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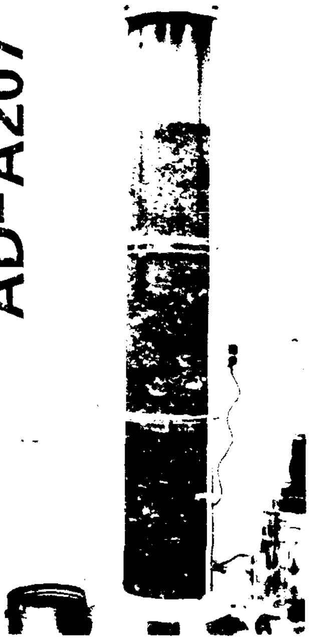

GEOTECHNICAL LABORATORY

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STUDY OF EMBANKMENT PERFORMANCE DURING OVERTOPPING AND THROUGHFLOW

Report 3

MODEL-PROTOTYPE COMPARISON STUDIES

by

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It was attempted to model two fairly well documented embankment overtoppings, the Clarence Cannon and Bloomington Lake overtopping failures. Additionally, the results of modeling in two dimensions versus modeling in three dimensions are investigated and discussed, as well as a study of the effects of nonhomogeneous embankment cross sections on erosion. Centrifuge modeling of Clarence Cannon and Bloomington Lake overtopping events consisted of four tests each at 1/80th full scale. Materials used were fairly noncohesive soils and fairly cohesive soils in an attempt to bracket prototype behavior rather than to duplicate a precise event. During testing, the centrifuge was repeatedly started and stopped in order to take measurements of erosion resulting from overtopping.

Agreement between scale models and documented prototype behavior was not especially good, but the study did confirm that noncohesive soils are considerably more erodible than cohesive materials in terms of the ability to withstand extended periods of overtopping

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without destruction.

The attempt to evaluate appropriateness of two- versus three-dimensional embankment modeling was inconclusive because of the interaction of the two-dimensional models with the boundaries. The study did confirm that zoned embankment structures, once compromised, are much more erodible and susceptible to destruction than homogeneous structures.

PREFACE

This study was conducted by the Department of Civil, Environmental, and Architectural Engineering at the University of Colorado, Boulder, under Department of Army Contract No. DACW39-83-C-0011, "Study of Embankment Performance During Overtopping and Throughflow." The contract was carried out under Work Unit #31710 entitled "Failure Mechanisms in Earth and Rock Fills under Critical Flow Conditions," of the Civil Works Investigative Studies Program, sponsored by the US Army Corps of Engineers (USACE). Mr. Richard Davidson and Mr. Ralph Beane were the USACE technical monitors.

The investigation was conducted by Drs. Hon-Yim Ko and R. Jeffrey Dunn and Mr. Tom Hollingsworth during calendar years 1984 and 1985. Mr. S. Paul Miller of the US Army Engineer Waterways Experiment Station (WES) was the Contracting Officer's representative. The work was performed under the general supervision of Mr. C. L. McAnear, Chief, Soil Mechanics Division, Geotechnical Laboratory (GL), and Dr. W. F. Marcuson III, Chief, GL.

During publication of this report LTC Jack R. Stephens, EN, was Acting Commander and Director of WES. Dr. Robert W. Whalin was Technical Director.

CONTENTS

CONVERSION TABLE

Multiply	By	To Obtain
inches	25.4	millimeters
feet	0.3048	meters
cubic feet per second	0.0283	cubic meters per second
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	0.15709	kilonewtons per cubic meter

PART I

Introduction

(1) This report is a continuation of the test programs reported by Ko, et al. in Study of Embankment Performance During Overtopping and Throughflow [1], [2] for the Waterways Experiment Station (WES) of the Department of the Army. In the first of the test programs, the feasibility study, two sets of three-dimensional tests were run. In one set, a crushed rock embankment was modeled. In the other, an earth embankment was modeled. The first test program showed that using a centrifuge to model earth embankments during overtopping events is valid. In the second of the test programs, two types of embankments were tested, rigid and erodable. The rigid embankment tests were used to measure hydraulic parameters, flow quantity and velocity, and to verify the scaling relationships. The erodable embankment tests modeled Simons, Li and Associates' full-scale tests in an attempt to validate the modeling. In the third and last stage of the study, one which is being reported herein, there were three objectives. The first objective was to model actual overtopping events, which was achieved by modeling Clarence Cannon and Bloomington Lake Cofferdam overtopping failures. The second objective was to determine the effects of modeling in two dimensions versus modeling in three dimensions. This was accomplished by modeling the previous 2-d tests of Ko, et al. [1,2] in 3-d and previous 3-d tests in 2-d. The third objective was to study the effects of nonhomogeneous embankment cross-sections on erosion, which was accomplished by modeling an embankment with a blanket drain and an embankment with a rock toe. The test program was defined with input from Paul Miller of WES.

Outline of Report

(2) In Part II of this report, the system used in running the test program is described. This system includes the centrifuge, the water conveyance system, and the embankment erosion monitoring and measurement system. In Part III, the modeling of Clarence Cannon Cofferdam is discussed. This includes the soils, soil preparation, test procedure, results and conclusions. In Part IV, the modeling of Bloomington Lake Cofferdam is described. In Part V, the modeling of the Simons, Li & Associates test embankment in three dimensions is described. In Part VI, the embankments that were previously modeled in three dimensions during the feasibility study are modeled in two dimensions and the results are described. In Part VII, the modeling of major embankments in two dimensions with different drainage filters is discussed. In Part VIII, general conclusions and recommendations for further study are presented.

PART II

Description of System

(3) The centrifuge used in this study was a 10 g-ton machine operated at the University of Colorado at Boulder. The specifications for the centrifuge are listed in Reference [1], Page 5. The sample container used to house the three-dimensional models was the same 17"x17" container reported in Reference [1], Page 5. The sample container used to house the two-dimensional models was the same container used to model the Simons, Li & Associates embankments reported in Reference [2], Page 35. The water conveyance system used was the same as described in Reference [1], Page 6. The method of measuring and monitoring embankment erosion for the three-dimensional tests was the same as described in Reference [1], Page 7. The method for measuring and monitoring embankment erosion of the two-dimensional models was the same method as described in Reference [2], Page 47.

PART III

Modeling of Clarence Cannon Cofferdam

Description

(4) Clarence Cannon Cofferdam was located near Hannibal, Missouri on the Salt River. It was a 45-foot high, random earth fill cofferdam. For the lower 37 feet of the cofferdam, the upstream and downstream slopes were placed at 3 H: 1 V. For the upper 8 feet of the cofferdam, the slopes were placed at 1 H: 1 V. A more complete description of the cofferdam and the overtopping event, as presented by Paul Miller of WES, is presented in Appendix A.

Test Materials and Preparation of Soils

Test Materials

(5) Two soil types were used in modeling Clarence Cannon Cofferdam. They were designated Soil 1 and Soil 2. Both soils had to meet the grain size criteria provided by Paul Miller of WES, who provided the specifications on the basis of personal observations of the characteristics of the soils found at the site. These criteria were that 50% of the soil is finer than the No. 200 sieve and that 50% of the particle sizes fall between the No. 8 and the No. 200 sieves.

(6) Soil 1 consisted of Bonny Loess silt provided by the U.S. Bureau of Reclamation Research Laboratory in Denver, Colorado. The Bonny Loess silt has 35% of the particle sizes coarser than the No. 200 sieve. So, in order to meet the criteria 15%, by weight, of coarse material was

added. The coarse material was a concrete aggregate obtained from the CU materials laboratory and was batched by equal portions of #30, #50 and #100 sand. Soil 1 contained 85% Bonny Loess silt and 15% coarse material by weight.

(7) The fine material in Soil 2 consisted of buckshot clay provided by WES. The buckshot clay was found to have a liquid limit of 64, a plastic limit of 22 and a plasticity index of 42. Hydrometer analysis results, as presented in Figure 1, show that buckshot clay is 100% finer than the #200 sieve. To meet the criteria, 50% coarse material by weight was added in equal portions on the #30, #50 and #100 sieves. Soil 2 was composed of 50% buckshot clay and 50% coarse material by weight. The grain size curves for Soil 1 and Soil 2 are also shown in Figure 1. Standard Proctor compaction tests were run on Soil 1 and Soil 2 to obtain the moisture-density relationships which are presented in Figures 2 and 3.

Water

(8) The water used in overtopping flow was potable water from domestic water supply provided by CU outlets.

Preparation of Model Embankments

(9) Soil 1 was mixed in a Hobart electric mixer with a sufficient quantity of water to obtain a standard Proctor optimum moisture of 12%. The soil was then compacted at a dry density of 109 pcf, which is 90% standard Proctor maximum dry density.

(10) Soil 2 was mixed with an adequate quantity of water to achieve standard Proctor optimum moisture of 16%. The soil was then compacted to a dry density of 109 pcf, which is 90% standard Proctor maximum dry density.

(11) A rectangular block of soil 11.5 in x 16 in. x 5 in. was compacted in 1-inch layers in the sample container using static compaction method and a hydraulic loading machine. Ladd's under-compaction method, Reference [3], was used to ensure the same density throughout the sample. Each layer was thoroughly scarified to ensure good contact between layers.

(12) In order to trim a two-dimensional model from the block of soil, two walls of the sample container were removed. The embankment slope was then trimmed from the rectangular block of soil by placing a template on one side of the block and trimming with a soil spatula and a knife. The block was trimmed to the configuration shown in Figure 4 which shows a longitudinal section of the model embankment over the middle 8 in. of the width of the model. In the transverse direction (not shown in Fig. 4), the last 4 inches on each side of the crest were increased in height by 1.5 inches to force the water to overtop the embankment in the middle 8 inches of the crest. This was done to ensure that the interface between the sample basket walls and the embankment did not interfere with the erosion, since previous experience had shown that water could easily follow the interface to cause piping erosion there, whic' would not be representative of erosion through soil. It is realized that the presence of this crest wings might alter the stress state in the embankment, but avoidance of piping erosion was the over-

riding consideration that led to their use. A 1.0-inch high, 16-inch long weir was installed to simulate the tailwater condition present in the field. Silicon sealant was placed on all soil-sample container contacts to prevent piping failure of the model along the container wall.

Test Procedure

(13) A total of four tests were run in modeling Clarence Cannon Cofferdam at 80g. Two tests were run using Soil 1, and two tests were run using Soil 2. The following test procedure was followed:

[1] Initial manual measurements of the embankment profile at eight points across the embankment cross-sections shown on Figure 5 were made with a machinist's scale.

[2] Initial photographs of the embankment were taken.

[3] The centrifuge was accelerated to 270 r.p.m.

[4] The video tape recorder was started to record the test.

[5] The overtopping flow was then started and the increase in weight slowed the centrifuge to 250 r.p.m. or approximately 80g where it remained for the test period.

[6] The depth of overtopping flow followed the following schedule:

Overtopping depth (in.)	Time (min.)
0.1	1.1
0.5	1.1
0.9	0.7
0.4	0.4

[7] After each increment, the centrifuge and the overtopping flow were stopped and photographs and manual measurements were taken of the embankment. (It is realized that ideally erosion measurements

should be made without stopping the centrifuge or the overtopping flow. However, it has been previously shown that no significant adverse effects were observed by so doing [1]).

[8] The centrifuge was accelerated back up to 250 r.p.m. and the overtopping flow was begun at the new depth.

[9] When the test was completed, the centrifuge and the overtopping flow were stopped and final photos and manual measurements were taken.

The tests on Soil 1 ran for a total of 0.8 to 1.1 minutes and tests on Soil 2 lasted 4.5 minutes without significant erosion.

Results

(14) Pairs of photographs of the test embankment were taken before and during overtopping. These photographs were used in analyzing the erosion.

(15) After the test was completed, erosion depth measurements were taken as described and the data plotted. The results for the two tests on Soil 1 are shown in Figures 6 and 7. The two tests on Soil 2 are shown in Figures 8 and 9. The volume of material eroded was calculated from the cross-sections in these figures. A plane planimeter was used to find the eroded area of the cross-sections and then the volume was found by averaging the areas and multiplying by the distance between the cross-sections. The data were presented as percent volume eroded which was calculated by dividing the eroded volume by the volume of material available, as defined in Figure 10. Figure 11 shows the test results for Soil 1 and Soil 2.

Discussion

(16) The description of the overtopping of Clarence Cannon Cofferdam provided by Paul Miller in Appendix A showed that breaching of the cofferdam occurred after 20 hours of overtopping. To compare the time to failure for the models to the prototype, the time scaling factor, $n^{1.38}$, found by Ko, et al. [2] was used. Soil 1 lasted only 7.8 hours and 5.87 hours (prototype times) for Tests 1 and 2, respectively, before breaching occurred, whereas Soil 2 sustained 30.3 hours and 20.44 hours of overtopping with minimal erosion for Tests 3 and 4, respectively. Unfortunately, an approximation of the volume of material eroded during overtopping of Clarence Cannon
Cofferdam was not available to compare with the model volumes of erosion.

Conclusions

(17) Soil types 1 and 2 were chosen in order to bracket the soil used in Clarence Cannon Cofferdam. Comparing the breaching times of Soils 1 and 2 to the time of breaching of Clarence Cannon Cofferdam shows that Soils 1 and 2 were successful in bracketing the response of the cofferdam. The results of the testing program provide a range of time for breaching that could be expected for an embankment consisting of a soil similar to the soil used in Clarence Cannon Cofferdam.

(18) If Soils 1 and 2 were supposed to exactly reproduce the response of the soil in Clarence Cannon Cofferdam, new time scaling factors, different than $n^{1.38}$ found by Ko, et al.,[2] could be calculated. The equation to be used is:

$$n^{S.F.} = t_p/t_m$$

where n = g level,

t_p = time of prototype,

t_m = time of model,

and S.F. = scale factor.

Such calculations lead to scaling factors for Soil 1 of 1.6 and 1.66 from Tests 1 and 2, respectively. The scaling factors for Soil 2 are 1.37 and 1.28 from Tests 3 and 4, respectively. The previously obtained scaling factor [2] of 1.38 is probably more accurate because it is based on more data.

PART IV

Modeling of Bloomington Lake Cofferdam

Description

(19) Bloomington Lake Cofferdam was located on the north branch of the Potomac River along the Maryland-Virginia border. The cofferdam was random earthfill consisting of clayey, sandy gravel. It had an upstream impervious blanket and cutoff and a downstream rock toe drain. The cofferdam had an upstream slope of 1 V: 3 H and a downstream slope of 1 V: 2.5 H. A more complete description of the cofferdam and the overtopping event is presented in Appendix A.

Test Materials and Preparation of Soils

Test Materials

(20) The two soils designated Soil 1 and Soil 2 that were used in modeling of Clarence Cannon Cofferdam were also used in modeling of Bloomington Lake Cofferdam.

Water

(21) The water used was as previously described.

Preparation of Model Embankment

(22) The preparation of the model of Bloomington Lake Cofferdam followed the samé procedure as that used in modeling of Clarence Cannon Cofferdam. The rectangular block of soil that was compacted was trimmed to the configuration shown in Figure 12. A toe weir was not installed because a tailwater condition was not wanted.

Test Procedure

(23) The same test procedure used for Clarence Cannon Cofferdam was used for Bloomington lake Cofferdam. Modeling was again performed at 80g. The two tests performed on Soil 1 ran a total time of 18-20 seconds before breaching occurred. The two tests performed on Soil 2 ran a total time of 4.5 minutes, but total breaching never occurred.

Results

(24) Stereo pairs of photographs of the test embankments were taken before and during overtopping. These photographs were used in analyzing the erosion.

(25) Erosion depth measurements were taken as previously described and the data were plotted to produce the cross-sections in Figures 13, 14, 15 and 16. Figures 13 and 14 are from tests using Soil 1, and Figures 15 and 16 are from tests using Soils 2. The percent volume eroded was computed as described previously, and the results were plotted. Figure 17 shows the available volume for erosion. Figure 18 shows the results of tests using Soil 1 and Soil 2.

Discussion

(26) According to the description of the overtopping of Bloomington Lake Cofferdam, breaching occurred after 10 hours. In the model tests using Soil 1, breaching occurred after 2.3 hours and 2.1 hours (projected prototype times) for Tests 1 and 2, and in the tests using Soil 2 breaching had not occurred after 31.7 hours of overtopping for Tests 3 and 4. Unfortunately, an approximation of the volume of material eroded

during overtopping of Bloomington Lake Cofferdam was not available to compare with the model volumes of erosion from the model tests.

Conclusions

(27) As in the modeling of Clarence Cannon Cofferdam, Soils 1 and 2 were chosen to bracket the response of the soil used in Bloomington Lake Cofferdam. The time for breaching of Soils 1 and 2 shows that the modeling was successful in bracketing the response of Bloomington Lake Cofferdam. This modeling gives a range of time for breaching for an embankment made of a soil similar to the soil used in Bloomington Lake Cofferdam.

(28) New time scaling factors can be calculated as was done for Clarence Cannon Cofferdam. For Soil 1 the scaling factors are 1.73 and 1.71 for Tests 1 and 2, respectively, and for Soil 2 the scaling factor is 1.13 from Tests 3 and 4. Previously obtained value of of 1.38 is more accurate because it was derived from more data.

PART V

Modelling of Simons, Li & Associates Test Embankment in Three Dimensions

Test Materials and Preparation of Soils

Test Materials

(28) The soil used in the modeling was provided by Simons, Li & Associates (SLA) and came from their test embankment located at the research facility on the Colorado State University campus. The soil was the same sandy clay soil used by Ko, et al., in Reference [2]. The Atterberg limits were determined to be: liquid limit of 39, plastic limit of 26 and plasticity index of 13. The gradation analysis and standard Proctor compaction results are shown in Figures 19 and 20.

Water

(30) The water used was as previously described.

Preparation of Model Embankment

(31) The soil obtained from SLA was at standard Proctor optimum moisture of 12%. The soil was compacted using static compaction on a hydraulic loading machine and Ladd's undercompaction method. It was placed in 1.0-inch layers directly in the sample container so that a dry density of 106 pcf, which is 90% of standard Proctor maximum, was obtained. A rectangular block of soil 4.0 in. x 11.5 in. x 16.0 in. was formed. A template was attached to one side of the soil block and was

used as a guide for trimming with a spatula the soil model to the configuration shown in Figure 21. The 4 inches on each side of the crest were again raised 1.5 inches for the same reasons mentioned in Part II. Silicon sealant was again placed on all soil-sample basket contacts.

Test Procedure

(32) Two tests were run in this phase of the study. The procedure used in running the tests was:

[1] Initial manual measurements of embankment profile in eight positions along the two cross-sections shown in Figure 22 were made with a machinist's scale.

[2] Initial photographs were taken of the embankment.

[3] The centrifuge was accelerated to 270 r.p.m.

[4] The video tape recorder was turned on.

[5] Overtopping flow was started and maintained for an overtopping depth of 0.33 inches.

[6] The added weight slowed the centrifuge to 230 r.p.m. or approximately 70g, where it remained for the remainder of the test.

[7] Overtopping flow was maintained until breaching of the model occurred.

[8] Overtopping flow was stopped and then the centrifuge was stopped.

[9] Final manual measurements and photographs were taken.

Results

(33) Stereo pairs of photographs of the embankments were taken before overtopping and after completion for each test and were used in analyzing the erosion.

The photographs from both tests show very similar erosion patterns. The erosion started at the toe of the embankment and progressed upward to the crest. The erosion in both tests formed gullies approximately 3 inches in width with steep sides. The erosion depths measured in eight positions along two cross sections were plotted and are shown in Figures 23 and 24. The profiles produced by the two tests are very similar. The fact that the erosion occurred on the left side in one test and on the right side in the other test is unimportant. It simply shows the random nature of erosion.

(34) In order to compare the three-dimensional tests run in this program to the two-dimensional tests conducted in the previous study, a basis for comparison had to be found. Two methods of comparison were utilized, volume eroded and erosion depth.

(35) The eroded volume comparison was done on a percentage basis versus time. The comparison is shown in Figure 25. The calculation of the percent eroded volume was made by the following calculation:

$$\%\text{ eroded volume} = \frac{\text{volume eroded}}{\text{volume available}}$$

The volume eroded was calculated by finding the area of the eroded cross-sections where depth measurements were taken (see Figure 21) and assuming erosion was linear between the two sections. The area of the eroded cross-sections was found by a polar planimeter. The volume was then found by averaging the two areas and multiplying by the distance between the cross-sections. The available volume was determined by multiplying the area shown in Figure 26 by 8 inches, the width of the spillway. The calculations of percent volume eroded for the two-dimensional tests were conducted as in Reference [2]. As can be seen from Figure 25, the erosion rates do not compare very well. The three-dimensional rate was much faster than the two-dimensional rate. The accuracy of the three-dimensional volume is questionable. The problem arises from the fact that to determine the eroded volume only two erosion profiles are known. The cross-sections do not encompass the entire slope and linear erosion is assumed between the two sections. Another problem is in defining what the available volume is. These problems lead to questionable accuracy in comparing the three-dimensional and two-dimensional models.

