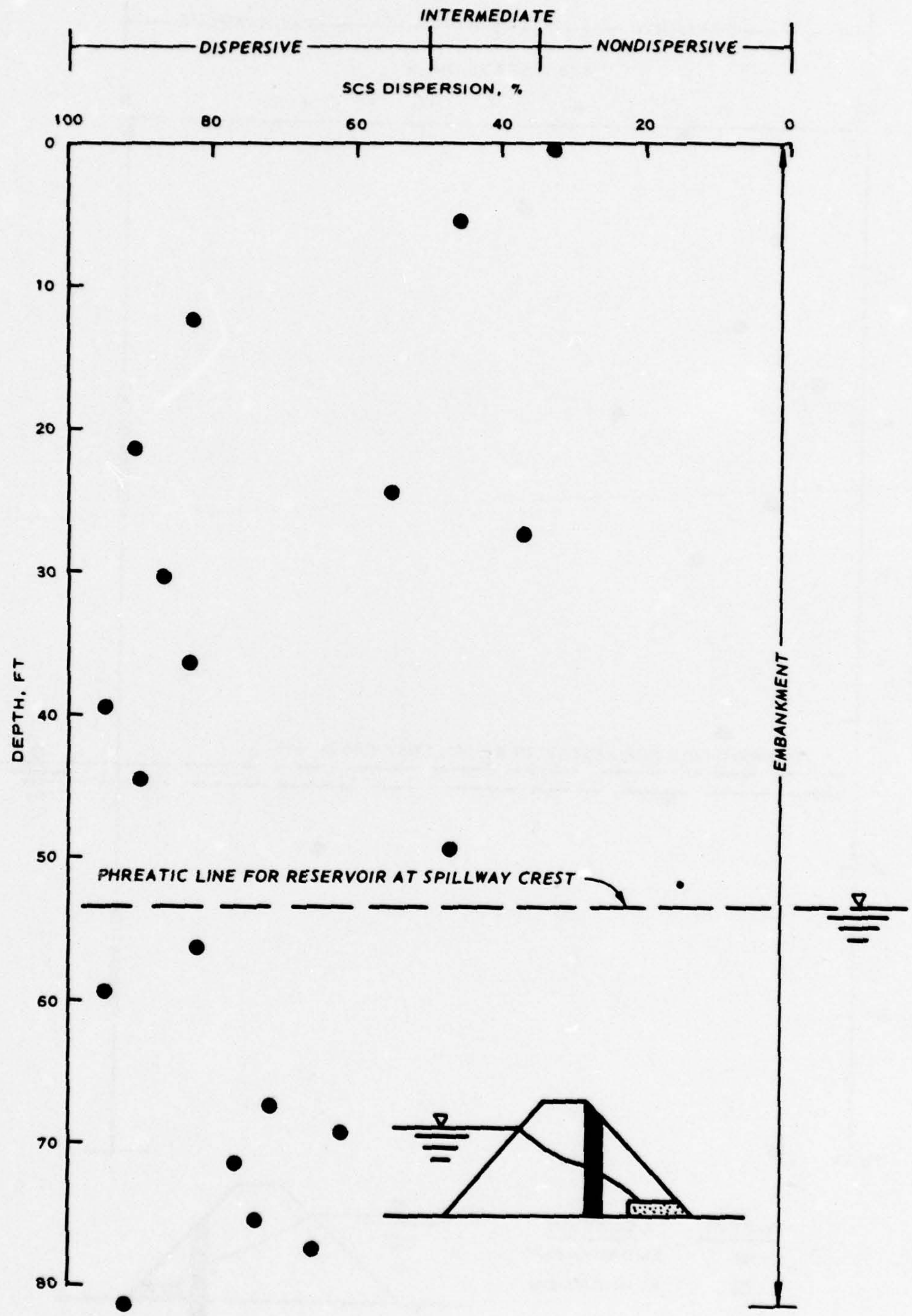


Figure 43. Dispersion versus depth from SCS dispersion test for borings 1-73 and 1-76

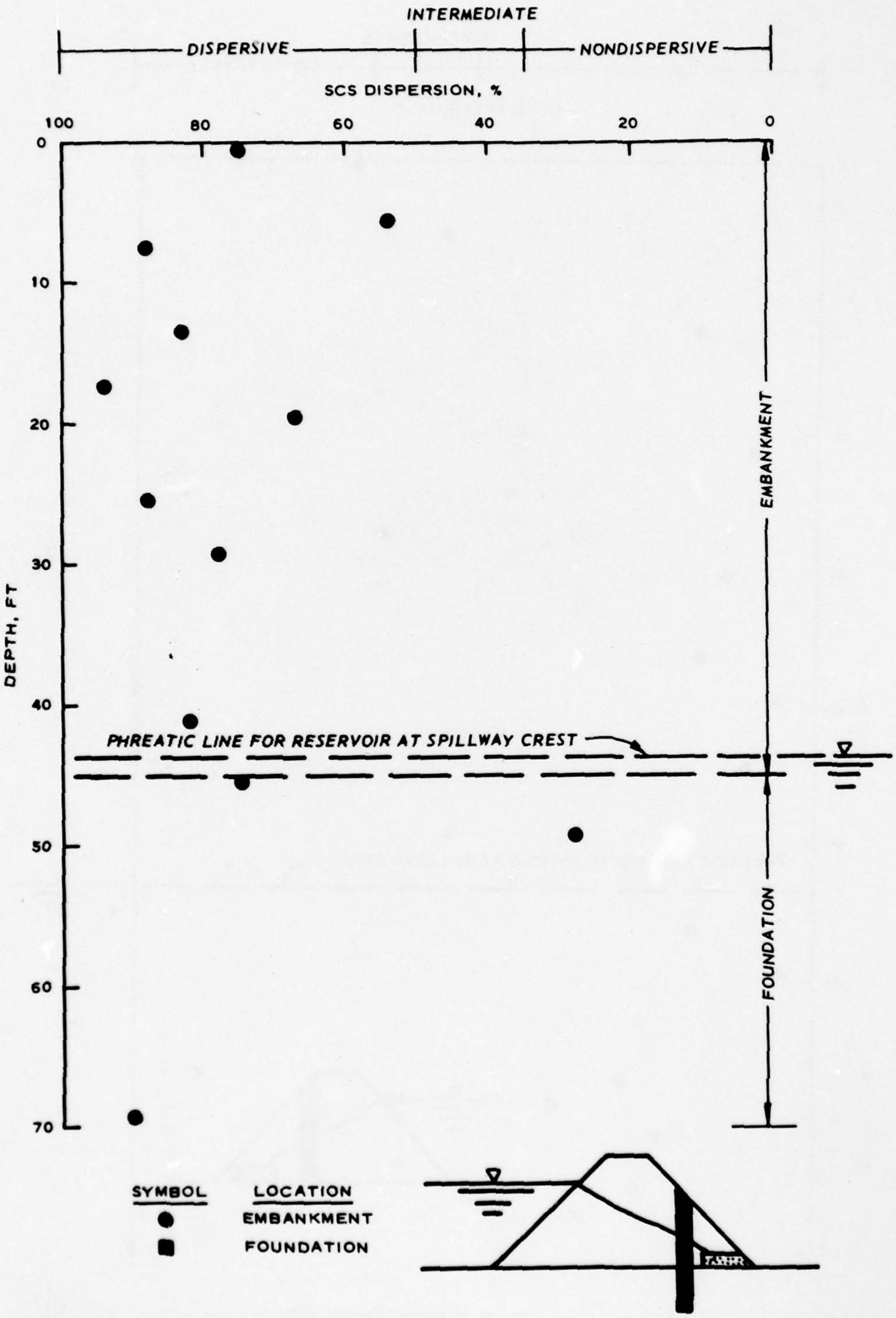


Figure 44. Dispersion versus depth from SCS dispersion test for borings 2-73 and 3-73

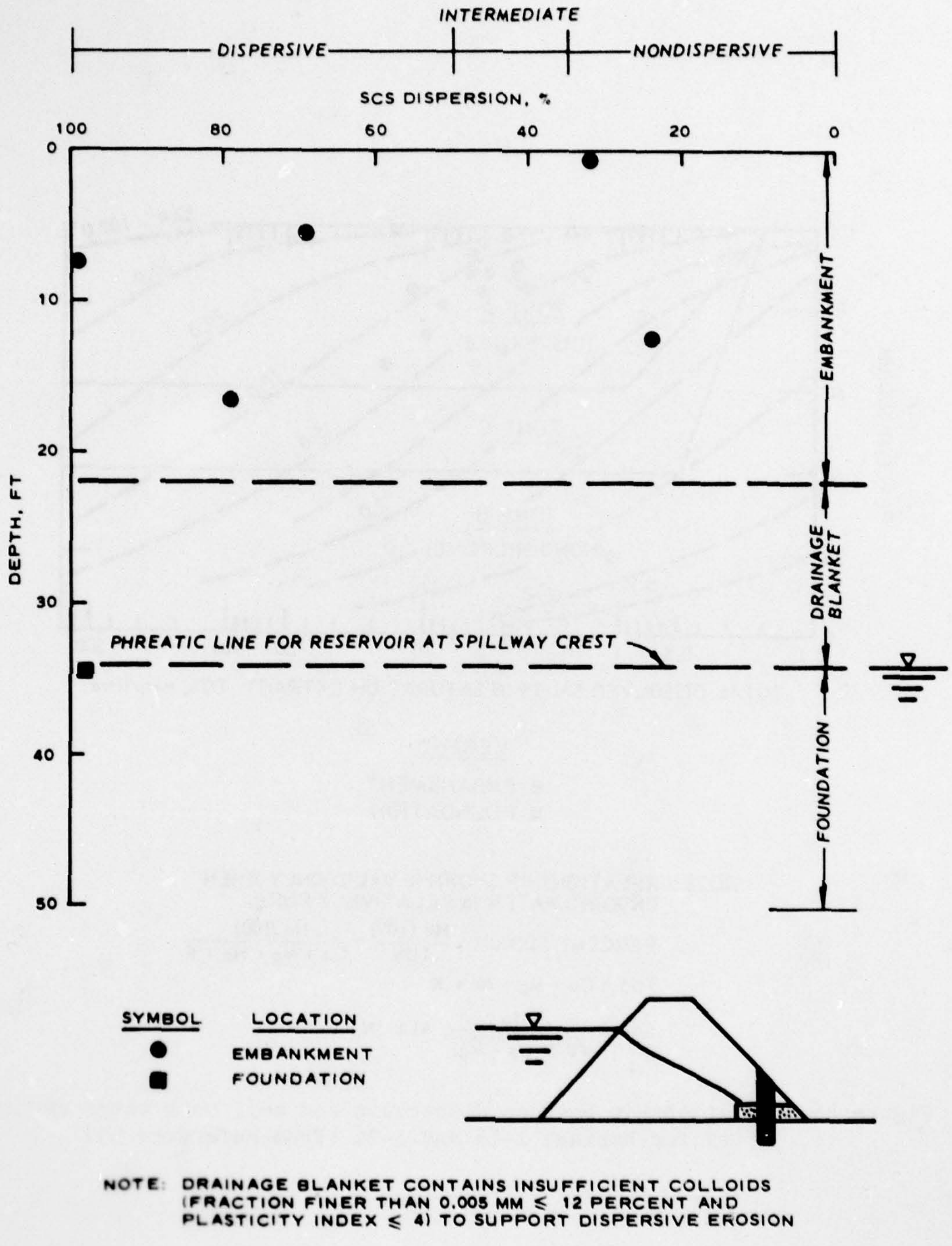
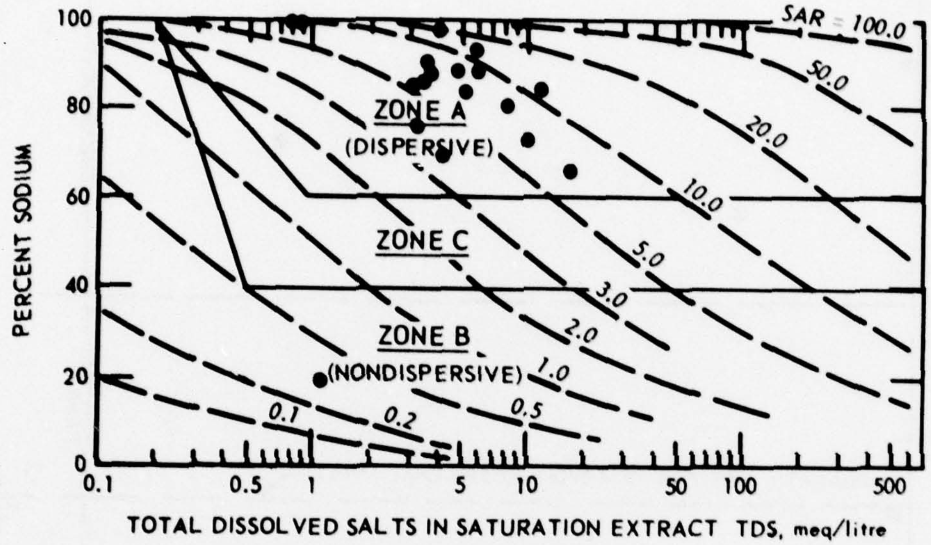


Figure 45. Dispersion versus depth from SCS dispersion test for borings 3-73 and 3-76



LEGEND

● EMBANKMENT

■ FOUNDATION

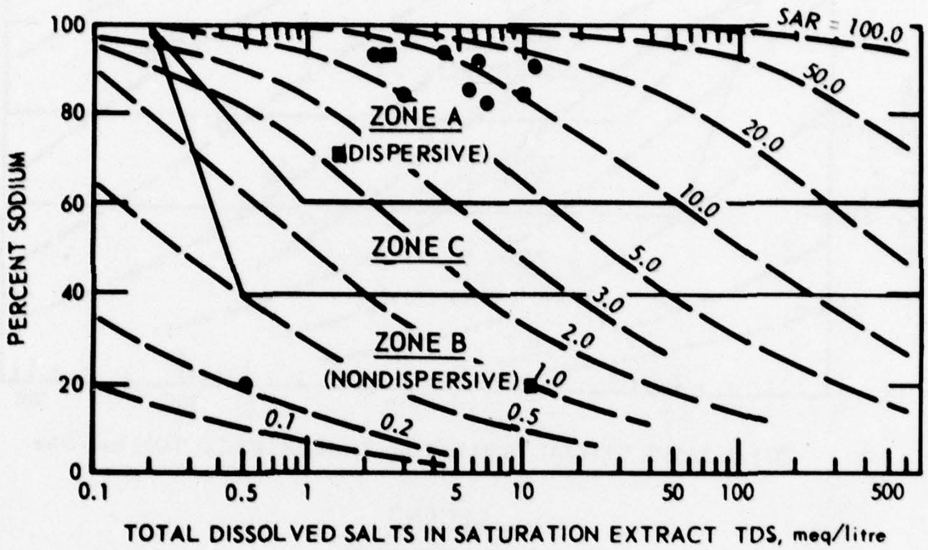
NOTE: RELATIONSHIP SHOWN IS VALID ONLY WHEN ERODING WATER IS RELATIVELY PURE.

$$\text{PERCENT SODIUM} = \frac{Na\ (100)}{TDS} = \frac{Na\ (100)}{Ca + Mg + Na + K}$$

$$TDS = Ca + Mg + Na + K$$

$$SAR = \frac{Na}{\sqrt{0.5\ (Ca + Mg)}}, \text{ ALL IN meq/litre}$$

Figure 46. Relationship between dispersion and soil pore water chemistry for borings 1-73 and 1-76 (from Reference 55)



LEGEND

● EMBANKMENT
■ FOUNDATION

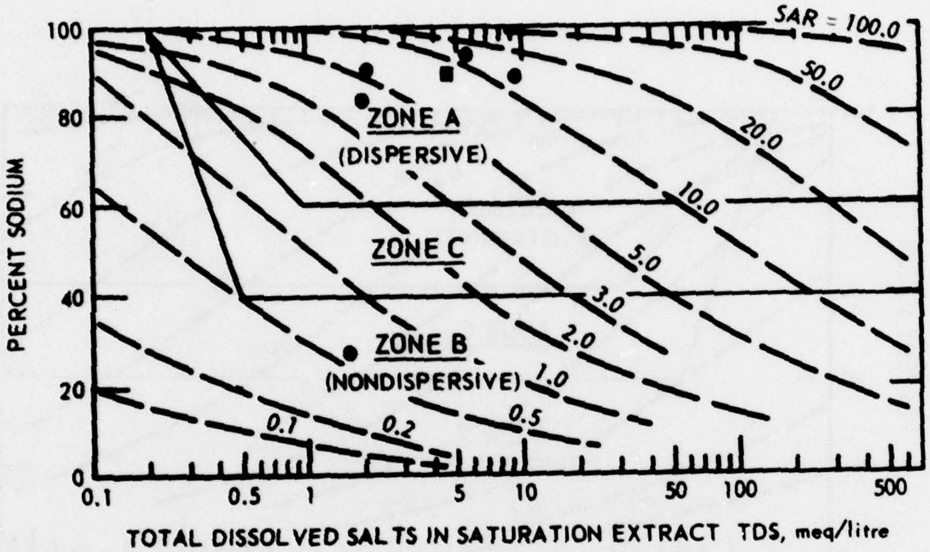
NOTE: RELATIONSHIP SHOWN IS VALID ONLY WHEN ERODING WATER IS RELATIVELY PURE.

$$\text{PERCENT SODIUM} = \frac{Na\,(100)}{TDS} = \frac{Na\,(100)}{Ca + Mg + Na + K}$$

$$TDS = Ca + Mg + Na + K$$

$$SAR = \frac{Na}{\sqrt{0.5\,(Ca + Mg)}}, \text{ ALL IN meq/litre}$$

Figure 47. Relationship between dispersion and soil pore water chemistry for borings 2-73 and 2-76 (from Reference 55)



LEGEND

● EMBANKMENT

■ FOUNDATION

NOTE: RELATIONSHIP SHOWN IS VALID ONLY WHEN ERODING WATER IS RELATIVELY PURE.

$$\text{PERCENT SODIUM} = \frac{Na\,(100)}{TDS} = \frac{Na\,(100)}{Ca + Mg + Na + K}$$

$$TDS = Ca + Mg + Na + K$$

$$SAR = \frac{Na}{\sqrt{0.5\,(Ca + Mg)}}, \text{ ALL IN meq/litre}$$

DRAINAGE BLANKET CONTAINS INSUFFICIENT COLLOIDS (FRACTION FINER THAN 0.005 MM < 12 PERCENT AND PLASTICITY INDEX < 4) TO SUPPORT DISPERSIVE EROSION

Figure 48. Relationship between dispersion and soil pore water chemistry for borings 3-73 and 3-76 (from Reference 55).

versus depth from the soil pore water chemistry correlation for borings 1-73 and 1-76, 2-73 and 2-76, and 3-73 and 3-76 are shown in Figures 49-51, respectively. The soil pore water chemistry correlation was nondispersive for the surface of the embankment, dispersive for the remaining portion of the embankment, and dispersive (3 of 4 readings) for the foundation.

