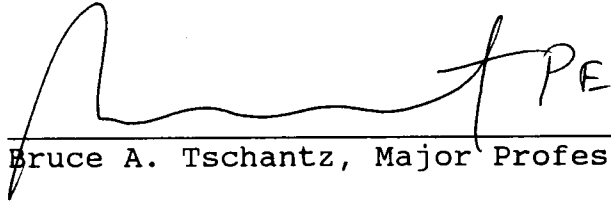


To the Graduate Council:

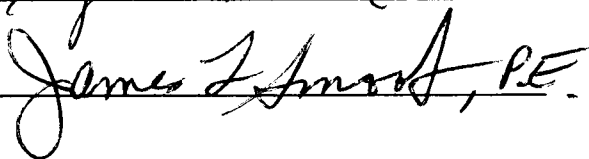
I am submitting herewith a thesis written by Susan R. Henderson entitled "Armoring Protection of Earthfill Dams". I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Environmental Engineering.

 PE

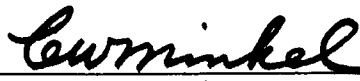
Bruce A. Tschantz, Major Professor

We have read this thesis
and recommend its acceptance





Accepted for the Council:



Associate Vice Chancellor
and Dean of The Graduate School

ARMORING PROTECTION
OF
EARTHFILL DAMS

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Susan Renee Henderson
May 1995

DEDICATION

This thesis is dedicated to:

my parents, Charles C. and Frances L. Henderson, with love and gratitude for their faith in my success;

Robert William Jacks III, with love and great thanks for his never failing encouragement;

and

my nieces and nephews, Melissa Renee Oubre, Caitlin Marie Kelly, Nathan Daniel Kelly, and Michael Christopher Oubre, with the hopes that it may be an inspiration to them to set and achieve high goals for their futures.

ACKNOWLEDGEMENTS

I would like to thank Dr. Bruce Tschantz for his patience and guidance over the past two years. I consider it a privilege and a great motivation to have been exposed to his engineering expertise.

I would also like to thank my brother, Stephen C. Henderson, for always being there when I needed him. Without his support I would have never made it this far.

I am also grateful to TVA for the use of its facilities and the opportunity to study Chickamauga Dam. Most helpful were the assistance and encouragement from Sam Shipman and the background studies by Louis Buck.

ABSTRACT

Because of the increased public awareness for dam safety, more accurate inflow design flood analysis procedures, updated inventories of state-regulated dams, and greater state attention to deficient spillways over the last fifteen years, many of the nation's existing dams are still considered to have inadequate spillway capacity. Expensive remedial measures of spillway enlargement, increased reservoir capacity and combination of these discourage many dam owners, especially *private* dam owners, from rehabilitating their dams. A possible alternative to expensive rehabilitation of earthen embankment dams, whose spillways are found inadequate to pass the required inflow design flood, is found in erosion protection of, or armoring, the crest and slopes. For this alternative one or more types of armor are placed on the crest and slopes, and the structure is allowed to overtop. Among the armoring options to be discussed in the paper are vegetation, geotextiles, riprap, gabions, cements (roller-compacted concrete and soil cement), and prefabricated concrete blocks (cellular, and wedge-shaped). This thesis includes a brief description of each method mentioned, a discussion of tests conducted by various agencies, a summary of benefits and limitations of each, and a recent survey of state dam safety officials about the use of armoring. This thesis will summarize the responses from each state to the following survey questions about allowing overtopping and the use of armoring: (1) Does your state have any procedures which allow embankment overtopping?, (2) Does your state allow the option of any kinds of embankment protection or armoring for inadequate spillway capacity?, and (3) If protection is allowable, are there any conditions or limitations to what could be done?.

In addition, this thesis will discuss the hydraulic and economic feasibility of using armor protection for the rehabilitation of the Tennessee Valley Authority's Chickamauga Dam.

ARMORING PROTECTION FOR EARTHFILL DAMS

TABLE OF CONTENTS

Chapter 1. Introduction

1.1 General	1
1.2 Overtopping Protection	3
1.3 Overtopping Hydraulics	5
1.3.1 Discharge Equations	7
1.3.2 Flow Regimes	8
1.3.3 Other Factors	11
1.4 Erosional Characteristics	11
1.5 Scope and Objectives	12

Chapter 2. Armoring Alternatives

2.1 General	14
2.2 Vegetation	15
2.3 Geotextiles	22
2.4 Riprap	30
2.5 Gabions	34
2.6 Cement	36
2.6.1 Soil Cement	36
2.6.2 RCC	39

2.7 Prefabricated Concrete Blocks	42
2.7.1 Cellular Blocks	42
2.7.2 Wedge-Shaped Blocks	50
 Chapter 3. ASDSO State Survey	
3.1 General	60
3.2 Results	61
 Chapter 4. TVA's Chickamauga Dam	
4.1 Statement of Problem	73
4.2 Project History and Description	73
4.2.1 Hydrologic Evaluation	76
4.2.2 Erosion Analysis Summary	77
4.2.3 Downstream Consequences of Erosion During PMF	78
4.3 Remediation Alternatives	81
4.3.1 Increase Spillway Capacity or Embankment Height	82
4.3.2 Do Nothing	83
4.3.3 Armoring Protection	83
 Chapter 5. Study Results	
5.1 Hydraulic Feasibility	85
5.2 Economic Feasibility	88

5.2.1 Armoring Benefits	88
5.2.2 General Armoring Costs	90
5.2.3 Common Costs	92
5.2.4 Roller Compacted Concrete Costs	94
5.2.5 Wedge-Shaped Blocks Costs	95
Chapter 6. Summary and Conclusions	
6.1 Literature Review	99
6.2 ASDSO Survey	100
6.3 Chickamauga Dam	101
6.3.1 Hydraulic Feasibility	101
6.3.2 Economic Feasibility	102
6.4 Conclusions and Recommendations	103
Cited References	105
Bibliography	112
Appendix A	121
Vita	141

LIST OF TABLES

Table 2.1.	ARS Reported Critical Velocity and Shear Stresses for Vegetation	21
Table 2.2.	CIRIA Reported Critical Velocity for Vegetative Cover	21
Table 2.3.	Functions of Geosynthetics in Dams	24
Table 2.4.	Permissible Shear Stresses for Selected Geotextiles	26
Table 2.5.	Critical Velocity and Shear Stress for Tested Geotextiles	29
Table 2.6.	FHWA Reported Permissible Shear Stress for Gravel and Rock Riprap	33
Table 2.7.	Permissible Shear Stresses for Cement Stabilized Soils	37
Table 2.8.	Summary of Characteristics of CIRIA-Tested Cellular Block Systems	44
Table 2.9.	Summary of Characteristics of SLA-Tested Cellular Block Systems	44
Table 2.10.	Comparison of CIRIA & SLA Reported Critical Velocities for Cellular Block Systems And SLA Reported Critical Shear Stresses	46
Table 2.11.	Characteristics of Wedge-Shaped Blocks	53
Table 3.1.	Responses to ASDSO State Survey	66
Table 3.2.	Known Dams Under State Jurisdiction Employing Some Type of Armoring Protection Against Overtopping	71
Table 5.1.	Summary of Estimated Costs for Armoring Chickamauga Dam's Earthen Embankments	97
Table A.1.	Summary of Armoring Alternatives	112
Table A.2.	Roller-Compacted Concrete Overtopping Protection Projects (USA)	114
Table A.3.	Dams with Cellular Concrete Block Overtopping Protection	116