(36) The other method of comparison was depth of erosion. In the three-dimensional test, direct measurement of erosion depth was done as previously explained. In the two-dimensional tests, erosion depths were measured at the same points along the slope as the three-dimensional model from the projection of the photograph negatives. This comparison is more direct and eliminates the errors introduced in the volume comparison. Figure 27 shows a typical cross-section with erosion depth plotted at the two measurement points. This method showed a much better comparison; however, it is still unsatisfactory. In general, the three-dimensional model eroded to greater depth than the two-dimensional model.

Conclusions

(37) The three-dimensional tests on the average eroded much more rapidly than the two-dimensional tests. The depth of erosion between the three-dimensional tests and the two-dimensional tests compared much better than the erosion volume; however, the three-dimensional tests eroded to a greater depth than the two-dimensional tests. From the results it would appear that in the two-dimensional tests there was interaction (adhesion) between the soil and the container walls. The three-dimensional tests did not reflect such effects, since the erosion channel did not touch the container wall. Three-dimensional tests would be a more appropriate method of testing, because container-soil interactions will not be reflected in the test results.

PART VI

Modeling of the Feasibility Study Embankments in Two Dimensions

Test Materials and Preparation of Soils

Test Materials

(38) Test Materials. There were two soils used in this phase, where were designated crushed rock and clay material. These soils are the same as reported in Reference [1]. The crushed rock was a silty sand and was classified as SM soil using the Unified Soil Classification. It was a mixture of 45% Bonny Loess Silt and 55% of sand with sizes from the #4, 8, 16, 30, 50, 100 and 140 sieves. The crushed rock had a maximum dry density of 121.0 pcf and an optimum moisture of 11.8%. The clay material was a sandy, silty clay and classified as CL. It has a liquid limit of 28 and a plasticity index of 15. The clay material has a maximum dry density of 120 pcf and an optimum moisture of 12.0%. It was provided by the Bureau of Reclamation.

Water

(39) The water used was the same as described before. In this series of tests the water was dyed blue by using Navy Blue Rit Dye in order to get more contrast in the photographs. This was necessary so that the overtopping depth could be observed.

Preparation of Model Embankment

(40) An appropriate amount of water was added to the crushed rock

material in a Hobart electric mixer to obtain a standard Proctor optimum moisture of 11.8%. The material was then compacted to a dry density of 109 pcf, 905 of standard Proctor maximum.

(41) In the case of the clay material, sufficient water was added to the soil to obtain a standard Proctor optimum moisture of 12%. The material was then compacted to a dry density of 108 pcf, 90% of standard Proctor maximum.

(42) The materials were statically compacted directly in the sample basket on a hydraulic loading machine using Ladd's undercompaction method. A rectangular block of soil 0.5 in. x 11.5 in. x 8.0 in. was formed in 0.5-inch layers with the surface of each layer being scarified thoroughly to ensure good contact between layers.

(43) The front plexiglass plate of the sample basket was then removed to allow trimming of the sample. A template was placed on the block of soil and the soil was trimmed using a soil spatula. The crushed rock material was trimmed to the configuration shown in Figure 28. The clay material was trimmed to the configuration shown in Figure 29.

Test Procedure

(44) A total of four tests were run in this phase of the test program. Two tests were run on the crushed rock material and two tests were run on the clay material. The following steps were used in running the tests:

[1] An initial photograph of the embankment was taken.

[2] The centrifuge was accelerated to 270 r.p.m. and overtopping flow was started. The added weight of the overtopping flow

slowed the centrifuge to 230 r.p.m. or 70g where it remained for the remainder of the test. The video tape recorder was turned on.

[3] The overtopping depth was maintained at 0.2 inch.

[4] Photos were taken at a predetermined time interval.

[5] The overtopping flow was stopped when breaching of the model had occurred and the centrifuge was brought to a stop.

The crushed rock embankments ran for a total of 12 to 13 seconds with a picture taken every second. The clay embankment ran for a total of 3.5 to 4.5 minutes with pictures taken every one minute.

Results

(45) A series of inflight photographs of the embankments during overtopping was taken for analysis of erosion. Erosion profiles of the eroded embankments were obtained by projecting the negatives of the inflight photographs onto tracing paper and tracing the image. This was done for both the crushed rock and clay embankment tests. From the erosion profiles the depth of erosion was measured at two points corresponding to the same locations used to determine profiles obtained in the three-dimensional tests conducted by Ko, et al. [1]. Figures 30 and 31 show the depth of erosion versus time for the crushed rock and clay embankments, respectively.

(46) A comparison of the erosion depths for the two-dimensional and three-dimensional tests is shown on cross-sections in Figures 32 and 33 for the crushed rock and clay embankments.

(47) The two-dimensional and three-dimensional tests were also compared on a percent of erodable volume basis. In the two-dimensional tests, the percentage of eroded volume was found by using a plane planimeter on the eroded cross-sections to find the eroded area. The eroded area was then multiplied by the thickness of the model to obtain the eroded volume. To obtain the percent volume eroded, the eroded volume was divided by the volume available. The volume available was the areas of the cross-sections shown in Figures 28 and 29 multiplied by the thickness, 0.5 inches. The volume of embankment eroded in the three-dimensional tests was found as previously stated from the cross-sections shown in Figure 34. The volume available for erosion is shown in Figure 35. The percent eroded volume for the two-dimensional and three-dimensional tests is plotted versus time on Figure 36 and Figure 37 for the crushed rock and clay material, respectively.

Conclusions

(48) The three-dimensional crushed rock embankment eroded at a slower rate and a smaller amount than the two-dimensional tests. The two-dimensional crushed rock embankment eroded to a greater depth than the three-dimensional tests. The three-dimensional tests would appear to be a better test method, because they are better representations of full scale events.

(49) The three-dimensional and two-dimensional clay embankments eroded at a similar rate and to a similar amount. The depth of erosion also compared very well. Even though the two tests compared favorably, the three-dimensional test method is still recommended, because erosion

in the prototype is a three-dimensional phenomenon, and attempting to model it in two dimensions could miss important features in some special cases.

PART VII

Modeling of Major Embankments in Two-Dimension

Description

(50) Two cross-sections were proposed to model a major embankment dam. One cross-section had a blanket drain, and one cross-section had a toe drain. The slopes and other dimensions were the same for both cross-sections.

Test Materials and Preparation of Soils

Test Materials

(51) The embankments were made out of the soil designated Soil 1 used in modeling of Clarence Cannon and Bloomington Lake cofferdams. The blanket drain was modeled by a fine sane, #100 sand, and the toe drain was modeled by a fine-medium sand, #50 sand. The sand was from the CU materials laboratory.

Water

(52) The water used for overtopping was the same as described in Part VI.

Preparation of Model Embankments

(53) Soil 1 was mixed as previously stated. It was then compacted to a dry density of 109 pcf, 90% standard Proctor maximum. The soil was compacted using static compaction on a hydraulic loading machine in 0.5-inch layers form a rectangular block of soil 0.5 in. x 11.5 in. x 4.0 in. The plexiglass front plate was removed to allow trimming.

A template was positioned on the block of soil and the embankment was trimmed using a soil spatula. The blanket drain and the rock toe were formed by cutting out the appropriate shape from the embankment and replacing the removed soil with sand. The sand was placed by pouring it in the void cut in the embankment and tamping it down with a rubber mallet. In order to keep the sand in place when positioning the sample basket in the centrifuge, a thin layer of soil 1 was placed along the slope face. This layer kept the sand in place, and it was felt that the thin layer would not significantly affect the test results. The embankment with the blanket drain had the configuration in Figure 38. The embankment with the rock toe had the configuration in Figure 39.

Test Procedure

(54) Two tests were run in this phase of the study, one blanket drain model and one rock toe model. The test procedure used was the same procedure used in modeling the embankments in the feasibility study in two dimensions. The blanket drain embankment ran for 18 seconds and the rock toe embankment ran for 23 seconds.

Results

(55) As the embankment with a blanket drain was overtopped, erosion started at the toe as expected. The thin layer of Soil 1 that was placed at the face of the blanket drain was quickly removed. Once the blanket drain was exposed, the overtopping water quickly removed the loose blanket drain within 2 to 3 seconds. The removal of the blanket drain left a void with the embankment material standing without any support. This was obviously due to adhesion between the soil and the sample basket

walls. In a prototype, the embankment probably would have collapsed at this point. In the model, the embankment remained intact and the erosion continued in the main embankment as if the blanket drain was never present. The remaining embankment eroded in the expected manner, erosion started at the toe and worked its way up the embankment. Erosion of the embankment took 16 seconds.

(56) The embankment with the rock toe behaved in a similar manner as the embankment with the blanket drain. As the embankment was overtopped, the thin layer of soil placed on the face of the rock toe was immediately removed. Once the soil layer was gone, the rock toe was washed out almost instantly. The rock toe was removed in the first few seconds of overtopping. With the removal of the rock toe, the embankment eroded in the typical manner, erosion starting at the toe and working up the embankment. It took 23 seconds for the embankment to fail.

Conclusions

(57) The addition of a sand drain to the embankment cross-section caused the embankment to fail sooner than a homogeneous embankment. The cohesionless drain material eroded quickly and undercut the embankment causing failure. In a prototype embankment, similar results could be expected.

(58) The addition of the rock toe to the embankment cross-section also caused failure to occur more rapidly than a homogeneous embankment. The rock toe failed quickly, thus causing failure of the embankment. A prototype embankment would react similarly if the composition of the rock toe was comparable to the scaled version that was tested.

PART VIII

Conclusions and Recommendations

(59) The modeling of Clarence Cannon and Bloomington Lake Cofferdams showed that centrifugal modeling could accurately reproduce the response of the prototype. Qualitatively speaking, in centrifuge model tests, erosion began at the toe and progressed upslope, just as in prototype embankments subjected to overtopping erosion. Quantitatively, the erosion rates measured in these centrifuge tests that model the actual cofferdam failures produced a range of times within which similar cofferdams can be expected to fail.

(60) The test program showed the effects of running two-dimensional versus three-dimensional tests. The results tend to show that the two-dimensional tests are not as accurate as the three-dimensional tests due to soil-container wall interactions. Three-dimensional tests are the recommended method for modeling overtopping erosion of actual prototypes which obviously have no lateral boundary constraints.

(61) Future research in modeling overtopping in the centrifuge should attempt at enlarging the data base by using different soil types and embankment configurations. Such a data base could be used for making predictions on the erosion rates to be expected in impending overtopping situations.

REFERENCES

1. Ko, H.Y., Dunn, R.J., and Simantob, E., "Study of Embankment Performance During Overtopping and Throughflow," Contract Report for the Department of the Army, Corps of Engineers, Waterways Experiment Station, Contract No. DACW 39-83-C-0011, 1984.

2. Ko, H.Y., Dunn, R.J., and Simantob, E., "Study of Embankment Performance During Overtopping and Throughflow," Contract Report for the Department of the Army, Corps of Engineers, Waterways Experiment Station, Contract No. DACW 39-83-C-0011, June, 1985.

3. Ladd, R.J., "Preparing Test Specimens Using Undercompaction," ASTM Geotechnical Testing Journal, Vol. 1, No. 1, March 1978, pp. 16-23.

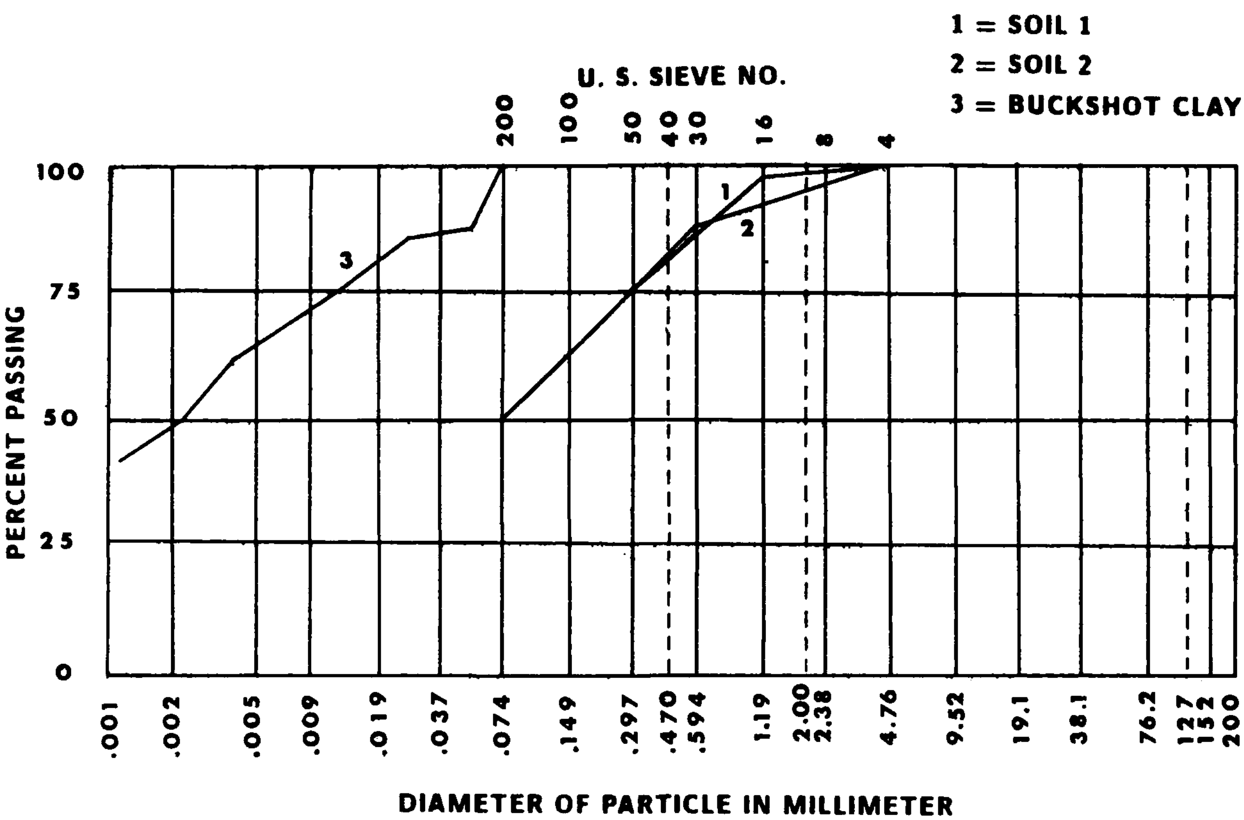


Figure 1 Grain size distribution Curves.

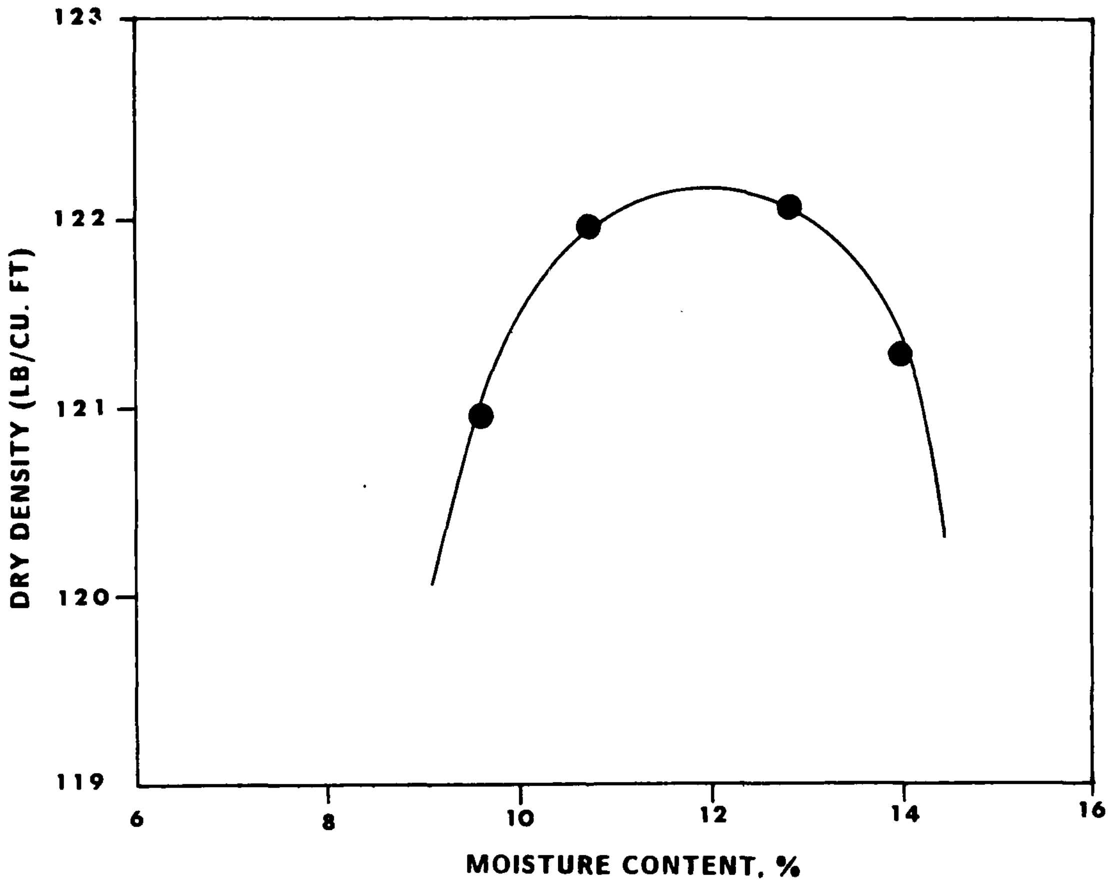


Figure 2 Moisture - density relationship for soil 1.

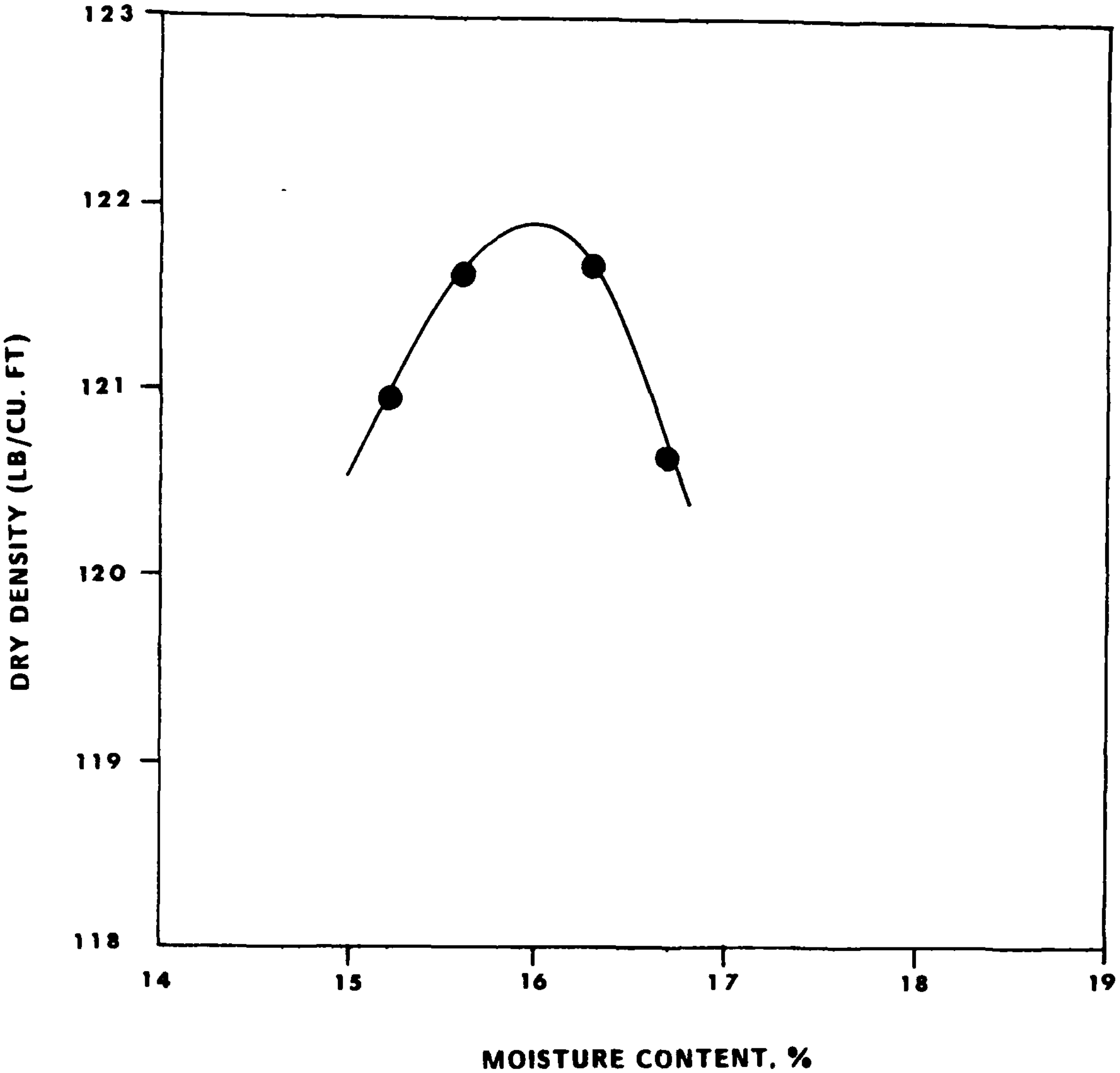


Figure 3 Moisture - density relationship for soil 2.

