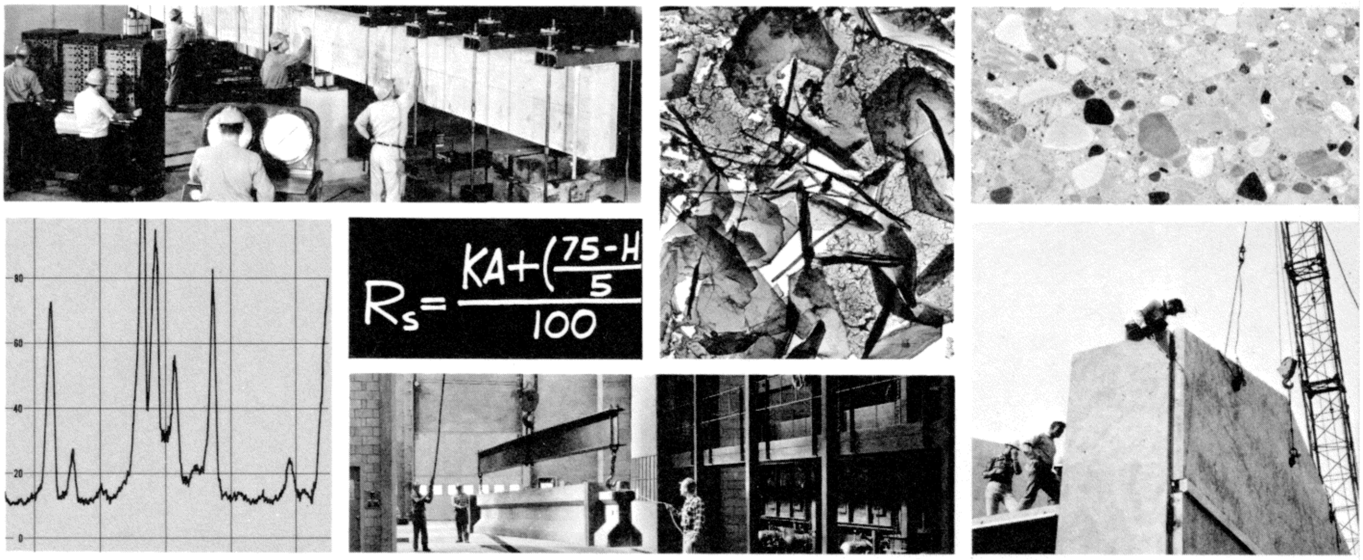
RESEARCH AND DEVELOPMENT BULLETIN



Dam Construction and Facing with Soil-Cement

By P. J. Nussbaum and B. E. Colley

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SYNOPSIS

This paper reports on laboratory tests to obtain design factors for the application of soil-cement in earth dams as slope protection, impermeable barriers, and as an erosion-resistant surface in areas of rapid flow. The stability of embankments constructed with cement-stabilized soils is also considered.

Severity of climatic exposure governs the amount of cement required to stabilize soil used for slope protection. Current practice of increasing the cement content 2 percentage points above that required by standard tests is desirable when the facing in the splash zone is exposed to freezing. In milder exposures, stabilization with the minimum amount of cement required to make soil-cement may be considered. When slope protection is exposed to rapid flow carrying stones or debris, the higher cement content and at least 20 percent gravel should be used in the soil-cement.

Soil-cement slope protection constructed in stepped layers will lessen wave run-up as compared to run-up on smooth embankment slopes. Methods to compute wave height and run-up are presented. Increases in slope steepness result in higher run-up.

Seepage through dams can be reduced by construction of soil-cement upstream blankets, core walls, or cutoff trenches. Seepage flow in the direction perpendicular to layering due to construction is considerably less than flow parallel to layering, although the use of a thin layer of cement grout at the interface of the compaction planes will reduce seepage significantly.

Key Words: cement content, cutoff walls, earth dams, erosion control, permeability, seepage, slope protection, soil-cement, triaxial shear, wave run-up.

INTRODUCTION

Cement has been mixed with soil to improve the engineering properties of pavement bases and subbases for many years. When the cement, soil, and water are proportioned to produce a hardened material meeting freeze-thaw and brush-loss criteria, the product is called soil-cement. It is used in the construction of base courses and subbases for streets, roads, highways, shoulders, airfield pavements, and parking areas to provide a firm, durable pavement layer with considerable bearing strength. Catton(1)** has described the use of soil-cement in road construction over the years, and Felt(2) has developed test methods and determined property parameters for a range of soil-cement mixtures.

The superior stability of soil-cement over natural soils with respect to erosion, permeability, and shear strength is desirable for earth dams. Therefore, it was logical that soil-cement would be considered for dam construction.

The first use of soil-cement as slope protection for earth dams was a test section on the south bank of Bonny Reservoir near Hale, Colo., in 1951. At this site, the material was subjected to severe exposure conditions and the adaptability of cement-stabilized slope protection for earth dams was established. A view of the stepped soil-cement slope protection at Bonny Reservoir is shown in Fig. 1. Details of construction procedures for soil-cement slope protection are provided in a PCA publication.(3)

Due in part to this successful application, the U.S. Bureau of Reclamation authorized use of soil-cement in 1961 for the Merritt and Cheney Dams.(4) Growing acceptance of this material is indicated by the fact that, to date, soil-cement slope protection has been used in more than 30 earth dams by both public and private agencies. It is being used also to provide erosion-resistant surfacings for a variety of earth slopes such as canal linings and shore protection, and it has been found effective in other parts of dams to provide impervious cutoff trenches and cores to reduce seepage flow.

Objectives of Test Program

The purpose of this investigation was to supplement existing data on the field performance of soil-cement in dam construction with laboratory data. The specific objectives were:

1. To determine the effect of various cement contents on the performance of cement-stabilized soils under exposure conditions similar to those encountered by a dam or canal.
2. To obtain wave run-up factors for soil-cement slope protection.
3. To investigate seepage flow for dams incorporating soil-cement as slope protection or as a core wall.
4. To obtain triaxial shear data for use in considering the construction of an earth dam in which all the soils are stabilized with cement.

SOILS AND CEMENT REQUIREMENTS

Properties of the soils used in the investigation are described. In addition, test procedures and data are presented to indicate methods used to evaluate cement requirements for the various types of exposure that might be encountered in dams or canals.

Soil Materials

A wide range of granular soils has been used for slope protection. Amounts passing a No. 40 mesh sieve have varied from 22 to 95 percent, and amounts passing a 200-mesh sieve from 5 to 35 percent.

Three soils were used in this investigation. The A-1-b and A-2-4 soils(5) are representative of the types most frequently used for past construction and are considered ideal for stabilization with cement. In addition, an A-4 soil was used to determine if a fine-grained soil could be used for slope protection in locations where more suitable materials are not

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**Superscript numbers in parentheses designate references on page 11.