Pinhole Erosion Test Results

47. The index properties of samples tested in the WES pinhole erosion apparatus are presented in Table 14. The pinhole erosion test results using distilled water and reservoir water as the eroding fluid are shown in Tables 15 and 16, respectively. As stated previously, the pinhole eorsion test is the most reliable test for identifying dispersive soils.[55,66] Dispersion versus depth from pinhole erosion test results for borings 1-73 and 1-76, 2-73 and 2-76, and 3-73 and 3-76 are shown in Figures 52-54, respectively. The pinhole erosion test results were nondispersive for the surface of the embankment, intermediate to dispersive (23 of 27 readings) for the remaining portion of the embankment, and nondispersive for the foundation (using reservoir water as eroding fluid).

Dispersion Characteristics of Embankment and Foundation Soils

48. A summary of the dispersion tests is shown on Table 17. Using the consensus of the SCS dispersion test results, soil pore water chemistry correlation, and pinhole erosion test results, a summary of dispersion versus depth for borings 1-73 and 1-76, 2-73 and 2-76, and 3-73 and 3-76 are shown in Figures 55-57, respectively. The embankment soil is nondispersive at the surface and dispersive below a depth of about 6 ft. Based upon the limited data obtained below the embankment, the foundation soil is dispersive at the surface and nondispersive to dispersive with depth.

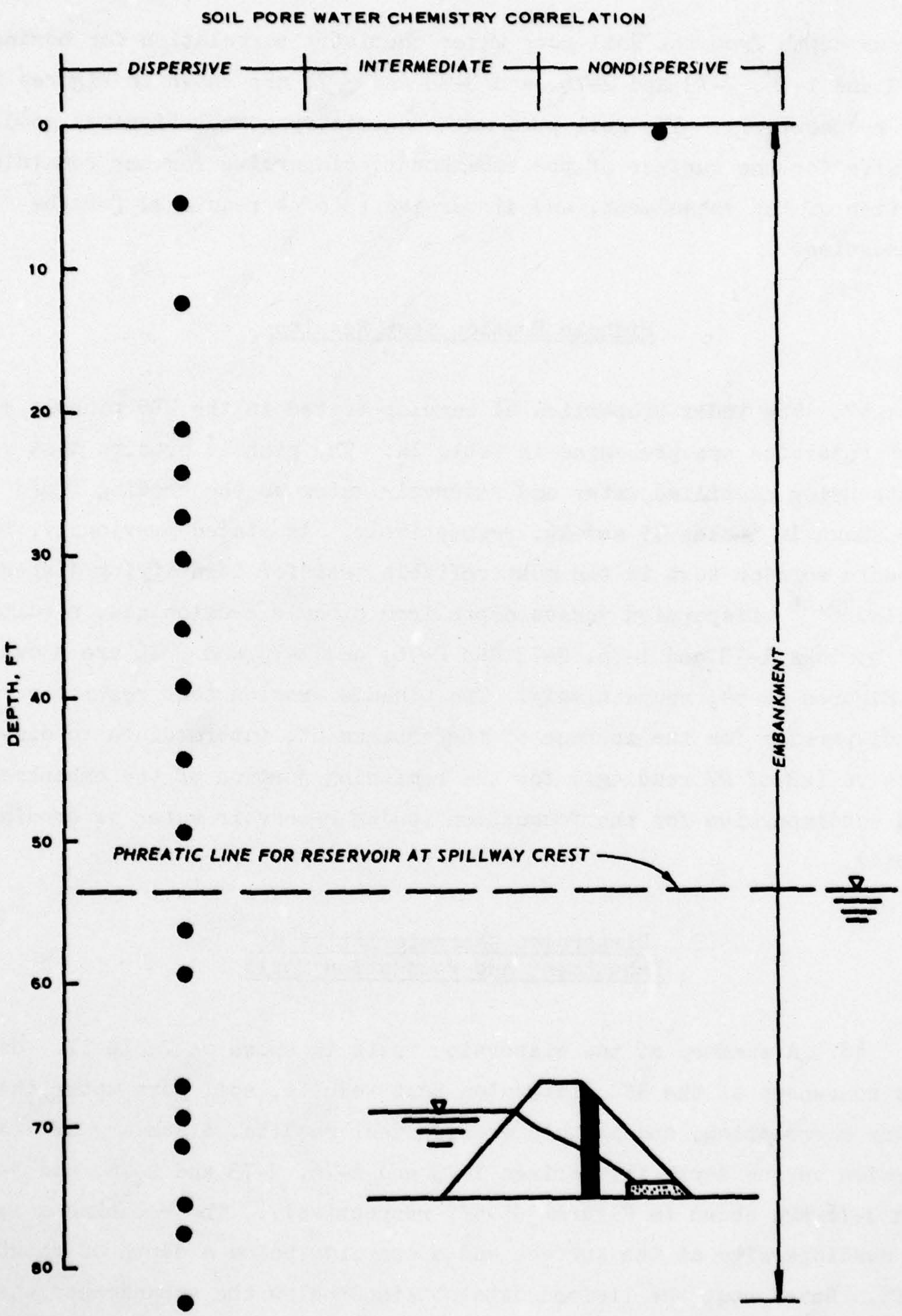


Figure 49. Dispersion versus depth from soil pore water chemistry correlation for borings 1-73 and 1-76

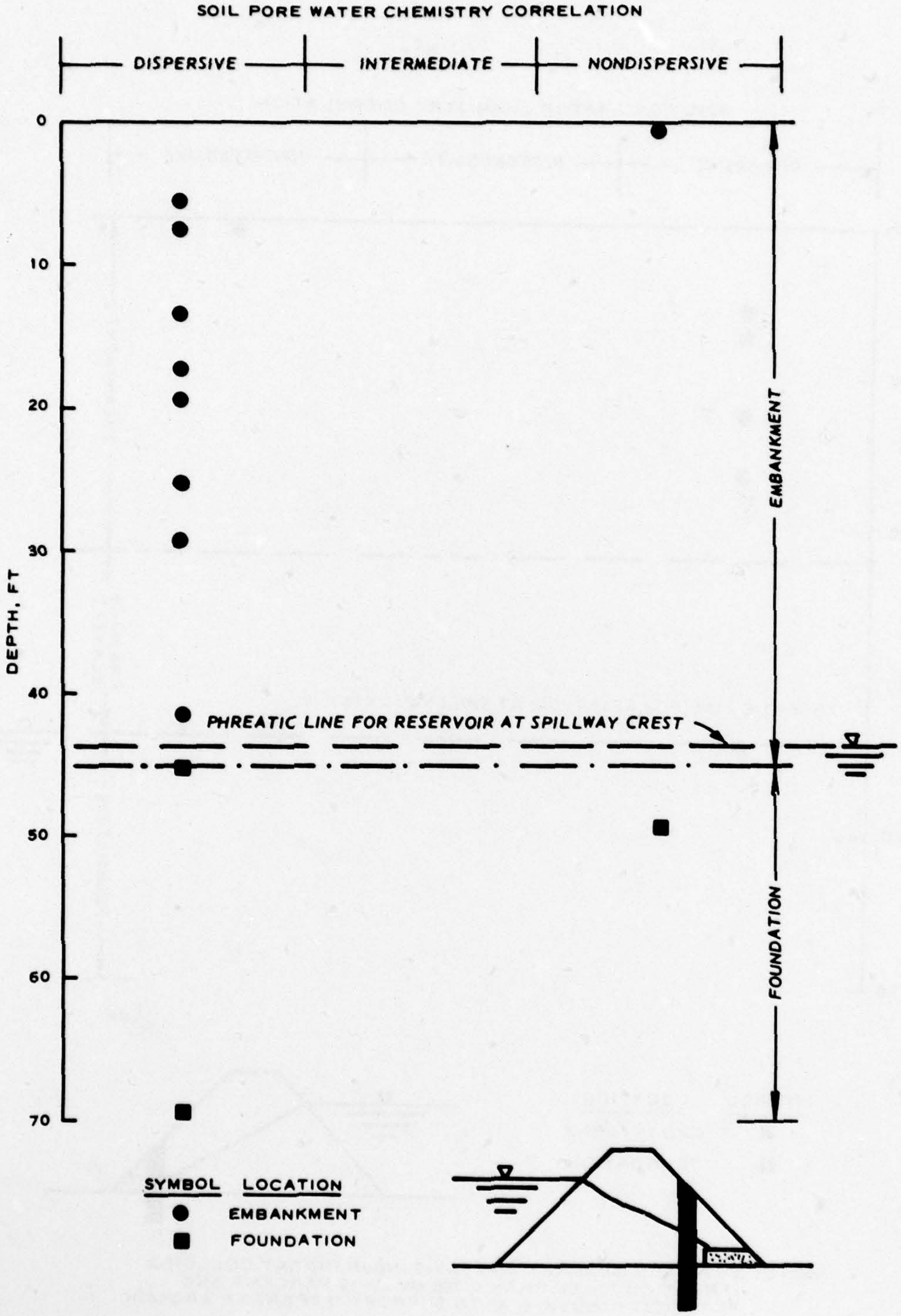


Figure 50. Dispersion versus depth from soil pore water chemistry correlation for borings 2-73 and 2-76

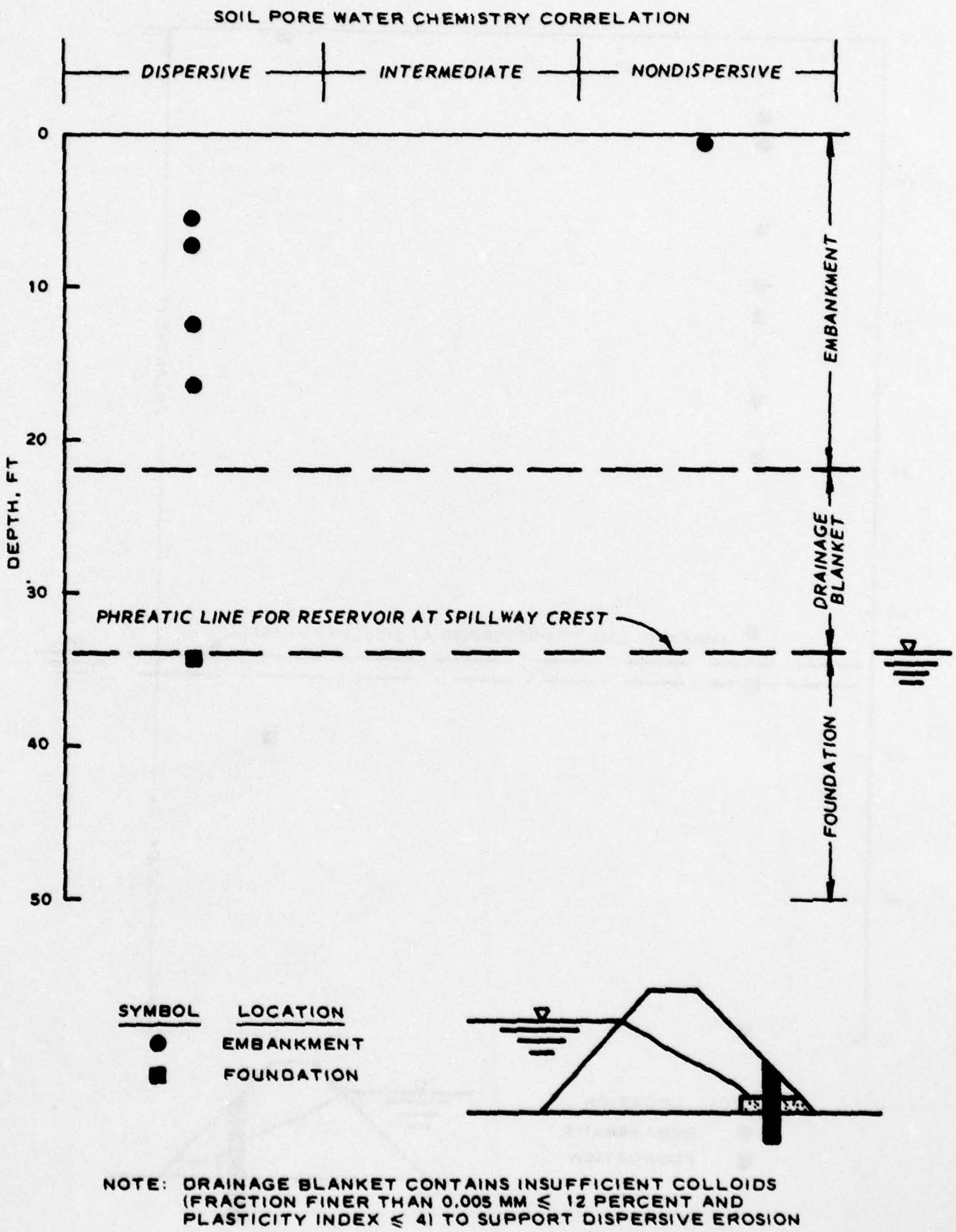


Figure 51. Dispersion versus depth from soil pore water chemistry correlation for borings 3-73 and 3-76

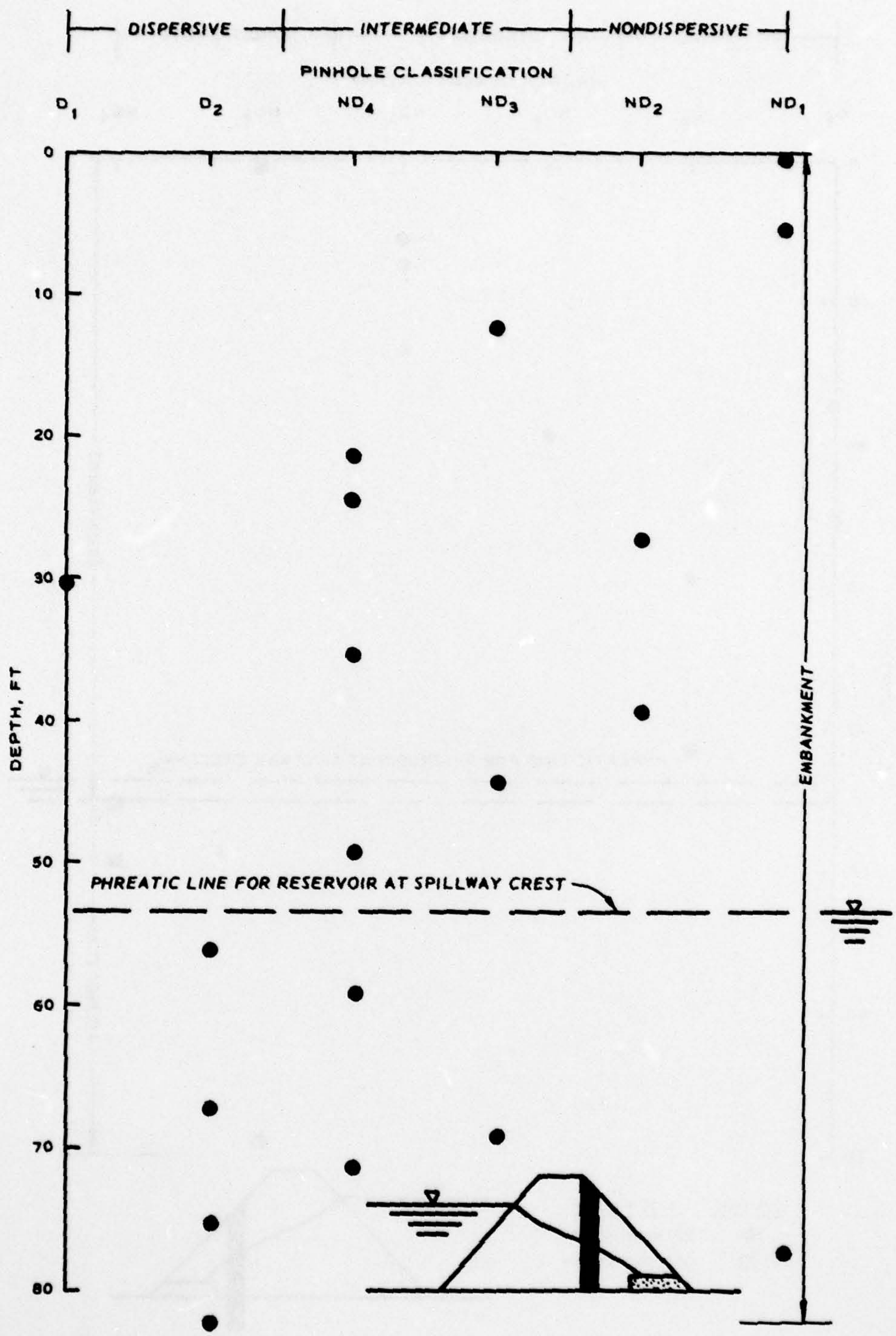


Figure 52. Dispersion versus depth from pinhole erosion test for borings 1-73 and 1-76