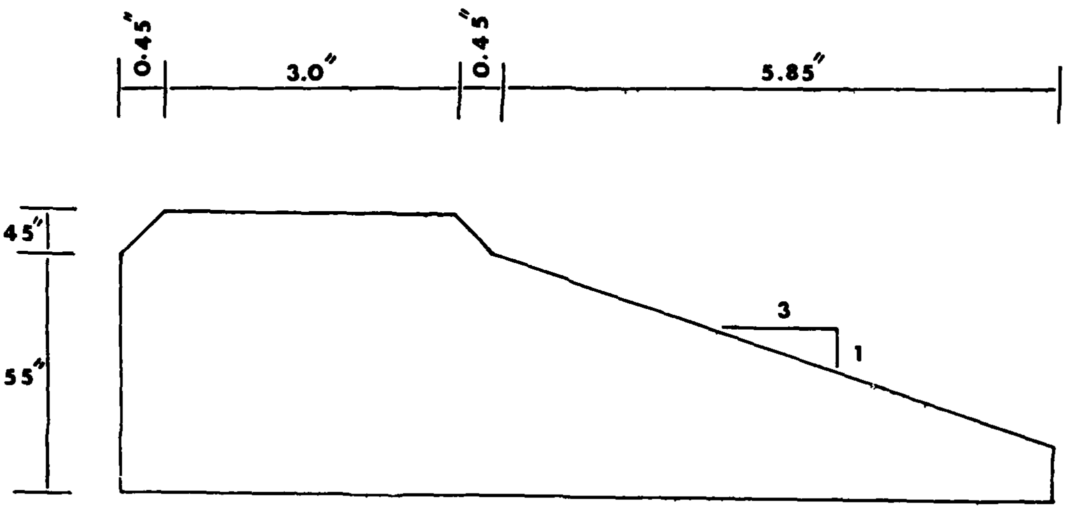


Figure 4 Configuration of Clarence Cannon Cofferdam.

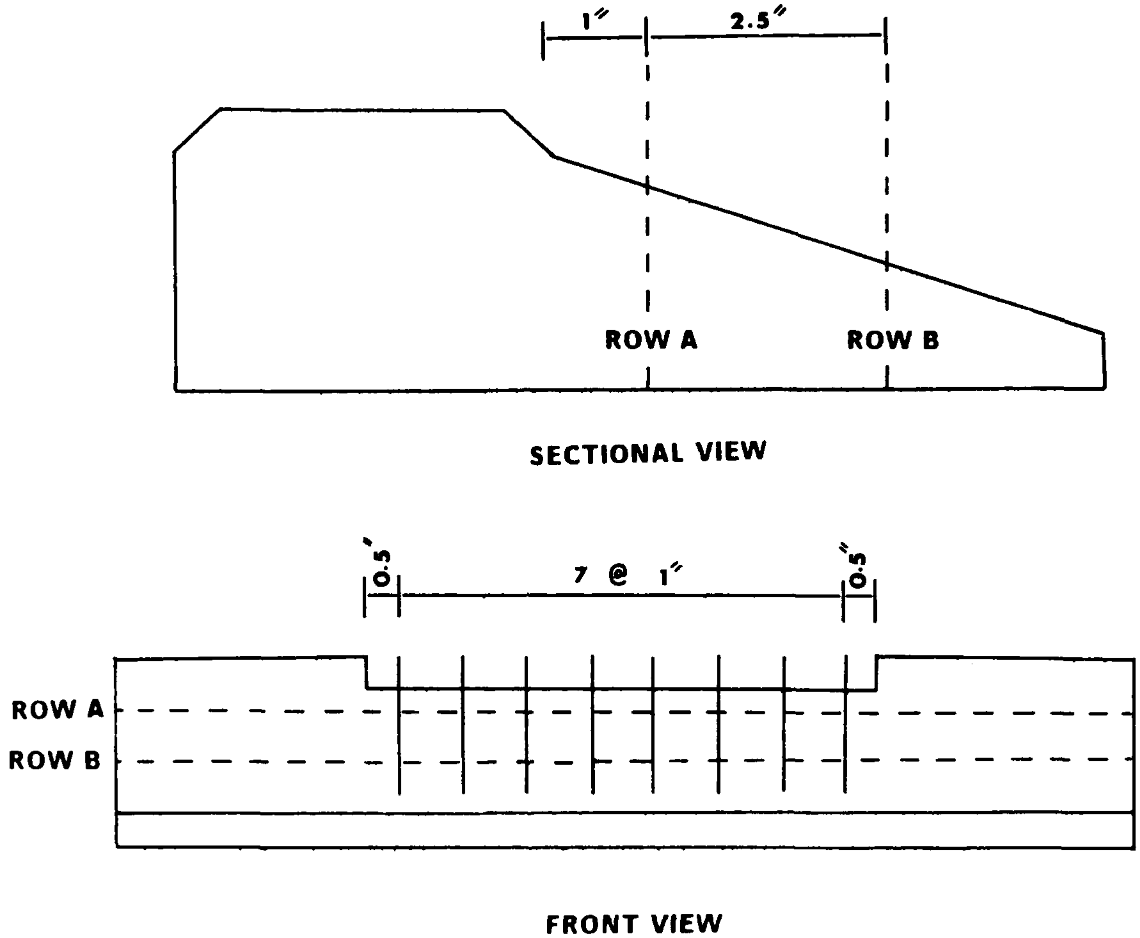


Figure 5 Manual measurement points for Clarence Cannon Cofferdam.

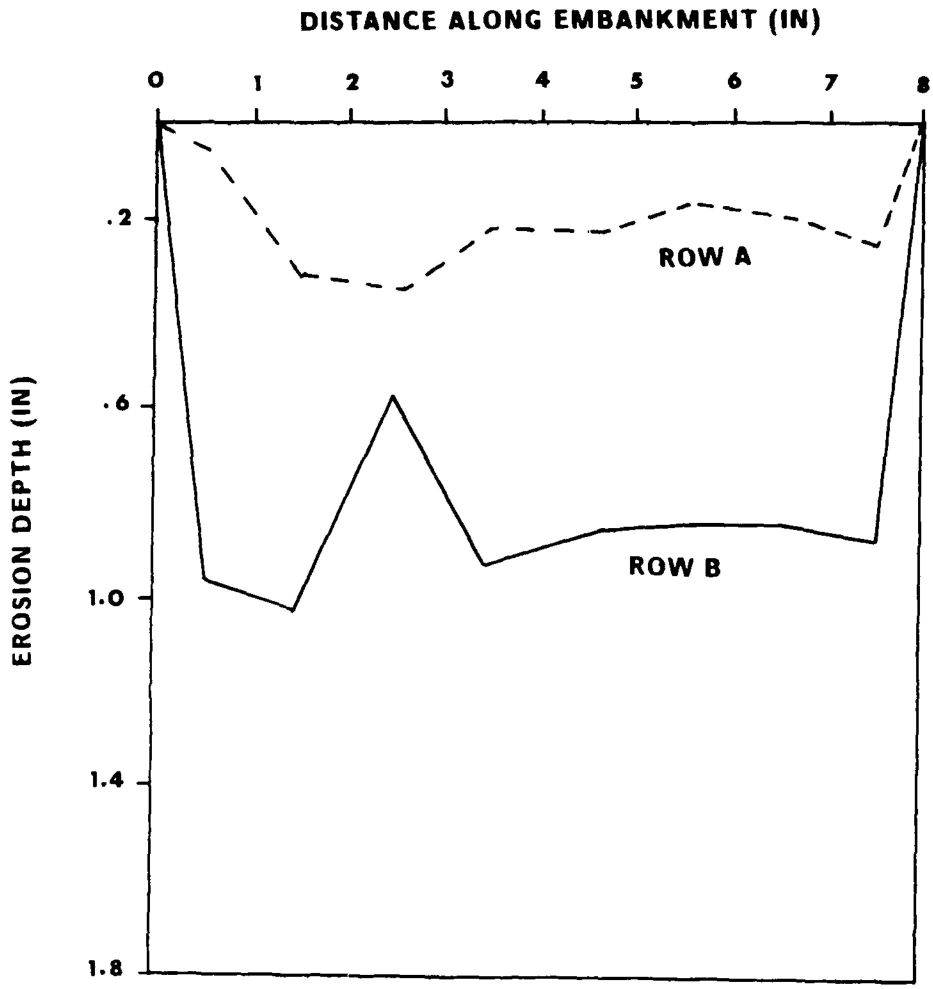


Figure 6 Erosion cross sections for Clarence Cannon Cofferdam. (soil 1, test 1).

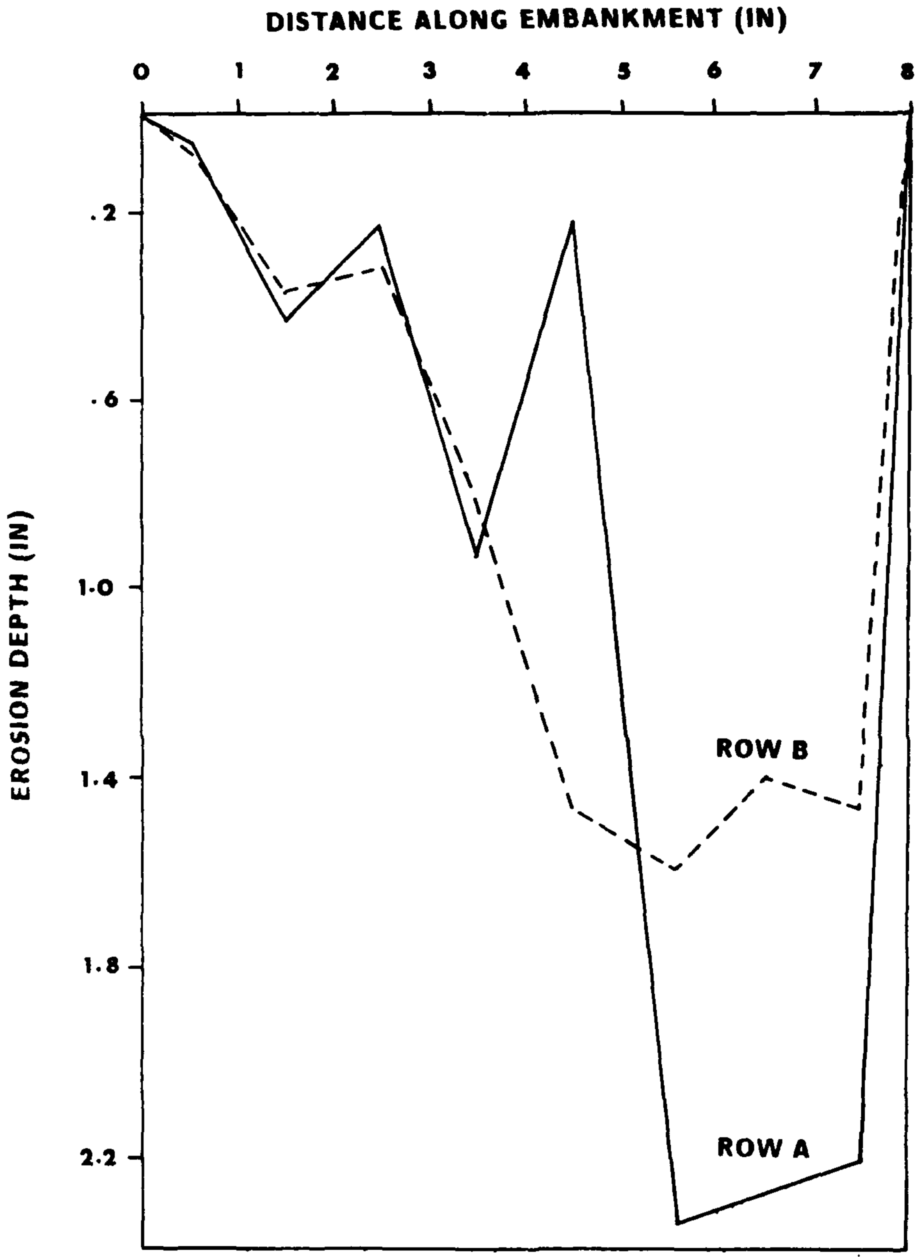


Figure 7 Erosion cross sections for Clarence Cannon Cofferdam. (soil 1, test 2).

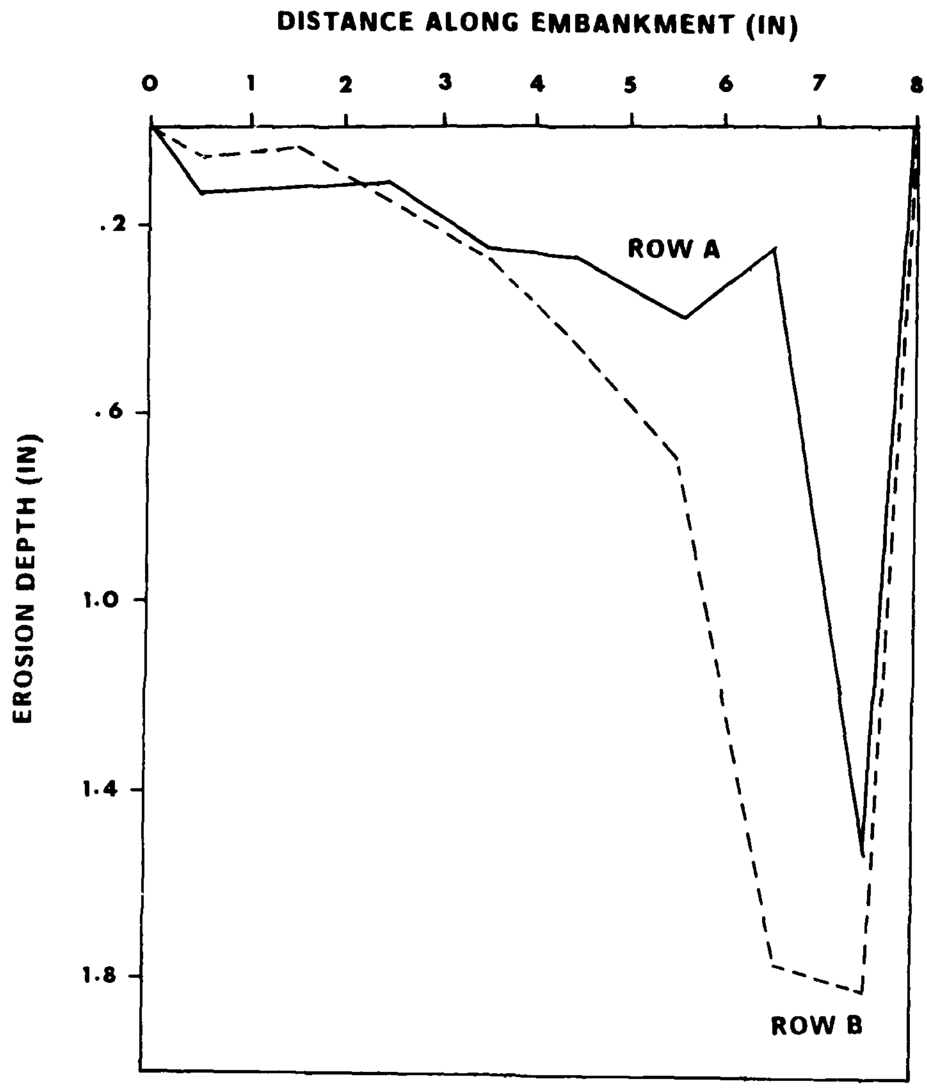


Figure 8 Erosion cross sections for Clarence Cannon Cofferdam. (soil 2, test 1).

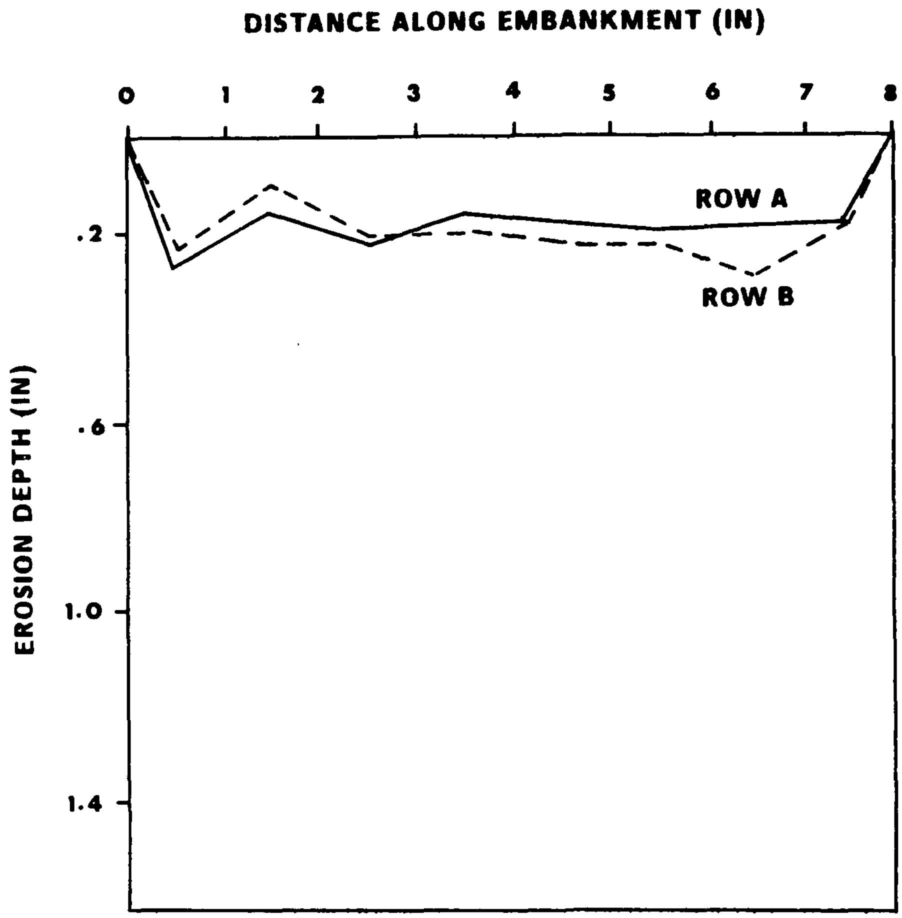
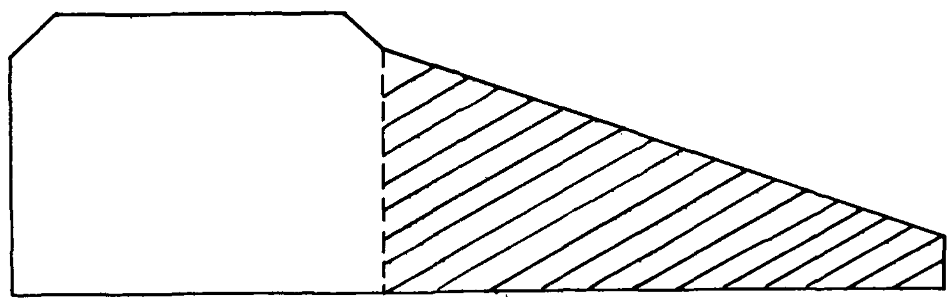


Figure 9 Erosion cross sections for Clarence Cannon Cofferdam. (soil 2, test 2).