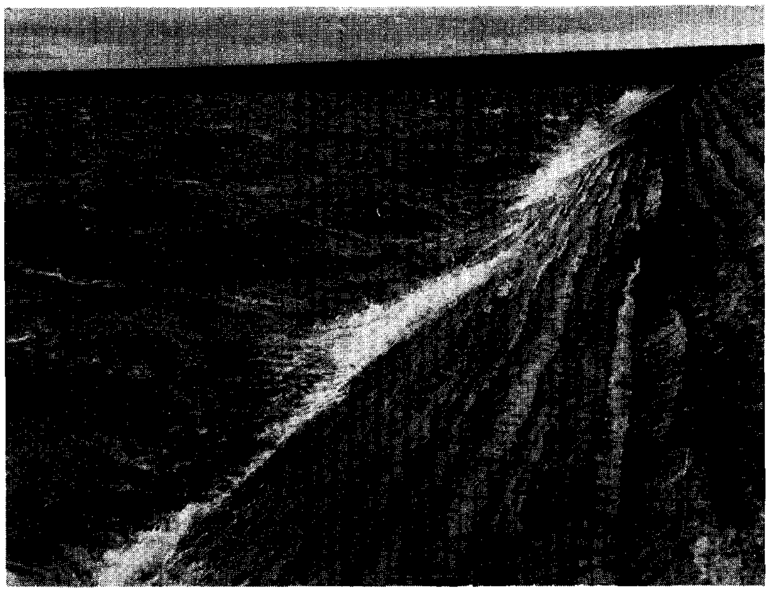

Fig. 1. Soil-cement slope protection.

TABLE 1. Soil Data

AASHO	Standard dry density,(8) pcf	Optimum moisture content, %	LL, %	PI, %	Percent passing 0.074 mm
A-2-4	124.5	10.5	19	4	22
A-1-b	138.5	7.8	NP	NP	10
A-4	114.5	13.2	NP	NP	74

NP (nonplastic)

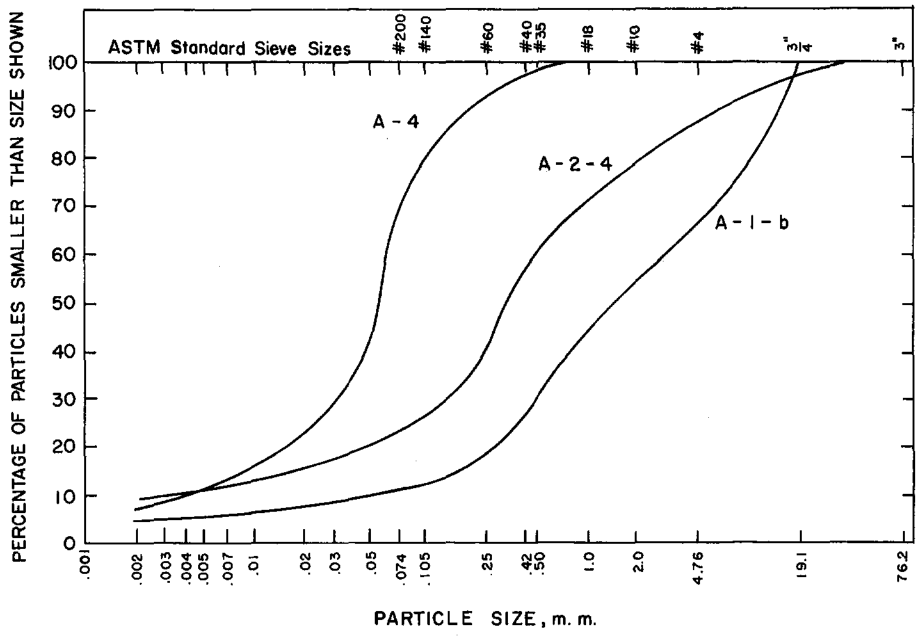

Fig. 2. Grain-size accumulation curve.

available and economics would permit the somewhat greater amounts of cement required for freeze-thaw durability and erosion resistance. Some of the characteristics of the soils included in this study are shown in Table 1 and grain-size curves are shown in Fig. 2.

Cement Requirements for Zones of Exposure

Cement requirements for slope protection have varied with the type of soil being stabilized. For some projects, the requirement has been established as 2 percentage points greater than the percentage necessary to meet ASTM(6) wet-dry, freeze-thaw tests plus the Portland Cement Association(7) brush-loss criteria as used for highway applications. On other projects, cement content has been based on the amount of cement required to give the soil being considered the same durability as the soils used at the Bonny Test Section. Neither approach considers the severity of exposure for a given dam location nor the variations of exposures for different portions of a dam facing.

A more economical design may be obtained if the face is constructed in zones, with materials treated with cement as needed for each exposure condition. For this procedure, the face of the dam is divided into three exposure zones: (1) the lower portion below the minimum pool elevation that is constantly exposed to water and only very rarely to freeze-thaw cycles, (2) the zone between minimum pool elevation and the normal splash zone that is exposed to severe changes of freezing and thawing in the presence of water, and (3) the topmost portion that is generally in a dry state but is exposed to the climatic environment. It is evident that zone 2 exposure is most severe, zone 3 intermediate, and zone 1 least affected.

To determine the required amount of cement, laboratory tests were made simulating the severity of exposure for each zone. Specimens were compacted in two lifts at standard density(8) and optimum moisture content. The top lift was placed 2 hours after compaction of the bottom lift. This procedure was followed to obtain a separation plane with a partial bond similar to the type encountered in field construction. Treatments at the interface included scarification of the bottom lift prior to placement of the top, spreading a cement grout on the bottom lift, and a wet or dry condition at the surface of the bottom lift. In addition, a study was made of the effects of delay times of 24 to 60 hours between placement of the top and bottom lifts.

After fabrication, specimens were cured for 7 days in a fog room and then subjected to exposure conditions. It should be noted that the 7-day curing period is considerably less than the time that might be expected in practice between construction and severe exposure conditions. For accelerated exposure, the cyclic treatment for zone 1 consisted of 17 hours of drying at 72 F and a relative humidity of 50 percent followed by 7 hours of exposure to a water jet from an 1/8-in.-diameter orifice at a pressure of 27 psi. This was computed to be a very severe condition, equivalent to a head of 62 ft of water or to the impact of a 16-ft-high wave. The water-jet erosion test was used to simulate the erosion forces of waves on the upstream dam facing. The high water pressure was selected to accelerate the laboratory testing. Zone 2 exposure consisted of 17 hours of freezing at minus 20 F and 7 hours of water-jet exposure. Zone 3 exposure was simulated by the standard freeze-thaw and brush-loss tests for soil-cement. All tests were repeated for 12 cycles. The weight loss in percent determined at the end of test is shown in Fig. 3 for the A-2-4 soil.