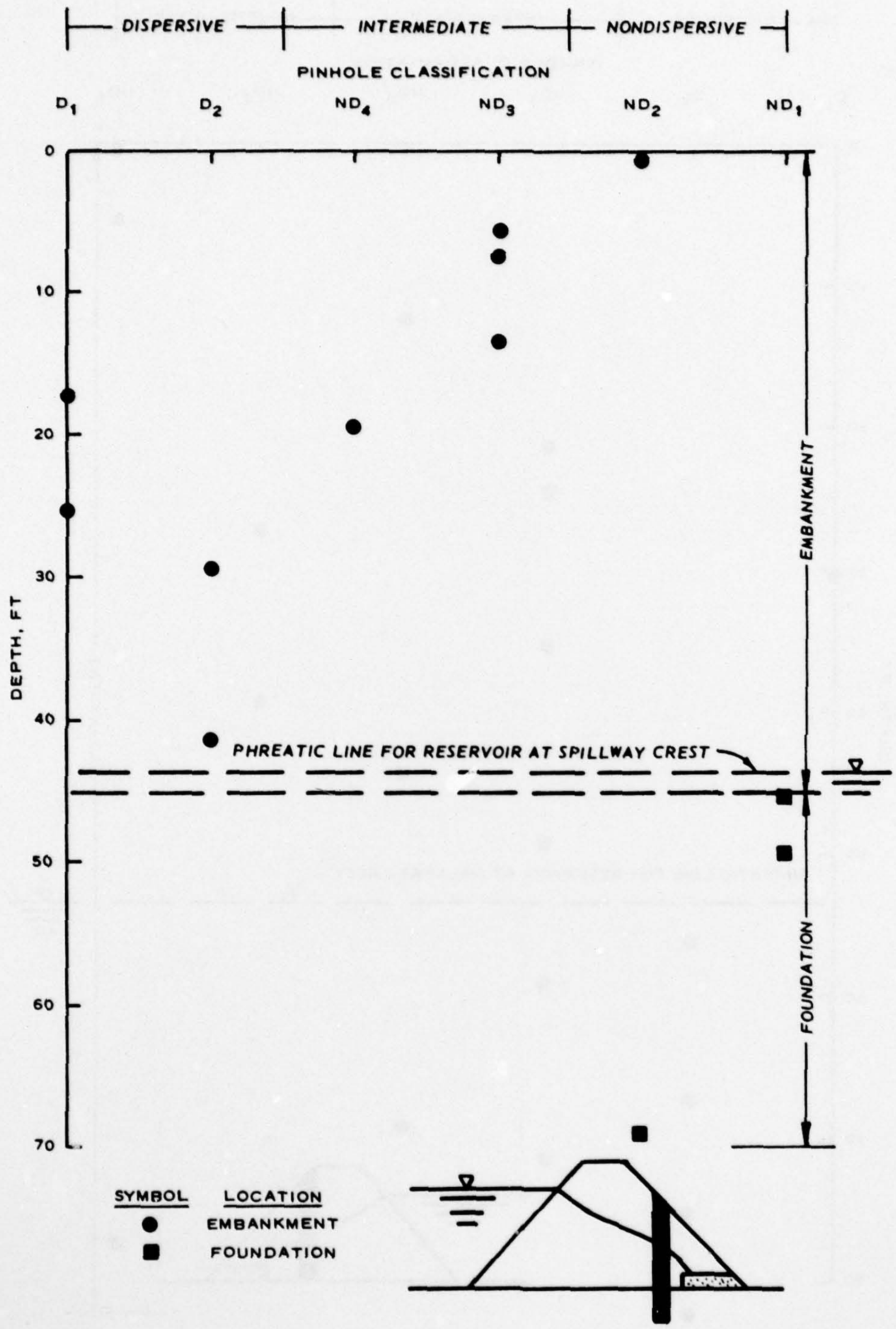
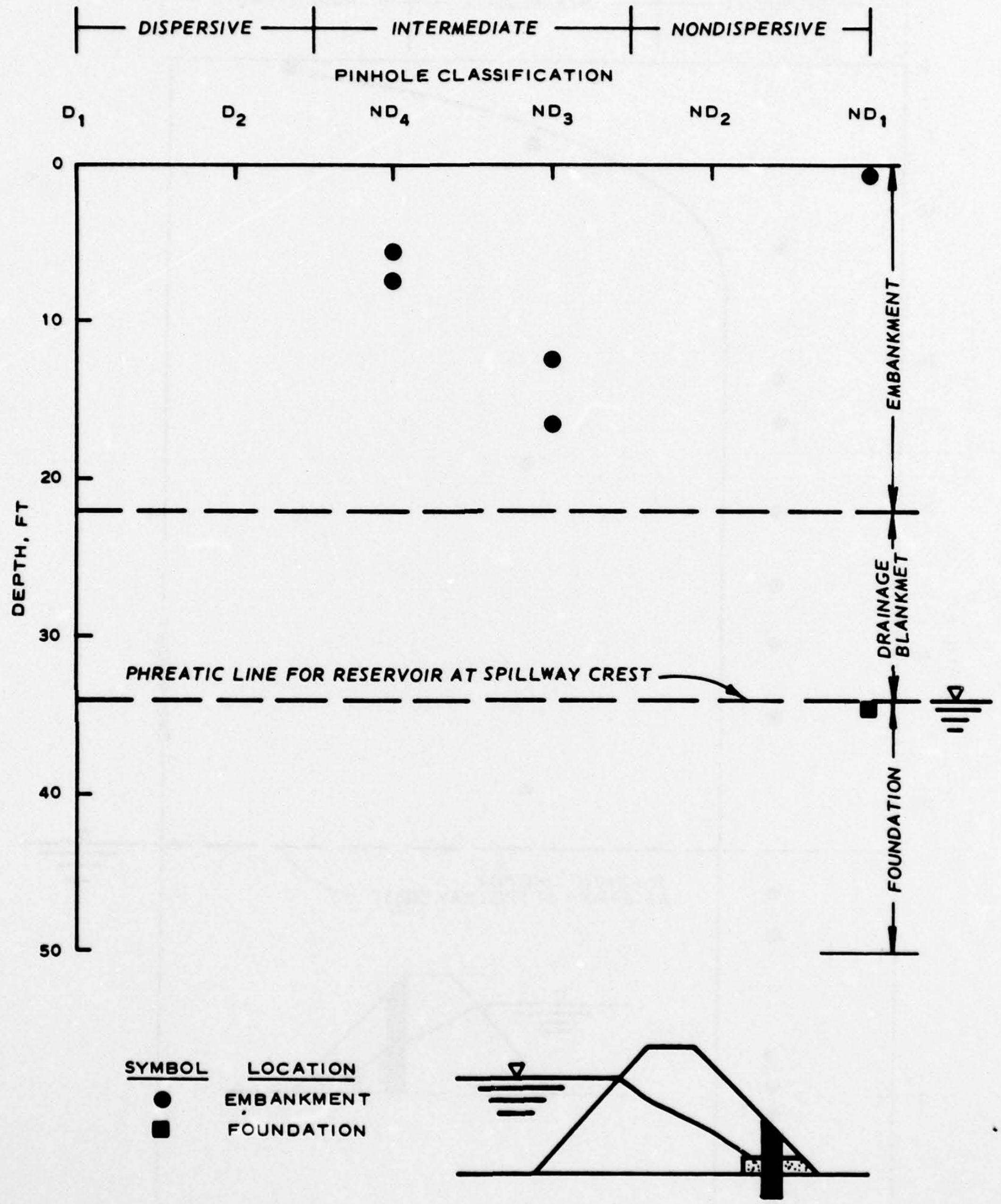


Figure 53. Dispersion versus depth from pinhole erosion test for borings 2-73 and 2-76



NOTE: DRAINAGE BLANKET CONTAINS INSUFFICIENT COLLOIDS (FRACTION FINER THAN 0.005 MM ≤ 12 PERCENT AND PLASTICITY INDEX ≤ 4) TO SUPPORT DISPERSIVE EROSION

Figure 54. Dispersion versus depth from pinhole erosion test for borings 3-73 and 3-76

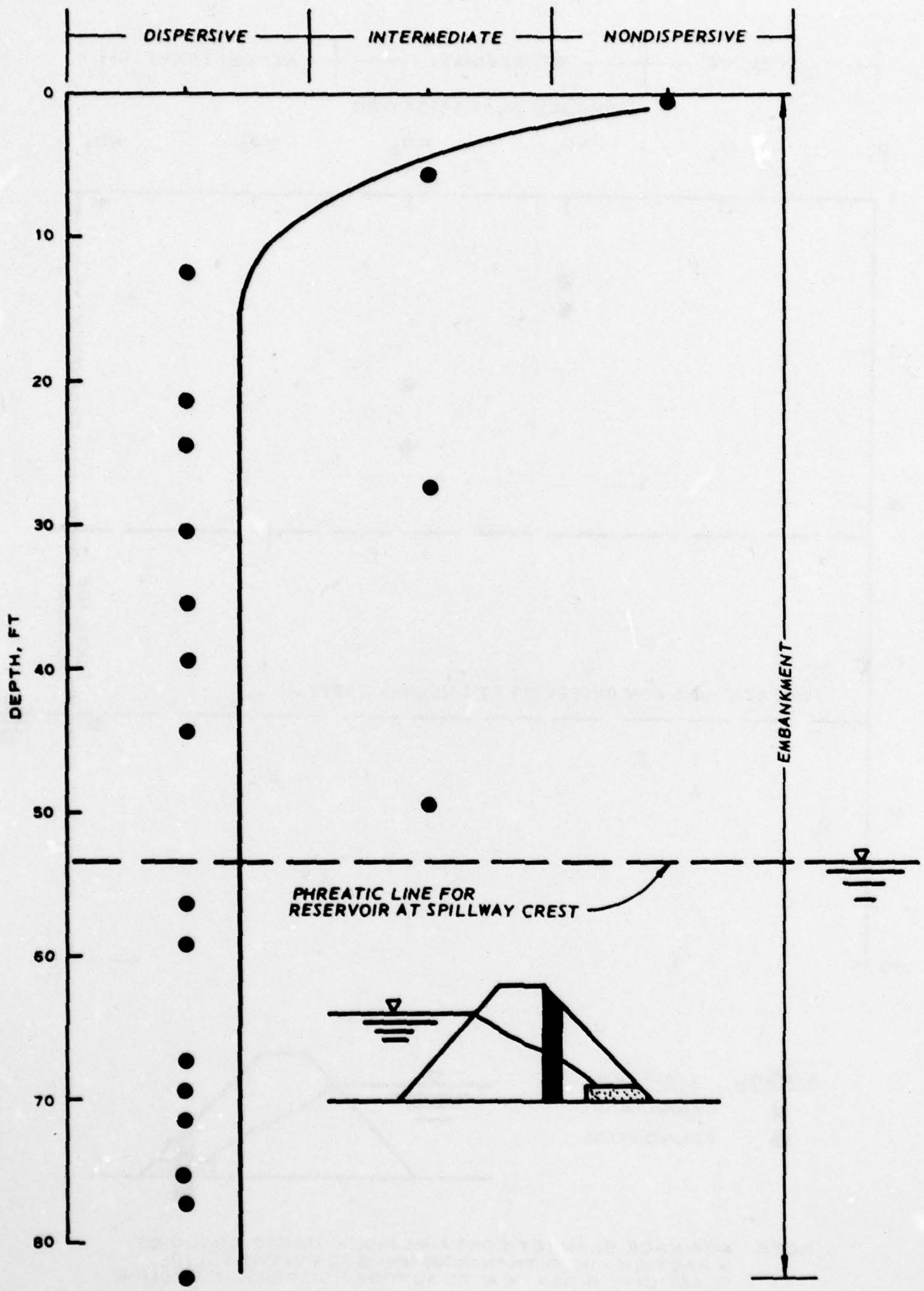


Figure 55. Summary of dispersion versus depth for borings 1-73 and 1-76

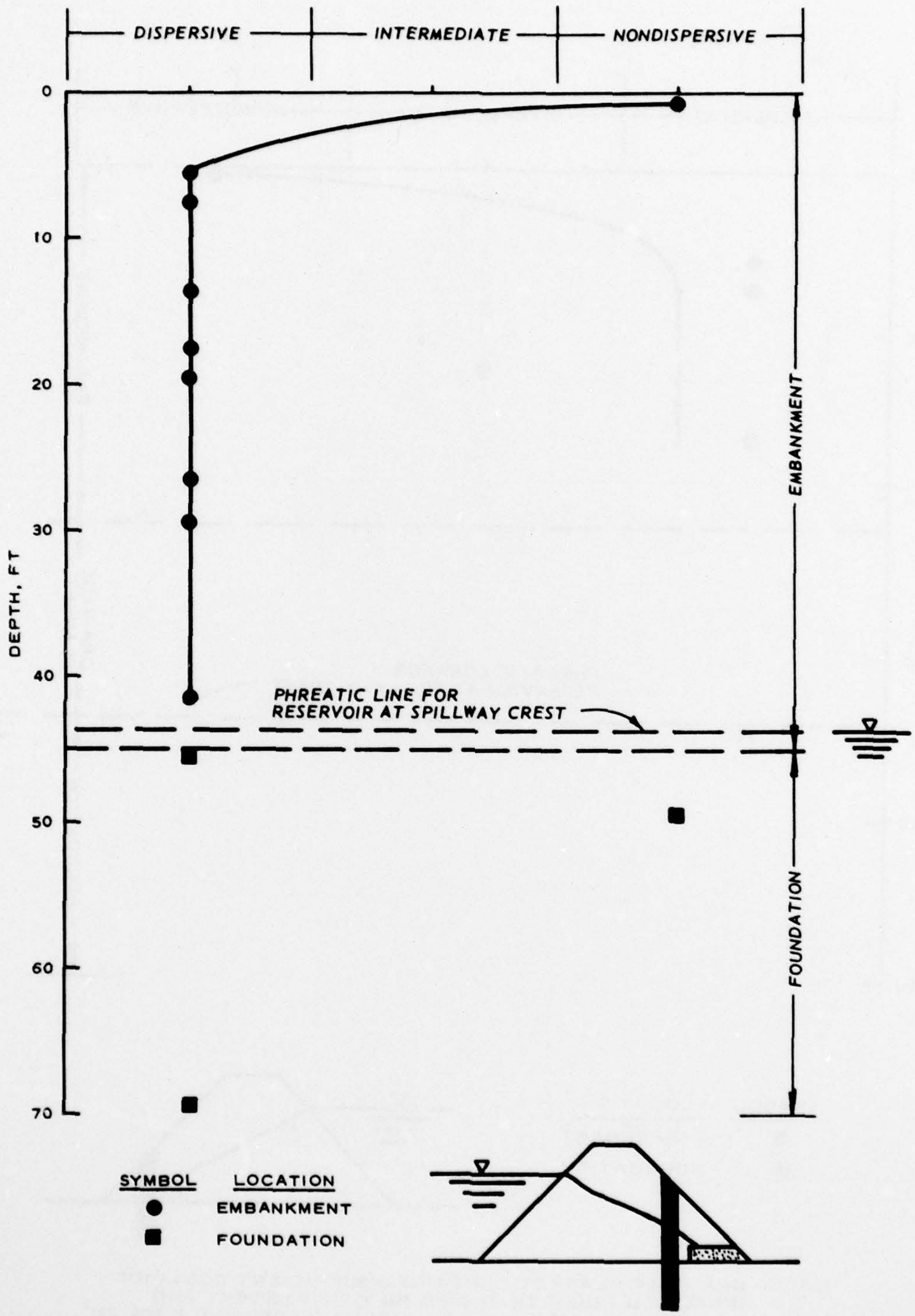
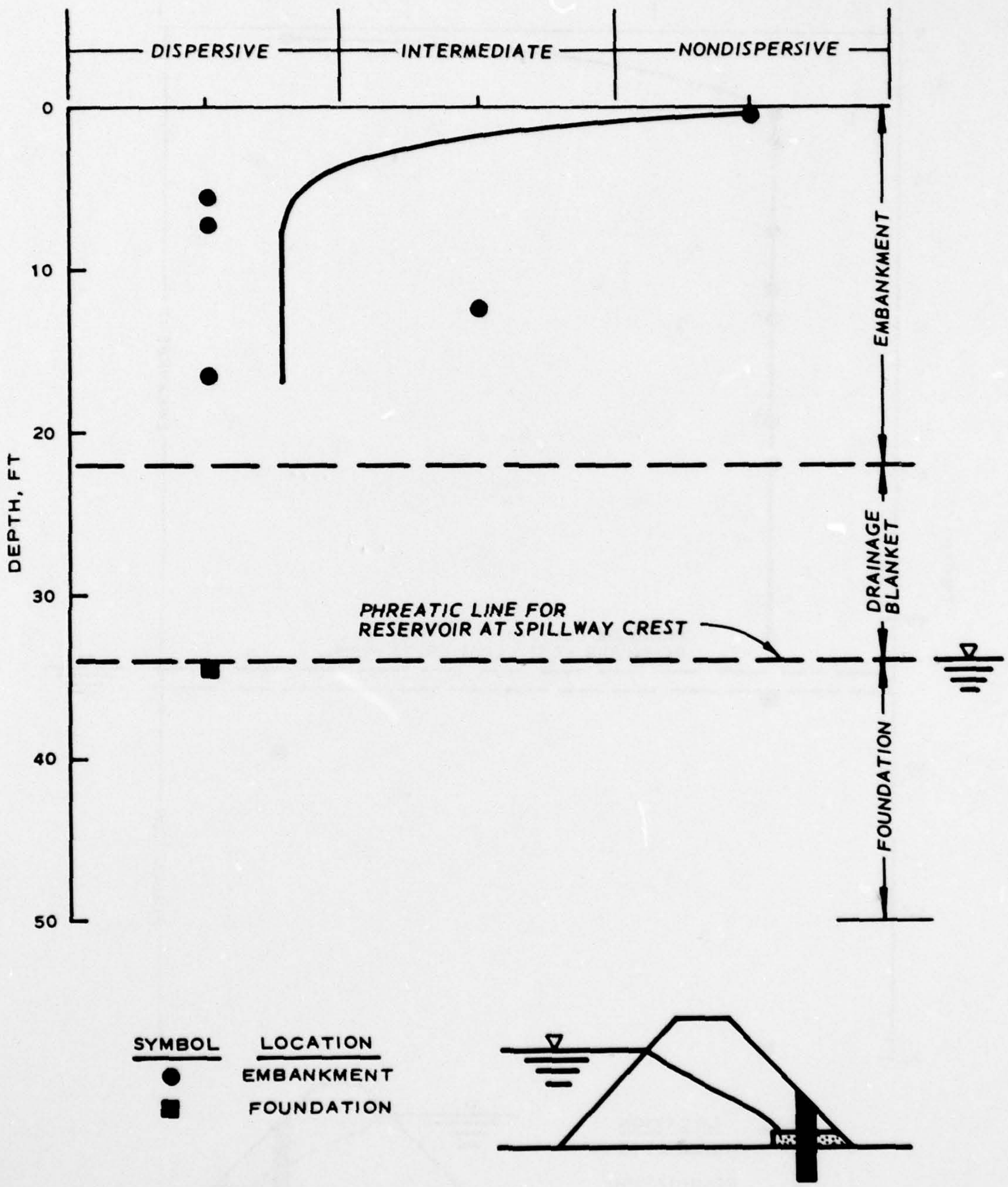


Figure 56. Summary of dispersion versus depth for borings 2-73 and 2-76



NOTE: DRAINAGE BLANKET CONTAINS INSUFFICIENT COLLOIDS (FRACTION FINER THAN 0.005 MM $\leq$ 12 PERCENT AND PLASTICITY INDEX $\leq$ 4) TO SUPPORT DISPERSIVE EROSION

Figure 57. Summary of dispersion versus depth for borings 3-73 and 3-76

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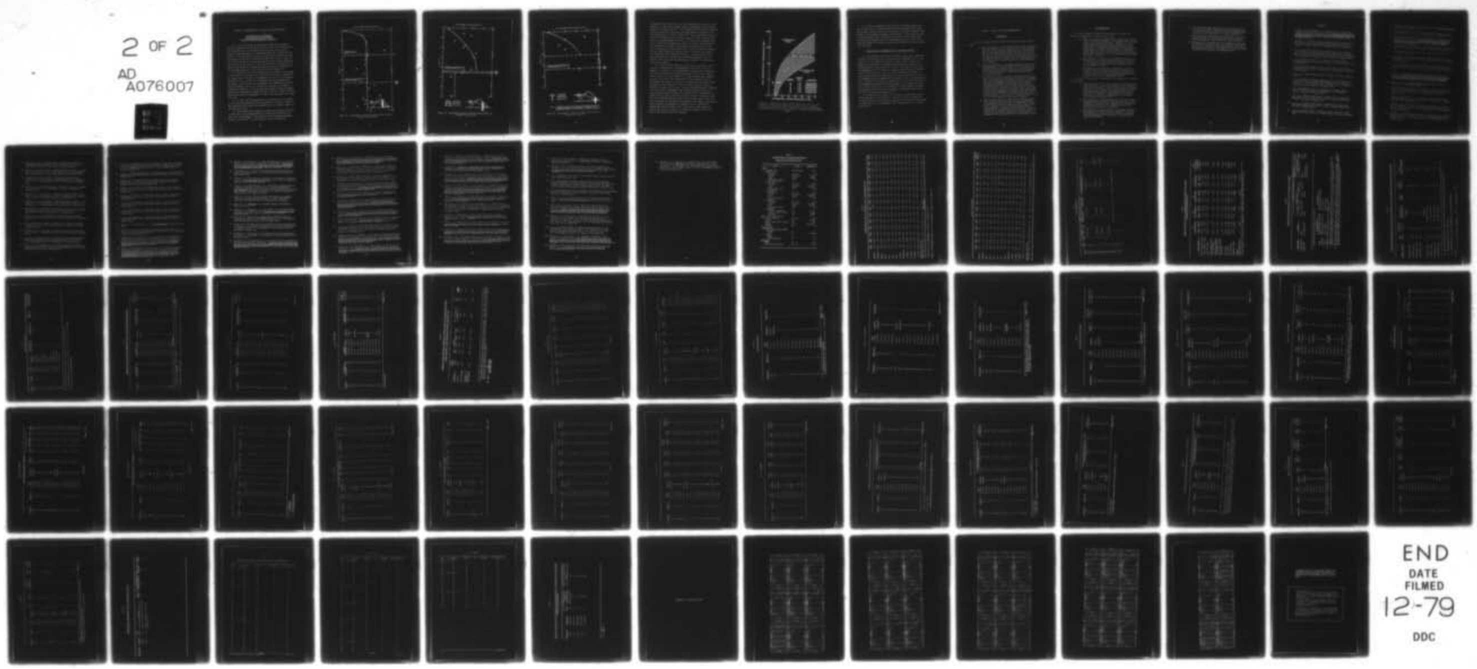

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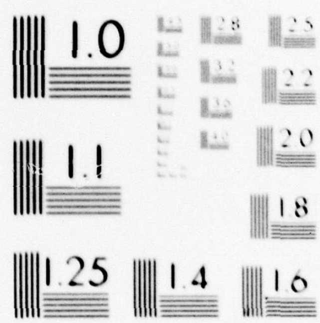

MICROCOPY RESOLUTION TEST CHART

PART VI: SUSCEPTIBILITY TO PIPING AND RAINFALL EROSION

Susceptibility of Embankment and Foundation Soils to Piping Using the Australian Method of Analysis

49. As shown in Figure 26, Australian workers have developed a method of analysis using the ESP of the soil, total ionic concentration of the eroding water, and predominate clay minerals of the soil to assess the susceptibility of a homogeneous earth dam to dispersive clay piping. If the dam contains cracks or sandy lenses traversing the width of the dam, the reservoir (eroding) water will have a path of rapid access across the dam, and the dam may fail by dispersive clay piping upon first filling of the reservoir. The only deterrence to failure by dispersive clay piping for a homogeneous earth dam would be the ability of the embankment material to swell and seal the flow channels.[45]

50. The ESP of the soil versus depth given in Table 13 for borings 1-73 and 1-76, 2-73 and 2-76, and 3-73 and 3-76 are shown in Figures 58-60, respectively. The ESP of the embankment soil is 2-3 at the surface, increases to 12-17 at 20-40 ft, and remains relatively constant below this depth. Since the ESP of the embankment soil immediately above and below the phreatic line for the reservoir at spillway crest showed no significant difference, it is concluded that seepage through the embankment has not significantly changed the ESP of the soil. Based on limited data obtained below the embankment, the ESP of the foundation soil ranged from 4-16.