(VOLUME IS EQUAL TO SHADED AREA TIMES EMBANKMENT WIDTH)

Figure 10 Volume available for erosion Clarence Cannon Cofferdam.

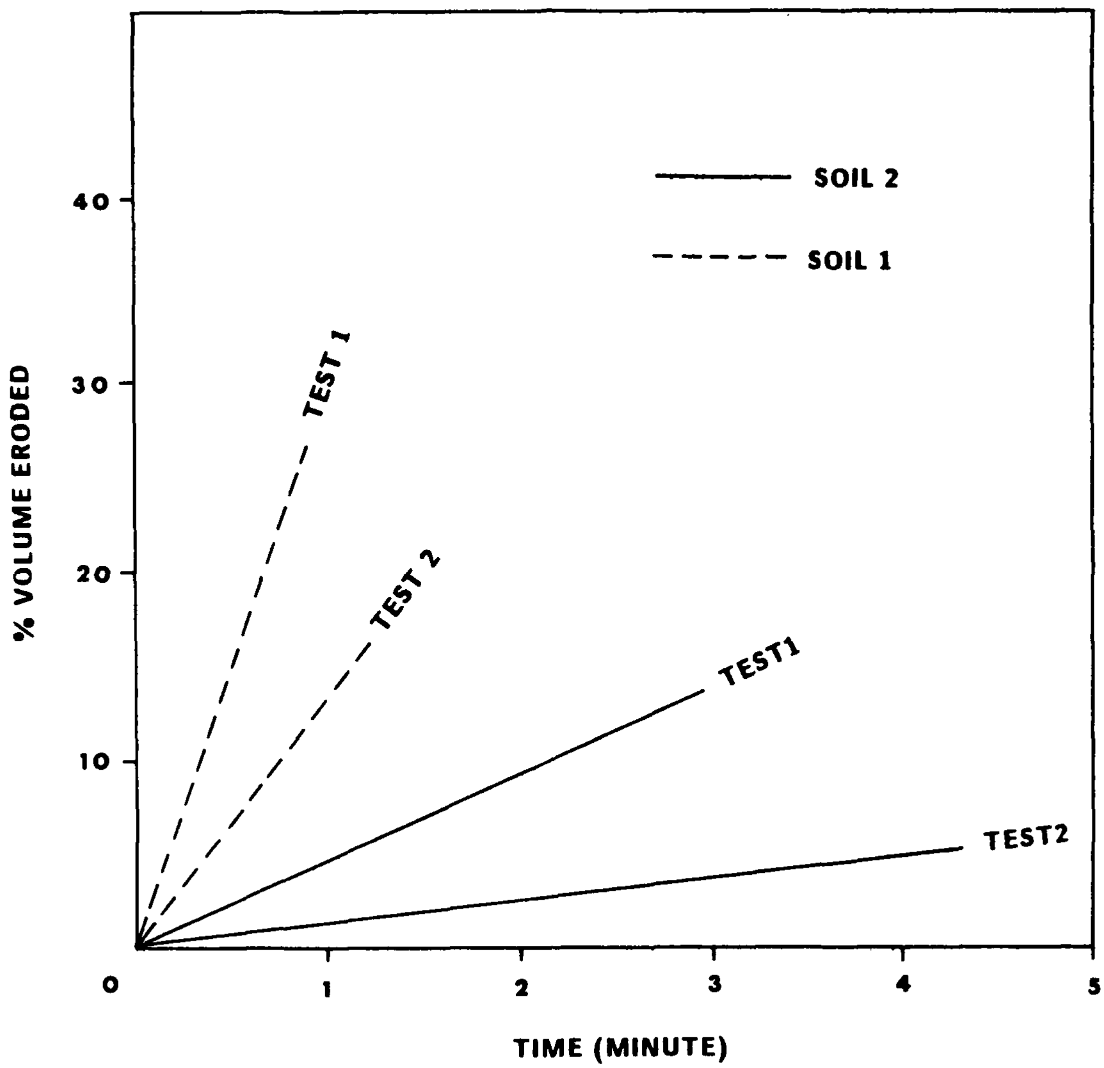


Figure 11 % Volume eroded vs. time for Clarence Cannon Cofferdam.

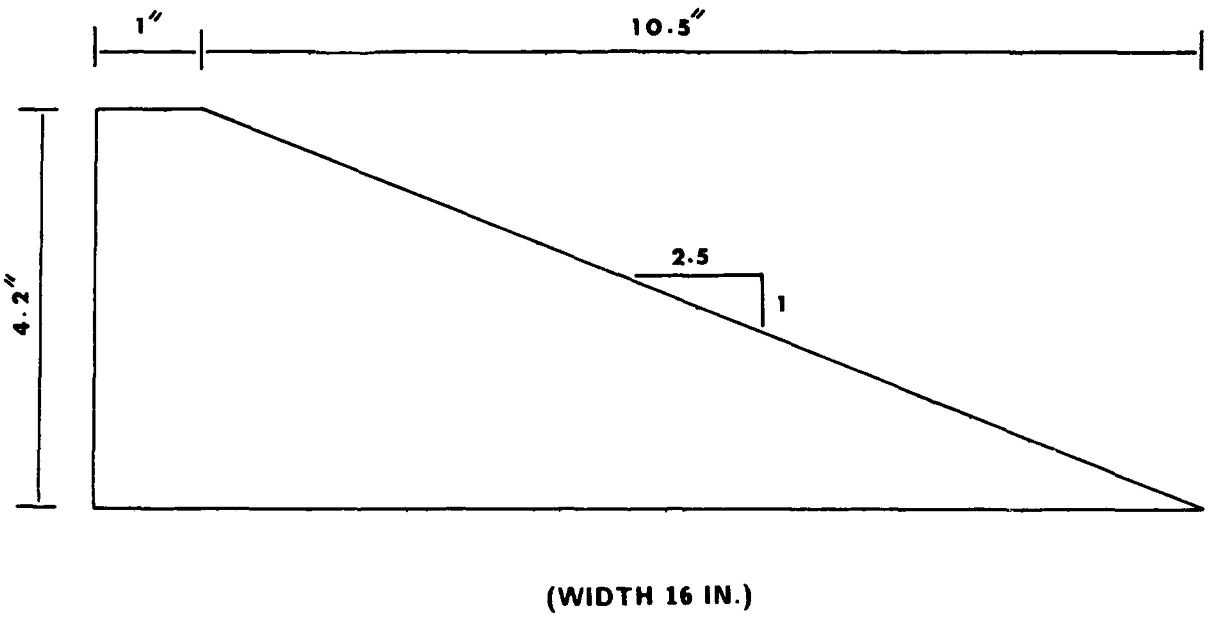


Figure 12 Configuration of Bloomington Lake Cofferdam.

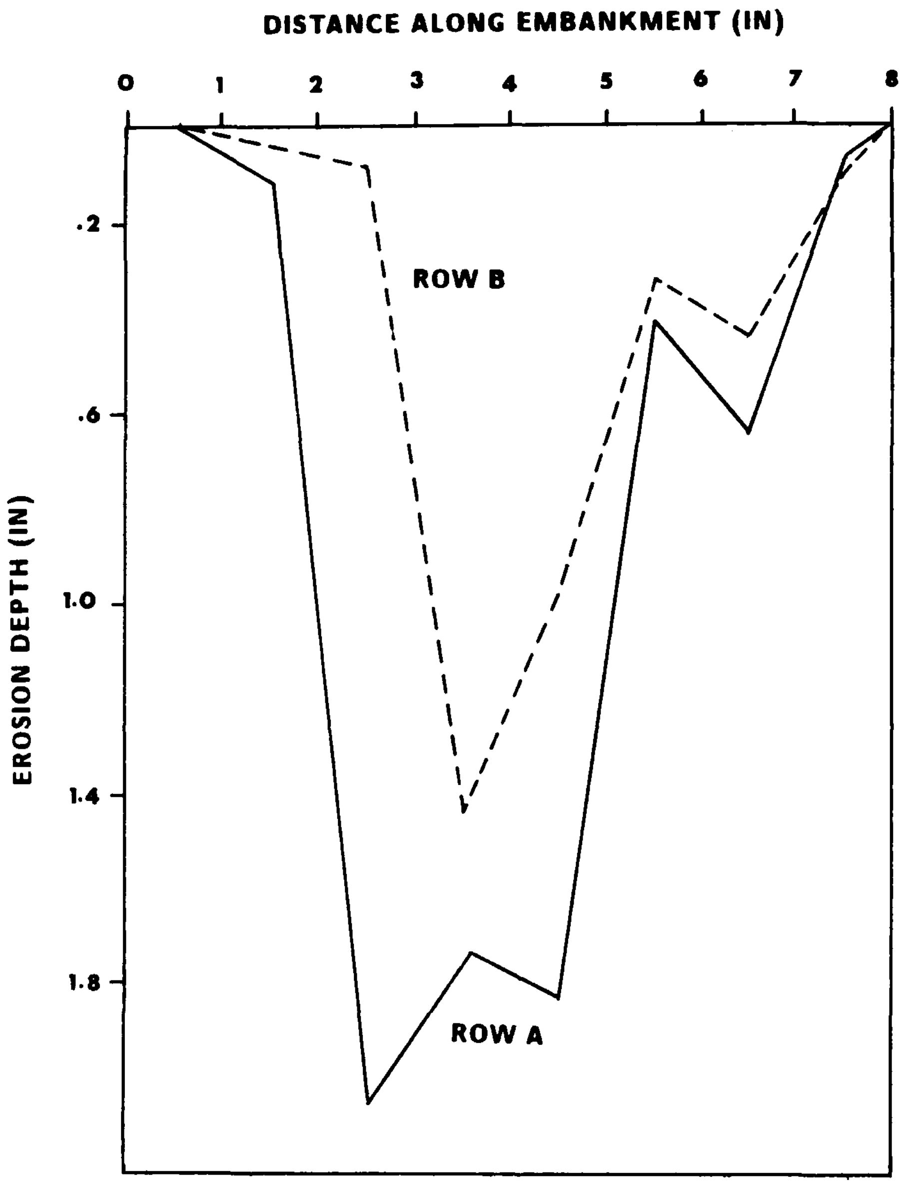


Figure 13 Erosion cross sections for Bloomington Lake Cofferdam. (soil 1, test 1).

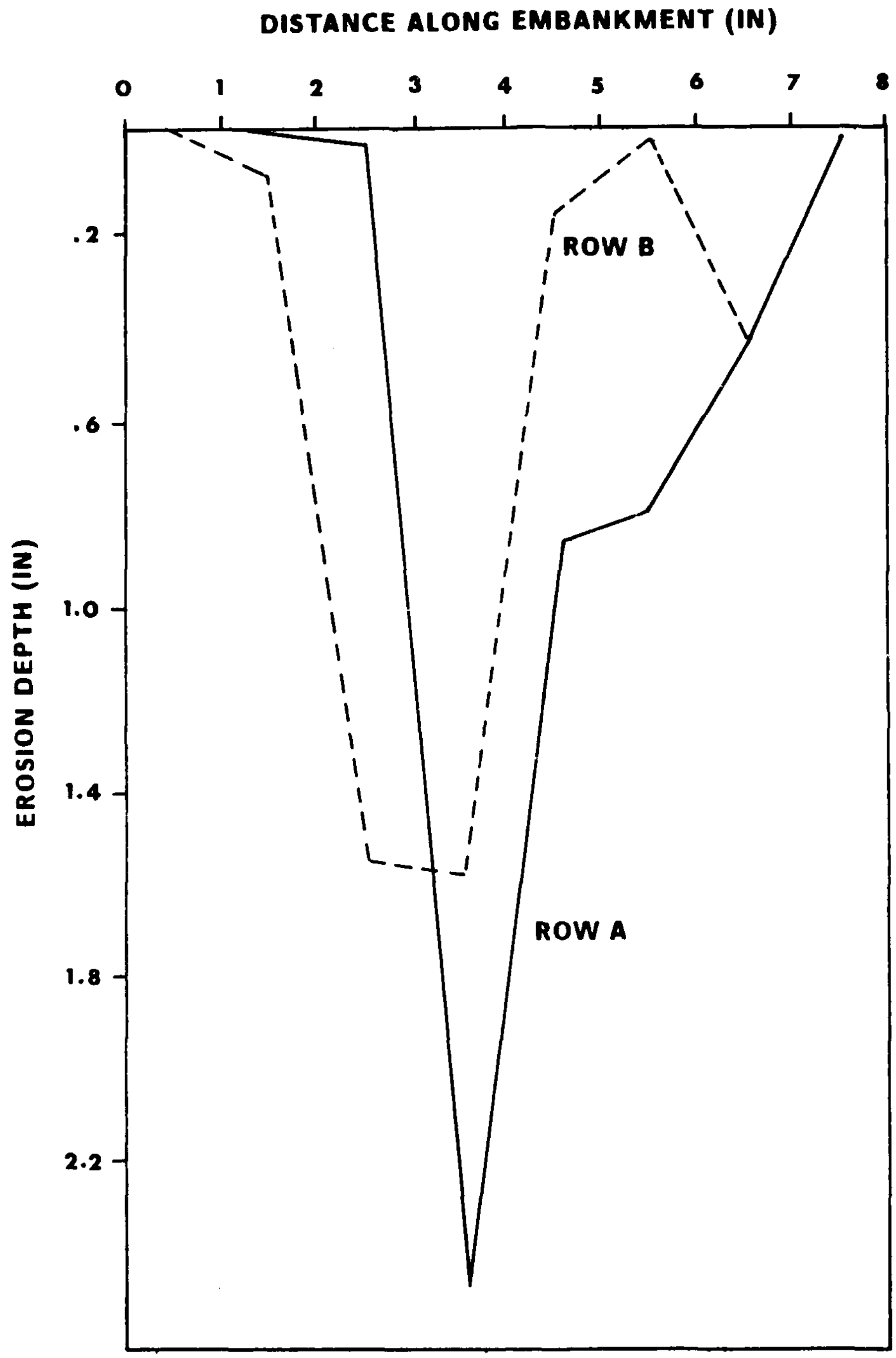


Figure 14 Erosion cross sections for Bloomington Lake Cofferdam. (soil 1, test 2).

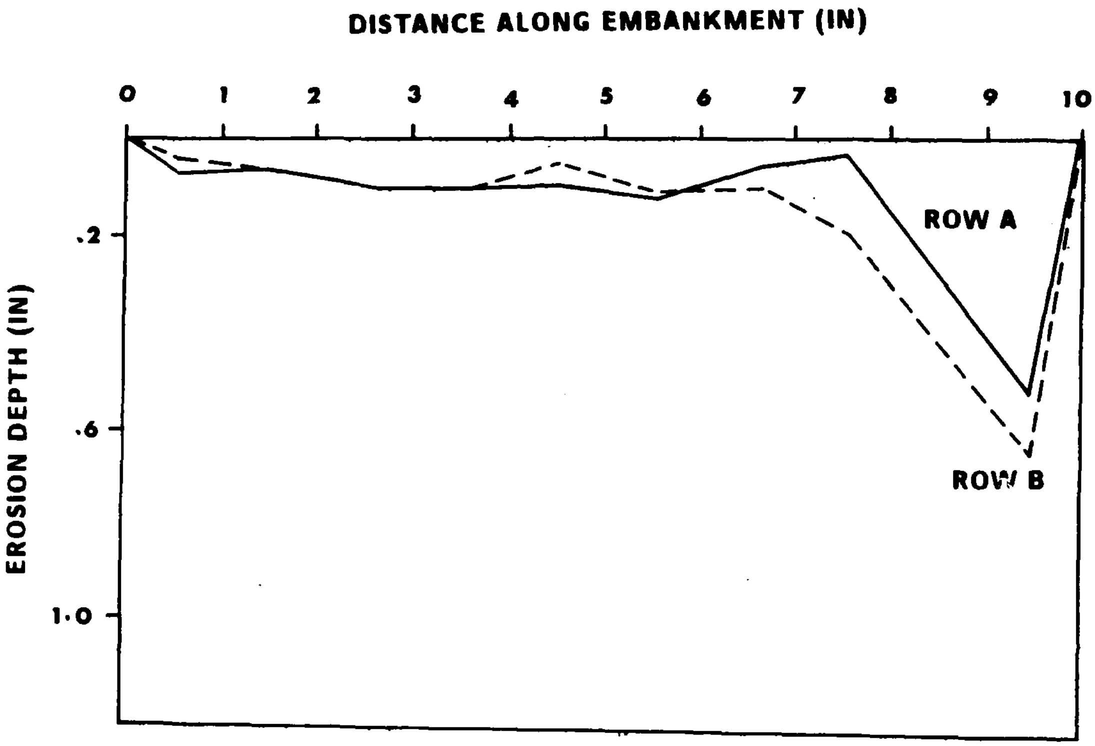


Figure 15 Erosion cross sections for Bloomington Lake Cofferdam. (soil 2, test 1).

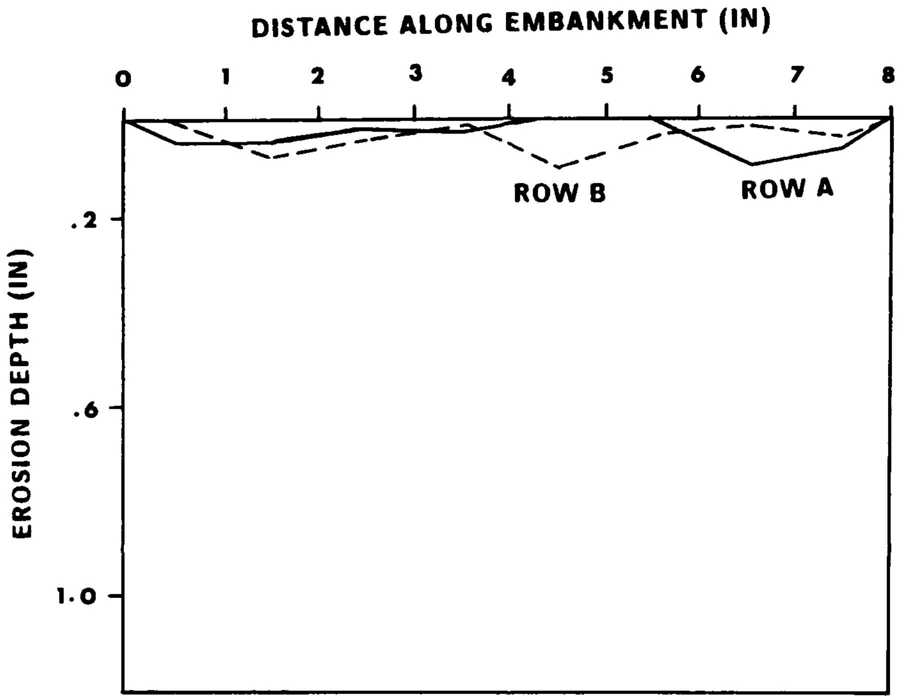
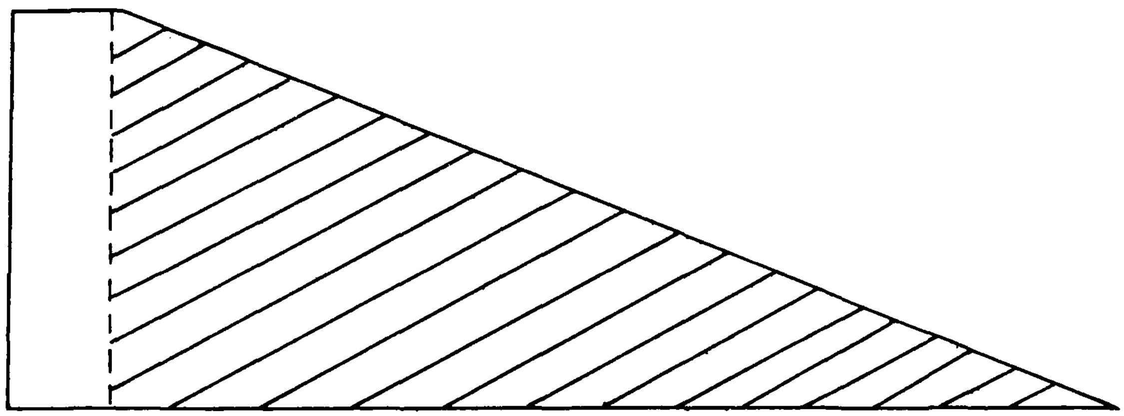


Figure 16 Erosion cross sections for Bloomington Lake Cofferdam. (soil 2, test 2).