For an allowable weight loss of 14 percent and a dry interface, the required amounts of cement as determined from best fit lines through the data for zones 1, 2, and 3, respectively, were about 0.7, 6, and 4 percent by weight. As the zone 2 requirement was 2 percent above that for zone 3 and because of the impracticality of effective stabilization with less than 2 percent cement, a practical guide for cement content for zone 3 is the amount of cement determined from standard freeze-thaw and brush-loss tests; for zone 2, this amount plus 2 percentage points and for zone 1, 2 percent less than that required from standard tests, but not less than 2 percent. Similar tests were made using the A-1-b and A-4 soils. Results from the A-1-b confirmed the finding that for granular soils a cement content 2 percent above that determined by standard tests will meet requirements for the severe exposure of zone 2. In addition, the results for tests simulating zones 1 and 3 confirmed the conclusions shown for the A-2-4 soil.

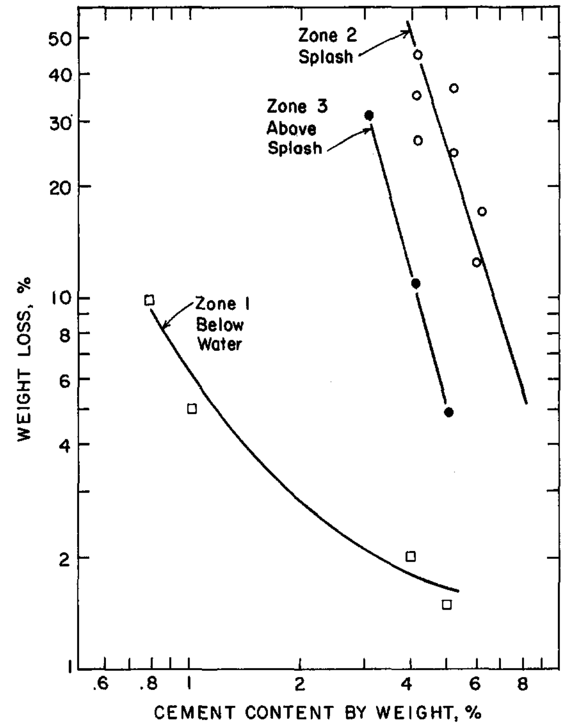

Fig. 3. Cement requirements for various exposures.

The cement requirements for the three zones, when using the three soils used in this investigation, are tabulated in Table 2. Based on these data, the current practice of increasing the amount of cement 2 percentage points above that required by standard tests appears warranted; however, it may be possible to achieve economy by reducing cement requirements currently used in zones 1 and 3.

The severity of the water-jet exposure was confirmed by the fact that with the A-4 soil, the water jet bored a hole through the specimen. Thus, this soil would not be recommended for use in very severe exposure conditions without modification. One type of modification of an A-4 soil by the addition of coarse material is described in the discussion of erosion resulting from streams carrying debris.

When the water jet was directed perpendicular to the compaction plane, no important differences in losses due to erosion were observed on any of the materials for various delay times between placing successive lifts of soil-cement. However, when the jet was directed on the interface and parallel to it, a significant reduction of erosion was achieved when a cement grout was placed on the bottom lift immediately prior to construction of the top lift even when delay time between successive layers of construction was 60 hours. For example, a grout layer placed at the interface of the A-2-4 soil reduced erosion loss from 14 to about 4 percent.

TABLE 2. Cement Requirements

AASHO	Cement, % by weight		
	Zone 1	Zone 2	Zone 3*
A-2-4	2.0	6.0	4.0
A-1-b	2.0	5.0	3.0
A-4	5.5	9.5	7.5

*Standard freeze-thaw testing with PCA brush-loss criteria.

Resistance to Abrasion by Water-Borne Particles

Cement-stabilized embankments are sometimes used in locations where swift-flowing, debris-laden streams abrade the slope-protecting materials. The cement and gradation requirements necessary to resist this erosion were investigated by exposing test specimens to flows of water carrying 1/8- to 1/4-in.-size gravel. Speci-

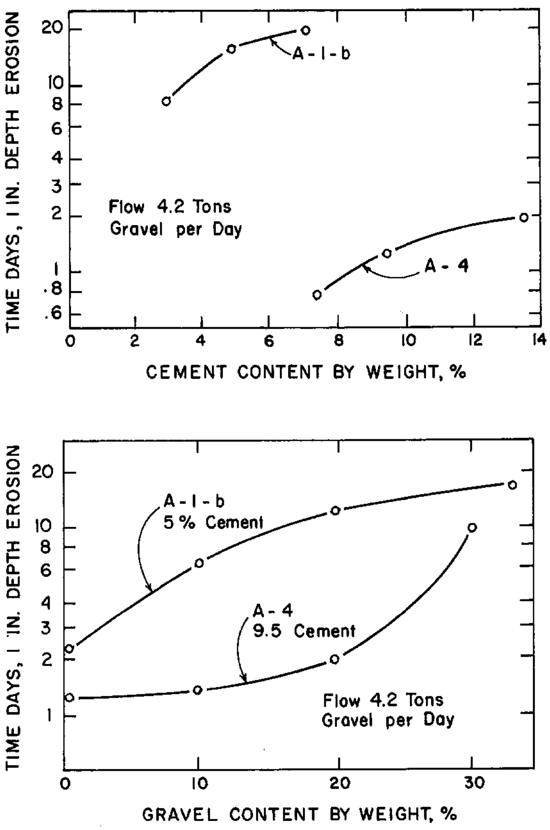

Fig. 4. Erosion resistance of stabilized soils.

mens were subjected to the abrasion test after 7 days of fog-curing. Approximately 8,000 gal of water per day carrying 4.2 tons of gravel flowed at 3.8 ft per second in a 1-in.-wide by ½-in.-deep stream across a specimen. To achieve maximum abrasive action, the flow rate was low and the stream of water was directed so that the gravel was carried in the lower portion of the flow.

Results from tests simulating stream bank erosion are shown in Fig. 4 as plots of time for 1-in. depth of erosion. The upper curves show the effect of cement content on abrasion resistance of the A-1-b and A-4 soils. The lower curves show the effect of the gravel component (plus ¼-in.-size component of the soil-cement materials). It is seen from Fig. 4 that the erosion resistance of the A-1-b material was excellent and superior to the A-4 soil for all cement contents tested. The time required to wear away a depth of 1 in. of A-4 material was less than two days even when the cement content was 13.5 percent. In contrast, the A-1-b soil exhibited good erosion resistance. When this soil was stabilized with 5 percent cement, it took 15 days to erode a depth of 1 in.