51. In 1957, Grim[81] determined the clay mineralogy of two samples of soil from the downstream slope of Grenada Dam. As shown in Table 18, the predominate clay mineral was montmorillonite with smaller amounts of kaolinite and illite.

52. Early limnological studies at Grenada Lake conducted from 1955 to 1965 did not include measurements of conductivity; therefore, the total ionic concentration of the reservoir water following impoundment in 1954 is not known.[82] No water quality data are available on

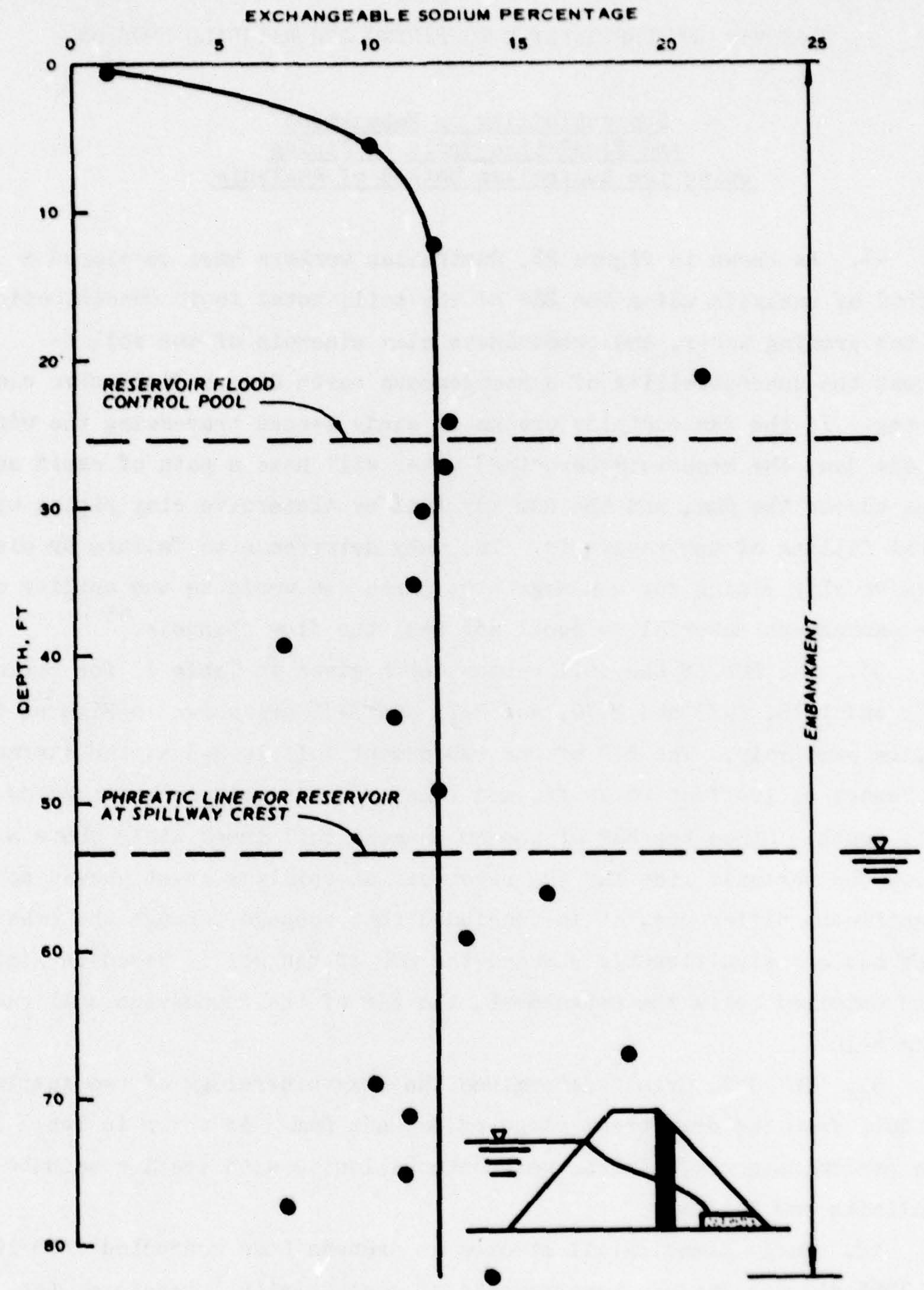

Figure 58. Exchangeable sodium percentage versus depth for borings 1-73 and 1-76

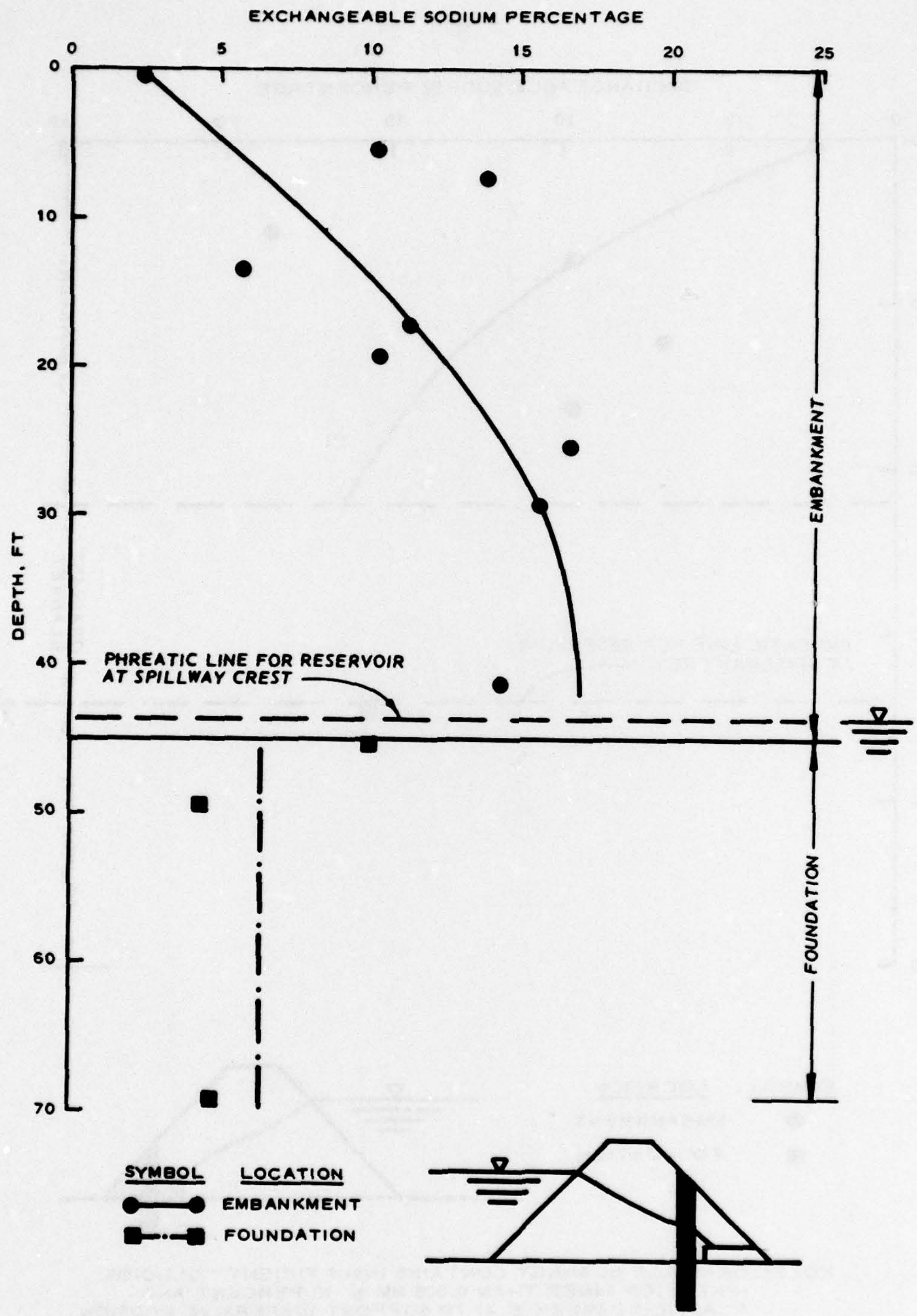

Figure 59. Exchangeable sodium percentage versus depth for borings 2-73 and 2-76

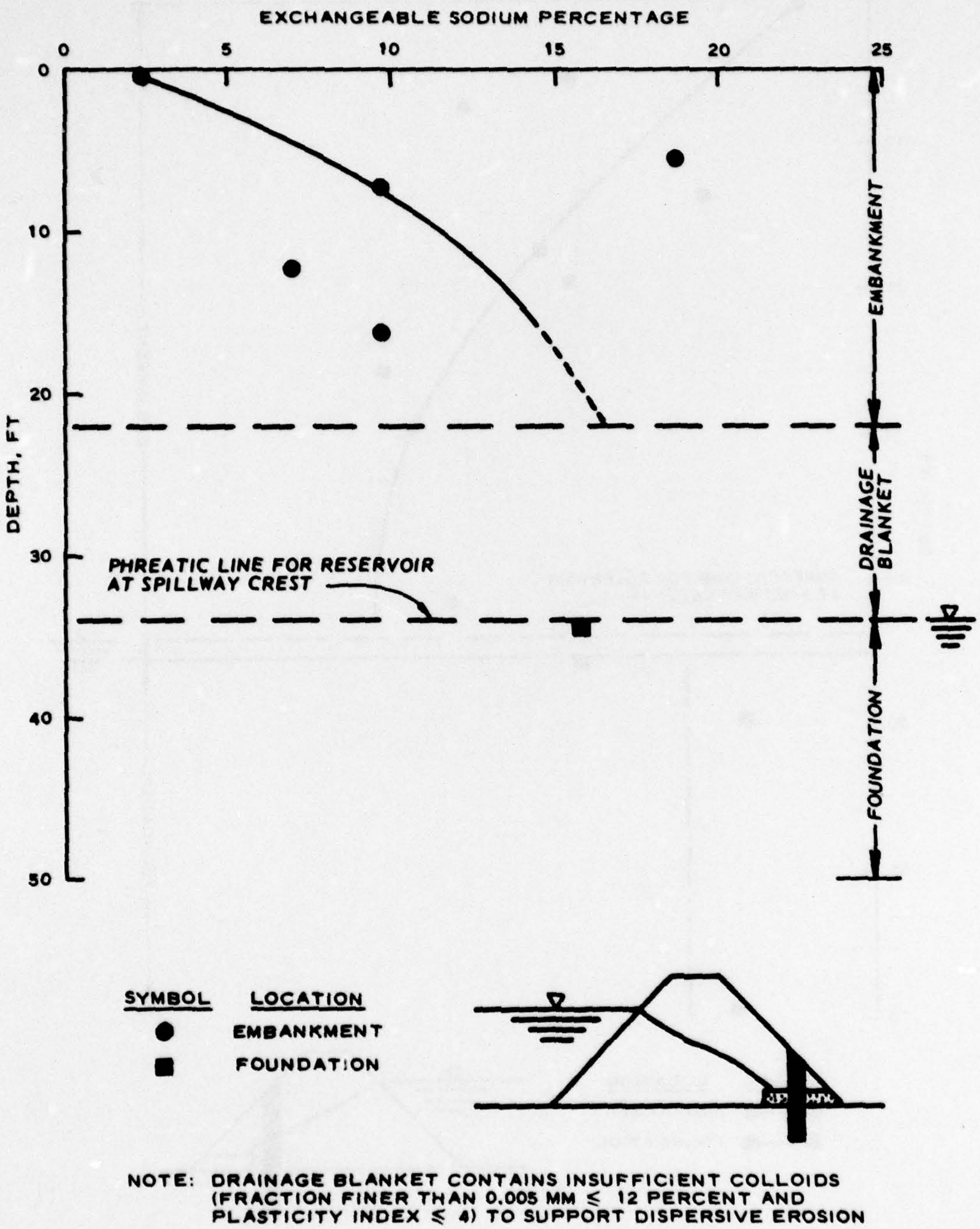

Figure 60. Exchangeable sodium percentage versus depth for borings 3-73 and 3-76

Yalobusha or Skuna Rivers prior to 1974; therefore, the total ionic concentration of the reservoir water following impoundment cannot be estimated from tributary data.[83,84] From December 1974 to present, limnological data, including conductivity, have been measured quarterly by the Vicksburg District as shown in Table 19.[85] These data indicate that the total ionic concentration of reservoir water, 0.25 miles from the dam opposite sta 154+00, ranged from 0.4 to 1.0 meq/ℓ with a maximum variation in total ionic concentration between the surface and the bottom of the reservoir of 0.2 meq/ℓ. This compares favorably with the total ionic concentration of reservoir water of 0.6 meq/ℓ (Table 13) obtained for this study in May 1976 from the surface of Grenada Lake opposite sta 136+00 about 100 ft from the shoreline. Although the total ionic concentration of the reservoir water following impoundment was not measured, the total ionic concentration was probably not significantly lower than 0.6 meq/ℓ.

53. Using the ESP of the soil, total ionic concentration of the reservoir (eroding) water, and predominate clay mineral of the soil summarized in Table 20, the susceptibility of Grenada Dam to dispersive clay piping, assuming the reservoir water has a path of rapid access across the dam, is shown in Figure 61. Since no appreciable changes in the ESP of the soil, total concentration of the reservoir water, or predominate clay mineral of the soil are believed to have occurred from impoundment of the reservoir in 1954 to present, Figure 61 would be applicable for any point in time. With the Australian method of analysis, the plot of both the embankment and foundation soils in the zone of potential deflocculation (Figure 61) indicates that if Grenada Dam contained cracks or sandy lenses traversing the width of the dam where the reservoir (eroding) water would have a path of rapid access across the dam, there would be a potential for dispersive clay piping. Since piping failure through the embankment or foundation has not occurred at Grenada Dam, either the dam is free from cracks or sandy lenses traversing the width of the dam, or if cracks are present, the soil is able to swell and seal the flow channels.

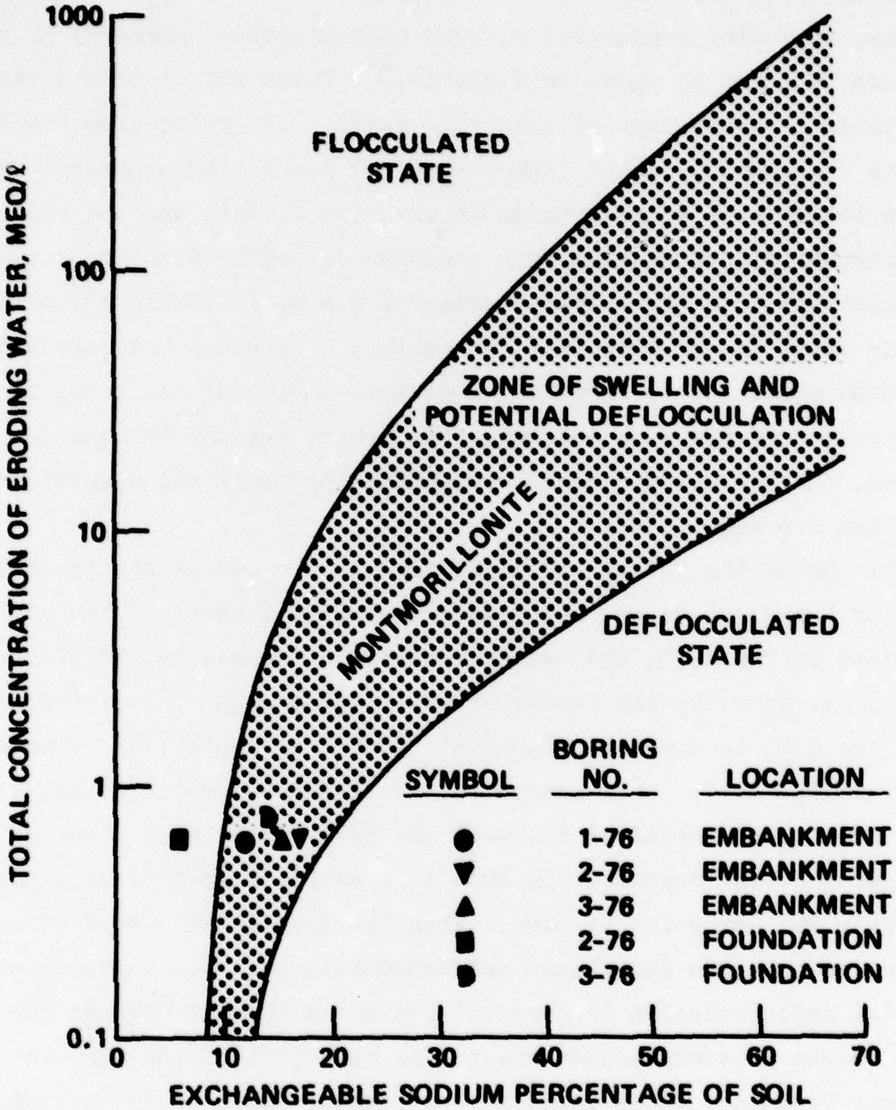

Figure 61. Prediction of piping failure using the total ionic concentration of the eroding water and the ESP of the soil assuming the reservior water has a path of rapid access across the dam (adapted from Reference 45 with the permission of the Publishers Butterworth, Sydney)

54. Piping of foundation soils into the toe drainage system collector pipe, prior to its replacement with an open-paved ditch in 1961, was of major concern because of the possibility of erosion channels being formed in the foundation leading to the reservoir. The dispersive nature of the foundation, shown by limited data obtained in this study, indicates that if backward erosion of concentrated leaks into the collector pipe had reached the reservoir, piping failure would have likely occurred in a few hours.