(VOLUME IS EQUAL TO SHADED AREA TIMES EMBANKMENT WIDTH)

Figure 17 Available volume for erosion for Bloomington Lake Cofferdam.

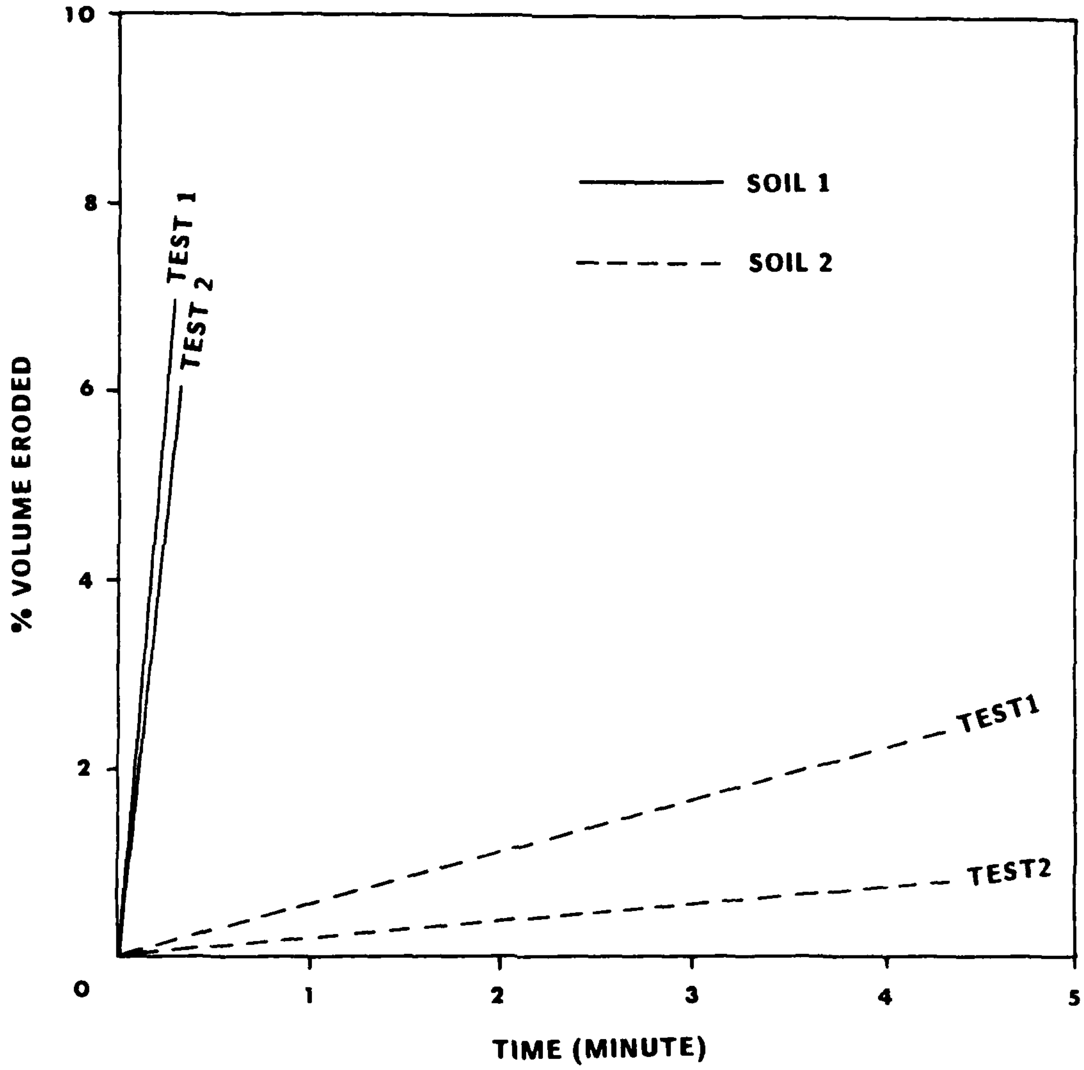


Figure 18 % Volume eroded versus time of Bloomington Lake Cofferdam.

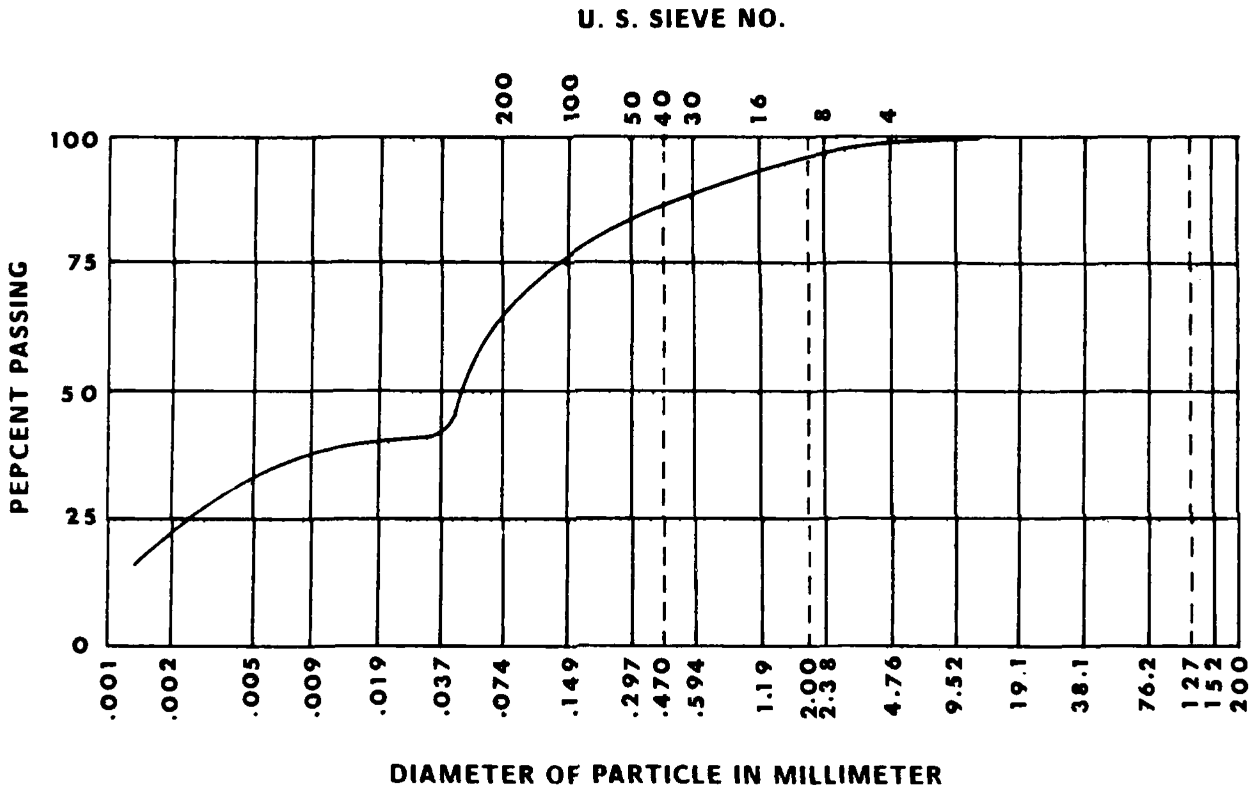


Figure 19 Grain size analysis for Simon, Li and Associates Soil.

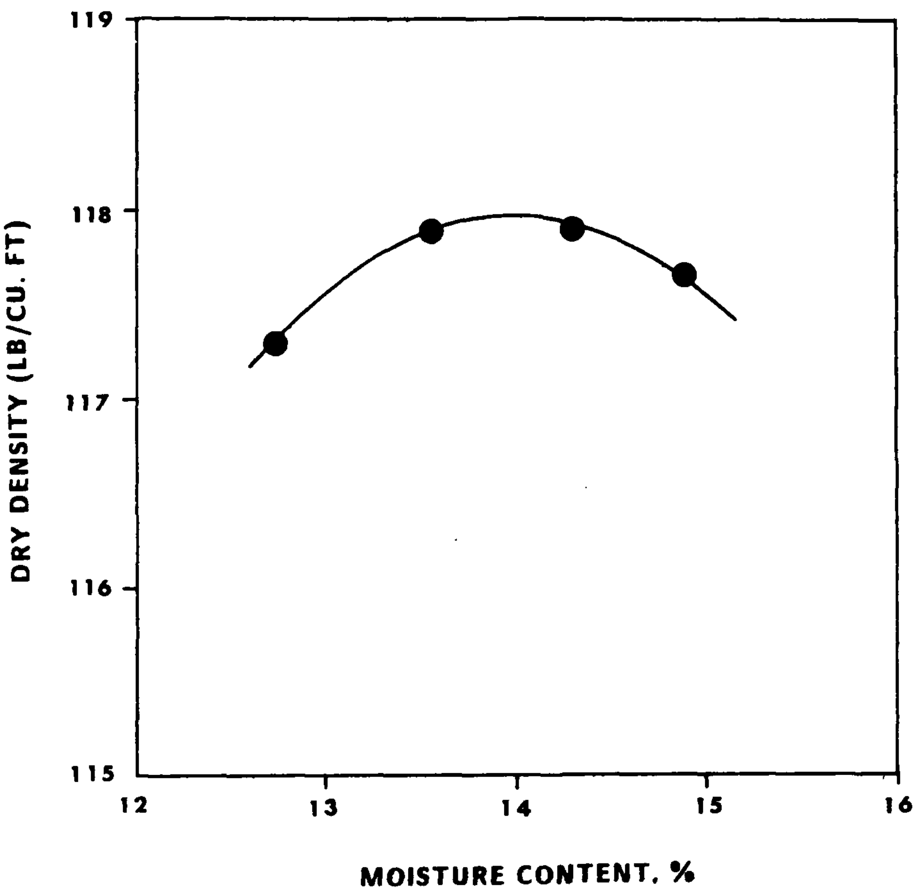


Figure 20 Moisture - density relationships for Simon, Li and Associates Soil.

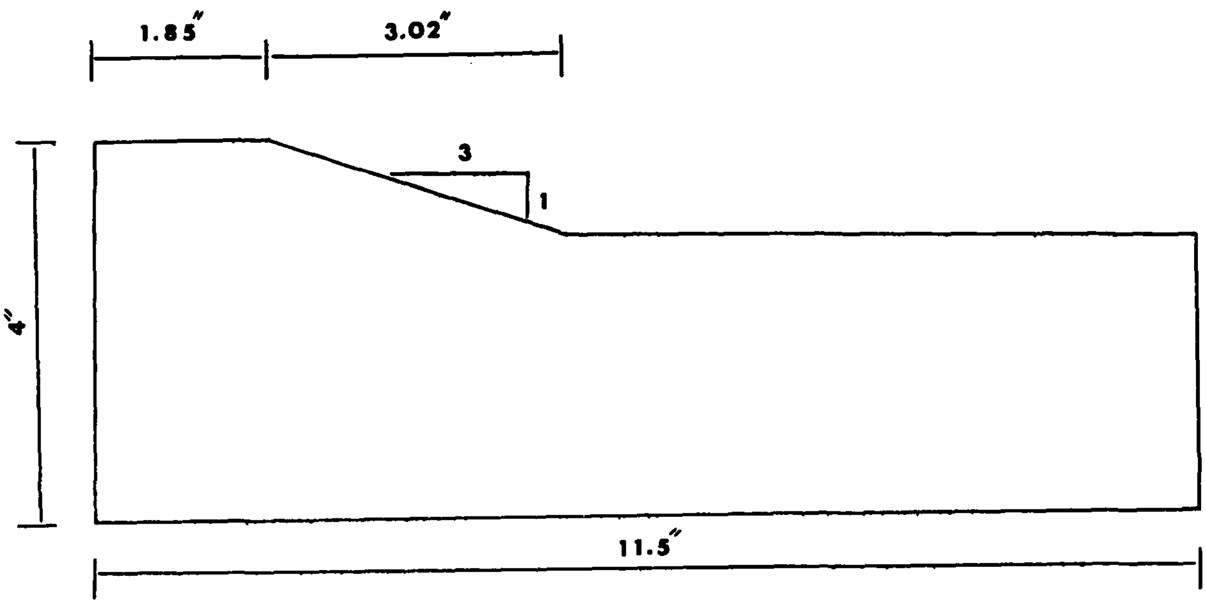


Figure 21 Configuration of the model of SLA embankment.

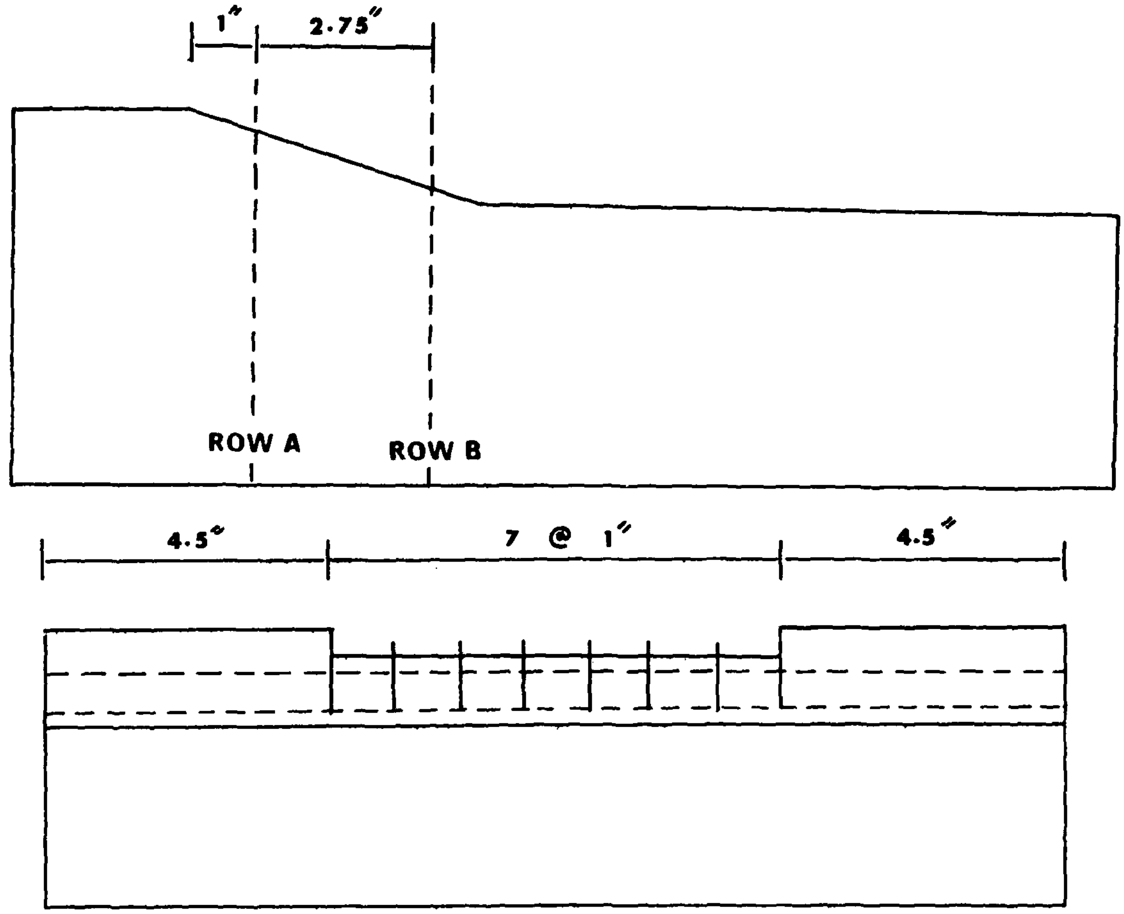


Figure 22 Manual measurement points for SLA embankment model.

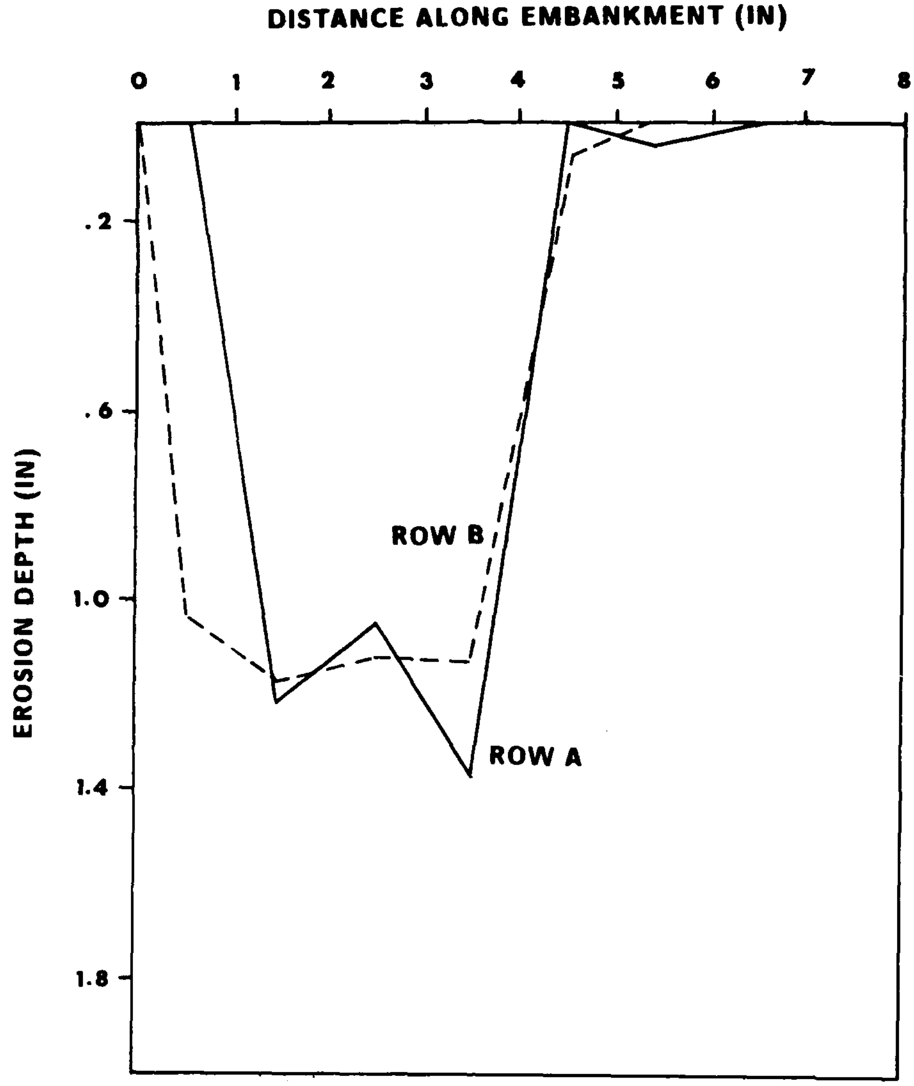


Figure 23 Erosion cross sections for SLA embankment model. (test 1).