Because of the better performance of the granular soil, additional tests were made by scalping various amounts of gravel from the A-1-b soil and by adding material greater than ¼ in. to the A-4 material. Results from these tests show that the percentage of gravel affects abrasion resistance significantly. For example, the time necessary to erode 1 in. of the modified A-4 soil-cement was significantly increased when the gravel component was more than 20 percent by weight. In fact, at 30 percent gravel and 9.5 percent cement, the modified A-4 soil was almost as resistant to erosion as the original A-1-b soil.

Gravel erosion tests were also made on a special low-strength gravel concrete to obtain a rough concept of the severity of the test. The water-cement ratio was 0.6 for this 2,000-psi, 28-day-strength concrete. After 7 days of moist-curing, this specimen was exposed to the water-borne-gravel test. Thirty-three days were required to erode away 1 in. of material. Thus, the abrasion resistance of this concrete was about double that of the A-1-b soil-cement at 7 percent cement, and it was concluded that the water-borne gravel test greatly accelerated abrasion.

The influence of strength gain by aging was evaluated by increasing curing time from 7 to 28 days. The abrasion resistance of the soil-cement made with the A-4 and A-1-b soil stabilized with 7.5 and 3 percent cement by weight, respectively, increased by 50 percent. When exposed for 6 days to a stream of water only (that is, a flow not carrying gravel) at a rate of 16,000 gal per day, no erosion was observed for the A-4 and A-1-b soil stabilized with 1.5 and 0.75 percent cement, respectively. This indicates that flows not carrying debris will of themselves have little or no erosive effect on soils stabilized with even minimal amounts of cement. Soil-cement canal linings are generally only exposed to flows not carrying sand and gravel loads or debris and thus may be constructed with the minimum amount of cement as determined from standard tests.

WAVE RUN-UP

Besides providing erosion protection, a dam facing may function as a buffer by breaking wave action and reducing wave run-up. Embankment slope and the roughness of the slope facing material are both important factors in establishing height of the dam above maximum pool elevation. Run-up factors for soil-cement slope facings were determined experimentally in a wave tank.

Wave-Tank Tests

Reduced scale soil-cement test slopes were constructed at one end of a 30-ft.-long wave tank. The tank was 12 in. wide and 36 in. deep, with the depth of water maintained at 21 in. Waves were formed with a piston-type wave generator. The height of wave and the wave period were varied by changing the travel distance and velocity of the piston bulkhead. The water depth to wave height ratio (*D*/*H*) was equal to or greater than 3 at the toe of the structure for all tests. Test variables were slope of the embankment and roughness of the slope facing.

Soil-cement slope facing configurations representing different degrees of surface roughness were tested for embankment slopes of 1 on 3 and 1 on 2. The 1 on 3 slope was constructed in increments of 1-in.-thick layers, with a 3-in. setback for each successive layer. In addition, tests were made using 2-in. layers with 6-in. setbacks. Surface roughness varied as a function of the type of edge on the slope; both a sharp-edge and a rounded-edge condition were tested. A facing made of concrete finished with a wood float was considered representative of a smooth surface, and data from this test are used as a basis for comparison with data obtained from the soil-cement facings.

A photograph of a test in progress is shown in Fig. 5; the type of measurements obtained is illustrated in Fig. 6. In this figure, wave height, *H*, is the vertical distance between peak and trough of the wave train; the run-up, *R*, is the vertical projection of the distance the water flows up the slope measured from the still water elevation; and the wave period, *T*, is a measure of the time between successive wave peaks.

The experimental data are reported in Figs. 7 and 8 and nondimensional plots of the run-up factors R/H versus wave steepness H/T^2 where *L*, the wave length, is equal to $5.12T^2$. About 110 individual wave run-up tests were used as the basis for drawing the best-fit curves. The standard error of estimate was 0.13. In terms of wave run-up, this represents about 0.6 ft for a 5-ft-high wave.

Recalling that freeboard height requirements decrease as the run-up factor decreases, it is apparent that all of the soil-cement facings inhibit run-up. As expected, both the 2 on 6 and 1 on 3 sharp-edged configurations were more effective in reducing run-up than the 1 on 3 rounded edge. Also, the 1 on 3 configuration was more effective than the 2 on 6.

In practice, the rounded edge results from erosion of inadequately compacted material at the upstream face of the lift. The inadequate compaction is due to lack of confinement at the edge during rolling operations. Because of the considerably better performance of the sharp edge, full-scale compaction tests were made using a pan vibrator to determine if it was feasible to construct an edge that would be more resistant to erosion. It was determined that soil-cement made from the A-2-4 and A-1-b soils used in this study was readily compacted to 100 percent of standard density with a pan vibrator. In addition, vibratory compaction of these soils contained within a movable wooden

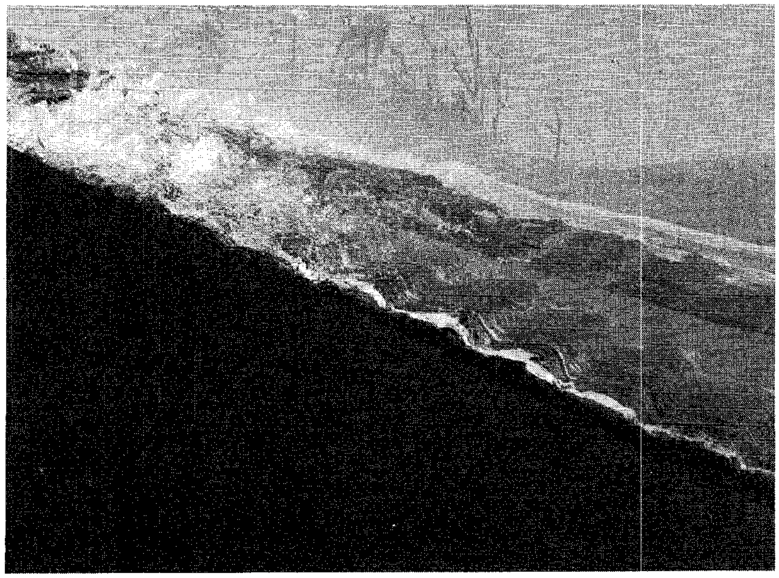

Fig. 5. Wave run-up on soil-cement.

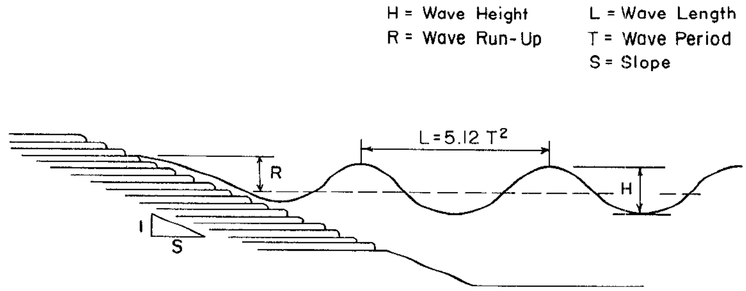

Fig. 6. Wave run-up measurements.