Susceptibility of Embankment Soil to Rainfall Erosion

55. The laboratory test results obtained in this study, summarized in Table 17 and Figures 55-57, indicate that the embankment soil is nondispersive at the surface and dispersive below a depth of about 6 ft. Based upon the history of occurrence of rainfall erosion tunnels on the downstream slope of the dam, given in Table 3 and Figure 19, apparently the embankment soil at the time of construction was dispersive from the surface (below the 12-in. layer of topsoil) throughout its depth. An amelioration process has taken place with time, and the upper portion of the embankment has been changed from a dispersive soil to a nondispersive soil. Changes in the dispersion characteristics of the upper portion of the embankment may result from seasonal moisture and temperature variation, biologic activity, and root growth.[86] Similar changes, described in paragraph 30, have occurred in homogeneous earth dams constructed by the SCS in north-central Mississippi.[58]

56. Based upon the results of this study, the downstream slope of Grenada Dam has low susceptibility to rainfall erosion at present due to an amelioration process that has taken place with time that has changed the upper portion of the embankment from a dispersive soil to a nondispersive soil.

PART VII: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

57. The following conclusions are drawn from the results of this study:

- a. Past performance of Grenada Dam indicates piping of embankment and foundation soils through joints of the collector pipe for the toe drainage system (prior to its replacement with an open-paved ditch in 1961) occurred. Also, rainfall erosion tunnels developed on the downstream slope of the dam, primarily in the valley section between sta 105+00 and 145+00, soon after completion of the main embankment in 1949 and have continued to develop at a decreasing rate to the present.
- b. Based upon laboratory test results obtained in this study, the embankment soil is nondispersive at the surface and dispersive below a depth of about 6 ft. Limited data obtained below the embankment indicate that the foundation soil is dispersive at the surface and nondispersive to dispersive with depth.
- c. The Australian method of analysis, using the ESP of the soil, total ionic concentration of the reservoir (eroding) water, and predominate clay mineral of the soil, indicates that both the embankment and foundation soils would be potentially susceptible to dispersive clay piping if Grenada Dam contained cracks or sandy lenses traversing the width of the dam where the reservoir water would have a path of rapid access across the dam. Since piping failure through the embankment or foundation has not occurred, either the dam is free from cracks or sandy lenses traversing the width of the dam, or if cracks are present, the soil is able to swell and seal the flow channels.
- d. Based upon the history of occurrence of rainfall erosion tunnels on the downstream slope of the dam, apparently the embankment soil at the time of construction was dispersive from the surface (below the 12-in. layer of topsoil) throughout its depth. The downstream slope of Grenada Dam has low susceptibility to rainfall erosion at present due to an amelioration process that has taken place with time that has changed the upper portion of the embankment from a dispersive soil to a nondispersive soil.

Recommendations

58. The following recommendations pertaining to Grenada Lake Project are suggested as a result of this study:

a. There is no reason to suspect that a concentrated leak will develop through the embankment or foundation under ordinary conditions of reservoir operation. However, if a concentrated leak developed under unusual conditions, such as cracking from earthquake loading, piping failure would likely occur in a few hours. Therefore, if a concentrated leak develops, provisions should be made to immediately notify public officials in the City of Grenada and Grenada County and evacuate the Corps of Engineers recreation areas downstream of the dam.

b. The occurrence of cave-ins on the downstream slope of the dam should be reported each quarter giving the location of each cave-in by station and offset, sketch of the cave-in with dimensions (Figure 10), and photographs of the cave-in (Figures 11 and 12).

c. The changes in the shoreline at Grenada Lake should be documented yearly by aerial photographs. Extensive erosion due to wave action has occurred along the shoreline of the reservoir in the natural (probably dispersive) soil deposits since reservoir impoundment in 1954.

59. The following general recommendations are suggested as a result of this study:

a. Laboratory tests for the identification of dispersive clays should be made as a part of the subsurface soils investigation for earth dams.[67] Since the distribution of dispersive clays in a soil profile may vary with depth as well as in plane, thorough sampling of proposed borrow areas is required.[60] Dispersion of characteristics of soil from joint solution cavities in the foundation and abutment areas should be determined.[87-89]

b. When dispersive clays are identified, field trials should be performed where excavated slopes and test fills (both untreated and protected with sand-gravel and lime-modified blankets) are exposed to the elements to observe the susceptibility to rainfall erosion.[90]

c. When dispersive clays are utilized as embankment material, laboratory filter tests should be conducted to evaluate the adequacy of the proposed filter material concerning internal stability and effectiveness in protecting the base soil.[60,63,91,92]

d. In zoning earth dams, dispersive clay should not be used in critical locations that might develop concentrated leaks, such as around conduits, adjacent to rigid structures, and in sections of the dam subject to tensile cracking.[90]

e. Dispersive clays should not be placed directly on fractured rock foundations. A lime-modified zone may be required between a dispersive clay core and the rock foundation.[91]

f. The possibility of piping failure in homogeneous earth dams constructed of dispersive clay resulting from a decrease in the total ionic concentration of the reservoir water should be investigated when it is contemplated to change the source of water for the reservoir (see paragraph 26).

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Table 1

Selected Data on Grenada Lake Project

Features (from Reference 10)

Item	Item	Quantity
Lake:		
Drainage Area	Square miles	1,320
Minimum Pool		
Elevation	Ft, msl	193.0
Volume	Acre-foot	85,700
Area	Acres	9,800
Equivalent runoff	In.	1.2
Flood Control Pool		
Elevation	Ft, msl	231.0
Volume	Acre-foot	1,251,700
Area	Acres	64,600
Equivalent runoff	In.	17.8
Surcharge Pool		
Elevation	Ft, msl	249.7
Volume	Acre-foot	1,384,700
Area	Acres	106,100
Equivalent runoff	In.	19.7
Freeboard		
Elevation	Ft, msl	256.0
Volume	Acre-foot	1,012,400
Area	Acres	131,600
Equivalent runoff	In.	14.4
Discharges		
Conduit, regulated	cfs	5,100
Conduit, flood control	cfs	10,000
Spillway	cfs	52,000
Dam (Rolled earthfill):		
Crest elevation	Ft, msl	256.0
Length	Ft	13,900
Height, above mean valley	Ft	80
Volume	Cu ft	9,600,000
Freeboard	Ft	6.3
Outlet Works:		
Gates (Caterpillar-type)		
Number	--	3
Size	Ft	7.5X14.0
Conduit		
Number	--	1
Size (diameter)	Ft	17.0
Spillway:		
Width	Ft	200
Wier crest elevation	Ft, msl	231.0

Table 2

Precipitation Data for Grenada, Miss. (from Reference 16)

	1948	1949	1950	1951	1952	1953	1954	1955	1956	1957	1958	1959	1960	1961	1962
January	4.25	13.95	7.15	9.04	4.30	3.53	7.09**	3.26	2.22	9.21	3.06	3.19	6.81	2.48	8.04
February	11.22	4.89	6.06	6.38	3.08	8.82	4.90	4.92	9.30	6.72	2.52	6.15	2.93	13.02	6.52
March	8.03	10.64	7.25	8.38	3.53	9.16	1.91	9.62	4.39	2.90	4.42	5.58	6.43	10.07	4.07
April	1.69	3.77	1.99	4.96	5.11	6.72	4.35	8.60	7.26	5.35	7.07	4.19	2.26	3.24	8.66
May	1.41	4.85	3.23	1.75	5.51	7.48	7.20	3.53	4.42	2.98	8.67	6.47	4.94	3.01	5.52
June	6.42	3.16	2.30	2.45	1.06	0.40	0.86	2.75	2.59	9.00	3.48	4.54	1.59	3.35	2.19
July	1.42	3.97	5.63	5.28	1.54	4.41	3.37	4.65	1.15	9.42	5.23	9.76	2.39	8.60	5.53
August	7.29	7.57	9.69	0.65	0.94	2.44	1.26	0.99	1.54	1.59	2.15	1.79	3.91	1.66	2.33
September	3.31	0.65*	3.47	2.37	0.57	0.97	1.38	2.96	1.50	4.66	16.15	6.20	1.65	2.50	3.60
October	0.43	6.69	2.77	1.61	0.06	0.75	3.62	0.92	2.71	5.02	1.57	3.40	3.16	1.49	1.18
November	18.01	0.08	4.98	4.45	4.96	0.52	3.28	2.46	5.00	13.50	5.96	2.83	3.44	17.70	3.25
December	3.32	2.68	4.43	9.05	4.47	6.09	7.64	2.55	6.79	2.64	1.35	8.28	4.47	10.57	2.15
Total	66.89	62.90	58.95	56.38	35.13	51.29	46.86	47.21	48.87	72.99	61.61	62.38	43.98	77.69	53.04
Departure from Long Term Mean	16.87	13.38	8.93	6.36	-14.89	1.27	-3.16	-2.81	-1.15	22.97	10.44	11.21	-7.19	26.52	0.74

(Continued)

*Main portion of embankment completed in September 1949.

**River and highway closure portions of embankment completed in January 1954.

Table 2 (Concluded)

	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	Long Term Mean
January	2.27	5.09	2.73	6.14	1.83	8.98	4.46	2.13	3.38	8.39	10.47	12.42	4.14	5.06	5.78
February	2.48	4.30	6.15	9.49	4.86	2.17	5.61	3.86	6.80	1.00	3.56	4.29	6.96	5.39	5.06
March	6.21	11.60	7.14	0.71	1.62	5.28	4.45	6.36	5.32	5.05	17.47	3.76	10.18	8.53	6.81
April	3.68	13.08	2.67	4.15	6.00	8.24	6.03	8.79	4.10	5.23	7.08	6.24	3.31	1.27	4.27
May	3.55	3.39	1.09	3.86	6.02	6.62	1.82	1.07	3.51	3.35	8.71	10.03	7.19	4.62	3.70
June	3.08	2.27	1.13	0.96	1.09	0.88	1.29	6.38	5.25	6.68	1.08	11.89	3.86	5.86	3.49
July	9.32	2.38	2.87	3.02	6.71	3.37	2.58	4.14	5.64	5.43	8.91	5.57	7.15	4.36	3.94
August	3.06	2.52	4.17	1.68	4.30	2.93	3.81	5.80	4.07	3.91	1.43	3.61	5.21	1.05	3.44
September	1.76	2.88	4.08	3.42	1.73	5.70	4.71	1.96	2.35	4.19	2.03	3.15	3.85	3.01	2.55
October	0.00	1.32	0.89	2.85	2.15	2.34	1.78	8.52	0.49	3.06	2.10	2.03	7.08	3.89	2.25
November	3.92	5.62	0.91	1.36	3.03	5.04	5.25	3.02	1.96	6.88	8.77	4.25	5.84	1.97	4.63
December	5.41	4.52	1.75	4.43	9.71	6.95	9.98	4.97	6.31	5.92	6.91	8.19	3.46	4.25	5.25
Total	44.74	58.97	35.58	42.07	49.05	58.50	51.77	57.00	49.18	59.09	78.52	75.43	68.25	49.26	50.17
Departure from Long Term Mean	-7.56	6.67	-16.72	-10.23	-3.25	6.20	0.53	4.70	-3.12	6.79	26.22	21.22	14.69	-4.30	

Table 3

Cave-ins on Downstream Slope

Year	Total No. of Cave-ins	Location: Slope, Upper 1 on 2.5	Slope, Middle 1 on 3	Slope, Lower 1 on 3.5	Sta, Curtain Wall Grouting 120+00 to 124+00	Sta, Clay-Filled Trenching 124+00 to 132+00	Cave-ins in Experimental Test Section / Total No. of Cave-ins x 100 percent	Reference
1977	4	2	1	1	0	1	25	36
1976	5	Slope Not Available			Sta Not Available		Slope Not Available	37
1975	14	11	1	2	1	2	21	38
1974	12	3	6	3	0	3	25	38
1973	7	3	0	4	1	1	29	38
1972	16	9	4	3	1	2	19	37
1971	27	8	10	9	6	2	29	39
1970		No Data Available			No Data Available		No Data Available	No Data Available
1969	20	10	6	4	0	0	0	40
1968	52	23	11	16	6	2	15	41
1967		No Data Available			No Data Available		No Data Available	No Data Available
1966	23	10	6	7	0	0	0	42
1965	17	13	3	1	0	0	0	43
1964	7	0	5	2	0	0	0	44
1963	33	Slope Not Available			0	0	0	38
1962	68				0	0	0	38
1961	46				2	3	11	38
1960	108				Experimental Test Section Completed June 1960		Experimental Test Section Completed June 1960	13
1959	105							13
1958	116							13
1957	111							13
1956	155							13

Table 4

Chemical Composition of Contiguous United States River Waters (adapted from Reference 50)

River Basin	Ca Concentration		Mg Concentration		Na Concentration		k Concentration		Total Concentration Ca+Mg+Na+k
	ppm	meq/ℓ*	ppm	meq/ℓ*	ppm	meq/ℓ*	ppm	meq/ℓ*	meq/ℓ
North Atlantic Coast	19.00	0.948	5.33	0.438	8.73	0.385	1.68	0.043	1.814
South Atlantic Coast	5.23	0.261	1.43	0.118	4.80	0.209	1.10	0.028	0.616
Ohio Main Stem	34.88	1.741	8.09	0.665	17.25	0.750	2.85	0.073	3.229
Mississippi System-Ohio Drainage	28.17	1.406	8.30	0.683	16.30	0.709	2.47	0.063	2.861
Mississippi System-Northwestern Part	44.08	2.199	11.97	0.965	54.46	2.369	6.11	0.156	5.689
Mississippi System-Lower Part	27.53	1.374	8.41	0.692	5.40	0.235	1.76	0.045	2.346
Rio Grande	46.25	2.308	8.38	0.689	27.50	1.196	4.20	0.107	4.300
Pecos River	442.20	22.066	121.20	9.970	470.00	20.445	12.78	0.327	52.808
Colorado River	78.80	3.932	24.80	2.040	77.60	3.376	4.12	0.105	9.453
Sacramento River	13.45	0.671	4.85	0.399	9.10	0.396	1.10	0.028	1.494
AVERAGE (Excluding Pecos)		1.649		0.743		1.069		0.072	

* ppm (or mg/ℓ) x k = meq/ℓ; k = 0.04990 (Ca), 0.08226 (Mg), 0.04350 (Na), and 0.02557 (k).

Table 5

Rainfall Erosion Potential for Various Types of Slopes*

Soil Type	Natural	Recently Established (≥ 1 year) Slope			
		Nonvegetated		Minimum (6-12 in.) Topsoil and Vegetated	
		Cut	Fill	Cut	Fill
Nondispersive	Small	Small to Severe**	Small to Severe**	Small	Small
Dispersive	Not Applicable+	Severe	Severe	Insufficient Data Available++	Severe

* Relative degrees of erosion as follows:

	Nondispersive	Dispersive
Small	Raindrop impact erosion	Raindrop impact erosion
Moderate	Sheet and rill erosion	Rill erosion
Severe	Gully erosion	Tunnel erosion

** Depends on index properties of soil (Figure 27).[57]

\+ Dispersive soils are usually not present in the topsoil (A-horizon) because the clay fraction has generally been removed downward into the soil profile by the process of eluviation.[1,58-60]

++ SCS has recently constructed (Aug 1976) experimental test sections (including no treatment, vegetated, and topsoil plus vegetated) in a 1500-ft reach of an excavated channel with 1V on 3H side slopes in dispersive clay at Caney Creek near Wynne, Ark., which are currently being monitored.[61]

Table 6

Dispersion Test Results for SCS Piney Creek Sites 21 and 35 (adapted from References 58 and 62)*

Sample Location	Depth ft	Finer than 0.005 mm %	Unified Soil Classification	Crumb Test**	SCS Dispersion %+	Pinhole Classification++
			Site 21			
Upstream slope	0-1	25	CL	3	52	ND_2
Upstream slope	2-3	19	CL	2	63	I
Downstream slope	0-1	25	CL	2	28	ND_2
Downstream slope	2-3	24	CL	3	50	I_2
			Site 35			
Upstream slope	0-1	24	No Data Available	2	42	ND_2
Upstream slope	2-3	21	No Data Available	2	33	D_1
Downstream slope	0-1	21	No Data Available	3	50	D_2
Downstream slope	2-3	18	No Data Available	2	58	D_2

* Dams constructed in 1960, samples obtained and tests conducted from March to April 1975.

** 1-2 nondispersive and 3-4 dispersive.

\+ 0-35% nondispersive, 30-50% intermediate, and 50-100% dispersive.

++ D_1 — Dispersive — D_2 | I Intermediate | ND_2 — Nondispersive — ND_1

Table 7

Laboratory Test Program

Boring No.	No. of Samples	Depth ft	Location	Classification and Index Tests*	Soil/Water Chemistry Tests**	Dispersion Tests+	Pinhole Test Using Reservoir Water As Eroding Fluid
1-76	1	0.0-1.0	Surface	x	x	x	
1-73	18	5.0-82.9	Embankment	x	x	x	
2-76	1	0.0-1.0	Surface	x	x	x	
2-73	8	5.0-41.9	Embankment	x	x	x	
2-73	3	45.0-69.6	Foundation	x	x	x	x
3-76	1	0.0-1.0	Surface	x	x	x	
3-73	4	5.0-16.8	Embankment	x	x	x	
	1	22.0-22.7	Drainage Blanket	x	x	x	
	3	24.0-28.5	Drainage Blanket	x	x	x	x
	1	34.0-34.7	Foundation	x	x	x	x

* Atterberg limits, grain-size analysis, specific gravity, unit weight, and water content.

** Cation exchange capacity, cations in soil pore water (or reservoir water) extract, and pH.

\+ Crumb, SCS dispersion, and pinhole erosion.

Table 8

Comparison of Water Content When Sampled With Water Content After Storage in Warehouse

Boring No.	Sample No. Undisturbed	Sample No. Jar	Depth, ft Undisturbed	Depth, ft Jar	Location	Water Content, % Undisturbed*	Water Content, % Jar**	Increase (+) Decrease (-) With Storage %
1-73	1	1A	5.0-5.9	5.9-6.0	Embankment	18.3	12	+6.3
	7	7A	12.0-12.8	12.8-12.9		18.7	17	+1.7
	16	16A	21.0-21.6	21.6-21.8		17.2	18	-0.8
	19	19A	24.0-24.7	24.7-24.8		23.7	15	+8.7
	22	22A	27.0-27.7	27.7-27.9		14.0	19	-5.0
	25	25A	30.0-30.5	30.5-30.7		20.8	17	+3.8
	30	30A	35.0-35.7	35.7-35.9		17.4	20	-2.6
	34	34A	39.0-39.8	39.8-39.9		17.6	24	-6.4
	39	39A	44.0-44.6	44.6-44.8		17.8	13	+4.8
	44	44A	49.0-49.6	49.6-49.7		17.3	18	-0.7
	51	51A	56.0-56.6	56.6-56.7		18.5	17	+1.5

(Continued)

* Undisturbed soil samples taken from 30 January to 7 February 1973 by the Vicksburg District and tested from 6 April to 24 June 1976 at the WES.