LIST OF FIGURES

Figure 1.1.	Flow Patterns	6
Figure 1.2.	Flow Regimes and Zones	9
Figure 2.1.	CIRIA Model Wedge-Shaped Blocks	55
Figure 2.2.	Step Geometries Tested by USBR	57
Figure 4.1.	Chickamauga Dam General Plan, Elevations and Cross-sections	75
Figure 4.2.	Chickamauga Dam North Embankment Erosion, Velocity, and Water Surface Profile	79
Figure 4.3.	Chickamauga Dam South Embankment Erosion, Velocity, and Water Surface Profile	80
Figure 5.1.	Averted Damage-Frequency Diagram	91
Figure A.1.	CIRIA Velocity Duration Curve	130
Figure A.2.	Downstream slope of Belle Fourche Dam protected by grass	131
Figure A.3.	Hand-placed riprap on Indian Creek Dike, Utah	132
Figure A.4.	Riprap on upstream slope of Cold Springs Dam, Oregon	133
Figure A.5.	Gabions	134
Figure A.6.	Placement of Soil Cement protection on Starvation Dam, Utah	135
Figure A.7.	Roller-Compacted Concrete placement on Goose Lake Dam, Colorado	136
Figure A.8.	Cellular Concrete Block Sketch	137
Figure A.9.	Helical Anchor Attachment for Cellular Concrete Blocks	138
Figure A.10.	Placing CCMs at Bass Lake Dam	139
Figure A.11.	CCMs on Trout Lake Dam	140
Figure A.12.	Original Wedge-Shape Blocks	141
Figure A.13.	Overlapping Wedge-Shaped Blocks	142
Figure A.14.	ASDSO Survey Form	143

CHAPTER 1

INTRODUCTION

1.1 General

Because of the increased public awareness for dam safety, more accurate inflow design flood analysis procedures, updated inventories of state-regulated dams, and greater state attention to deficient spillways over the last fifteen years, many of the nation's existing dams are still considered to have inadequate spillway capacity. According to the latest information compiled by the Association of State Dam Safety Officials (ASDSO), there are 100,781 state regulated dams in the United States and of these, several thousand were originally found to be unsafe. As of 1994, 1,798 were reported to remain unsafe [1]. The required spillway capacity of a dam is determined mainly by the hydraulic potential created by the sizes of the dam and reservoir, and by the potential hazard that the dam imposes. Dams are classified into a hazard category of either *high*, *significant*, or *low* depending on which state has jurisdiction over the non-federal dams and which federal agency is responsible for a federal dam (see criteria noted in The Federal Guidelines for Dam Safety [2]). Some of the criteria considered in the classification of dams by both Federal and state agencies, as noted in a 1989 Report on Review of State Non-federal Dam Safety Programs [3] are as follows: 1) number of downstream potential lives lost, 2) number of potential habitable structures destroyed, 3) potential economic loss or damage to downstream property or reservoir/dam site, 4)

potential downstream traffic or vehicles on flooded roadways or bridge wash-outs, 5) potential environmental damage; destruction of wetlands, hazardous waste impoundment releases, etc., and 6) other considerations, including interruption of critical public services or security, hazard to school bus routes, potential nuclear power plant or similar facility flooding, velocity and depth [3]. The inflow design flood (IDF) is determined through consideration of the hazard classification and the size of the dam and its associated reservoir. Typically, if the failure of the structure will result in the possible loss of life, and or significant property damage, it is placed in a high hazard category and, therefore, should be able to safely pass the full probable maximum flood, or PMF. The PMF is defined as the probable maximum flood that could occur under a combination of meteorologic and hydrologic conditions in a specific area [4]. This flood should have virtually no chance of being exceeded. If the dam is classified as low hazard, where the only significant damage due to failure is to the structure itself, it is generally designed for one-half of the PMF or less. Significant hazard includes dams with a hazard potential anywhere between high hazard and low hazard, and the design flood should be determined by careful consideration of potential damage. Some of the need for rehabilitation of many dams stems from the fact that a significant amount of development has occurred downstream since the construction of the dam. This increases the hazard potential (i.e. low to significant or significant to high) and could require the safe passage of a full PMF when in fact only one-half PMF was originally used for design.

In the past, rehabilitation of dams to withstand the design flood have been limited

to increasing the height of the dam, increasing the spillway capacity or a combination of the two [5,6]. In many cases, however, the costs of implementing these modifications are too great or they may simply be physically impossible. The cost to upgrade an existing small dam spillway can typically range from \$100,000 to \$300,000 [7] and most existing dams have private owners without the necessary resources for this costly measure. Until recently, overtopping was considered paramount to failure of the dam. Today, the use of overtopping protection, or armoring, is gaining attention and being applied to the crest and slopes of some earthen embankments to enable the structure to withstand overtopping flow. This method of rehabilitation may be a cost effective way to prevent, or limit, erosion of the embankment which may lead to the potential failure of the embankment, and incrementally increased damage downstream. In effect, the entire length of the armored structure serves as an emergency spillway over which the flood waters can pass.

1.2 Overtopping Protection

Several different types of overtopping protection, or armoring, can be effective in preventing erosion, depending on the characteristics of the dam and design flood. Those characteristics include, the height of the dam, the downstream slope, the design flood duration, the overtopping flow velocity, and the depth of overtopping flow. To date, the more common armoring methods are vegetation, geotextiles, riprap, gabions, cements (roller-compacted concrete and soil cement), and pre-fabricated concrete blocks (cable-tied, interlocking, wedge-shaped). Chapter two contains descriptions of these methods,

along with references to tests conducted, limitations, and applications of each.

Many of the above mentioned alternatives have typically been used on the downstream side of earthen embankments and construction sites as protection from erosion due to rainfall impact or runoff. Erosion and sediment control at construction sites is a requirement, and therefore many times vegetation, geotextiles, riprap, gabions or some combination of these methods are used for protection from the erosion potential that rainfall would impose on exposed soils. Clearly, the event of overtopping flow across a dams crest has the potential for greater erosion than that of runoff, due to the energy supplied by the overtopping head and the slope of the downstream side. However, tests have been conducted by several agencies, both nationally and internationally showing that these methods can be successful in withstanding flows of an overtopping nature. Of course, there are limitations for each of these methods and their design is contingent on site specifics such as the maximum velocity and duration of the overtopping flow, and the headwater/tailwater differential.