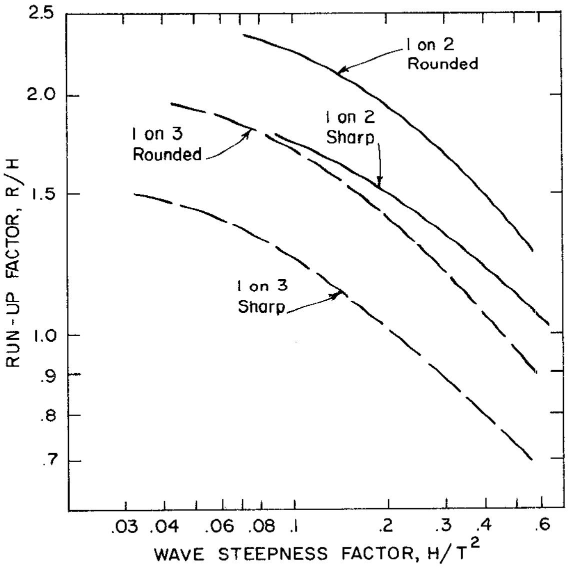

Fig. 7. Wave run-up on soil-cement slope facing.

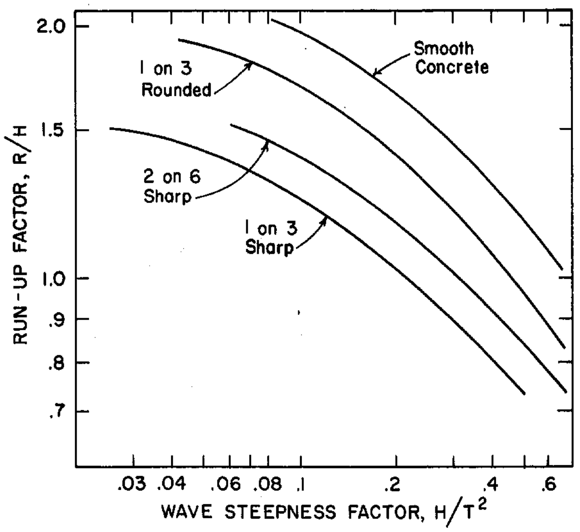

Fig. 8. Effect of slope steepness on wave run-up.

form produced a sharp edge with an equally high degree of compaction. Vibration of a second lift on top of a newly placed bottom lift did not crack the bottom layer. It would appear that a trial field installation should be considered.

In addition to the significance of sharp versus rounded edges, the data showed that the larger steps of the 2 on 6 slope were not as effective in reducing run-up as the smaller but more frequent steps of the 1 on 3 slope. Also, as shown in Fig. 8, the 1 on 3 slope was considerably more effective than the steeper 1 on 2 slope.

Analysis of Run-Up Data

To illustrate the use of the data in Fig. 7, the following sample computation is presented for determination of freeboard height.

A 1 on 3 rounded slope is assumed for a reservoir with:

F_e = effective unobstructed fetch length, 19,000 ft
F_s = total fetch, 8 miles
V = wind velocity over water, 93 ft/sec or 63.5 mph
D = average depth of reservoir, 50 ft
$g = 32.2$ ft/sec^2

Wave height H is computed from the Bretschneider revision of the Sverdrup-Munk equation, weighted for data from inland reservoirs as presented by Saville.(9)

$$H = \frac{V^2}{g}\left[0.0026\left(\frac{gF_e}{V^2}\right)^{0.47}\right] = 5.1\text{ ft}$$

Wave period T is also computed from the Bretschneider equation, weighted for data from inland reservoirs as presented by Saville.(9)

$$T = \frac{V}{g}\left[0.46\left(\frac{gF_e}{V^2}\right)^{0.28}\right] = 4.4\text{ sec}$$

The wave height and period are used to calculate a value of 0.26 for wave steepness, H/T^2. Entering Fig. 7 with a wave steepness of 0.26, a value for R/H of 1.25 is obtained; therefore the wave run-up for a 5.1-ft-high wave is 6.4 ft.

Freeboard height to prevent overtopping is measured from the maximum operating pool elevation and consists of a requirement for wave run-up, R, plus a height for wind-tide effects, S, that is computed from the equation:(9)

$$S = \frac{V^2F_s}{1{,}400D} = 0.45\text{ ft; use } 0.5\text{ ft}$$

Adding wave run-up and wind-tide values gives a freeboard requirement of 6.9 ft. In the same manner, freeboard requirement is computed to be 5 ft for the sharp-edged soil-cement and 7.6 ft for the smooth concrete.

SEEPAGE THROUGH DAMS

Seepage is an important consideration in the stability of earth dams. Seepage can be controlled by proper selection of embankment soils, by drainage methods, or by construction of impervious barriers. Flow of water through the dam can be inhibited by construction of relatively impermeable zones such as cores, cutoffs, or upstream blankets. The materials for the impermeable zones are generally selected from the least permeable soils near the damsite. However, in areas where lower-permeability soils are not available, it is possible to reduce permeability by stabilization with cement.

Test data are reported to show the effects of cement content and delayed compaction time on permeability. Because shrinkage cracks develop in soil-cement slopes, data from model tests are used to illustrate the influence of cracking on seepage.

Permeability

To facilitate testing with the flow of water directed both normal and parallel to the compaction plane, constant head permeability tests were made in specially constructed square molds. Specimens were compacted dynamically in two lifts to standard density(8) at optimum moisture content, then cured in a fog room for 7 days prior to testing. The effects of cement content and direction of flow on permeability of the A-1-b, A-2-4, and A-4 soils for conditions of no time delay between compacting the lifts are shown in Table 3.

For these soils and test conditions permeability decreased as cement content increased. For example, when the cement content met standard soil-cement requirements, tests normal to the compaction plane gave permeabilities that were only 1.2 to 12 percent of the values for the soils without cement. When the flow was parallel to the compaction plane, permeabilities were reduced also as cement content was increased. However, permeabilities for flow parallel to the compaction plane were 2 to 20 times larger than values for flow normal to the compaction plane.

To simulate field construction practice, tests were made also on specimens prepared with a time delay between compaction of the two layers. For time delays of 0 to 6 hours and flow normal to the compaction plane, permeabilities were relatively unchanged from the values reported in Table 3. However, as shown in Table 4, when the flow was parallel to the compaction plane permeability increased with increased time delay. For example, permeability for the A-2-4 soil stabilized with 3 percent cement by weight increased from 0.4 to 1.2 ft per year when the time delay was varied from 0 to 6 hours. Similarly, for the A-1-b soil stabilized with 3 percent cement, permeability increased from 0.6 to 12 ft per year. In the last example, the application of a neat cement paste to the bottom lift just prior to placement of the second lift reduced permeability to a value comparable to that obtained for flows normal to the compaction plane. Although not as effective as the cement paste layer, seepage was also reduced by a mechanical process. In this method, the surface of the bottom lift was scarified to a minimum depth of about 0.5 in. with a spike-tooth instrument. The second lift was then compacted on top of the first lift to obtain an interlocking of the two layers.