** Jar samples taken from 30 January to 7 February 1973 and tested 7-14 February 1973 by the Vicksburg District.

Table 8 (Continued)

Boring No.	Sample No.		Depth, ft		Location	Water Content, %		Increase (+) Decrease (-) With Storage %
	Undisturbed	Jar	Undisturbed	Jar		Undisturbed*	Jar**	
1-73	54	54A	59.0-59.4	59.4-59.5	Embankment	17.8	18	-0.2
	62	62A	67.0-67.6	67.6-67.7		16.9	16	+0.9
	64	64A	69.0-69.6	69.6-69.8		20.9	15	+5.9
	66	66A	71.0-71.8	71.8-71.9		18.1	19	-0.9
	70	70A	75.0-75.9	75.9-76.0		16.6	17	-0.4
	72	72A	77.0-77.9	77.9-78.0		28.6	19	+9.6
	77	77A	82.0-82.9	82.9-83.0		23.1	20	+3.1
2-73	1	1A	5.0-5.9	5.9-6.0	Embankment	23.8	35	-11.2
	3	3A	7.0-7.9	7.9-8.0		24.0	21	+3.0
	9	9A	13.0-13.8	13.8-13.9		16.9	20	-3.1
	13	13A	17.0-17.5	17.5-17.6		15.3	18	-2.7
	15	15A	19.0-19.7	19.7-19.8		18.3	16	+2.3
	21	21A	25.0-25.6	25.6-25.7		18.5	17	+1.5
	25	25A	29.0-29.6	29.6-29.7		15.8	14	+1.8
	37	37A	41.0-41.9	41.9-42.0		17.8	16	+1.8
	41	41A	45.0-45.4	45.4-45.5	Foundation	19.8	18	+1.8

(Continued)

Table 8 (Concluded)

Boring No.	Sample No. Undisturbed	Sample No. Jar	Depth, ft Undisturbed	Depth, ft Jar	Location	Water Content, % Undisturbed*	Water Content, % Jar**	Increase (+) Decrease (-) With Storage %
2-73	45	45A	49.0-49.7	49.7-49.8	Foundation	30.6	+	--
	63	63A	69.0-69.6	69.6-69.7		25.3	24	+1.3
3-73	1	1A	5.0-5.8	5.8-5.9	Embankment	16.1	17	-0.9
	3	3A	7.0-7.5	7.5-7.6		17.7	16	+1.7
	8	8A	12.0-12.6	12.6-12.7		18.7	18	+0.7
	12	12A	16.0-16.8	16.8-16.9		20.4	16	+4.4
	18	18A	22.0-22.7	22.7-22.9	Drainage Blanket	20.1	18	+2.1
	20	20A	24.0-24.7	24.7-24.9		29.5	28	+1.5
	22	22A	26.0-26.5	26.5-26.8		33.3	+	--
	24	24A	28.0-28.5	28.5-28.6		27.1	+	--
	30	30A	34.0-34.7	34.7-34.8	Foundation	26.7	23	+3.7

+ No data obtained.

(Sheet 3 of 3)

Table 9

Changes in Soil Pore Water Chemistry for Waxed Leda Clay Sample* After Three Months Storage at Room Temperature (after Reference 79)

Condition	Water Content %	meq/1 Na	K	Mg	Ca	Na + K + Mg + Ca meq/1	Na %	SAR**
Initial	58.5	1.31	0.56	0.67	1.00	3.54	37	1.4
After Storage	50.4	0.87	0.74	1.32	1.55	4.48	19	0.7
Increase (+) Decrease (-) With Storage	-8.1	-0.44	+0.18	+0.65	+0.55	+0.94	-18	-0.7

* Soil sample taken from site of landslide north of Chelsea, Quebec. Leda clay (CL) is a post-glacial marine clay with natural water content between 55 and 65 percent, liquid limit from 37 to 41 percent and plastic limit from 21 to 25 percent.

** $SAR = \frac{Na}{\sqrt{0.5\ (Ca + Mg)}}$

Table 10

Classification Test Results

Boring No.	Sample No.	Depth ft	Location	Liquid Limit	Plastic Limit	Plasticity Index	Finer Than 0.005 mm %	Unified Soil Classification
1-76	1	0.0-1.0	Surface	41	19	22	33	lean clay (CL)
1-73	1	5.0-5.9	Embankment	32	14	18	27	sandy clay (CL)
	7	12.0-12.8		34	18	16	29	sandy clay (CL)
	16	21.0-21.6		37	18	19	27	sandy clay (CL)
	19	24.0-24.7		51	19	32	48	plastic clay (CH)
	22	27.0-27.7		39	17	22	37	sandy clay (CL)
	25	30.0-30.5		47	18	29	34	lean clay (CL)
	30	35.0-35.7		36	17	19	27	sandy clay (CL)
	34	39.0-39.8		36	19	17	24	sandy clay (CL)
	39	44.0-44.6		29	20	9	20	clay (CL)
	44	49.0-49.6		38	18	20	56	lean clay (CL)
	51	56.0-56.6		40	19	21	31	lean clay (CL)
	54	59.0-59.4		34	20	14	23	lean clay (CL)
	62	67.0-67.6		36	16	20	31	lean clay (CL)
	64	69.0-69.6		65	22	43	47	sandy clay (CH)
	66	71.0-71.8		30	16	14	28	sandy clay (CH)
	70	75.0-75.9		32	14	18	19	sandy clay (CL)
	72	77.0-77.9		48	26	22	26	clayey sand (SC)
	77	82.0-82.9		43	19	24	38	lean clay (CL)
2-76	1	0.0-1.0	Surface	39	22	17	32	lean clay (CL)
2-73	1	5.0-5.9	Embankment	42	19	23	39	lean clay (CL)

(Continued)

Table 10 (Concluded)

Boring No.	Sample No.	Depth ft	Location	Liquid Limit	Plastic Limit	Plasticity Index	Finer Than 0.005 mm %	Unified Soil Classification
2-73	3	7.0-7.9	Embankment	47	19	28	39	lean clay (CL)
	9	13.0-13.8		36	18	18	20	sandy clay (CL)
	13	17.0-17.5		30	17	13	23	sandy clay (CL)
	15	19.0-19.7		37	16	21	31	lean clay (CL)
	21	25.0-25.6		39	18	21	10	sandy, clayey sand (SC)
	25	29.0-29.6		35	16	19	29	sandy clay (CL)
	37	41.0-41.9		29	18	11	27	sandy clay (CL)
	41	45.0-45.4	Foundation	37	19	18	20	sandy clay (CL)
	45	49.0-49.7		51	18	33	28	clayey sand (SC)
	63	69.0-69.6		33	23	10	22	silty clay (CL)
3-76	1	0.0-1.0	Surface	38	21	17	30	lean clay (CL)
3-73	1	5.0-5.8	Embankment	46	17	29	33	sandy clay (CL)
	3	7.0-7.5		36	17	19	27	sandy clay (CL)
	8	12.0-12.6		32	20	12	21	silty clay (CL)
	12	16.0-16.8		39	20	19	25	sandy clay (CL)
	18	22.0-22.7	Drainage Blanket	--	--	NP	10	silty sand (SM)
	20	24.0-24.7		--	--	NP	17	silty sand (SM)
	22	26.0-26.5		--	--	NP	9	silty sand (SM)
	24	28.0-28.5		--	--	NP	8	silty sand (SM)
	30	34.0-34.7	Foundation	37	18	19	28	lean clay (CL)

Table 11

Crumb Test Results

Boring No.	Sample No.	Depth ft	Location	Crumb Test*
1-76	1	0.0-1.0	Surface	1
1-73	1	5.0-5.9	Embankment	1
	7	12.0-12.8		4
	16	21.0-21.6		1
	19	24.0-24.7		1
	22	27.0-27.7		1
	25	30.0-30.5		2
	30	35.0-35.7		1
	34	39.0-39.8		4
	39	44.0-44.6		4
	44	49.0-49.6		2
	51	56.0-56.6		1
	54	59.0-59.4		4
	62	67.0-67.6		4

(Continued)

(Sheet 1 of 3)

* 1-2 **nondispersive and 3-4 dispersive.**
Dispersive soils may show nondispersive reaction to Crumb Test.

Table 11 (Continued)

Boring No.	Sample No.	Depth ft	Location	Crumb Test*
1-73	64	69.0-69.6	Embankment	4
	66	71.0-71.8		4
	70	75.0-75.9		1
	72	77.0-77.9		1
	77	82.0-82.9		2
2-76	1	0.0-1.0	Surface	1
2-73	1	5.0-5.9	Embankment	1
	3	7.0-7.9		4
	9	13.0-13.8		2
	13	17.0-17.5		4
	15	19.0-19.7		4
	21	25.0-25.6		3
	25	29.0-29.6		3
	37	41.0-41.9		4
	41	45.0-45.4	Foundation	4
	45	49.0-49.7		1

(Continued)

Table 11 (Concluded)

Boring No.	Sample No.	Depth ft	Location	Crumb Test*
2-73	63	69.0-69.6	Foundation	2
3-76	1	0.0-1.0	Surface	1
3-73	1	5.0-5.8	Embankment	3
	3	7.0-7.5		4
	8	12.0-12.6		4
	12	16.0-16.8		4
	18	22.0-22.7	Drainage Blanket**	--
	20	24.0-24.7		--
	22	26.0-26.5		--
	24	28.0-28.5		--
	30	34.0-34.7	Foundation	4

** Drainage blanket contains insufficient colloids (fraction finer than 0.005 mm $\leq$ 12 percent and plasticity index $\leq$ 4) to support dispersive erosion.

(Sheet 3 of 3)

Table 12

SCS Dispersion Test Results

Boring No.	Sample No.	Depth ft	Location	Plasticity Index	Finer Than 0.005 mm %	SCS Dispersion* %
1-76	1	0.0-1.0	Surface	22	33	33
1-73	1	5.0-5.9	Embankment	18	27	46
	7	12.0-12.8		16	29	83
	16	21.0-21.6		19	27	91
	19	24.0-24.7		32	48	55
	22	27.0-27.7		22	37	37
	25	30.0-30.5		29	34	87
	30	35.0-35.7		19	27	83
	34	39.0-39.8		17	24	95
	39	44.0-44.6		9	20	90
	44	49.0-49.6		20	56	47
	51	56.0-56.6		21	31	82
	54	59.0-59.4		14	23	95

(Continued)

* 0-35% nondispersive, 35-50% intermediate, and 50-100% dispersive.

Table 12(Continued)

Boring No.	Sample No.	Depth ft	Location	Plasticity Index	Finer Than 0.005 mm %	SCS Dispersion*
1-73	62	67.0-67.6	Embankment	20	31	72
↓	64	69.0-69.6	↓	43	47	62
↓	66	71.0-71.8	↓	14	28	77
↓	70	75.0-75.9	↓	18	19	74
↓	72	77.0-77.9	↓	22	26	66
↓	77	82.0-82.9	↓	24	38	92
2-76	1	0.0-1.0	Surface	17	32	75
2-73	1	5.0-5.9	Embankment	23	39	54
↓	3	7.0-7.9	↓	28	39	88
↓	9	13.0-13.8	↓	18	20	83
↓	13	17.0-17.5	↓	13	23	94
↓	15	19.0-19.7	↓	21	31	67
↓	21	25.0-25.6	↓	21	10	88
↓	25	29.0-29.6	↓	19	29	78
↓	37	41.0-41.9	↓	11	27	82
↓	41	45.0-45.4	Foundation	18	20	75

(Continued)

Table 12 (Concluded)

Boring No.	Sample No.	Depth ft	Location	Plasticity Index	Finer Than 0.005 mm %	SCS Dispersion* %
2-73	45	49.0-49.7	Foundation	33	28	28
↓	63	69.0-69.6	↓	10	22	90
3-76	1	0.0-1.0	Surface	17	30	32
3-73	1	5.0-5.8	Embankment	29	33	69
	3	7.0-7.5		19	27	99
	8	12.0-12.6		12	21	24
	12	16.0-16.8	↓	19	25	79
	18	22.0-22.7	Drainage Blanket**	NP	10	--
	20	24.0-24.7		NP	17	--
	22	26.0-26.5		NP	9	--
	24	28.0-28.5	↓	NP	8	--
↓	30	34.0-34.7	Foundation	19	28	98

** Drainage blanket contains insufficient colloids (fraction finer than 0.005 mm $\leq$ 12 percent and plasticity index $\leq$ 4) to support dispersive erosion.

Table 13

Soil Chemistry Test Results

Cation Exchange Capacity

Boring No.	Sample No.	Depth ft	Location	CEC meq/100g	NH_4 OAC Exchangeable Base meq/100g Na	K	Mg	Ca	ESP*
1-76	1	0.0-1.0	Surface	16.6	0.4	0.2	3.0	8.3	2.4
1-73	1	5.0-5.9	Embankment	16.0	1.6	0.1	4.4	6.6	10.0
	7	12.0-12.8		9.8	1.2	0.1	2.2	2.6	12.2
	16	21.0-21.6		10.9	2.3	0.1	4.0	3.9	21.1
	19	24.0-24.7		23.1	2.9	0.1	6.7	7.1	12.6
	22	27.0-27.7		12.1	1.5	0.1	3.5	3.2	12.4
	25	30.0-30.5		18.8	2.2	0.2	6.5	6.8	11.7
	30	35.0-35.7		17.7	2.0	0.1	5.6	4.4	11.3
	34	39.0-39.8		10.0	0.7	0.1	3.2	2.5	7.0
	39	44.0-44.6		8.3	0.9	0.1	3.4	2.3	10.8
	44	49.0-49.6		22.1	2.7	0.2	6.5	8.4	12.2
	51	56.0-56.6		16.5	2.6	0.1	4.6	4.6	15.8

(Continued)

* $ESP = \frac{\overline{Na}}{CEC}(100)$

Table 13 (Continued)

Cation Exchange Capacity (Continued)

Boring No.	Sample No.	Depth ft	Location	CEC meq/100g	NH_4 OAC Exchangeable Base meq/100g Na	K	Mg	Ca	ESP*
1-73	54	59.0-59.4	Embankment	12.2	1.6	0.1	4.2	3.9	13.1
↓	62	67.0-67.6	↓	11.9	2.2	0.1	4.0	5.6	18.5
	64	69.0-69.6		19.0	1.9	0.2	5.9	10.0	10.0
	66	71.0-71.8		12.5	1.4	0.2	4.1	6.2	11.2
	70	75.0-75.9		10.0	1.1	0.1	3.3	3.9	11.0
	72	77.0-77.9		12.6	0.9	0.2	3.1	4.7	7.1
	77	82.0-82.9		14.4	2.0	0.2	5.6	3.8	13.9
2-76	1	0.1-1.0	Surface	16.4	0.4	0.5	2.0	8.6	2.4
2-73	1	5.0-5.9	Embankment	16.7	1.7	0.2	4.7	3.6	10.2
↓	3	7.0-7.9	↓	17.4	2.4	0.1	4.5	5.0	13.8
	9	13.0-13.8		15.7	0.9	0.1	3.0	3.0	5.7
	13	17.0-17.5		10.6	1.2	0.1	2.8	3.8	11.3
	15	19.0-19.7		11.7	1.2	0.1	4.7	2.8	10.3
	21	25.0-25.6		13.9	2.3	0.1	4.2	5.3	16.6
	25	29.0-29.6		16.7	2.6	0.1	6.5	9.6	15.6
	37	41.0-41.9		13.3	1.9	0.2	4.6	5.7	14.3

(Continued)

Table 13 (Continued)

Cation Exchange Capacity (Concluded)

Boring No.	Sample No.	Depth ft	Location	CEC meq/100g	NH_4 OAC Exchangeable Base meq/100g Na	K	Mg	Ca	ESP*
2-73	41	45.0-45.4	Foundation	11.1	1.1	0.2	2.5	3.2	9.9
	45	49.0-49.7		13.8	0.6	0.2	5.2	5.4	4.4
	63	69.0-69.6		12.6	0.6	0.1	3.3	2.1	4.8
3-76	1	0.0-1.0	Surface	16.9	0.4	0.3	2.1	4.7	2.4
3-73	1	5.0-5.8	Embankment	11.8	2.2	0.1	3.4	5.8	18.6
	3	7.0-7.5		14.6	1.4	0.1	4.7	3.4	9.6
	8	12.0-12.6		13.0	0.9	0.2	4.1	4.4	6.9
	12	16.0-16.8		14.6	1.4	0.1	4.7	3.4	9.6
	18	22.0-22.7	Drainage Blanket	5.6	0.5	0.1	1.5	3.6	8.9
	20	24.0-24.7		16.6	0.7	0.2	4.0	4.7	4.2
	22	26.0-26.5		10.4	0.4	0.2	1.4	1.9	3.9
	24	28.0-28.5		14.7	0.5	0.3	2.7	3.4	3.4
	30	34.0-34.7	Foundation	12.7	2.0	0.1	3.4	3.4	15.8

(Continued)

Table 13 (Continued)

Soil Pore Water Extract

Boring No.	Sample No.	Depth ft	Location	pH	meq/l Na	meq/l K	meq/l Mg	meq/l Ca	Na + K + Mg + Ca meq/l	Na %	SAR*
1-76	1	0.0-1.0	Surface	6.0	0.2	0.0	0.3	0.6	1.1	18	0.3
1-73	1	5.0-5.9	Embankment	6.2	6.8	0.0	1.1	1.7	9.6	71	5.8
	7	12.0-12.8		5.0	2.8	0.0	0.0	0.1	2.9	97	12.5
	16	21.0-21.6		5.6	10.3	0.0	1.2	0.8	12.3	84	10.3
	19	24.0-24.7		5.3	9.9	0.1	2.5	2.6	15.1	66	6.2
	22	27.0-27.7		5.2	3.2	0.0	0.2	0.1	3.5	91	8.3
	25	30.0-30.5		6.0	4.4	0.0	0.3	0.2	4.9	90	8.8
	30	35.0-35.7		5.3	6.5	0.0	0.9	0.6	8.0	81	7.5
	34	39.0-39.8		5.4	0.9	0.0	0.0	0.0	0.9	100	--
	39	44.0-44.6		5.7	0.8	0.0	0.0	0.0	0.8	100	--
	44	49.0-49.6		5.7	4.4	0.0	0.2	0.5	5.1	86	7.4
	51	56.0-56.6		5.3	5.5	0.0	0.2	0.1	5.8	95	14.2
	54	59.0-59.4		6.4	3.4	0.0	0.2	0.2	3.8	89	7.6
	62	67.0-67.6		7.3	5.1	0.0	0.3	0.4	5.8	89	8.6
	64	69.0-69.6		6.3	3.1	0.0	0.2	0.2	3.5	89	6.9
	66	71.0-71.8		6.1	2.5	0.0	0.2	0.5	3.2	78	4.2