Concrete has long been believed to withstand the highly turbulent flows resulting from overtopping, which is evident in the past construction of concrete spillways. Over the past ten to twenty years, new application of concrete has been investigated and in several cases, implemented. Roller Compacted Concrete (RCC) has been used on spillways as well as for over thirty earthen dams, in this country alone, as a means of overtopping protection [8]. In contrast to a flat concrete slab, rolling the concrete in a step formation, either horizontal steps or angled steps, can dissipate much of the energy, in effect, lessening the erosion potential. Also under investigation are prefabricated

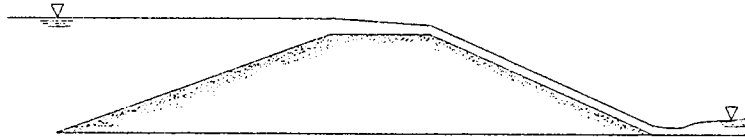
concrete blocks with varying shapes placed on the downstream slope. Depending on the hydraulics of the overtopping flow expected, many alternatives may be available.

1.3 Overtopping Hydraulics

In order to properly design armor protection for an earthen embankment, it is essential to have an understanding of the hydraulics and erosion potential of the expected overtopping flow. When a storm occurs that is significant enough to cause overtopping of the dam, the water level in the reservoir, or upstream side, begins to rise. Once the elevation of the surface of the water exceeds the elevation of the crest, an overtopping head is formed. The head is defined as the difference in elevation between the crest of the dam and the elevation of the reservoir water surface. As the water travels over the crest and approaches the downstream edge it may either be classified as free flow or submerged flow as shown in Figure 1.1.

The United States Geological Survey's (USGS) Water Supply Paper by Kindsvater [9] presents the discharge characteristics of embankment shaped weirs. The paper defines free flow as the flow pattern that occurs when the discharge is controlled by the upstream head, or headwater. When the head over the embankment crest reaches critical depth, and the tailwater is low, the conditions are right for free flow. The discharge may either plunge below the surface of the tailwater (free-plunging flow), creating a submerged hydraulic jump on the downstream slope, or ride out over the surface of the tailwater (free-surface flow), without creating a hydraulic jump on the downstream slope.

FREE FLOW



SUBMERGED FLOW

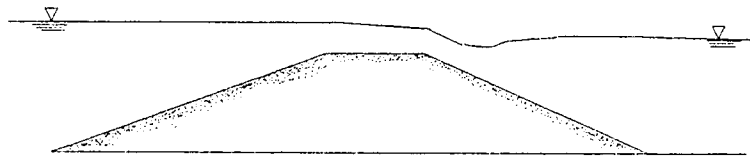


Figure 1.1. Flow Patterns

As the head increases to a depth greater than critical depth, the tailwater begins to rise and the discharge is controlled by both headwater and tailwater. Once the tailwater reaches a depth where the discharge is controlled by tailwater alone, the discharge is considered to be submerged, and will behave as surface flow. The flow pattern just before the transition from free flow to submerged flow occurs is known as incipient submergence and can occur very quickly in rising tailwaters.

1.3.1 Discharge Equations

The equation for flow over a rectangular shaped weir can be used to compute the discharge over an embankment for the free flow condition [10].

$$q = CH_1^{3/2}$$

Where: q = discharge per unit width

C = an experimentally determined coefficient

H_1 = total head above embankment

For submerged flow, the discharge can be calculated by the following equation.

$$Q = CLH^{3/2} \frac{C_s}{C}$$

Where: L = length of crest

H = total upstream head w/respect to crown

C = coefficient of discharge for free flow

C_s = coefficient of discharge for submerged flow

1.3.2 Flow Regimes

Overtopping flow with little or no tailwater goes through three distinct zones, which are depicted in figure 1.2. The three zones corresponding to overtopping flow and figure 1.2 can be described as follows [10]:

1. ZONE 1 - Subcritical flow over the embankment crest. From the upstream edge of the crest to the point of critical depth, the flow over the embankment is subcritical. As the flow crosses the embankment crest, the velocity increases. The water level is drawn down due to the velocity head ($V^2/2g$) until critical depth is reached. If the crest is rounded, critical depth will be near the centerline. If the crest is sloped, critical depth will occur near the edge with a lower elevation. The critical depth will also shift with changes in overtopping head. The greater the head, the closer critical depth will be to the upstream edge [10].

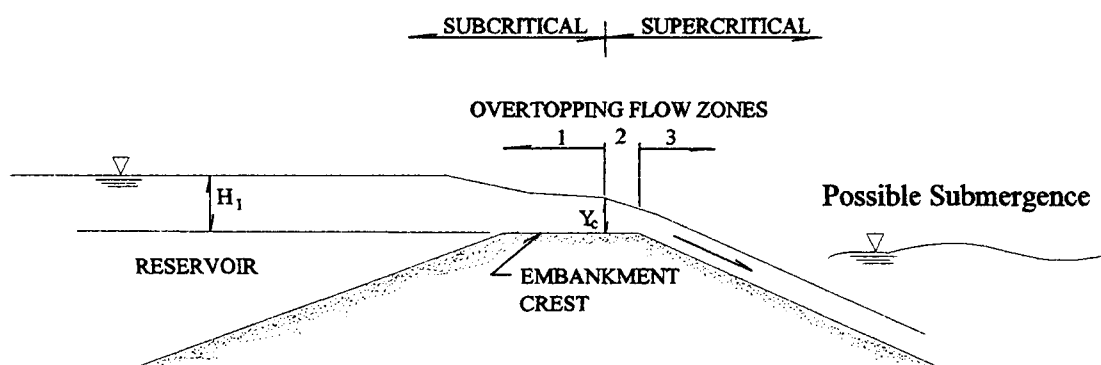


Figure 1.2. Flow Regimes and Zones

$$V^2/2g = 1/3 H + h_f$$

where: h_f = friction losses (usually negligible)

H = overtopping head

g = acceleration due to gravity

V = velocity

2. ZONE 2 - Supercritical flow over the embankment crest. From the point of critical depth to the downstream edge of the crest, the flow is supercritical [10].

$$Y_c = 2/3 H$$

$$V = (gY_c)^{1/2}$$

where: Y_c = critical depth

3. ZONE 3 - Supercritical flow on the downstream slope of the embankment. From the downstream edge of the embankment crest and down the slope of the embankment, the flow remains supercritical due to the steepness of the slope. At this point the flow will be also be turbulent due to the roughness of the surface. In addition, necessary corrections should be made for the effects of the slope angle and air entrainment encountered in this area [10]. If the increase in tailwater depth is significant, the dam may be completely submerged.

Other Factors that should also be considered in the hydraulic analysis of overtopping flow are uplift pressure, negative pressure, velocity of flow, and shear stresses. A brief description of each of these factors follows.