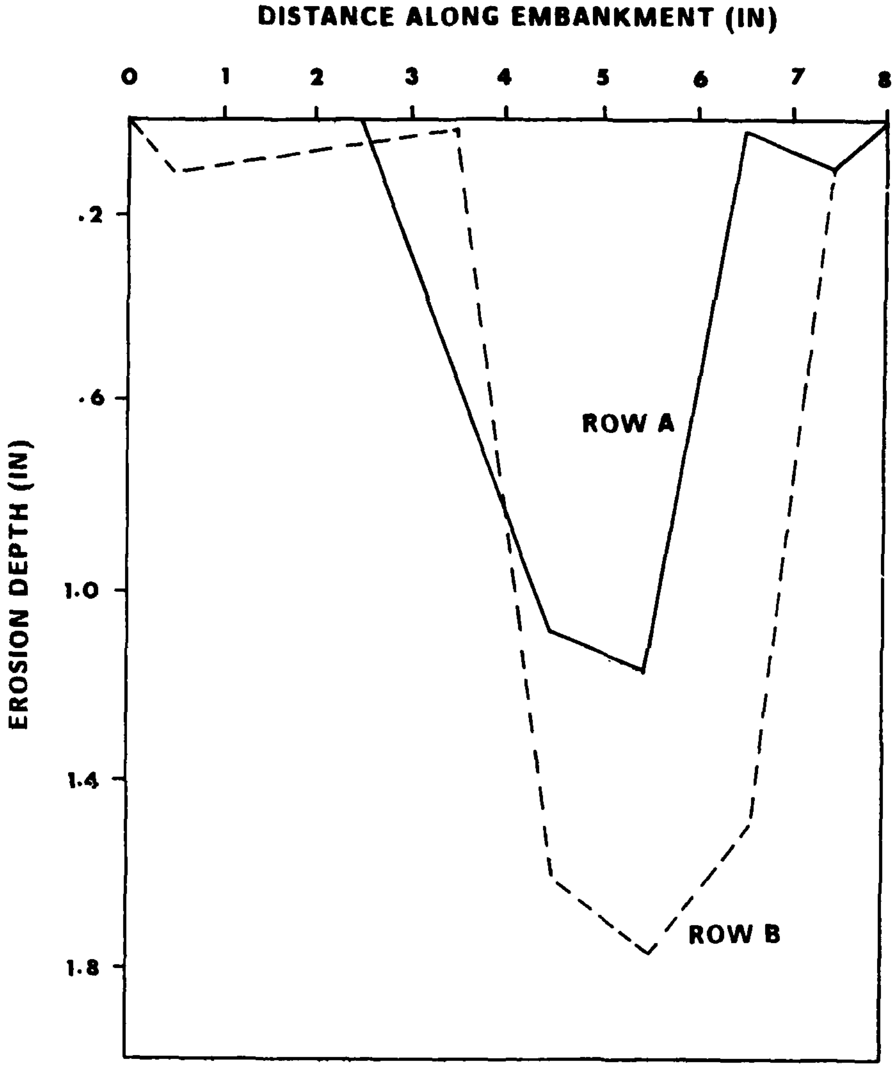


Figure 24 Erosion cross sections for SLA embankment model. (test 2).

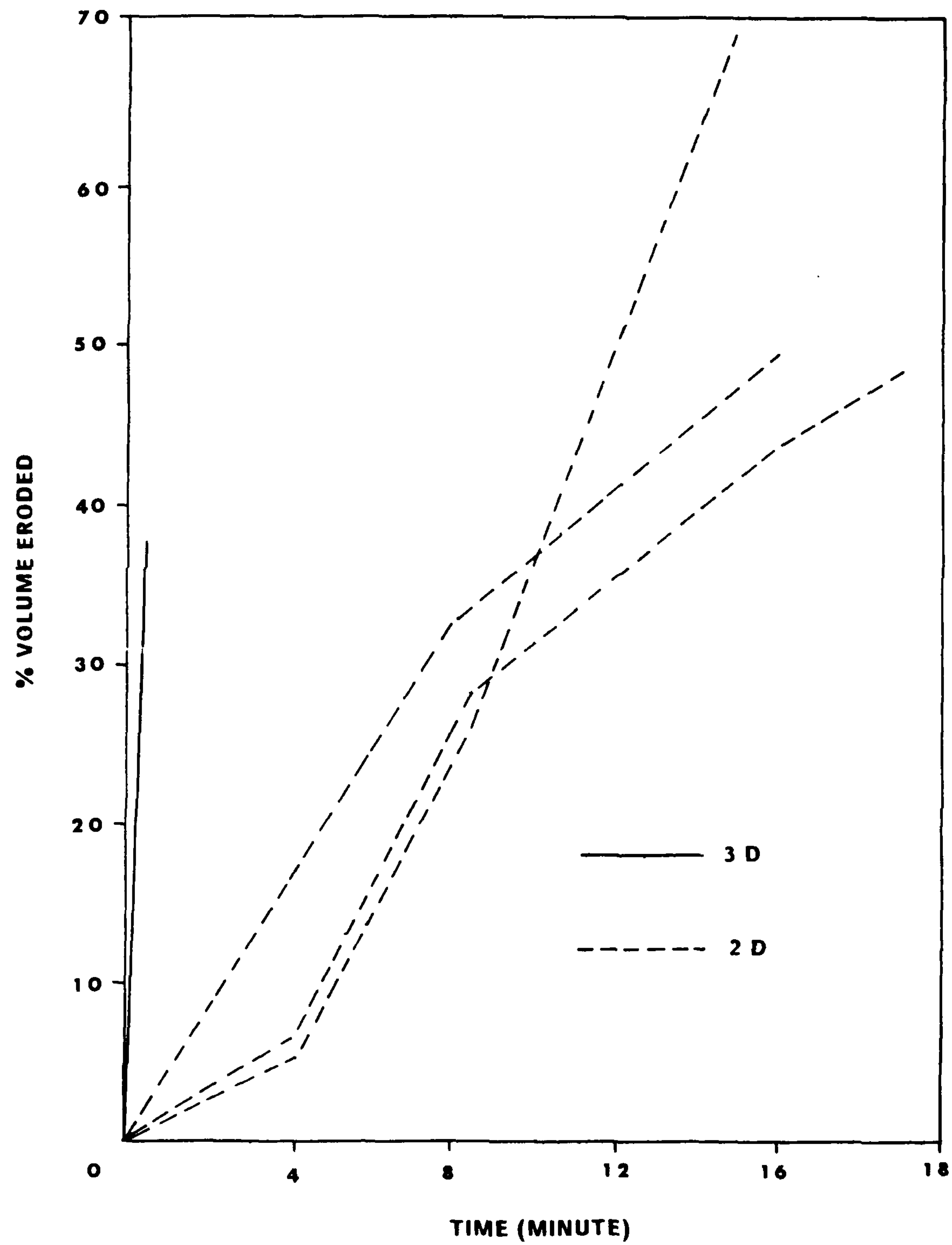
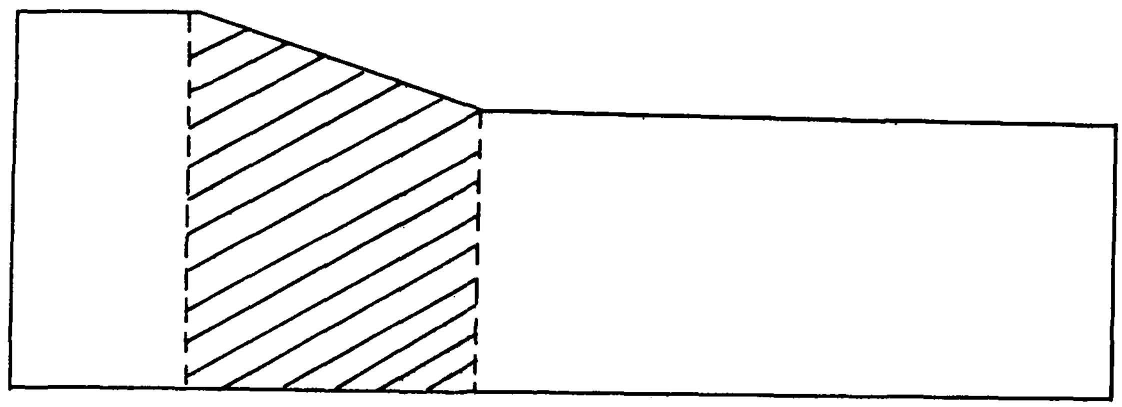


Figure 25 % Volume eroded vs. time SLA embankment. (2D vs. 3D).



(VOLUME IS EQUAL TO SHADED AREA TIMES EMBANKMENT WIDTH)

Figure 26 Volume available for erosion SLA embankment.

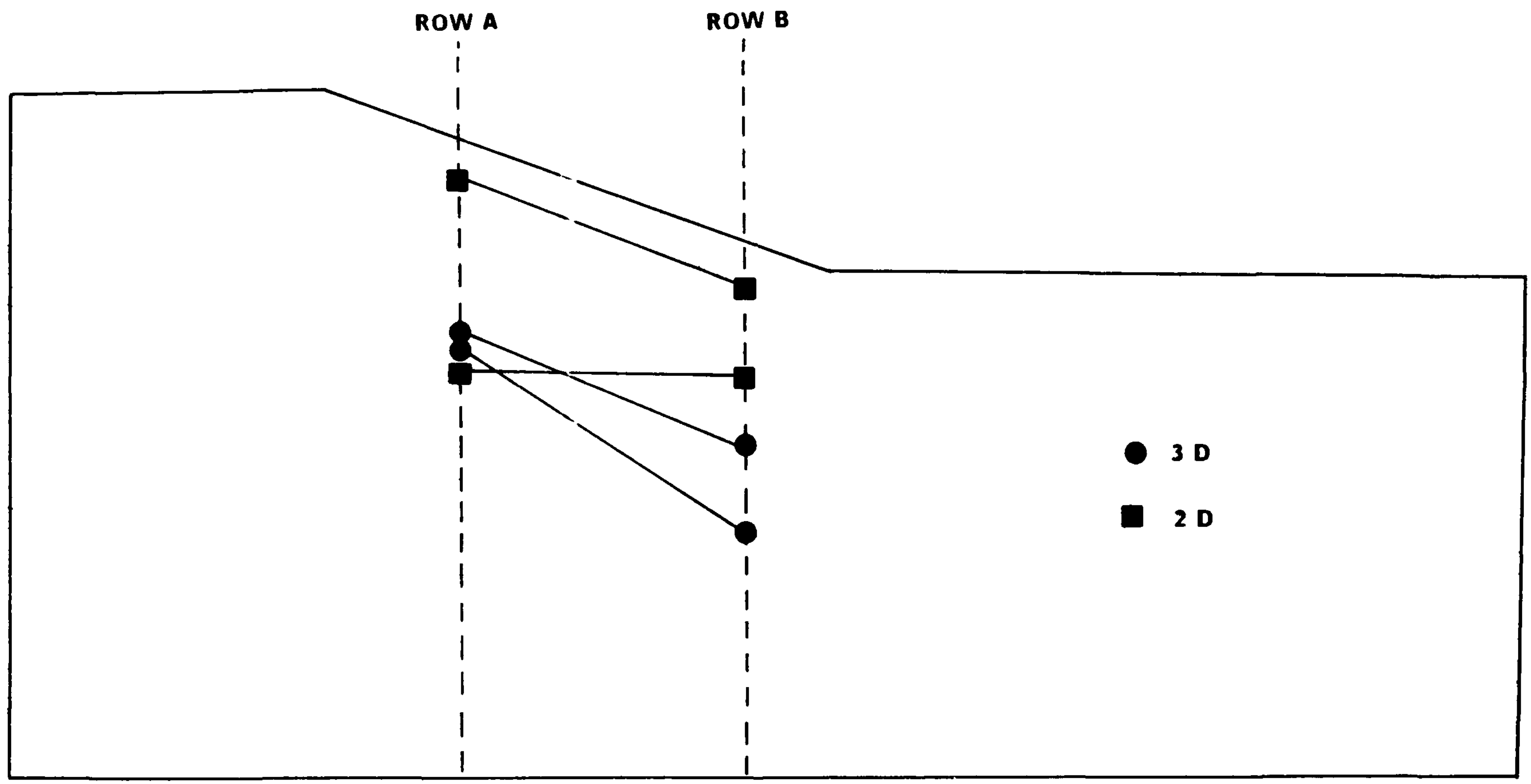


Figure 27 Erosion depth SLA embankment. (2D vs. 3D).

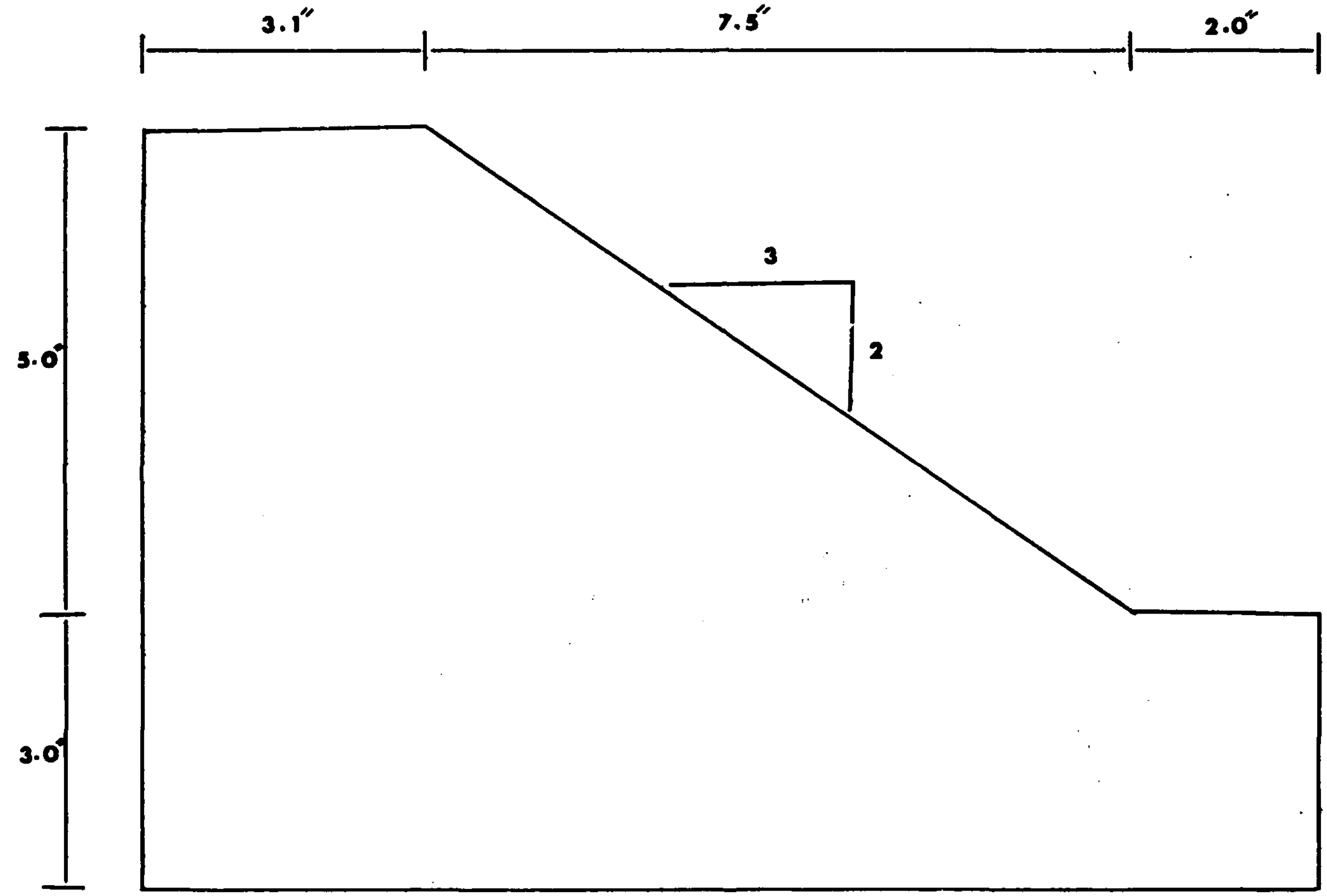


Figure 28 Configuration of crushed rock embankment for feasibility study in 2- dimensions.

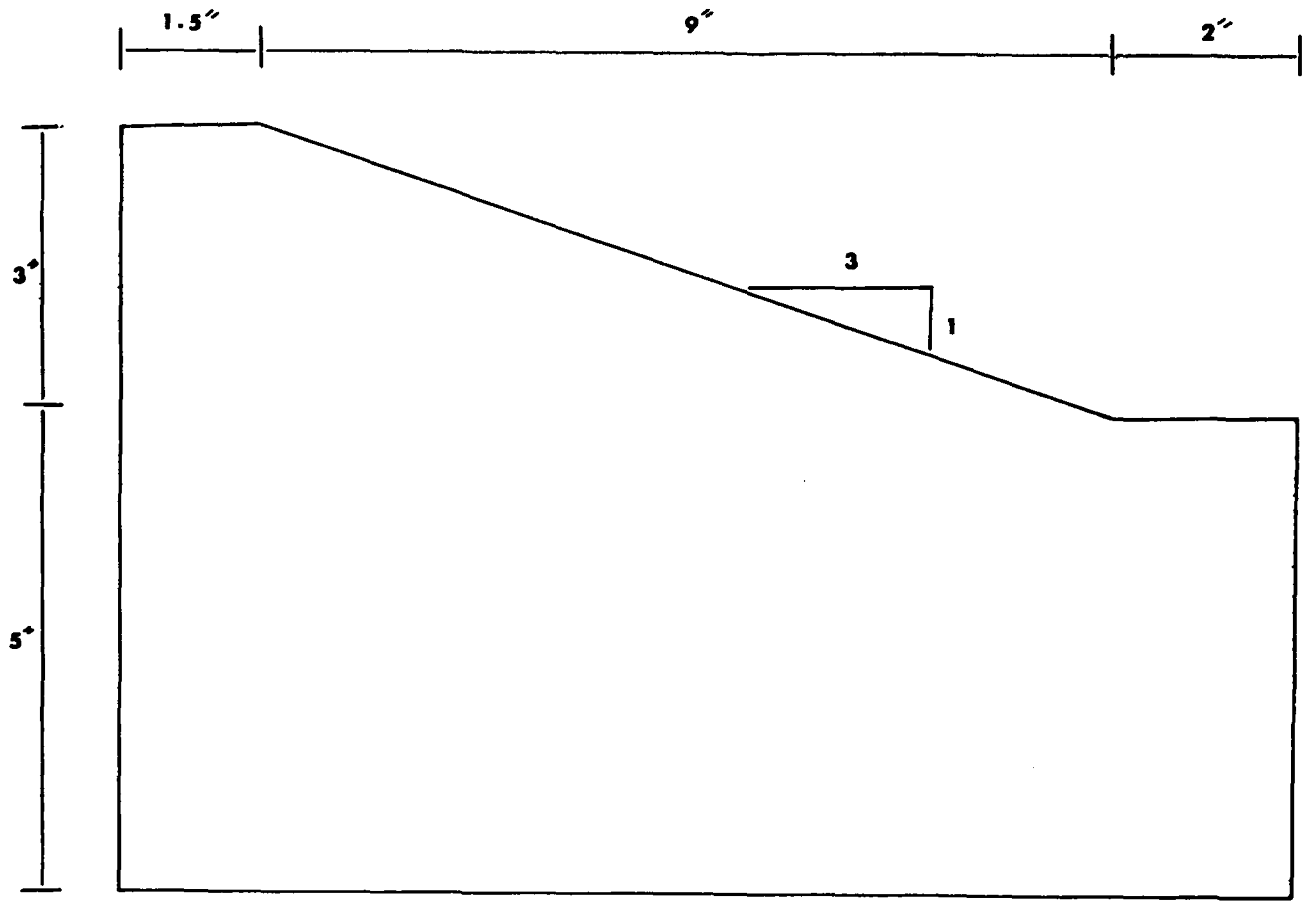


Figure 29 Configuration of clay embankment for feasibility study in 2-dimensions.