Seepage

Seepage tests were made in a model flume to determine the effect of cracks in an impervious upstream blanket on the amount of flow through the retaining structure. In addition, the effectiveness of cutoff trenches was evaluated. The model flume was 7 ft long, 1 ft wide, 2 ft deep. Permeability coefficients of materials used in the model flume tests were 3 X 10^6 ft per year for the embankment, k_e, and 3 X 10^4 and 3 X 10^3 ft per year for two facing or cutoff materials, k_c.

Seepage rate measurements to determine the effect of cracks were made for 2.5 to 1 embankment slopes with the width of crack opening varied from 0.03 to 0.25 in. by a metal frame and gage block system. Results of the seepage tests are expressed in Fig. 9 in terms of percentage of crack area to solid upstream facing and in percentages of increased flow when compared to seepage quantities in the absence of a formed crack. Tests were made for two conditions: (1) for a dam on impervious foundation, and (2) for a dam with cutoff trench extending from base of dam to a lower impervious boundary. It is noted that flow increased exponentially from 3 percent to 150 percent as crack area was increased from 0.2 to 2.5 percent.

To interpret these data, it is necessary to consider field observations from in-service facings that show shrinkage crack openings of 1/8 to 1/4 in. at spacings of 9 to 18 ft. Assuming the larger opening at the shorter spacing gives an open area of only 0.23 percent of the slope. Entering Fig. 9 with 0.23 percent open area yields an increased seepage quantity of about 5

TABLE 3. Permeability, Ft/Yr, No Time Delay Between Lifts

Cement content, % by weight	A-1-b		A-2-4		A-4	
	Normal	Parallel	Normal	Parallel	Normal	Parallel
0	3.0	2.3	0.5	1.0	0.5	1.2
1	0.7	1.1	0.06	0.2	0.3	–
3	0.07	0.6	0.02	0.4	0.2	0.4
5	0.01	0.1	0.02	0.2	0.1	–
7.5	–	–	–	–	0.06	0.1
9.5	–	–	–	–	0.05	–
11.5	–	–	–	–	0.03	0.08

TABLE 4. Permeability, Ft/Yr, Time Delay Between Lifts

Cement content, % by weight	Delay hours	A-1-b		A-2-4		A-4	
		Normal	Parallel	Normal	Parallel	Normal	Parallel
1	0	0.7	1.1	0.06	0.2	–	–
	6	0.8	13	0.08	0.9	–	–
3	0	0.07	0.6	0.02	0.4	0.2	0.4
	6	0.06	12	0.03	1.2	0.2	0.5
5	0	–	–	0.02	0.2	–	–
	6	–	–	0.01	2.1	–	–
7.5	0	–	–	–	–	0.06	0.1
	6	–	–	–	–	0.05	2.0

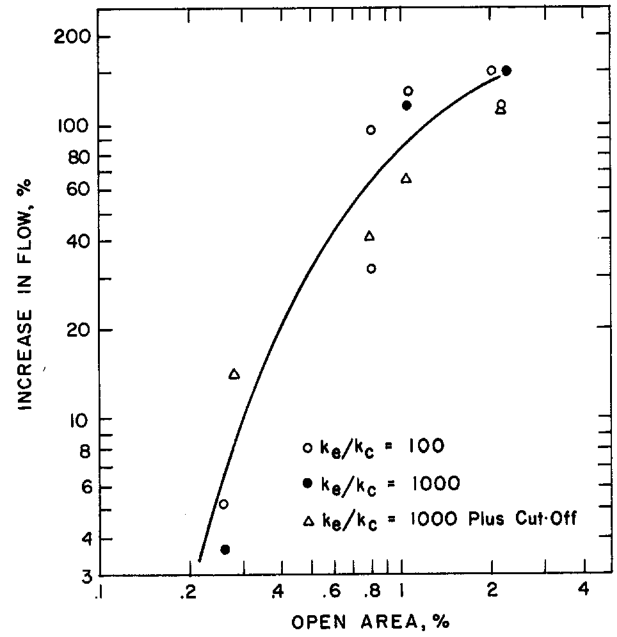

Fig. 9. Effect of crack opening on seepage.

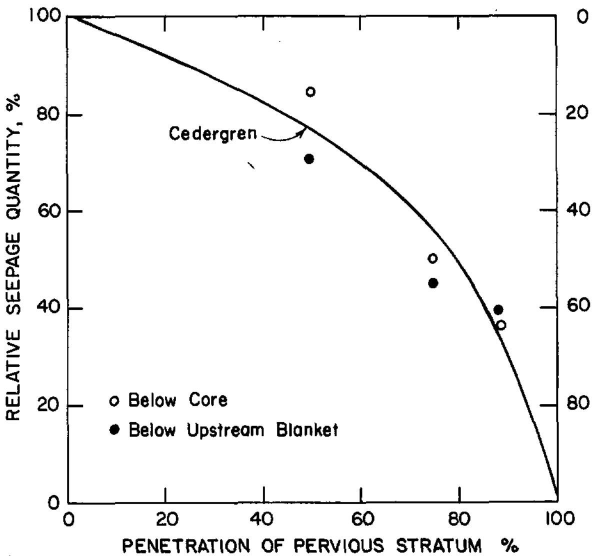

Fig. 10. Effectiveness of cutoff trench.

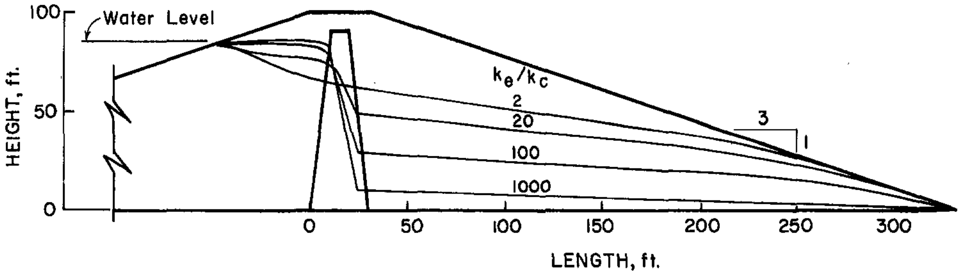

Fig. 11. Effectiveness of core wall in lowering saturation line.

percent. This indicates that increases in flow due to shrinkage cracking of the soil-cement are probably not significant. Therefore, when the permeability of the facing is less than the permeability of the interior, the facing contributes to dam stability.