1.3.3 Other Factors

If the crest is paved or covered with an impermeable layer, the uplift pressures on the pavement or layer may be great enough to lift it off of the embankment crest, causing floatation. The maximum potential uplift is equal to the velocity head at critical depth or the total head, H_1 as shown in figure 2.1, minus the load from the weight of the water at critical depth, $2/3H_1$, which is equal to $1/3H_1$ [10].

Because the flow is carried out over the downstream slope for a distance (depending on the velocity of the flow and the geometry of the embankment), a zone of subatmospheric pressure is developed. The steeper the slope, the greater the negative pressure as well as the area in which it acts. The greater the overtopping head, the greater the negative pressure.

The velocity of the flow will vary with headwater and tailwater depths. If there is no effect due to tailwater, critical velocity is equal to $(2/3 g H_1)^{1/2}$ [10]. The faster the velocity the greater the shear stress, and the velocity is affected by both the headwater and tailwater depths. If the velocity of the water induces a shear stress greater than the critical shear stress of the embankment soil, erosion will begin. All materials have a distinct critical shear stress.

1.4 Erosional Characteristics

The extent of erosion, due to overtopping, that may occur on an earthen embankment is dependant on the characteristics of both the water and the embankment. The velocity of the water will induce a shear stress on the soil. The magnitude of this

component in relation to the maximum allowable shear stress of the embankment, will determine the probable erosion. The range of velocity is related to the headwater and tailwater depths during overtopping. The erodibility of the soil is a major component in overall erosion, as well as the duration of the event [10].

In reference to the three zones identified in the section 1.3.2, Figure 1.2, the following erosion patterns are identified [10]:

Zone 1 - generally minor erosion

Zone 2 - minor to significant erosion (crest will begin to erode at downstream edge)

Zone 3 - erosion may begin at a point of slope discontinuity, (the toe of the dam), or anywhere along the embankment, depending on the slope, tailwater depth and soil type. After the surface of the embankment is penetrated, a scour hole will form. If there are no effects due to tailwater, the scour hole will move upstream, deepen, and begin to propagate laterally. If the scour hole is submerged by tailwater, the result is a small energy level change between headwater and tailwater, thus greatly reducing the erosion potential.

1.5 Scope and Objectives

The objective of this thesis is to review the literature and to discuss armoring alternatives for the protection of earthen embankments, in the event of overtopping. From information obtained through research of associated literature, the aforementioned alternatives are described in Chapter 2. For each alternative, a summary of the benefits

and limitations, and a discussion of how and where each has been tested is included.

From a recent survey of state dam safety officials, chapter three will summarize the responses from each state to the following survey questions about the allowance of overtopping and the use of armoring: (1) Does your state have any procedures which allow embankment overtopping?, (2) Does your state allow the option of any kinds of embankment protection or armoring for inadequate spillway capacity?, and (3) If protection is allowable, are there any conditions or limitations to what could be done?.

A major focus of this thesis is the application of the above findings to a real situation -- the hydraulic and economic feasibility of armor protecting the Tennessee Valley Authority's Chickamauga Dam, located in Chattanooga Tennessee. At present, Chickamauga is in need of rehabilitation because its spillway capacity is incapable of passing the required design flood. Armoring the earthen embankments of Chickamauga and allowing them to overtop is being considered as an alternative to more traditional and expensive rehabilitation methods such as increasing the spillway capacity and raising the height of the dam. Chapters 4 and 5 discuss the Chickamauga project and investigate which types of armoring would be feasible both economically and hydraulically.

CHAPTER 2

ARMORING ALTERNATIVES

2.1 General

Several different alternatives are available for armor protection. Depending on the physical and hydraulic characteristics of the site in question, some methods may or may not be feasible. This chapter will summarize seven tested armor materials with a brief description of each and discussions of limitations, considerations, and approximate unit costs. The research and facilities of the various agencies, in the United States and two other countries, that have been involved in the testing of armor protection are briefly discussed in the section representing the alternative they tested. Because many series of tests involved the analysis of several different alternatives, the agencies facilities will be discussed in the alternative section that appears first and referred back to, when necessary, to avoid repetition. In Appendix A, a summary table (Table A.1) is presented which lists each armor type, a brief description, applications, limitations, unit costs, and agencies involved in the testing of each alternative.

Much of the research that has been done related to armor protection, in the United States, can be traced to the following organizations or agencies, either individually or in joint association:

- 1) The U. S. Department of Agriculture (USDA)
 - a) Agricultural Research Service (ARS)
 - b) Soil Conservation Service (SCS)
 - c) Forest Service (USFS)

- 2) The U. S. Department of Transportation (USDOT)
Federal Highway Administration (FHWA)
- 3) The U. S. Bureau of Reclamation (USBR)
- 4) The U.S. Geological Survey (USGS)
- 5) The U.S. Army Corps of Engineers (USACE)
- 6) Simons, Li and Associates (SLA)

Outside of the United States, the two main countries which have been involved in extensive research and testing, dealing with the subject of overtopping protection, are Great Britain and Russia. In Great Britain, the agency responsible for much input in the study of overtopping protection is the Construction Industry Research and Information Association (CIRIA). Russia's input came mainly from Professor Yuri Pravdivets of the Moscow Institute of Civil Engineering and his work with concrete blocks [31,32].

This chapter is divided into sections which discuss each of the 6 armoring alternatives: vegetation, geotextiles, riprap, gabions, cements, and prefabricated concrete blocks.

2.2 Vegetation

Vegetative cover has long been used as erosion control from runoff and rainfall impact on slopes and construction sites. The main objective is to prevent the exposure of bare soil to the elements. Various types of vegetation have been proven effective to a certain degree. Grass growing on the down stream slope can prevent the detachment of

soil particles due to rainfall impact, if the cover is complete. However, during overtopping flow conditions the performance of vegetation is limited by the maximum shear stress and the maximum velocity of the flow. Other considerations which will determine the effectiveness of vegetation, as well as the type of cover, are the type of vegetation used, the erodibility of the soil, whether or not the root structure is well established, and the length of the grass.

Over the years, extensive studies have been conducted concerning the stability of grass lined channels in order to study the permissible velocities of vegetation. The United States Department of Agriculture's (USDA) Agricultural Research Service (ARS) and Soil Conservation Service (SCS) have been involved in much of the research. The SCS developed an empirical permissible velocity procedure for the design of grassed lined channels in 1954 as SCS TP-61 [1].

Because of renewed interest in grass linings for use in floodways, urban drainageways, and reservoir emergency spillways, the available data was reanalyzed in the late 1970's [2]. These studies resulted in the semiempirical effective stress design approach which improved the separation of independent variables in design relations.

In 1987, the USDA published a handbook entitled "Stability Design of Grass-Lined Open Channels" [2] professing it to be "the state-of-the-art in grassed waterway design." This handbook combines the appropriate soil erodibility relations with past data, resulting in more flexible procedures which are "consistent with current nonvegetated channel design practices" [3]. Included in the USDA's report are the theory and logic underlying the effective stress approach to the design of vegetated channels;

considerations in selecting, establishing, and maintaining grass channel linings; methodology for estimating design parameters; the numerical effective stress design procedure; and graphical design aids.