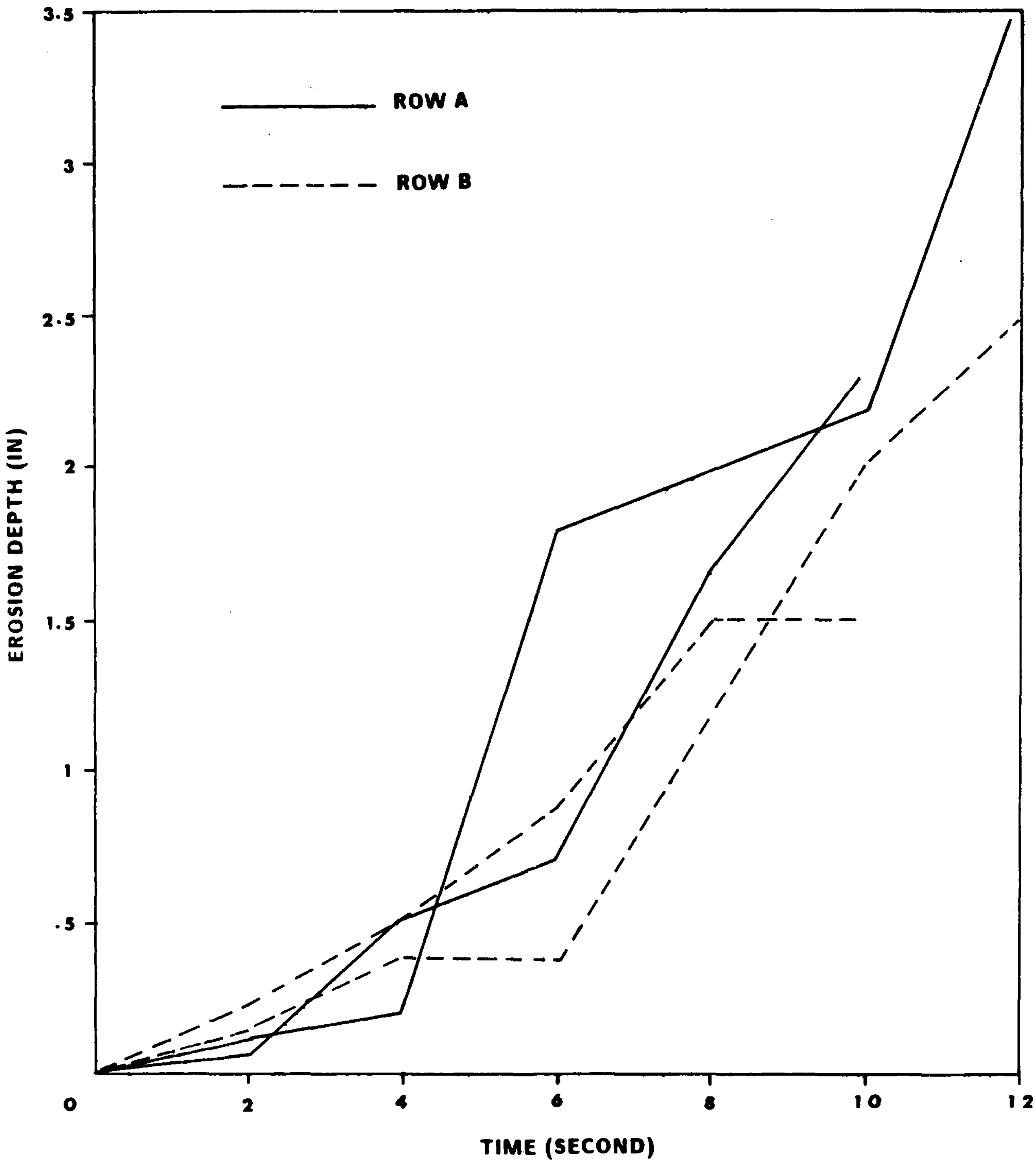


Figure 30 Erosion depth vs. time for crushed rock material.

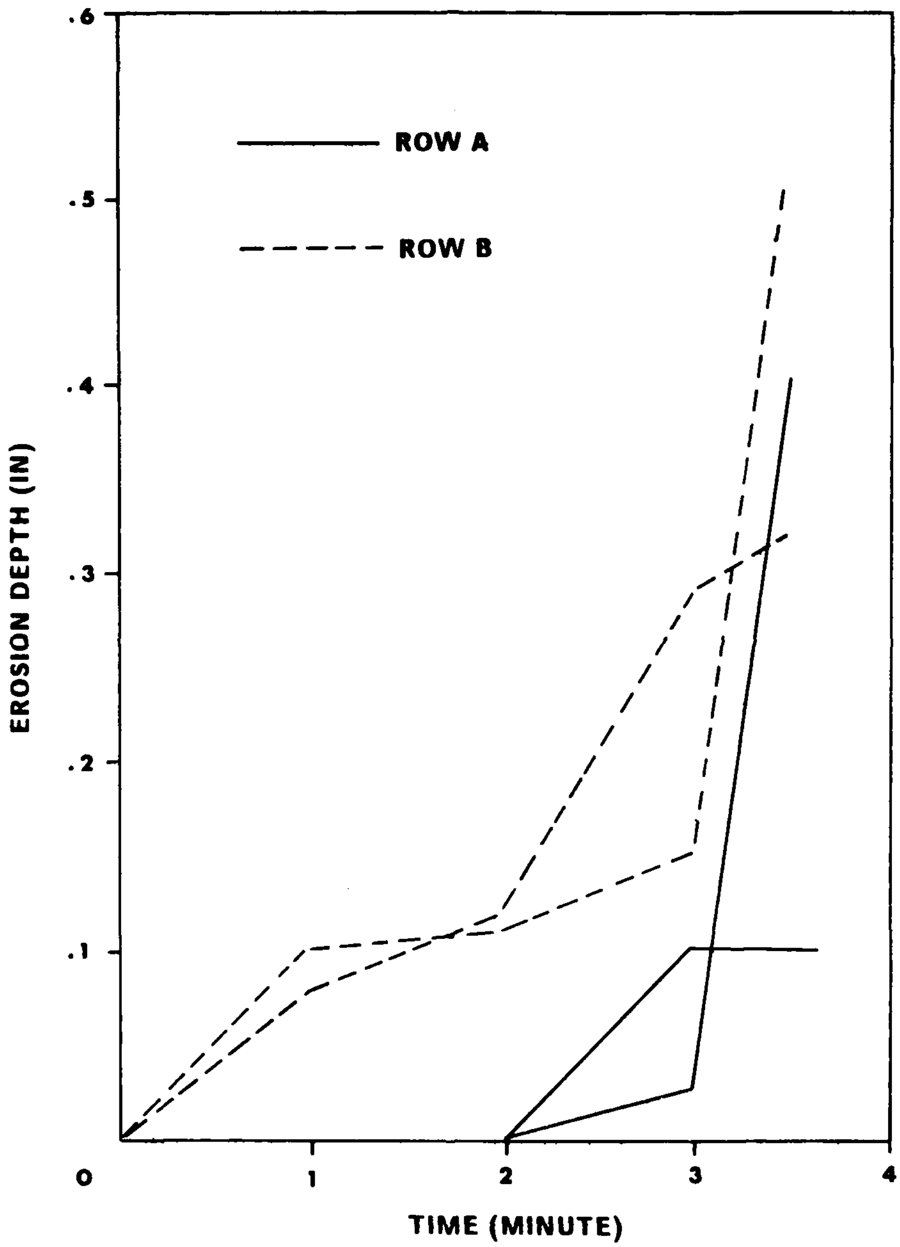


Figure 31 Erosion depth vs. time for clay material.

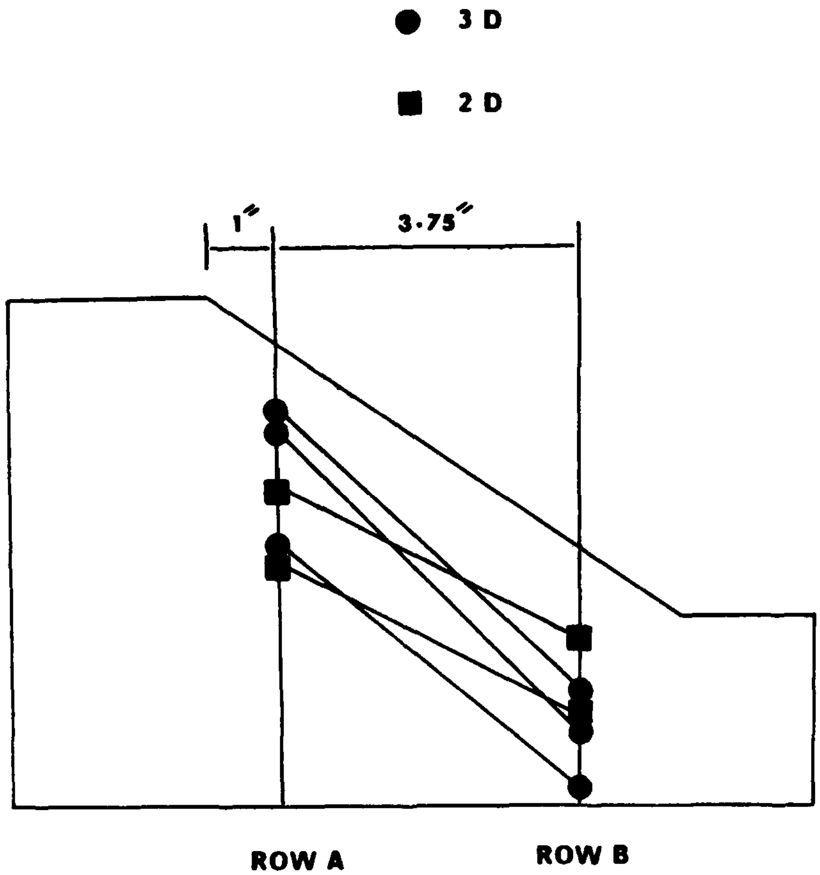


Figure 32 Comparison of the depth of erosion for the 2D and 3D crushed rock embankments.

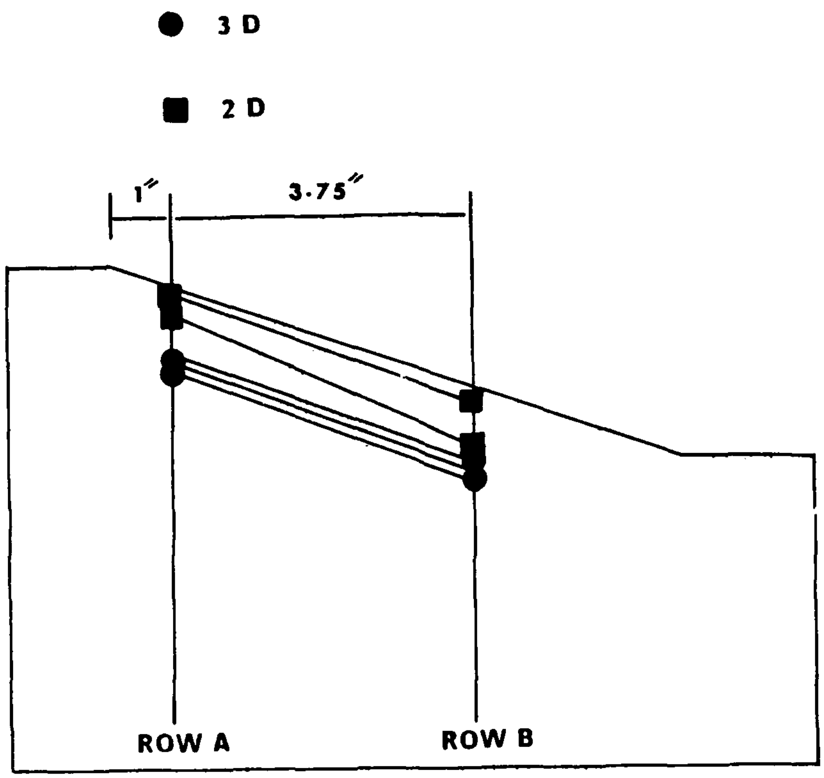


Figure 33 Comparison of the depth of erosion for the 2D and 3D clay embankments.

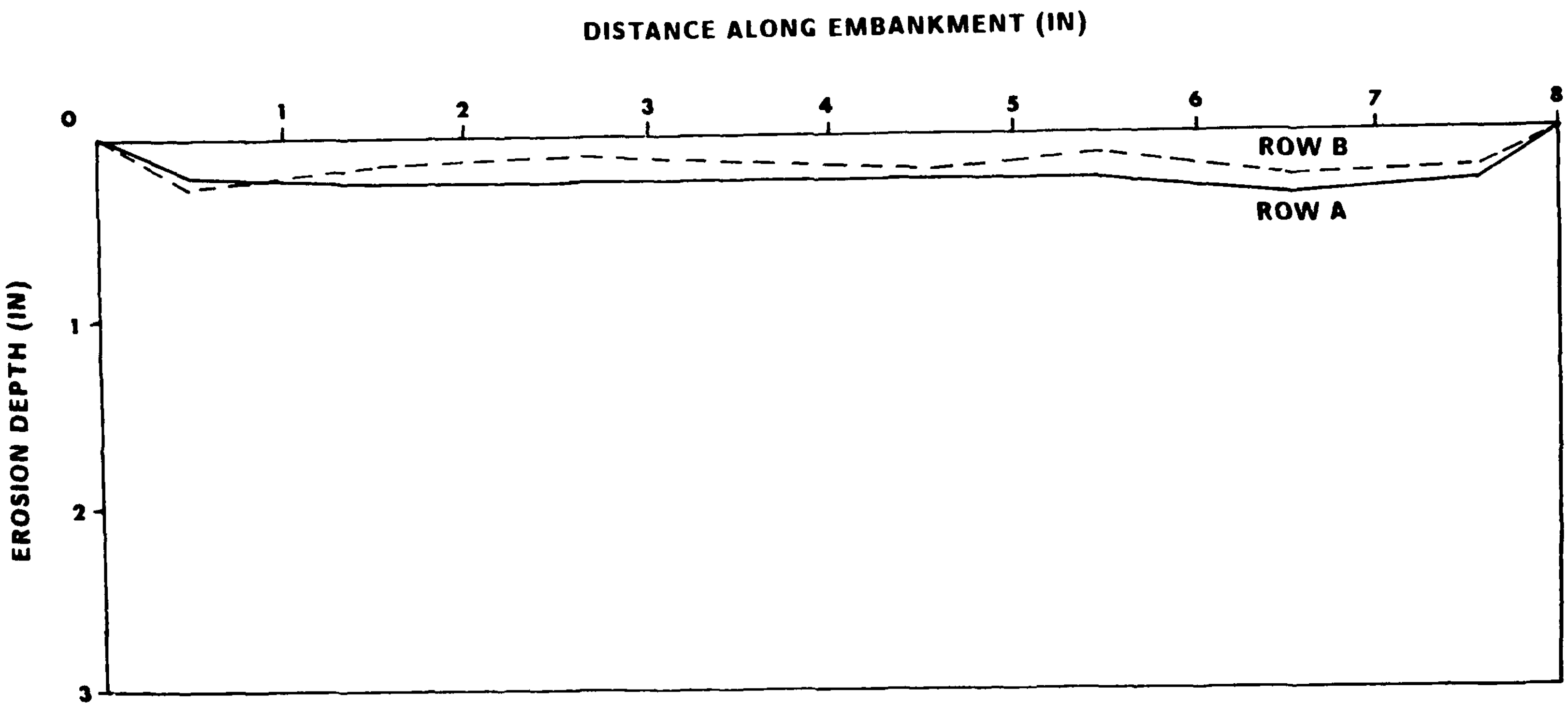


Figure 34a Erosion profile for clay embankment in 3- dimensions.

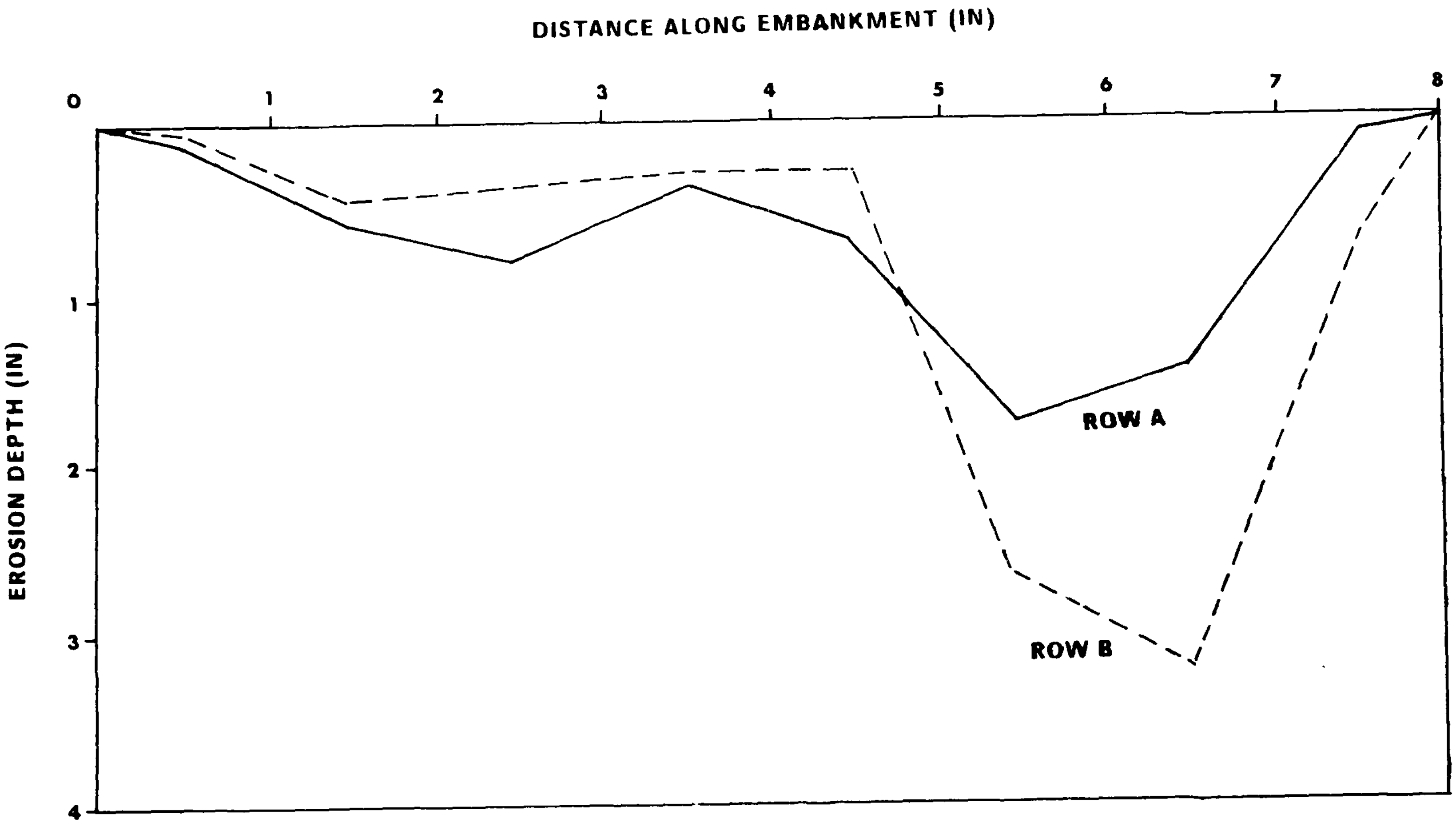
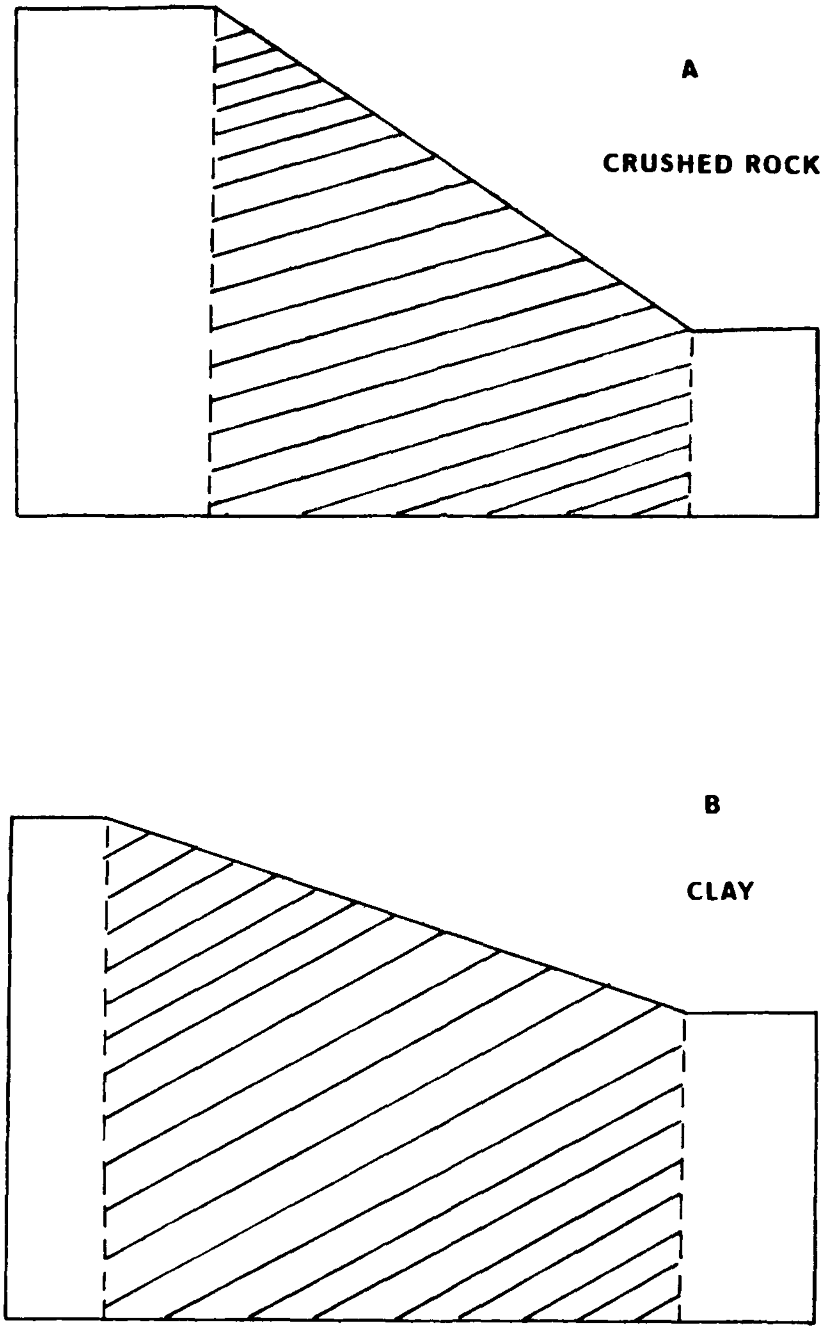


Figure 34b Erosion profile for crushed rock material in 3- dimensions.