Results from seepage flow tests to evaluate the applicability of soil-cement as a cutoff trench are plotted in Fig. 10. These data are for a permeability ratio of pervious foundation to cutoff trench of 50 and for designs where the cutoffs extend downward below a core or below the toe of an upstream blanket. The data points agree well with the work of Cedergren(10) and demonstrate that seepage through pervious dam foundations can be reduced significantly by construction of soil-cement cutoff trenches.

Core walls can be constructed from soil-cement concurrently with placing and compacting the earth embankment. The resulting lowering of the saturation line depends on the permeability ratio of the embankment soil to the cement-stabilized core and has been computed on the basis of equations presented by Pavlovsky(11) for an example of a 100-ft-high dam with an average core width of 20 ft. As shown in Fig. 11, the saturation line is lowered significantly for embankment to core permeability ratios greater than 20. Lowering the saturation line enhances the stability of the dam and increases the economic benefits by reducing seepage losses.

Rapid Drawdown

Provisions for drainage in back of relatively impermeable upstream slope protection blankets need only be made for unusual operating conditions where rapid drawdown could lead to piping failure. One method of designing an upstream drainage layer requires calculation of the time, t, required to lower the saturation line within the embankment:(12)

$$t = \frac{Cn_e}{2k} L \left[F\left(\frac{H}{H_i}\right)\right]$$

where

C = correction factor, 1.0
n_e = effective soil porosity, 0.20
k = coefficient of permeability, 35 ft per year
L = length from upstream toe to downstream drainage structure, 500 ft
H = height of saturation line above base of dam, variable/increment
H_i = drawdown, 120 ft
$F\left(\frac{H}{H_i}\right)$ = function of saturation line elevation to magnitude of drawdown (obtained from graphs in Reference 12)

This equation applies to homogeneous earth dam embankments constructed on an impervious foundation. The values assigned to the various factors were used for the following example of application of the drawdown expression. A solution for the time required to incrementally lower the saturation line and a plotting of its loci for four stages of drawdown, as indicated by the H/H_i ratio in percent, is shown in Fig. 12. For an embankment of a very fine sand and silt with a k value of 35 ft per year, the time to lower the saturation line by 40 percent of total drawdown was 300 days; it was about 8 hours for a sand with a k value of 3.5×10^4 ft per year. The coefficients of permeability, k, used in this example are used solely to illustrate the nature of the calculations and are not intended to be representative of the types of materials to be used in a 150-ft high dam. For a dam with the dimensions shown in Fig. 12 and an embankment with a k value of 3.5×10^3 ft per year, the amount of water, Q, drained toward the upstream slope from the center of the dam is about 340 cu ft per day when the line of saturation is lowered 20 percent. The greatest flow rate, 360 cu ft per day, was noted for the incremental lowering of the saturation line from 20 to 40 percent. This greatest rate of flow is used for the design of the drainage layer in accordance with the equation:

$$A = \frac{QL}{thk}$$

The head, h, for this example is 10 ft, which is conservative. A sand and gravel with a k value of 3.5×10^6 ft per year is selected for the filter blanket. The length of the drain, L, is about 350 ft and the value of Q/t is 360 cu ft per day. The thickness of the drainage layer at the upstream slope is thus about 1.3 ft. However, from construction considerations a minimum thickness of 2 to 3 ft may be required.

The integrity of soil-cement slope protection without a drainage layer is assured for earth dams when the reservoir operating conditions preclude sudden drawdowns that would create pressures greater than those that can be counterbalanced by the weight of the soil-cement. When sudden drawdowns are expected, a drainage layer is required for all types of slope protection. The soil-cement slope protection placed over the drainage layer protects it from becoming clogged by debris and suspended fines carried in the water and also protects it from displacement by wave forces.

CEMENT-STABILIZED DAMS

Stabilization with cement of the entire dam embankment can result in significant savings in materials. The considerable increases in compressive and shear strength of soils stabilized with even small amounts of cement can be exploited through the use of steeper slopes, resulting in a reduction of the total volume of material handling and placing as well as in construction time. Additional advantages can be gained by a reduction in the length of diversion structures and spillways. Also, overtopping due to unexpected flooding during construction or during the life of the structure would not be disastrous, as it might be for an embankment compacted with unstabilized soils.

Triaxial Strength

Triaxial strength tests were made on 2.8x5.6-in. cylindrical specimens compacted to standard density(8) at optimum moisture content. Specimens were cured in a fog room at 72 F until testing. The triaxial tests were undrained with a rate of loading such that the test was completed in about 10 minutes. Even at the lowest cement contents, the total strain at failure was less than 2 percent. Data from the triaxial tests on the untreated and cement-stabilized A-1-b, A-2-4, and A-4 soils are presented in Tables 5 and 6.

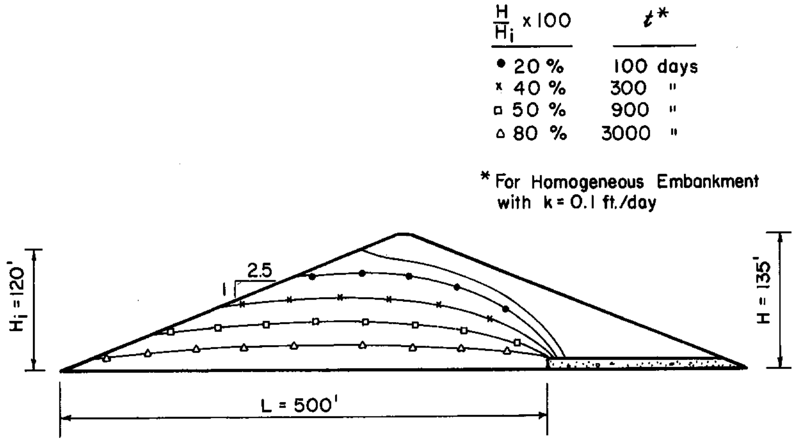

Fig. 12. Nonsteady state of flow for rapid drawdown.

TABLE 5. Triaxial Strength and Cement Content After 28 Days Curing

Soil	Percent cement by wt.	Cohesion, psi	Slope angle, deg, ϕ
A-2-4	0	20	29
	2	50	41
	3	58	44
	4	70	44
	6	90	48
	8	100	49
A-1-b	0	10	38
	1	27	45
	2	37	49
	3	50	51
	4	72	52
	5	95	55
A-4	0	5	37
	2.5	30	46
	5.5	65	45
	7.5	85	45
	9.5	125	45

The stabilized soils showed substantial increases in the coefficient of internal friction and cohesion when compared to the untreated soils. It was noted that cohesion increased with either cement content or curing time. Also, considerable increases in the angle of internal friction values were observed when the stabilized soils were compared with the untreated soils; however, only small changes were noted when the amount of cement used to stabilize the soil was varied. The increases in cohesion with increased amounts of cement is significant in considerations of slope steepness and stability.