R.C. Riley, of the SCS, presented a paper at the 1986 Annual Meeting of the Association of State Dam Safety Officials (ASDSO) in Austin, Texas, in 1986, entitled "A Procedure for Evaluating Permissible Limits of Overtopping of Vegetated Earthen Embankments"[4]. In this paper the author attempts to predict the erosion potential of flows over earthen embankments using SCS stability criteria by examining two aspects of embankment overtopping. First, alternative procedures for hydraulically rating embankment overtopping are examined, and second, procedures and guidelines for evaluating the potential erosion associated with overtopping are discussed.

Through joint efforts, the ARS and SCS developed a new computational procedure for use in design and analysis of vegetated earth spillways to be incorporated into design/analysis software, DAMS2 [5], for use beginning in 1994. In this procedure the erosion process on vegetated spillways is divided into three phases. These three phases are described as vegetal cover removal, headcut development, and headcut advance. Data for this study was compiled through joint research and field data acquisition programs described below.

In 1983, the SCS formed an Emergency Spillway Flow Study Task Group (ESFSTG) in order to gather field data from existing spillways. Of interest were earth spillways experiencing over 3 feet of head and/or flood related damage. As of 1993, data from 83 sites representing 13 flood events in 10 states were collected and compiled. The

ARS assisted by conducting laboratory research to quantify the associated erosion process. In 1991, a Design and Analysis of Earth Spillways (DAES) team was formed jointly between SCS and ARS. Their task was to develop and document new technology for use in the design and analysis of earth spillways.

The laboratory research was conducted in cooperation with the Oklahoma Agricultural Experiment Station in Stillwater, Oklahoma. Their facilities include numerous prototype spillways and three flumes. The major flume has 12 channels which are 3 feet wide with slopes 20H:1V with a discharge range of 7 to 20 cfs/ft and the duration of testing has reached up to 275 hours. Although studies are ongoing, many results thus far are being used as the basic guidelines for predicting the life of grassed-lined spillways.

The field data was obtained from 21 sites which were damaged by flooding in 1982, 1983 and 1985; identified by the FHWA, located in Arkansas, Missouri, Wyoming, Colorado and Arizona. The USGS was contracted to estimate the flood conditions at several sites damaged in 1983, except for Wyoming and Colorado. Estimated parameters included peak flow, maximum depth, velocity, duration, head and tailwater elevations, and maximum depth over the roadway.

From these studies, the DAES reported that *uniform* vegetal cover has the greatest impact on overall performance when the exit slopes are less than 6 percent, the maximum head on the spillway is less than 3 feet, and flow duration is less than 100 hours [6].

The FHWA, along with the USFS, contracted SLA to develop a procedure for

determining embankment erosion due to flood overtopping, and to assess protection measures. The information gathered for this study was gained through a literature review, collection of field data, and by conducting laboratory tests. As a result of their analysis, the computer model EMBANK [7] was created and made available in 1986 to determine the erosion associated with embankment overtopping. A table of Critical shear stress for channels lined with vegetation is presented in their final report, "Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping" [7]. This table classifies different types of vegetative cover and the critical shear stress associated with stated conditions. The maximum critical shear stress is reported as 3.7 pounds per square feet (psf) for Class A, Weeping Lovegrass and Yellow Bluestem *Ischaemum* with excellent stand and average heights of 30 inches and 36 inches respectively.

SLA's laboratory testing was conducted as Phase I of an extensive engineering research project, at the Engineering Research Center (ERC) of Colorado State University, using a full scale model of a roadway embankment [7]. The facility consisted of an outdoor, large scale flume, 11 feet high, 90 feet long, and 4 feet wide. Contained within the flume was a 6 feet high compacted soil embankment with a crest length of 20 feet, in the direction of flow, and downstream slopes of 2H:1V to 4H:1V. The maximum unit discharge of the facility was 25 cubic feet per second per foot (cfs/ft). Other protection alternatives tested along with vegetation were geotextiles, riprap and gabions.

In 1984, CIRIA constructed a testing facility using a 35-foot high abandoned clay embankment dam in Blackburn, UK, known as Jackhouse Dam [8]. The structure was

lined with 9 trapezoidal channels with lengths of 82 feet. The channels had a nominal base width of 3.2 feet with 1:1 side slopes and a mean longitudinal slope of 2.5H:1V. The maximum discharge of the facility was about 11 cfs/ft yielding a maximum overtopping head of 2.5 feet with velocities reaching 26 feet per second (fps). Armor alternatives tested at this facility were plain grass lined channels, as well as geotextile systems and concrete block systems with grass cover. The plain grass began showing signs of stress after 3 hours of a continuous discharge less than 1.0 cfs/ft, and failed after 25 minutes under 2.0 cfs/ft and 15 minutes under 2.5 cfs/ft.

Results from testing at Jackhouse Dam resulted in the publication of CIRIA report 116, Design of Reinforced Grass Waterways [8]. In this publication, failure modes of plain grass are identified, and methods of reinforcement using geotextiles and concrete blocks are discussed. CIRIA also provides a velocity-duration diagram (Appendix A, Figure A.1) which identifies their recommended limiting values for erosion resistance of plain and reinforced grass. For plain grass with good cover the limiting velocity is identified as approximately 15 ft/s with a duration of 1 hour. Table 2.1 lists the critical velocities and critical shear stresses for different classifications of grass as reported by ARS. Table 2.2 lists the critical velocities of grass with respect to coverage reported by CIRIA as a result of their testing.

Table 2.1. ARS Reported Critical Velocity and Shear Stresses for Vegetation [9]

VEGETATION	UNDERLYING SOIL	CRITICAL VELOCITY (fps)	CRITICAL SHEAR STRESS (psf)
Class A	Erosion Resistant	6-8	3.7
	Erodible	4-6	3.7
Class B	Erosion Resistant	5-7	2.1
	Erodible	3-5	2.1
Class C	Erosion Resistant	4-5	1.0
	Erodible	3-4	1.0
Class D	Erosion Resistant	3.5	0.6
	Erodible	2.5	0.6
Class E	Erosion Resistant	3.5	0.35
	Erodible	2.5	0.35

Table 2.2. CIRIA Reported Critical Velocity for Vegetative Cover [9]

TYPE OF COVER	UNDERLYING SOIL	CRITICAL VELOCITY* (fps)
Good	Clay	7-15
Average	Clay	5-12
Poor	Clay	3-10

* Critical Velocity is dependant on test duration

Vegetation provides an inexpensive and aesthetically pleasing alternative if the associated conditions at the site in question do not exceed the limitations with respect to critical shear stresses and velocities. Another concern is the region in which the embankment is located. If the area is arid, good grass growth may not be feasible at all times, thus leaving the structure at risk for erosion. For all vegetation methods, routine maintenance must be conducted to insure good cover and performance (i.e. mowing and fertilizing). The maintenance required for vegetated embankments would be the most costly factor. Initial costs would depend on the type or types of vegetation used. Figure A.2, in Appendix A, shows a photograph of the downstream slope of Belle Fourche Dam which is protected by grass [15].