(Continued)

$$* \quad SAR = \frac{Na}{\sqrt{0.5\,(Ca + Mg)}}$$

$$ESP = \frac{100\,(-0.0126 + 0.01475\ SAR)}{1 + (-0.0126 + 0.01475\ SAR)}$$

Table 13 (Continued)

Boring No.	Sample No.	Depth ft	Location	pH	Soil Pore Water Extract (Continued) meq/l Na	K	Mg	Ca	Na + K + Mg + Ca meq/l	Na %	SAR*
1-73	70	75.0-75.9	Embankment	6.0	2.6	0.1	0.2	0.2	3.1	84	5.8
	72	77.0-77.9		4.9	2.9	0.0	0.5	0.6	4.0	73	3.9
	77	82.0-82.9		5.5	2.5	0.0	0.2	0.2	2.9	86	5.6
2-76	1	0.0-1.0	Surface	6.3	0.1	0.1	0.2	0.1	0.5	20	0.3
2-73	1	5.0-5.9	Embankment	5.5	4.7	0.0	0.3	0.3	5.3	89	8.6
	3	7.0-7.9		5.6	7.9	0.0	0.6	0.5	9.0	88	10.7
	9	13.0-13.8		5.2	2.5	0.0	0.2	0.2	2.9	86	5.6
	13	17.0-17.5		6.2	2.0	0.0	0.0	0.1	2.1	95	8.9
	15	19.0-19.7		6.0	4.0	0.0	0.1	0.2	4.3	93	10.3
	21	25.0-25.6		6.0	5.4	0.0	0.3	0.2	5.9	92	10.8
	25	29.0-29.6		8.0	9.6	0.0	0.1	0.7	10.4	92	15.2
	37	41.0-41.9		6.3	5.4	0.2	0.3	0.5	6.4	84	8.5
	41	45.0-45.4	Foundation	5.5	2.4	0.0	0.0	0.1	2.5	96	10.7
	45	49.0-49.7		4.8	2.4	0.0	5.4	4.2	12.0	20	0.8
	63	69.0-69.6		4.8	1.0	0.0	0.2	0.2	1.4	71	2.2
3-76	1	0.0-1.0	Surface	5.2	0.5	0.1	0.9	0.2	1.7	29	0.7
3-73	1	5.0-5.8	Embankment	7.1	7.8	0.0	0.5	0.5	8.8	89	11.0

(Continued)

Table 13 (Concluded)

(Soil Pore WAter Extract (Concluded)

Boring No.	Sample No.	Depth ft	Location	pH	meq/1 Na	K	Mg	Ca	Na + K + Mg + Ca meq/1	Na %	SAR*
3-73	3	7.0-7.5	Embankment	5.5	5.1	0.0	0.2	0.2	5.5	93	11.4
	8	12.0-12.6		5.6	1.6	0.0	0.1	0.2	1.9	84	4.1
	12	16.0-16.8		5.2	1.8	0.0	0.1	0.1	2.0	90	5.7
	18	22.0-22.7	Drainage Blanket	7.5	1.3	0.1	0.3	0.4	2.1	62	2.2
	20	24.0-24.7		4.5	1.1	0.1	1.3	1.3	3.8	29	1.0
	22	26.0-26.5		4.4	0.8	0.0	0.4	0.4	1.6	50	1.3
	24	28.0-28.5		4.1	1.2	0.3	4.0	3.7	9.2	13	0.6
	30	34.0-34.7	Foundation	5.2	4.3	0.0	0.2	0.2	4.7	91	9.6
Reservoir Water	1	0.0-1.0	Surface	6.1	0.2	0.1	0.1	0.2	0.6	33	0.5
	2			6.3	0.2	0.1	0.1	0.2	0.6	33	0.5
	3			6.1	0.2	0.1	0.1	0.2	0.6	33	0.5
	4			6.9	0.2	0.1	0.1	0.2	0.6	33	0.5

Table 14

Index Properties of Samples Tested in WES Pinhole Erosion Apparatus

Boring No.	Sample No.	Depth ft	Location	Wet Unit Weight pcf	Water Content %	Dry Unit Weight pcf	Specific Gravity of Solids	Void Ratio	Degree of Saturation %
1-76	1	0.0-1.0	Surface	119.0	16.7	102.0	2.68	0.640	70
1-73	1	5.0-5.9	Embankment	116.7	18.3	98.6	2.68	0.696	70
	7	12.0-12.8		131.8	18.7	111.0	2.67	0.501	100
	16	21.0-21.6		128.8	17.2	109.9	2.69	0.527	88
	19	24.0-24.7		119.7	23.7	96.8	2.70	0.741	86
	22	27.0-27.7		125.4	14.0	110.0	2.69	0.526	72
	25	30.0-30.5		127.1	20.8	105.2	2.69	0.596	94
	30	35.0-35.7		124.1	17.4	105.7	2.70	0.594	79
	34	39.0-39.8		129.0	17.6	109.7	2.67	0.519	91
	39	44.0-44.6		119.2	17.8	101.2	2.70	0.665	72
	44	49.0-49.6		130.2	17.3	111.0	2.69	0.512	91
	51	56.0-56.6		127.1	18.5	107.3	2.68	0.559	89
	54	59.0-59.4		117.6	17.8	99.8	2.69	0.682	70
	62	67.0-67.6		128.5	16.9	109.9	2.73	0.550	84
	64	69.0-69.6		121.0	20.9	100.1	2.70	0.683	83

(Continued)

Table 14 (Continued)

Boring No.	Sample No.	Depth ft	Location	Wet Unit Weight pcf	Water Content %	Dry Unit Weight pcf	Specific Gravity of Solids	Void Ratio	Degree of Saturation %
1-73	66	71.0-71.8	Embankment	125.7	18.1	106.4	2.73	0.601	82
	70	75.0-75.9		130.4	16.6	111.8	2.72	0.518	87
	72	77.0-77.9		110.2	28.6	85.7	2.65	0.930	82
	77	82.0-82.9		127.4	23.1	103.5	2.72	0.640	98
2-76	1	0.0-1.0	Surface	106.0	27.9	82.9	2.66	1.002	74
2-73	1	5.0-5.9	Embankment	127.9	23.8	103.3	2.68	0.619	100
	3	7.0-7.9		120.8	24.0	97.4	2.66	0.704	91
	9	13.0-13.8		129.4	16.9	110.7	2.67	0.505	89
	13	17.0-17.5		130.7	15.3	113.4	2.69	0.480	86
	15	19.0-19.7		129.4	18.3	109.4	2.69	0.534	92
	21	25.0-25.6		130.7	18.5	110.3	2.69	0.522	95
	25	29.0-29.6		128.2	15.8	110.7	2.69	0.516	82
	37	41.0-41.9		128.9	17.8	109.4	2.68	0.529	90
	41	45.0-45.4	Foundation	119.2	19.8	99.5	2.66	0.668	79
	45	49.0-49.7		102.5	30.6	78.5	2.68	1.130	73
	63	69.0-69.6		124.3	25.3	99.2	2.68	0.686	99

(Continued)

Table 14 (Concluded)

Boring No.	Sample No.	Depth ft	Location	Wet Unit Weight pcf	Water Content %	Dry Unit Weight pcf	Specific Gravity of Solids	Void Ratio	Degree of Saturation %
3-76	1	0.0-1.0	Surface	114.7	18.2	97.0	2.67	0.718	68
3-73	1	5.0-5.8	Embankment	128.2	16.1	110.4	2.69	0.520	83
	3	7.0-7.5		131.7	17.7	111.9	2.69	0.500	95
	8	12.0-12.6		129.7	18.7	109.3	2.69	0.536	94
	12	16.0-16.8		120.8	20.4	100.3	2.70	0.680	81
	18	22.0-22.7	Drainage Blanket	110.3	20.1	91.8	2.68	0.822	66
	20	24.0-24.7		106.4	29.5	82.2	2.65	1.012	77
	22	26.0-26.5		106.2	33.3	79.7	2.62	1.051	83
	24	28.0-28.5		109.5	27.1	86.2	2.61	0.889	80
	30	34.0-34.7	Foundation	122.4	26.7	96.6	2.68	0.731	98

(Sheet 3 of 3)

Table 15

Pinhole Erosion Test Results Using Distilled Water as Eroding Fluid

Boring No.	Sample No.	Depth ft	Location	Flow Through Specimen After 10 min, ml/sec Initial Hydraulic Head, in. 2	7	15	Pinhole Classification*
1-76	1	0.0-1.0	Surface	0.37	1.30	1.61	ND_1
1-73	1	5.0-5.9	Embankment	0.52	1.20	1.97	ND_1
	7	12.0-12.8		0.29	1.34	2.45	ND_3
	16	21.0-21.6		0.95	**	**	ND_4
	19	24.0-24.7		1.07	5.05	10.55	ND_4
	22	27.0-27.7		0.23	0.78	2.79	ND_2
	25	30.0-30.5		4.38	--	--	D_1
	30	35.0-35.7		1.03	4.59	7.58	ND_4
	34	39.0-39.8		0.62	1.66	2.67	ND_2
	39	44.0-44.6		0.60	5.40	+	ND_3
	44	49.0-49.6		0.74	16.39	--	ND_4
	51	56.0-56.6		1.95	**	**	D_2
	54	59.0-59.4		0.89	10.2	--	ND_4

(Continued)

(Sheet 1 of 3)

* D_1 — Dispersive — D_2 | ND_4 — Intermediate — ND_3 | ND_2 — Nondispersive — ND_1

** No data obtained.

+ **Exceeded hydraulic capacity of pinhole erosion apparatus (≈ 17.5 ml/sec).**

Table 15 (Continued)

Boring No.	Sample No.	Depth ft	Location	Flow Through Specimen After 10 min, ml/sec Initial Hydraulic Head, in. 2	7	15	Pinhole Classification*
1-73	62	67.0-67.6	Embankment	1.68	+	--	D_2
	64	69.0-69.6		0.57	3.79	**	ND_3
	66	71.0-71.8		0.99	**	**	ND_4
	70	75.0-75.9		2.36	--	--	D_2
	72	77.0-77.9		0.30	0.61	0.95	ND_1
	77	82.0-82.9		2.08	--	--	D_2
2-76	1	0.0-1.0	Surface	0.27	1.37	2.98	ND_2
2-73	1	5.0-5.9	Embankment	0.53	2.03	3.88	ND_3
	3	7.0-7.9		0.28	13.30	--	ND_3
	9	13.0-13.8		0.74	1.74	2.73	ND_3
	13	17.0-17.5		6.41	--	--	D_1
	15	19.0-19.7		0.78	**	**	ND_4
	21	25.0-25.6		9.34	--	--	D_1
	25	29.0-29.6		1.51	--	--	D_2
	37	41.0-41.9		1.66	--	--	D_2

(Continued)

(Sheet 2 of 3)

** No data obtained.

\+ Exceeded hydraulic capacity of pinhole erosion apparatus (≈ 17.5 ml/sec).

Table 15 (Concluded)

Boring No.	Sample No.	Depth ft	Location	Flow Through Specimen After 10 min, ml/sec Initial Hydraulic Head, in. 2	7	15	Pinhole Classification*
3-76	1	0.0-1.0	Surface	0.33	1.18	2.69	ND_1
3-73	1	5.0-5.8	Embankment	1.00	+	--	ND_4
	3	7.0-7.5		1.01	+	--	ND_4
	8	12.0-12.6		0.45	1.81	5.26	ND_3
	12	16.0-16.8		0.57	1.76	2.38	ND_3
	18	22.0-22.7	Drainage Blanket	0.55	**	**	**

(Sheet 3 of 3)

** No data obtained.

+ Exceeded hydraulic capacity of pinhole erosion apparatus (≈ 17.5 ml/sec).

Table 16

Pinhole Erosion Test Results Using Reservoir Water as Eroding Fluid

Boring No.	Sample No.	Depth ft	Location	Flow Through Specimen After 10 min, ml/sec, Initial Hydraulic Head, in. 2	7	15	Pinhole Classification*
2-73	41	45.0-45.4	Foundation	0.41	0.95	1.52	ND_1
↓	45	49.0-49.7	↓	0.23	0.58	1.05	ND_1
↓	63	69.0-69.6	↓	0.50	1.33	3.01	ND_2
3-73	20	24.0-24.7	Drainage Blanket	0.48	1.05	1.63	ND_1
↓	22	26.0-26.5	↓	0.60	1.32	2.10	ND_1
↓	24	28.0-28.5	↓	0.38	0.93	1.58	ND_1
↓	30	34.0-34.7	Foundation	0.34	1.25	2.30	ND_1

* D_1 — Dispersive — D_2 | ND_4 — Intermediate — ND_3 | ND_2 — Nondispersive — ND_1

When using the pinhole erosion test to identify dispersive clays distilled water (total dissolved salts ≈ 0.0 meq/l) is used as eroding fluid. The low total dissolved salts (0.6 meq/l) of the reservoir water, used as eroding fluid for these tests, is not believed to have significantly influenced the pinhole classification.

Table 17

Summary of Dispersion Tests

Boring No.	Sample No.	Depth ft	Location	Crumb Test*	SCS Dispersion Test	Soil Pore Water Chemistry Correlation (From Figure 29)	Pinhole Erosion Test	Consensus of Tests (Excluding Crumb)
1-76	1	0.0-1.0	Surface	ND	ND	ND	ND	ND
1-73	1	5.0-5.9	Embankment	ND	I	D	ND	I
	7	12.0-12.8		D	D	D	I	D
	16	21.0-21.6		ND	D	D	I	D
	19	24.0-24.7		ND	D	D	I	D
	22	27.0-27.7		ND	I	D	ND	I
	25	30.0-30.5		ND	D	D	D	D
	30	35.0-35.7		ND	D	D	I	D

(Continued)

Note: ND = nondispersive, I = intermediate, and D = dispersive.

* Dispersive soils may show nondispersive reaction to crumb test.[69]

Table 17 (Continued)

Boring No.	Sample No.	Depth ft	Location	Crumb Test*	SCS Dispersion Test	Soil Pore Water Chemistry Correlation (From Figure 29)	Pinhole Erosion Test	Consensus of Tests (Excluding Crumb)
1-73	34	39.0-39.8	Embankment	D	D	D	ND	D
↓	39	44.0-44.6	↓	D	D	D	I	D
↓	44	49.0-49.6	↓	ND	I	D	I	I
↓	51	56.0-56.6	↓	ND	D	D	D	D
↓	54	59.0-59.4	↓	D	D	D	I	D
↓	62	67.0-67.6	↓	D	D	D	D	D
↓	64	69.0-69.6	↓	D	D	D	I	D
↓	66	71.0-71.8	↓	D	D	D	I	D
↓	70	75.0-75.9	↓	ND	D	D	D	D
↓	72	77.0-77.9	↓	ND	D	D	ND	D
↓	77	82.0-82.9	↓	ND	D	D	D	D
2-76	1	0.0-1.0	Surface	ND	D	ND	ND	ND
2-73	1	5.0-5.9	Embankment	ND	D	D	I	D
↓	3	7.0-7.9	↓	D	D	D	I	D
↓	9	13.0-13.8	↓	ND	D	D	I	D
↓	13	17.0-17.5	↓	D	D	D	D	D

(Continued)

Table 17 (Concluded)

Boring No.	Sample No.	Depth ft	Location	Crumb Test*	SCS Dispersion Test	Soil Pore Water Chemistry Correlation (From Figure 29)	Pinhole Erosion Test	Consensus of Tests (Excluding Crumb)
2-73	15	19.0-19.7	Embankment	D	D	D	I	D
	21	25.0-25.6		D	D	D	D	D
	25	29.0-29.6		D	D	D	D	D
	37	41.0-41.9		D	D	D	D	D
	41	45.0-45.4	Foundation	D	D	D	ND+	D
	45	49.0-49.7		ND	ND	ND	ND+	ND
	63	69.0-69.6		ND	D	D	ND+	D
3-76	1	0.0-1.0	Surface	ND	ND	ND	ND	ND
3-73	1	5.0-5.8	Embankment	D	D	D	I	D
	3	7.0-7.5		D	D	D	I	D
	8	12.0-12.6		D	ND	D	I	I
	12	16.0-16.8		D	D	D	I	D
	18	22.0-22.7	Drainage Blanket**	--	--	--	--	--
	20	24.0-24.7		--	--	--	--	--
	22	26.0-26.5		--	--	--	--	--
	24	28.0-28.5		--	--	--	--	--
	30	34.0-34.7	Foundation	D	D	D	ND+	D

(Sheet 3 of 3)

** Drainage blanket contains insufficient colloids (fraction finer than 0.005 mm ≤ 12 percent and plasticity index ≤ 4) to support dispersive erosion.

+ Reservoir water (total dissolved salts = 0.6 meq/1) used as eroding fluid.

Table 18

Mineralogical Composition of Soil (from Reference 81)

Sample No.	Station	Depth ft	Location	Constituents, % Quartz	Montmorillonite	Kaolinite	Illite
1	54+00	0-1	Sample from area of no erosion	40	40	10	10
2	128+00	0-1	Sample from wall of vertical erosion tunnel	30	40	15	15

Table 19

Limnological Data from Grenada Lake 0.25 Miles from Dam Opposite Sta 154+00 (from Reference 83)

Date	Climate	Air Temperature °C	Depth m	Conductivity micro mhos/cm	Total Ionic Concentration * meq/l
12-19-74	Partly Cloudy, Calm	6.5	0	80	0.80
			1	80	0.80
			2	82	0.82
			3	80	0.80
			4	80	0.80
			5	80	0.80
			6	80	0.80
			7	80	0.80
			8	82	0.82
3-22-75	Overcast, Wind 15-20 mph	17.0	0	55	0.55
			1	58	0.58
			2	58	0.58
			3	58	0.58
			4	58	0.58
			5	58	0.58
			6	58	0.58
			7	58	0.58
			8	58	0.58
			9	58	0.58
			10	58	0.58
			11	58	0.58
			12	60	0.60
			13	60	0.60
			14	58	0.58
			15	58	0.58
6-5-75	Cloudy, Wind 20-25 mph	22.5	0	45	0.45
			1	40	0.40
			2	50	0.50
			3	48	0.48
			4	50	0.50
			5	48	0.48
			6	48	0.48
			7	50	0.50
			8	50	0.50
			9	50	0.50
			10	50	0.50
			11	48	0.48
			12	48	0.48
			13	48	0.48
			14	50	0.50

(Continued)

* Concentration (meq/l) = 0.01 conductivity (micro mhos/cm).