The data from triaxial tests after 28 days of curing were used to compute the slope angle, i, permissible for cement-stabilized embankment construction. The slope angle was computed based on a submerged case and analyzed as a simple

TABLE 6. Triaxial Strength and Curing Time

Soil	Percent cement by wt.	Cohesion, psi	Slope angle, deg, ϕ	Age, days
A-2-4	2	10	43	7
	2	50	41	28
	2	40	40	90
	6	75	48	7
	6	90	48	28
	6	95	53	90
A-1-b	1	12	47.5	7
	1	27	45.5	28
	1	35	45.5	90
	3	33	49	7
	3	50	51	28
	3	85	46	90
A-4	2.5	25	43	7
	2.5	30	46	28
	2.5	55	42	90
	5.5	35	48	7
	5.5	65	45	28
	5.5	100	41	90

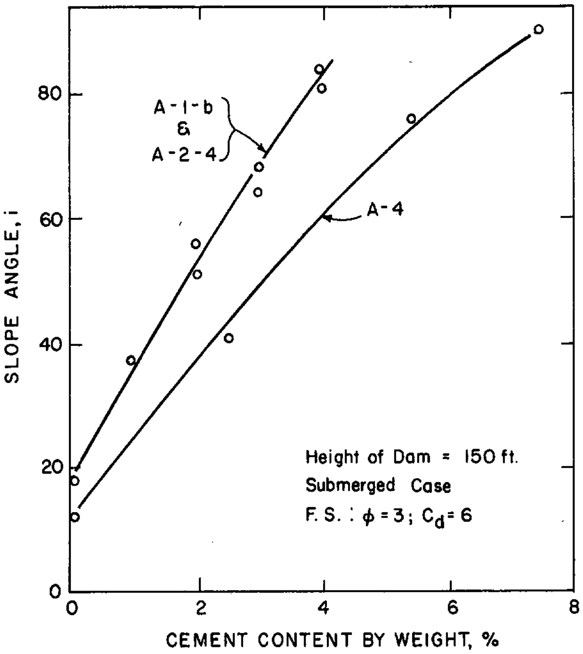

Fig. 13. Allowable slopes for cement-stabilized embankments.

slope with circular failure, having a safety factor of 3 for friction angle and 6 for cohesion. The relationships between slope angle and percent cement are shown in Fig. 13. It is seen that by stabilizing the soils with only 2 percent cement by weight, the allowable slope angle is about 54 deg for the A-1-b and A-2-4 soils and about 38 deg for the A-4 soil. These relationships were used to compute the volume of material required to build a 150-ft-high dam for comparison with the volume of material required for the same height dam made of unstabilized material on a 3 to 1 slope. In Fig. 14, it is seen that the volume of material placed and compacted in a dam section may be reduced by 60 to 70 percent when 2 percent cement by weight is used to stabilize soil materials of the type used in this investigation.

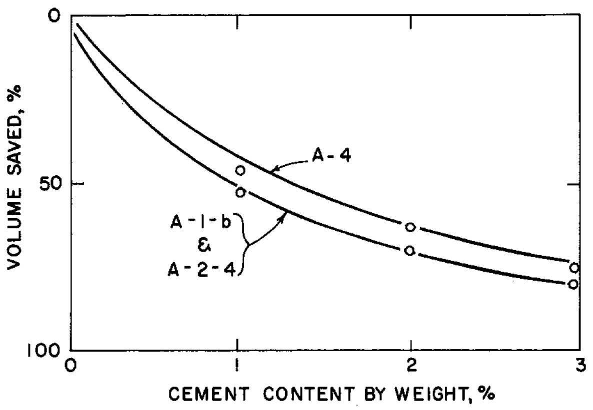

Fig. 14. Embankment volume for cement-stabilized soils.

CONCLUSIONS

1. A wide range of granular soils has been used successfully for soil-cement slope protection in earth dams. Data from this investigation show also that fine-grained, nonplastic soils can be used where more suitable soils are unavailable and economics permit use of the greater amounts of cement required for durability and erosion resistance.

2. Cement requirements for various portions of a dam facing may be varied with exposure. Requirements for surfaces exposed to freezing in the splash zone are about 2 percentage points greater than those determined from standard tests.(6,7) Areas exposed to freezing, but above the splash zone can be stabilized with the amount of cement required by standard tests. Those areas not exposed to freezing can be stabilized with about 2 percentage points less than that required by standard tests, but not less than a total of 2 percent.

3. When soil-cement is used in areas exposed to rapid stream flow carrying sand or gravel or other debris, the cement content should be 2 percentage points greater than the minimum required by standard tests. In addition, the soil selected should have a gravel component

exceeding 20 percent. When exposed to flows without debris, such as canal linings, the amount of cement may be the minimum required by standard tests and the material need not have a gravel component.

4. Wave run-up factors were obtained for soil-cement slope facing materials. Slope profiles that inhibited wave run-up to the greatest extent were those with sharp-edged steps. Run-up factors for a concrete slope were about 1.5 times greater than those for sharp-edged stepped surfaces. Run-up factors for steep slopes are greater than those for flatter slopes. Sharp-edged steps can be constructed with granular soils using a pan vibrator and a sliding form on the upstream face.

5. The permeability of soils generally used for dam facing is reduced considerably when stabilized with cement. When soil-cement is to be used for impervious barriers, scarification of the bottom lift with a spike-tooth instrument to a minimum depth of 0.5 in. and removal of the excess material prior to compacting the next lift will decrease seepage at the interface. A thin neat cement grout placed between the horizontal lifts reduced seepage to an amount equal to that determined from permeability tests made with the flow perpendicular to the compaction plane.

6. Shrinkage cracks in upstream soil-cement facings do not increase seepage through the dam by appreciable quantities when compared to a condition without cracks. Cutoff trench excavations and placement of the impervious cutoff trench materials should be carried through pervious foundation layers.

7. Triaxial strength factors and therefore the stability of slopes of embankments built with cement-stabilized soils increase with cement content and age. When compared with untreated soils, both the angle of internal friction and cohesion increased considerably even when small amounts of cement were used.

8. Volume of dam embankments can be reduced when the entire embankment is constructed with soils stabilized with small amounts of cement.

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KEYWORDS: cement content, cutoff walls, earth dams, erosion control, permeability, seepage, slope protection, soil-cement, triaxial shear, wave run-up.

ABSTRACT: Discusses laboratory tests to obtain design factors for application of soil-cement in earth dams as slope protection, impermeable barriers, and erosion-resistant surfaces in areas of rapid flow. Also discusses stability of embankments constructed with cement-stabilized soils. Methods to compute wave height and run-up are included.

REFERENCE: Nussbaum, P. J., and Colley, B. E., *Dam Construction and Facing with Soil-Cement* (RD010.01W), Portland Cement Association, 1971.

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