2.3 Geotextiles

Geotextiles are just one subcategory of a broader range of geosynthetics. There are a wide variety of geosynthetics on the market today with unlimited possibilities of usage in many disciplines. Geosynthetics are generally subdivided into different categories such as geotextiles, geomembranes, geonets, geogrids, geomats, geocells, geocomposites, etc. In much of the literature, these subdivisions are often used interchangeably causing some confusion in the possible usage of particular types. Dr. J. P. Giroud, technical director of Geo-Services Inc. and President of the International Geotextile Society (IGS) (1990) is responsible for developing many of the design methods used for geosynthetics, and is given credit for coining the terms "geotextile" and "geomembrane". In 1990, Giroud published a paper discussing the functions and

applications of geosynthetics in dams [10].

In his paper, Giroud provides a table (Table 2.3) listing traditional functions for the usage of geosynthetics in dams and the geosynthetic typically used for each application, as well as the succeeding description of each function.

In Giroud's discussion of the typical applications of geosynthetics in dams, five categories are outlined, some with the capability of joint application. These five categories are drainage, transitions in zoned dams, reinforcement, waterproofing, and erosion control/slope protection. The erosion control examples cited in Giroud's paper mainly dealt with the protection of the upstream slope and the use of geosynthetics as transition elements. However, the possibility of designing a "protection which can withstand a certain amount of flow caused by overtopping of the dam" is noted [10].

In addition to the paper previously cited, Dr. Giroud has published several other papers dealing with the use of geosynthetics in dams, over 100 papers on geosynthetics in general, and prepared a two-volume report for the USBR entitled "Geomembranes in Embankment Dams". In fact, Giroud was the first to use a geotextile in the design of an earthen dam (Valcros Dam, France, 1970).

Table 2.3. Functions of Geosynthetics in Dams [10]

Category of Function	Function	Geosynthetic Typically Used
Hydraulic Functions	Fluid Barrier	Geomembranes
	Filtration	Non-woven and monofilament woven geotextiles
	Transmission	Geonets, geomats, cores of drainage geocomposites, and, to a lesser degree, thick needle punched Non-woven geotextiles
Interface Functions	Separation	Geotextiles
	Interlayer	Geotextiles (and, sometimes geonets or thin geomembranes)
Reinforcement Functions	Macro-Reinforcement	Geotextiles, geogrids
	Micro-Reinforcement	Fibres, yarns, and microgrids
	Surface Reinforcement	Geomats, geocells

FUNCTION DESCRIPTIONS [10]

Fluid Barrier - a quasi-impermeable geosynthetic (that is a geomembrane) is used to contain a fluid.

Filtration - A geotextile located between a soil and an open draining material (for example: gravel, transmissive geosynthetic, perforated pipe) allows a liquid to pass while controlling the migration of soil particles.

Transmission - A thick and permeable geosynthetic (that is a transmissive geosynthetic, such as a geonet, a geomat, or a drainage geocomposite) conveys fluids within its plane.

Separation - A geotextile located between a soil containing fine particles and a material (either natural or manmade) with large openings (ie: aggregate, fractured rock, concrete panelling with joints) prevents intermixing of the soils and the adjacent material.

Interlayer - A geosynthetic located between two materials controls stress and strain transfer between the two materials, thereby preventing mutually inflicted damage.

Macro-Reinforcement - One, or several layers, of geosynthetics placed in a soil, or between two soils, reinforces the soil(s) as a result of stress transfer from the soil(s) to the geosynthetic.

Micro-Reinforcement - Fibres, yarns or small element with various shapes (such as microgrids) intimately mixed with soil particles form a reinforced soil mass.

Surface Reinforcement - A geomat, a geocell (that is, a synthetic honeycomb structure filled with soil), a geogrid or any other appropriate geosynthetic, placed at the surface of a soil mass and closely interlocked with a layer of soil particles, prevents surface erosion. Mats or loose grid-like fabrics made with natural fibres (for example, geojute) are used where biodegradation is desirable.

Extensive studies have been done on the subject of geosynthetics for applications other than dams, with a more direct focus on geotextiles. In the late sixties and early seventies, at least two studies were conducted by the FHWA on the performance of geotextile linings [11,12]. From a USGS study [13] completed for the FHWA, in 1985, the FHWA compiled a report entitled "Design of Roadside Channels with flexible linings" [14]. In this report, the FHWA reports permissible shear stresses and Manning's roughness coefficients (n) for selected geotextiles, categorized as temporary linings, which range from 0.15 to 2.00 pounds per square feet (psf) and 0.015 to 0.066 respectively. Manning's n values are adjusted for different depth ranges, and can be used with Mannings equation to determine the maximum allowable velocities. Table 2.4 summarizes the reported shear stresses.

Table 2.4. Permissible Shear Stresses for Selected Geotextiles. [14]

GEOTEXTILE	PERMISSIBLE SHEAR STRESS (psf)
Woven Paper Net	0.15
Jute Net	0.45
Fiberglass Roving: Single	0.60
Fiberglass Roving: Double	0.85
Straw with Net	1.45
Curled Wood Mat	1.55
Synthetic Mat	2.00

Most geotextiles available for embankment protection are synthetic fabrics, mats, or large-scale cells, subdivided into three groups: 2-dimensional- natural or synthetic woven fabric and meshes, 3-dimensional open - synthetic mats filled with topsoil and seeded, 3-dimensional filled- synthetic mat filled with bitumen bound gravel [9]. Typically, the geotextile is embedded in the soil and vegetation covers the entire surface area. The roots then bind around the geotextile forming a geotextile/soil/root mat.

During CIRIA's analysis at Jackhouse Dam (section 2.2), four geotextiles were tested and determined to provide effective overtopping protection when the flow velocity is less than 15 to 20 fps, depending on the expected duration. The four types of geotextiles tested were a 2-dimensional woven fabric, a 2-dimensional mesh, an open 3-dimensional mat (soil filled), and a bitumen-bound 3-dimensional mat. Testing conditions were durations of 10 minutes to 3 hours, discharges of 2 to 38 cfs, and

velocities of 8 to 27 fps. The 2-dimensional mesh and the soil filled, open 3-dimensional mats were reported to have the best performance, with uplift failure occurring after 1.5 hours, under a velocity of about 27 fps. The 2-dimensional woven fabric failed after 45 minutes under a velocity of about 21 fps. The Bitumen bound 3-dimensional mat failed after 1 hour, under a velocity of 19 fps, and also after 10 minutes under a velocity of 21 fps. CIRIA's recommended limiting values for velocity and duration are shown on the aforementioned velocity-duration diagram in Appendix A, Figure A.1.