(VOLUME IS EQUAL TO SHADED AREA TIMES SPILLWAY LENGTH)

Figure 35 Volume of material available for erosion in feasibility study in 3 dimensions.

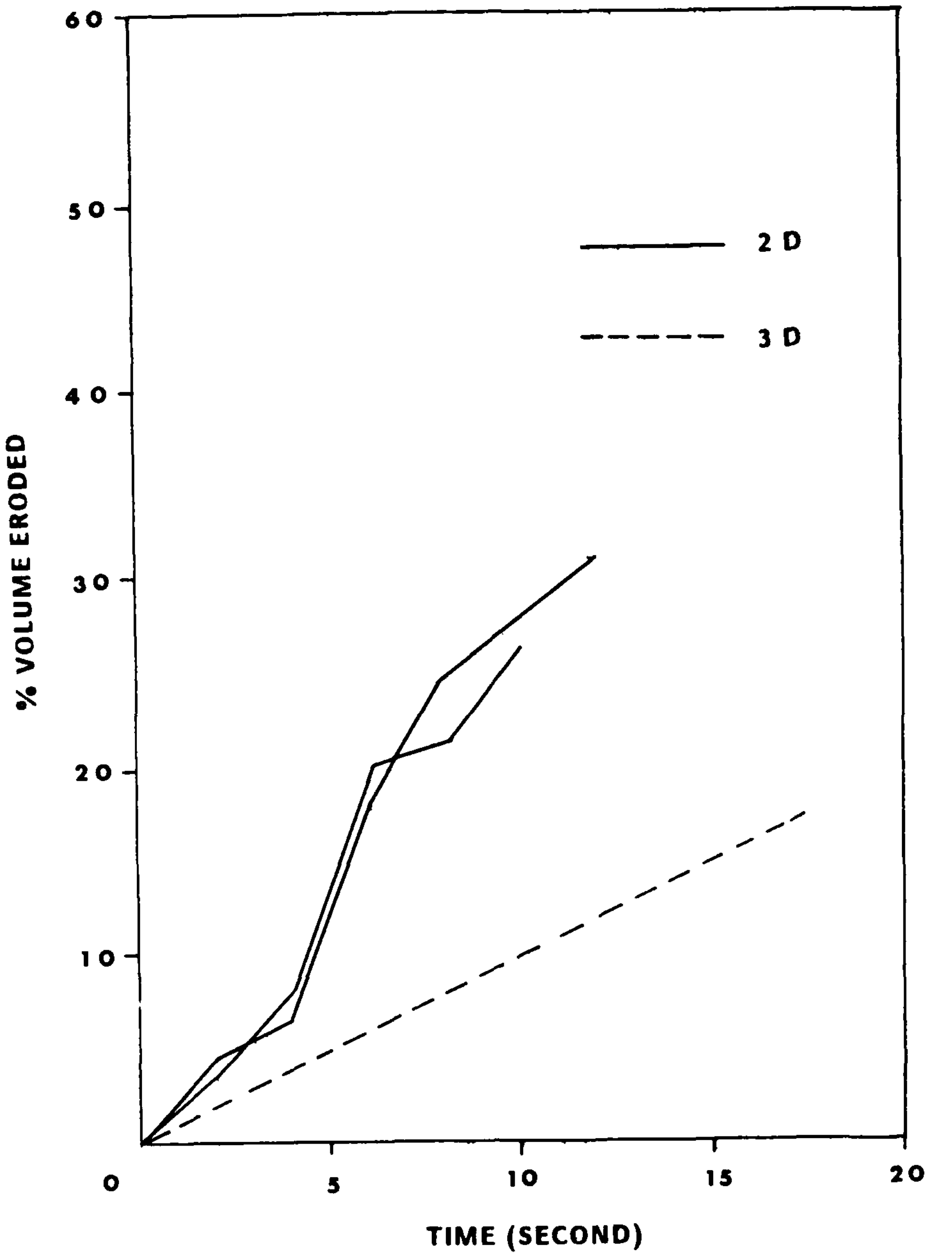


Figure 36 % Volume eroded vs. time for crushed rock material 2D vs. 3D.

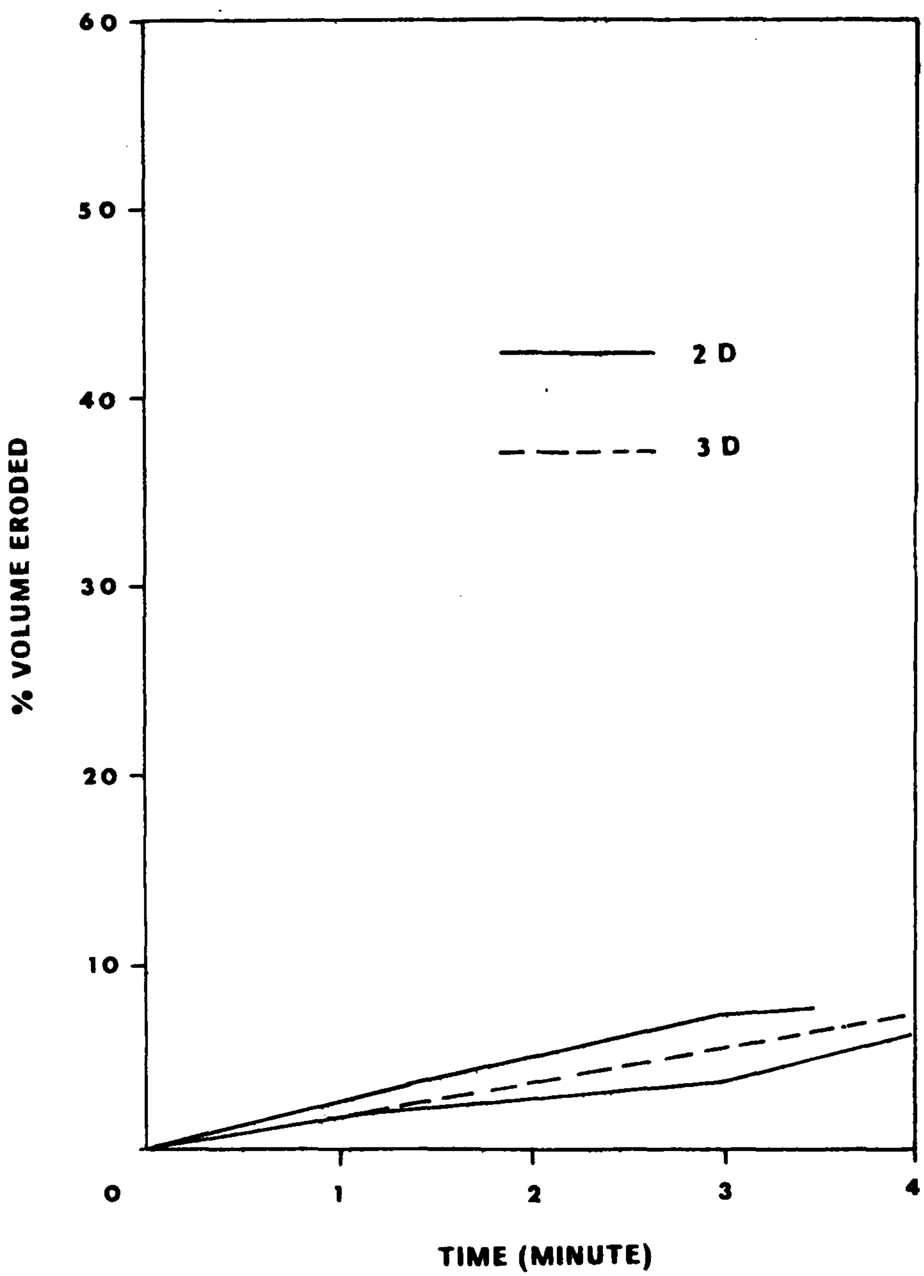


Figure 37 % Volume eroded vs. time for clay material 2D vs. 3D.

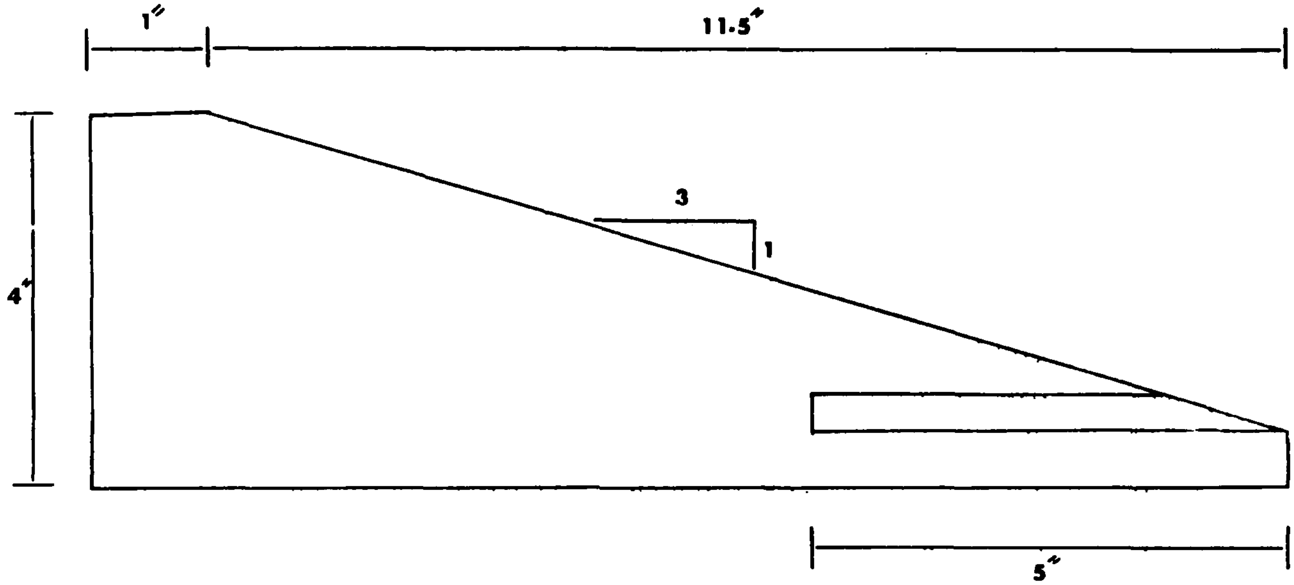


Figure 38 Configuration of major embankment with blanket drain.

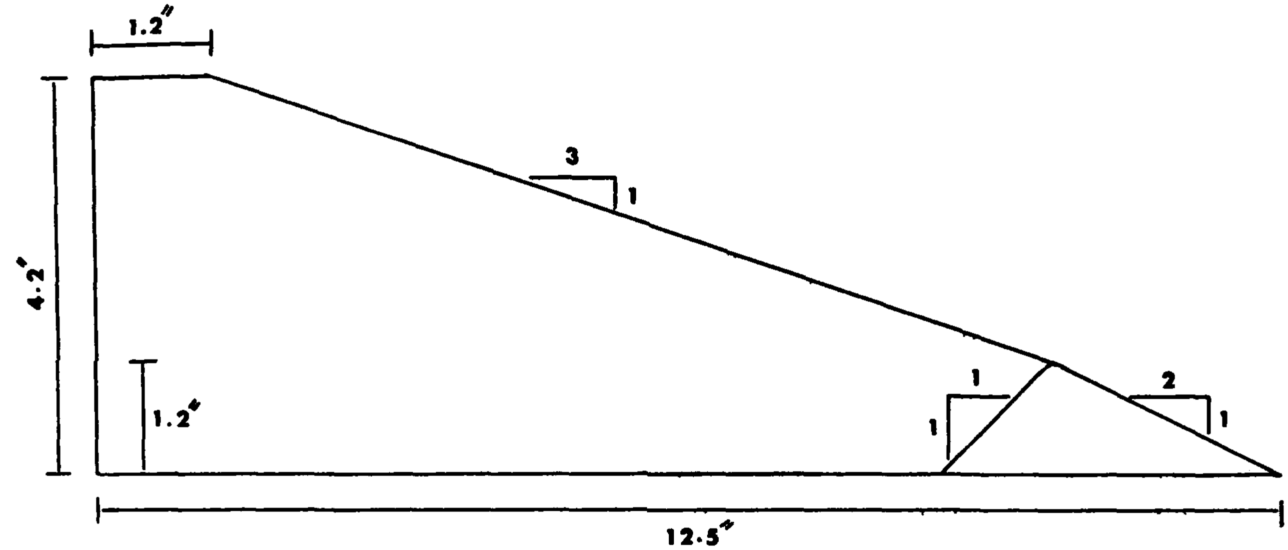


Figure 39 Configuration of major embankment with rock toe.

APPENDIX A

(1) The descriptions of Clarence Cannon and Bloomington Lake cofferdams which were used in the modeling were provided by Mr. S. Paul Miller of the Waterways Experiment station. Figures A1 and A2 show cross-sections of Clarence Cannon and Bloomington Lake cofferdams, respectively, from which the model cross-sections were built.

Clarence Cannon Cofferdam and Main Embankment

(2) General Description. Located near Hannibal, Missouri, on the Salt River, Clarence Cannon Dam is a concrete and earth-fill dam with a height of 138 ft. and crest length of 1940 ft. In July of 1981, the concrete section, nearly complete, was being used for river diversion while a 45-ft-high random earth cofferdam protected earth embankment construction. The partially complete main earth embankment, located about 900 ft. downstream of the cofferdam, had a crest elevation 15 ft. lower than the cofferdam crest. A horizontal drainage blanket underlying the downstream portion of the main embankment was connected to a vertical chimney drain which ran along the embankment centerline. At the time of overtopping, the top of the chimney drain was covered by 3 ft. of cohesive fill. The lower 37 ft. of the cofferdam had 1V:1H slopes, while the upper 8 ft. had been placed at angle of repose (1V:1H) and compacted by equipment traffic. When overtopping became imminent, a notch was made 400 ft. along the cofferdam from the left abutment. Notch elevation was 5 ft. below the cofferdam crest. Material in this area of the cofferdam was a stiff clay which would be resistant to erosion, and the location would divert most of the flow over the partially complete main embankment and away from the concrete structure.

(3) Overtopping Data: Date 27-30 July 1981; maximum depth - 3 ft; duration - 3 days.

(4) Damage. Overtopping flow was primarily contained in the notch area, with breaching in the center of the notch occurring after 20 hours of overtopping. Maximum depth was reached about this time. Limited flow over other areas of the cofferdam occurred after a few hours prior to notch breaching. The partially complete earth embankment, with very shallow slopes, allowed spread of the flow over a crest length of 850 ft. Apparently, the main embankment sustained very little damage until about 8 hours after notch breaching. At this time erosion reached the horizontal drainage blanket at the downstream main embankment toe and started removing the granular blanket materials from beneath the central portion of the downstream slope. This undercutting of the overlying cohesive fill by removal of the draining blanket continued for 8 to 10 ours until the chimney drain was reached. The chimney drain was removed from the left abutment to the concrete dam section without appreciable damage to the upstream portion of the main embankment. The chimney drain and 30 to 40 percent of the downstream embankment fill were lost.

(5) The cofferdam of cohesive soil (stiff clay) withstood 20 hours of overtopping at depths up to 5 ft. (upstream stage minus notch elevation). Tailwater, maintained to within 8 to 10 ft. of the notch crest, probably delayed removal of toe material and subsequent breaching. Control of the overtopping location was successful--the notch area probably provided the most durable and uniform performance compared to putting water over the whole cofferdam which had poorly compacted material in the upper 8 ft.

Bloomington Lake Diversion Cofferdam

(6) General Description. Sited on the North Branch of the Potomac River along the Maryland-Virginia border, two diversion structures for Bloomington Dam were overtopped in 1978. An upstream diversion dike, approximately 30 ft. high, provided initial diversion of the river while a diversion cofferdam was being constructed approximately 400 ft. downstream. The diversion cofferdam was a random earth (clayey sandy gravel) fill with an upstream impervious blanket and cutoff and a downstream rock toe filter. The earth fill specs limited maximum size to 8 in., with 20 percent or more passing the No. 200 sieve, and required 12-in. loose lifts compacted by a 50-ton rubber tired roller. With upstream slopes of 1V:3H and downstream slopes of 1V:2..5H, the cofferdam's upstream 30 ft. of crest had been completed to an elevation 4 ft above the dikecrest (el 1270) while downstream 70 ft. was at a level 2 ft. below dike crest.

(7) Overtopping Data: Date - 3 July 1978; maximum depth - 5 ft; duration - unknown (10 hours before breaching).

(8) Damage. Flood waters overtopped the dike for about 25 minutes before the cofferdam was overtopped. After 4 hours of overtopping, erosion began adjacent to the right abutment which was lower than the rest of the embankment. The erosion continued through the cofferdam until full breaching occurred approximately 10 hours after overtopping started. The breach was 70 ft. wide and eroded to foundation level with vertical faces. The remaining portion of the embankment's downstream slopes had vertical faces 15 to 20 ft. high. Breaching of the diversion dike followed cofferdam breaching by 10 minutes.

(9) Breaching, which occurred at the lower part of the embankment and at the more abrupt abutment, took 4 hours to start and 10 hours to complete. The relatively long crest (100 ft +) increased breaching time. The more easily eroded rock toe probably accelerated undercutting of the downstream slope and subsequent breaching. The random earth, fine-grained compacted embankment should have withstood breaching longer, especially with such a long crest length. Other factors could have been concentration of flow at the right abutment and poor rock abutment-fill contact. Loss of tailwater and saturation probably prompted quick breaching of the upstream dike after cofferdam breaching. If the left portion of the embankment had been lower and received the major portion of flow, breaching might have been delayed further because of the shallow sloped left abutment. This would have provided a large earth mass for erosion.

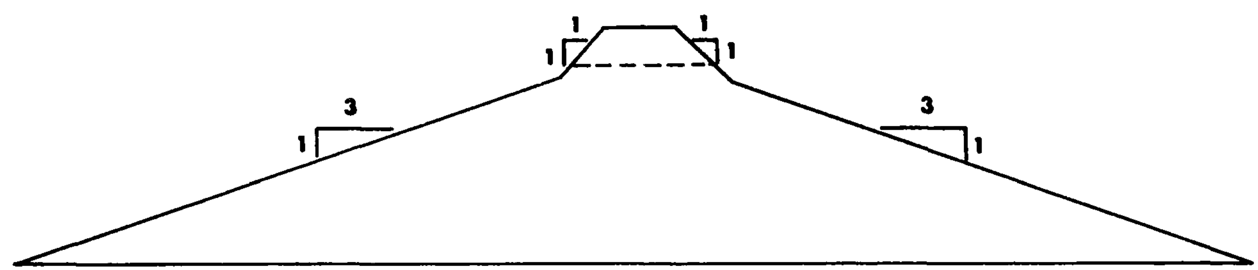


Figure A.1. Cross section of Clarence Cannon Cofferdam scale: 1 in.= 300 ft.

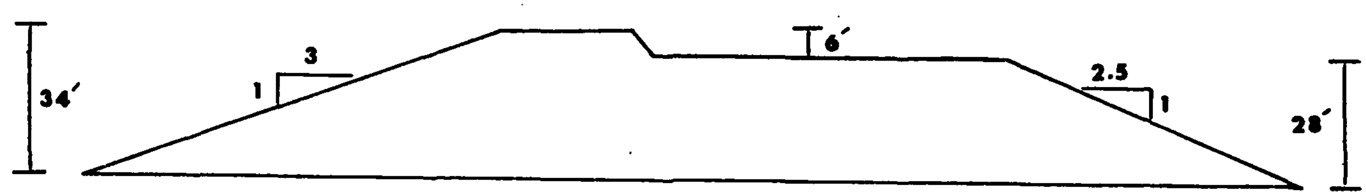


Figure A.2. Cross section of Bloomington Lake Cofferdam.