Table 19 (Continued)

Date	Climate	Air Temperature °C	Depth m	Conductivity micro mhos/cm	Total Ionic Concentration * meq/l
7-7-76	Partly Cloudy, Calm	30	0	75	0.75
			1	75	0.75
			2	75	0.75
			3	75	0.75
			4	75	0.75
			5	70	0.70
			6	75	0.75
			7	75	0.75
			8	75	0.75
			9	70	0.70
			10	75	0.75
			11	75	0.75
10-12-76	Clear, Calm	22.0	0	80	0.80
			1	80	0.80
			2	83	0.83
			3	85	0.85
			4	90	0.90
			5	95	0.95
			6	98	0.98
			7	100	1.00
1-12-77	Partly Cloudy, Calm, Ice	3	0	75	0.75
			1	80	0.80
			2	85	0.85
			3	90	0.90
			4	90	0.90
			5	90	0.90
			6	95	0.95
4-15-77	Partly Cloudy, Calm	28	0	70	0.70
			1	70	0.70
			2	70	0.70
			3	70	0.70
			4	70	0.70
			5	70	0.70
			6	70	0.70
			7	70	0.70
			8	70	0.70
			9	75	0.75
			10	75	0.75
7-20-77	Partly Cloudy, Calm	32	0	80	0.80
			1	80	0.80
			2	80	0.80
			3	80	0.80
			4	80	0.80

(Continued)

Table 19 (Concluded)

Date	Climate	Air Temperature °C	Depth m	Conductivity micro mhos/cm	Total Ionic Concentration * meq/1
7-20-77 (continued)	Partly Cloudy, Calm	32	5	80	0.80
			6	75	0.75
			7	85	0.85
			8	80	0.80
			9	85	0.85
			10	85	0.85
10-19-77	Partly Cloudy, Calm	24	0	85	0.85
			1	85	0.85
			2	90	0.90
			3	90	0.90
			4	80	0.80
			5	80	0.80
			6	90	0.90
			7	100	1.00
			8	100	1.00
2-3-78	Partly Cloudy, Calm	5	0	70	0.70
			1	75	0.75
			2	75	0.75
			3	75	0.75
			4	85	0.85
			5	80	0.80
			6	90	0.90
			7	95	0.95

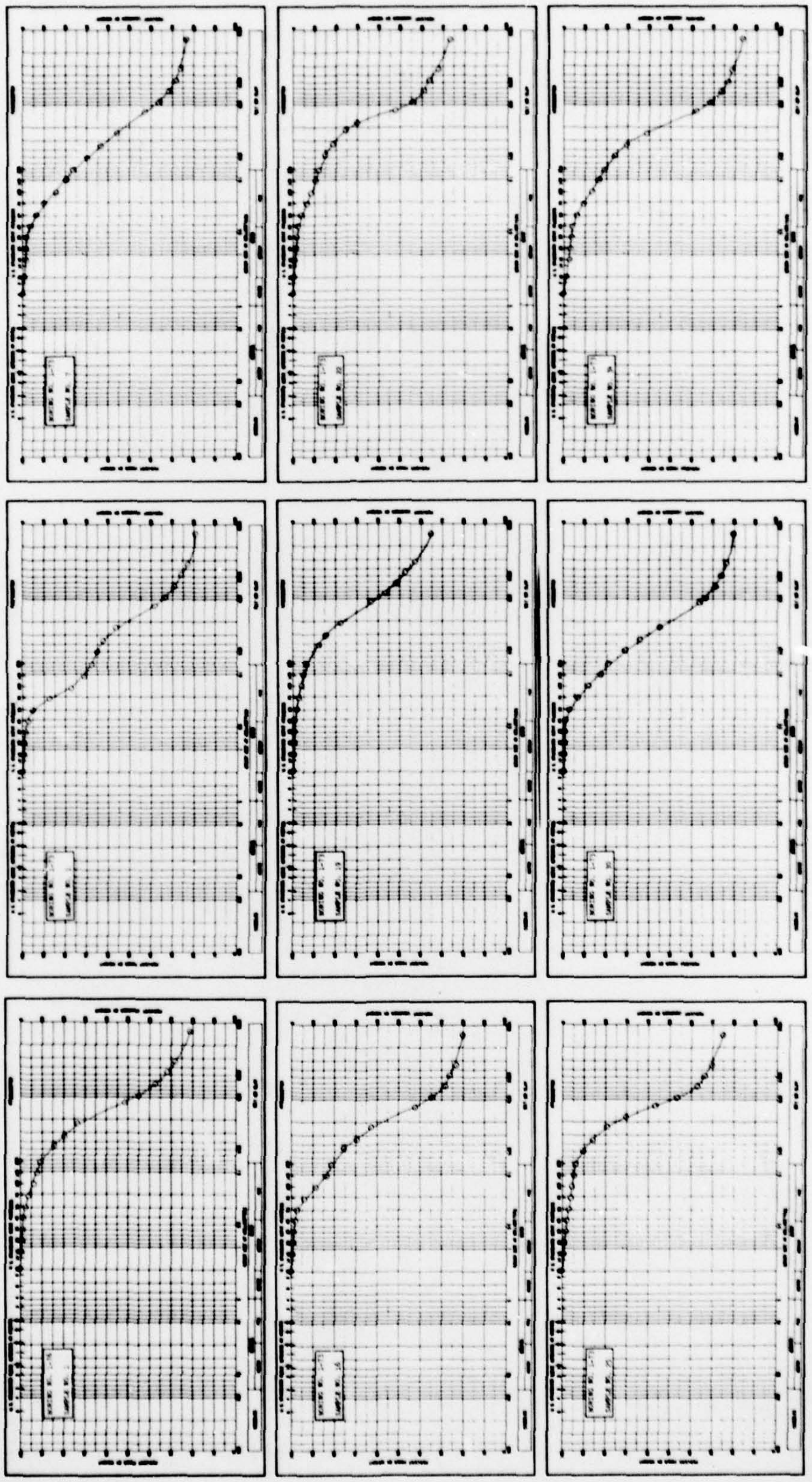
Table 20

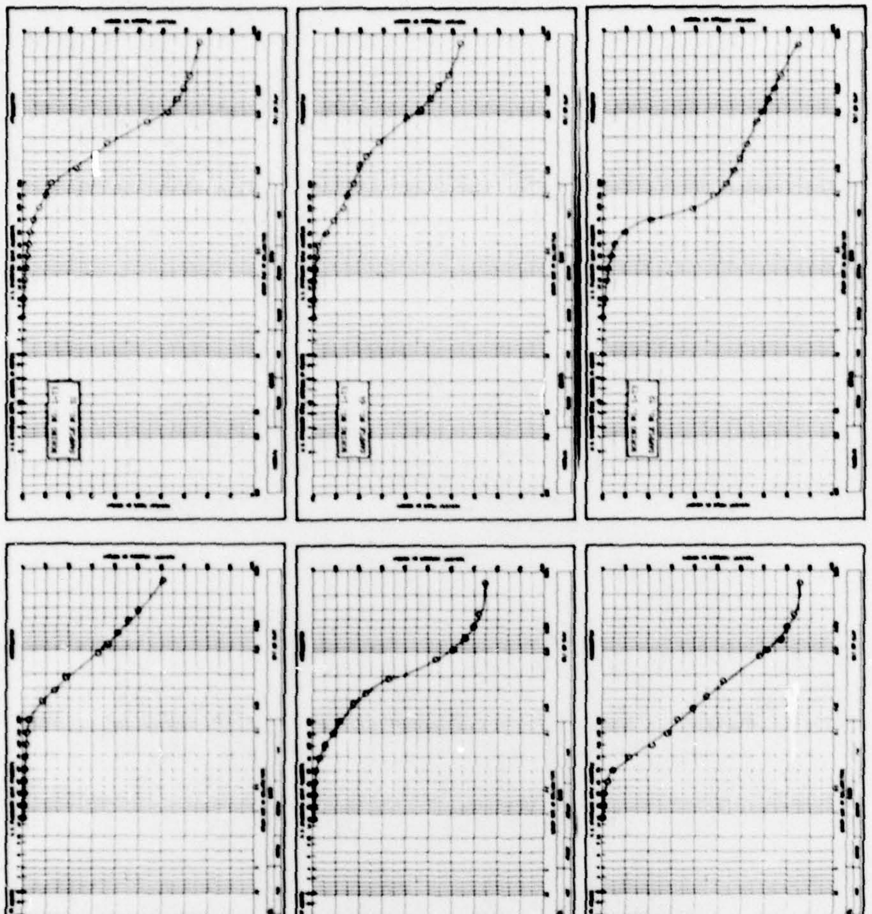
Summary of Parameters for Prediction of Susceptibility of Embankment and Foundation Soils to Piping Failure

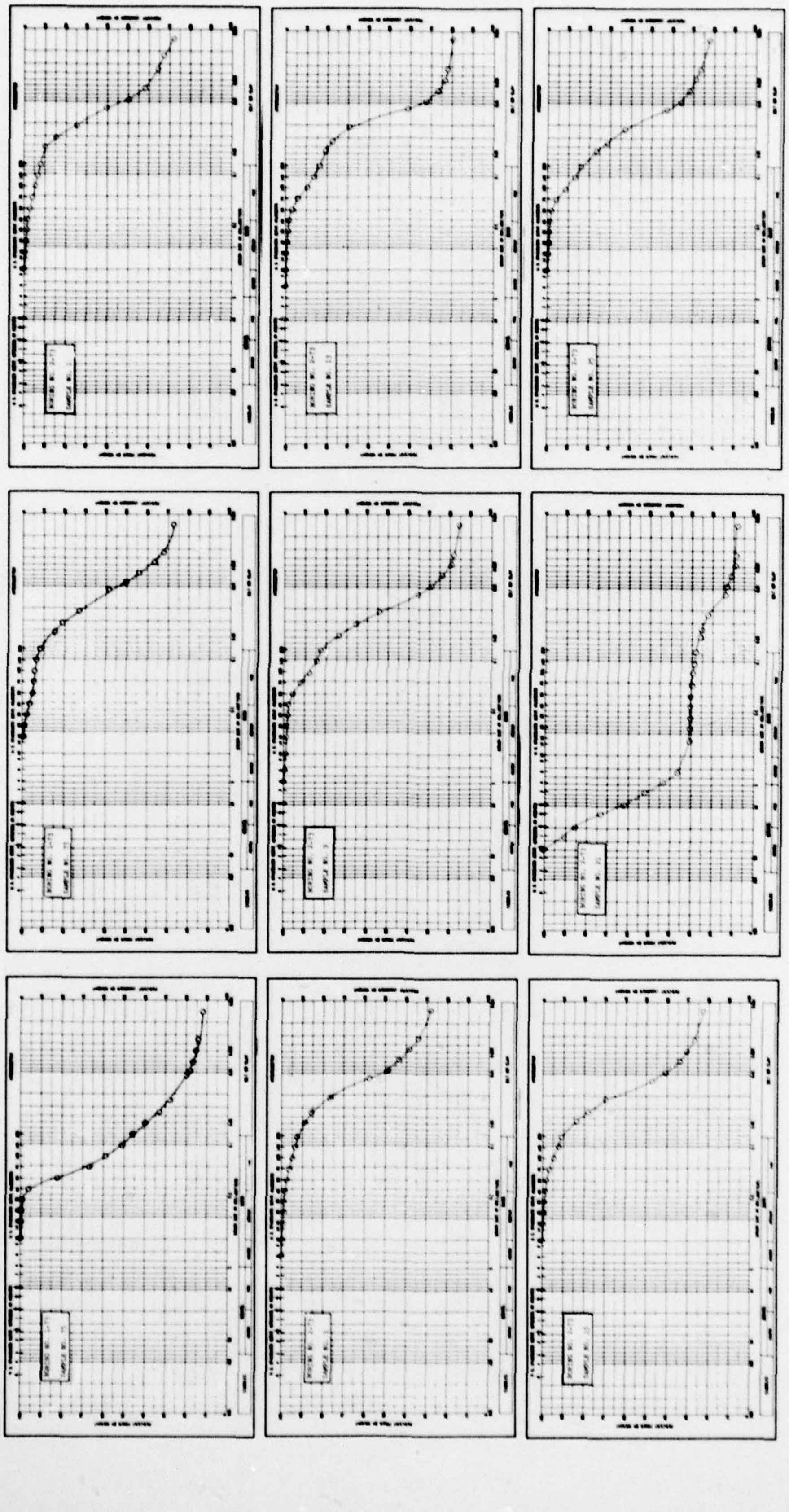
		Soil		Reservoir (Eroding) Water
Boring No.	Location	ESP*	Predominate Clay Mineral	Total Ionic Concentration meq/l
1-76	Embankment	12	Montmorillonite	0.6
2-76	Embankment	17		
3-76	Embankment	16		
2-76	Foundation	6		
3-76	Foundation	16		

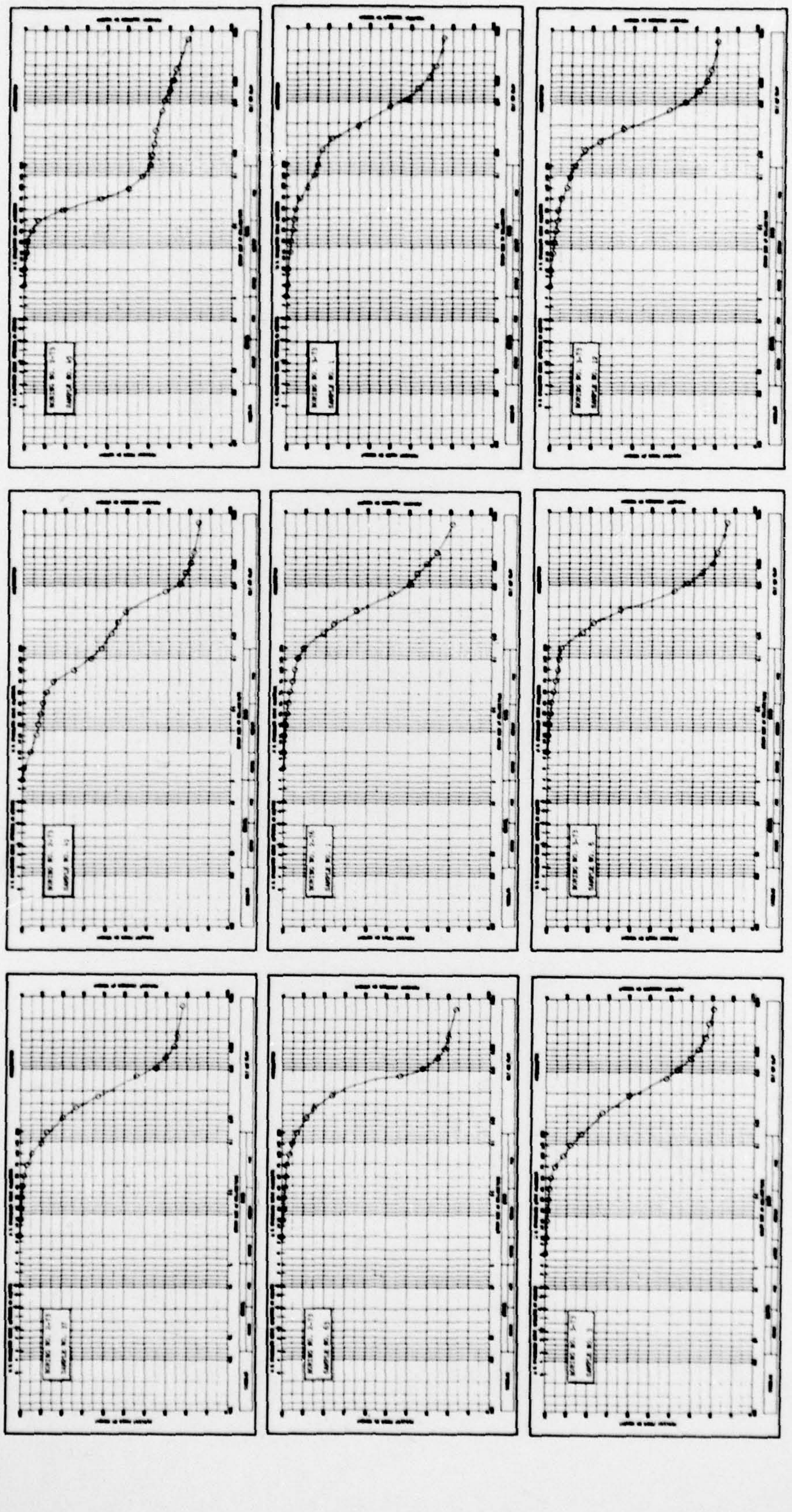
* $ESP = \frac{\overline{Na}}{CEC} (100)$

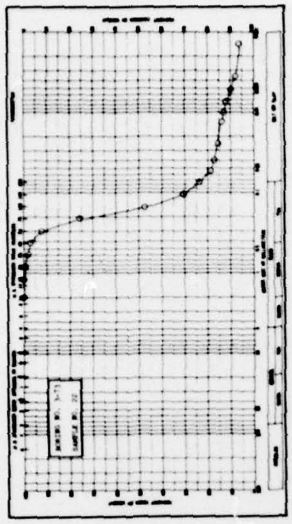
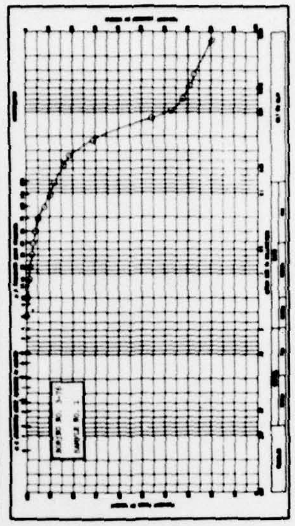
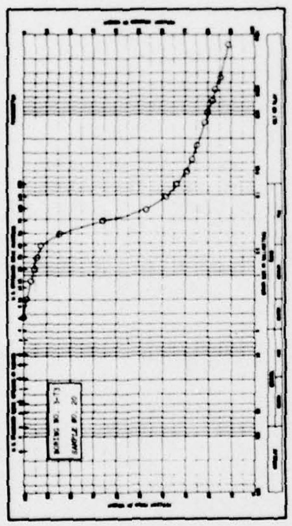
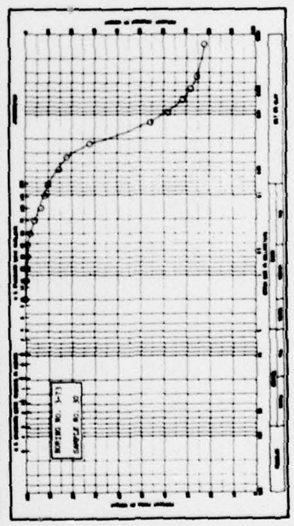
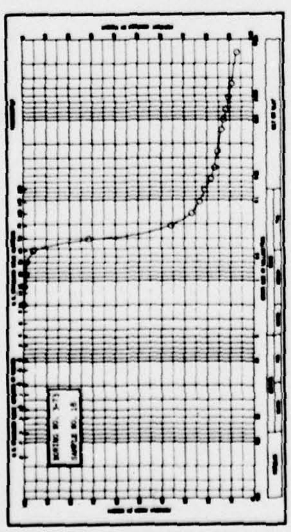
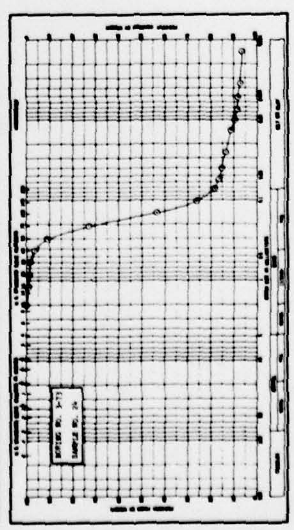
APPENDIX A: GRADATION CURVES









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.

Perry, Edward Belk
Susceptibility of dispersive clay at Grenada Dam, Mississippi, to piping and rainfall erosion / by Edward B. Perry. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1979.
111, [47] p. : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; GL-79-14)
Prepared for U. S. Army Engineer Division, Lower Mississippi Valley, Vicksburg, Miss.
References: p. 103-111.

1. Grenada Dam. 2. Dispersive clays. 3. Piping (Erosion). 4. Rainfall erosion. I. United States. Army. Corps of Engineers. Lower Mississippi Valley Division. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; GL-79-14.
TA7.W34 no.GL-79-14