In Phase I of SLA's testing (section 2.2), 2 specific geotextiles were tested, Geoweb and Enkamat. Geoweb is a grid confinement system of high-density polyethylene that has been used for erosion control along lake shores and river banks, while Enkamat is a matting made from heavy nylon monofilaments fused at their intersections. The critical shear stresses and velocities for Geoweb and Enkamat were report to be 0.7 psf, 6.0 fps and 2.0 psf, 10.0 fps respectively. Because failure was believed to be partially attributable to improper installation, it was recommended that additional testing should be done, which lead to Phase II.

SLA continued their studies with the FHWA as phase II and were co-sponsored by the USBR. The efforts of testing were compiled in a report for the FHWA and USBR entitled "Minimizing Embankment Damage During Overtopping Flow" [9], in November of 1988. Using the same facilities as in Phase I (section 2.2) geotextiles were tested as armoring alternatives along with gabions, soil cement, and pre-fabricated concrete blocks.

In Phase II, both Geoweb and Enkamat were tested again with slightly different installations, but as in Phase I, failure was attributed to poor anchorage or stretching of the mat. Geoweb (4 inch) was stable under 1 foot of overtopping depth on a 2H:1V slope and velocities near 10 fps, but failed under 2 and 4 feet overtopping depths within 1 hour and 10 minutes respectively. The Enkamat was torn in areas where the material was staked, during overtopping depths of 1 and 2 feet, thus allowing the soil beneath to be eroded. SLA reported that the Enkamat had too low a unit weight and was too flexible to resist deformation, thus Enkamat should not be used as bank protection by itself. It was also noted that the installation was labor intensive for both materials which may increase the cost of application. Table 2.5 summarizes the results with respect to critical velocity and shear stress for CIRIA and SLA Phase II tests.

The failure mode of geotextiles, as armor protection, is largely due to uplift. Therefore, the openings in the membrane should be large enough to relieve any uplift pressure. The velocity of the overtopping flow and the degree of turbulence also affect the performance. At higher velocities, uplift failure is more likely to occur. Therefore, velocities should be limited and the minimum geotextile openings should be 0.5 mm (500 microns) [8]. Good contact with the soil surface is essential to prevent water from flowing underneath the geotextile. According to CIRIA, when establishing vegetation, small openings and filled mats protect better than open mats, however, open mats allow greater relief of uplift pressures and form a good contact, because filling with topsoil will fill possible voids below the geomembrane. Openings need to be large enough to allow

plant growth, and a minimum mat thickness of 20 mm is recommended by CIRIA, for open mats, to ensure interaction with vegetation roots.

Table 2.5. Critical Velocity and Shear Stress for Tested Geotextiles [9]

SOURCE	GEOTEXTILE	UNDERLYING SOIL	CRITICAL VELOCITY (fps)	CRITICAL SHEAR STRESS (psi)
CIRIA	Nylon mat	Clay	14-19*	N/A
SLA - Phase II	Enkamat (3 in. asphalt)	Type I	13	13-16
SLA - Phase II	Enkamat (1 in. asphalt)	Type I	< 8.6	< 5
SLA - Phase II	4 in. Geoweb	Type I	9-10	10

* Critical Velocity is dependant on test duration

Possible vandalism, damage by human traffic or livestock, fire, and snagging during grass cutting are some of the other considerations to keep in mind when using geotextiles. Cut grass should be removed so it will not restrict the growth of fresh grass, and fertilizer should be used along with some type of weed control. The embankment should be checked regularly for exposed geotextile and if found, exposure must be repaired immediately. If any damages are found, the geotextile can be overlapped with additional reinforcement, pinned down, covered with topsoil and seeded. If the geotextile is stripped off of the subsoil and not much grass or soil erosion has taken place, temporary restraining pins can be used to hold the geotextile/soil/root mat in place while it re-roots itself [8].

Geosynthetics of many types have been used in various dam applications over the years, as previously mentioned, and more recently geotextiles are being installed as downstream slope protection for the prevention of erosion during overtopping. For example, Lake in the Sky Dam located in Blount County, Tennessee, was armor protected using Tensar Erosion Mat Turf. Tensar is a polyethylene, three-dimensional mat, and in this case was embedded into the embankment, filled with soil and seeded. The design storm for Lake in the Sky Dam is .5 PMP and during this storm the dam is expected to have overtopping depths between 3 and 6 feet. Tensar was installed in order to prevent erosion of the earthen dam when overtopped, but has not experienced overtopping at this time (April 1995).

Reported cost ranges (1986) for the Geoweb and Enkamat tested by SLA are \$0.75 to \$3.00 per square foot (/sq.ft.). TVA engineering suggested a cost for heavier geotextiles usually employed in dams as \$4 /sq.ft.

In many instances a geosynthetic or geotextile is used as an under lying filter material along with another, more durable form of overtopping protection, rather than the main armoring material. For the remainder of this chapter, geotextiles and other geosynthetics will be referred to in the context of underlying mats.

2.4 Riprap

The USBR describes riprap as "a layer of large, durable rock fragments placed on a slope to prevent erosion from wave action or from stream currents", in its technical publication *Design of Small Dams* [15]. In this publication, dumped riprap is said to

offer the best surface protection for upstream slopes at the lowest ultimate cost. Like vegetation, riprap has been used as an erosion control measure for embankment protection against rainfall impact and runoff, as well as for erosion control in channel stabilization and bank stabilization applications. Riprap can be hand-placed or dumped stone that has been sized to a specified diameter, grade and thickness. While hand-placed stone may be more aesthetically pleasing, greater stability is gained with dumped riprap (see Figures A.3 and A.4, in Appendix A, for photographs of hand placed and dumped riprap).

In the late 1940's, the U.S. Army Corps of Engineers examined approximately 100 dams across the United States with a variety of climatic conditions and wave severity, ranging from 5 to 50 years old [15]. Of the dams using dumped riprap as upstream slope protection, only 5 percent failed while 30 percent of those with hand-placed riprap failed. The failures of the dumped riprap were attributed to improper sizing of the stones, while hand-placed riprap failures were due to the usual method of single-course construction.

The efficiency of riprap protection, according to the USBR [15], is dependent on the quality of the rock, the thickness of the riprap layer, the weight or size of the individual pieces, the shape of the stones or rock fragments, the slope of the embankment on which the riprap is placed, and the stability and effectiveness of the filter layer upon which the riprap is placed.

Three different methods for designing riprap in channel stabilization applications have been identified by the FHWA [16], the SCS [17] and Colorado State University

(CSU) [18,19]. In the FHWA and SCS procedures, required stone diameter is a function of the depth of flow and channel slope. Both the FHWA and SCS procedures were based solely on experiments and field observations while the CSU method incorporates a factor of safety. This factor of safety is given by the ratio of critical to actual tractive force. Since the forces acting on a channel bank are different than those acting on a channel bed, the equations used to calculate the factor of safety for a channel bank incorporate a gravitational component. Unfortunately current design methods do not account for the combination of steep embankments, high velocity, and shallow flows associated with overtopping flow. Table 2.6 lists FHWA reported permissible shear stresses for gravel and rock riprap with respect to diameter size [14]. No information on critical velocities were reported.

Powledge and Dodge of the USBR, [20] conducted tests, in 1983, using a scaled laboratory flume with embankment slopes of 4H:1V and 6H:1V. The embankment protection alternatives examined were riprap and gabions. The flume was 3 feet wide by 4 feet high by 30 feet long containing a model embankment with a length scale of 1:15. The embankment was constructed out of clayey sand, and reportedly represented a prototype embankment of 32 feet in height. During testing, scaled velocities exceeded 30 fps and the scaled durations of overtopping were 1 hour and 5 hours (17 and 77 minutes).

The scaled riprap tested by Powledge and Dodge represented diameters of 6 to 24 inches. The rocks eroded out immediately on the 6H:1V slope, under a scaled unit discharge of 40 cfs/ft. In the literature, Powledge states that "studies indicate that the

minimum rock size required to resist movement during overflow of embankments is on the order of 1.5 m" [20], but no flow conditions are discussed.

Table 2.6. FHWA Reported Permissible Shear Stress for Gravel and Rock Riprap [14]

CATEGORY	DIAMETER (in)	PERMISSIBLE SHEAR STRESS (psf)
Gravel	1	0.40
Gravel	2	0.80
Rock	6	2.50
Rock	12	5.00

The most recent testing that the USBR has conducted was done using a 50-foot high, prototype embankment with a downstream slope of 2H:1V. The width of the structure was 10 feet and the maximum unit discharge was 40 cfs/ft. This facility was used to test the performance of riprap, and prefabricated concrete blocks. Although the testing has been completed, all of the results have not yet been published. According to Kathleen Frizell, a hydraulic engineer with the USBR, and director of said testing, this phase of riprap testing was no more effective than the tests in 1983 [21]. Failure of riprap with an average diameter of 15 inches occurred at 24 cfs. However, the riprap was designed to withstand a maximum discharge of 74 cfs.

Even though testing has not been very successful for determining the stability of riprap as overtopping protection on steep slopes, several dams in West Virginia with

relatively mild slopes have been protected from potential overtopping flows with riprap as an armoring alternative. For example, Lake Chewva Dam has a downstream slope of 6H:1V and is protected by 4-foot diameter riprap. Similarly, Laurel Lake Dam has a varying slope of 6H:1V to 10H:1V and is protected with 2.5 to 3-foot diameter riprap and Lake of Woods Dam has a slope of 4H:1V with 3-foot diameter riprap. All three of these dams are expected to be overtopped by 2 to 3 ft during the full PMP.

The cost of riprap is largely dependent on the location of the site. If no appropriate material is available in the area of the dam, the greatest cost associated would probably be the transportation costs. TVA estimating suggested that a typical cost for riprap transportation is about \$6/cu.yd.

2.5 Gabions

Gabions or reno mattresses, have been used for over 100 years for stabilizing embankments and controlling erosion and have been very successful for the stabilization of channel banks [9]. Essentially, gabions are wire baskets encasing uniformly graded stone. The dimensions of the wire basket and gradation of the stone may vary according to the desired usage. Figure A.5 in Appendix A depicts some sketches of gabion revetment systems.

As mentioned in sections 22.2 and 2.3, gabions have been tested for their stability as overtopping protection by the USBR and by SLA. Quantitative results for the limiting velocities of gabions subjected to overtopping flow are not available at this time, because in most tests, the limiting factor was the maximum discharge of the testing facility,

therefore the gabions were not tested to failure other than failure attributed to poor anchorage. However, some of the limitations of gabions were determined. Deformation of the wire baskets with thicker mattresses occurred with the displacement of the rockfill, and scour downstream from gabions was noticed. If gabions are used, they must be sized properly, packed tightly and well anchored. Gabions have not been tested for overtopping velocities greater than 24 fps.

During SLA's testing, 6 inch gabions were tested on a 2H:1V slope, with an overtopping head of 4 feet. The gabions were stable under velocities between 14 and 15 fps and a maximum discharge of 96 cfs, but the baskets had deformed. Failure of the gabions occurred at the crest anchor under the same discharge on a 3H:1V slope with velocities between 14 and 17 fps. Limiting shear stresses for the 6 inch gabions were reported to be between 15 and 25 psf by SLA.

Gabions have been installed as overtopping protection on a several small dams in the United States. One such dam is a 25-foot structure at Bartholomew Park Regional Detention Pond, in Texas. The maximum velocities originally predicted for this dam during its design flood of 0.5 PMF were 23 fps. Before Texas would allow the installation of gabions at Bartholomew Park, the state required that the velocities be limited to 17 fps. In order to meet this requirement the downstream slope of the dam had to be modified from the original 3H:1V to a milder 5H:1V slope.

From information received in response to the ASDSO survey discussed in Chapter 3, an estimated cost range for gabions is about \$30 to \$60/sq.yd.

2.6 Cement

Both soil cement and Roller Compacted Concrete (RCC) are possible alternatives for the use of cement as overtopping protection. Soil cement is formed by creating a mix of soil and cement, while RCC combines a mix of sand, gravel and concrete. Both methods of overtopping protection can be applied using typical earth moving equipment. A greater percentage of concrete is typically required for RCC in order to achieve a higher compressive strength [9]. Figure A.6 in Appendix A is a photograph of the placement of soil-cement slope protection on Starvation Dam in Utah [15] and Figure A.7 is a photograph of the RCC construction on Goose Lake Dam in Colorado [24]. Soil cement and RCC are addressed separately in this section.

2.6.1 Soil Cement

The USBR has been responsible for the testing of soil cement as erosion resistance against wave action and as a more cost effective protection measure for dam facing than riprap, beginning studies as early as 1965 [22]. The more recent considered application of use for overtopping protection was tested in both Phase I and II studies by SLA for the FHWA (section 2.2 and 2.3).

Dr. M. R. Acky studied the erodibility of cement stabilized soil for his Ph.D. dissertation at the University of California in 1974. Permissible shear stresses for cement stabilized soil with low cement content, as reported by Dr. Acky [23], are tabulated in Table 2.7.

Table 2.7. Permissible Shear Stresses for Cement Stabilized Soils [23]

PERMISSIBLE SHEAR STRESS		
CEMENT PERCENT	UNCYCLED SAMPLE (psf)	12 FREEZE-THAW CYCLES (psf)
3.0	No failure at 4.5	0.51
2.0	2.36	0.47
1.5	2.05	0.45
1.0	1.54	0.18

For SLA's Phase I testing [7], the soil cement tested was prepared commercially by a local ready-mix contractor and delivered to the testing site. The specifications for the soil cement were a cement content of 11 percent and a 10 percent moisture content. Plaster and sand made up the remainder of ingredients in the soil cement mixture. Once delivered, the soil cement was placed as a 1-foot layer on top of the embankment crest and on the downstream side. Below the soil cement layer, embedded in the embankment, was a non woven filter fabric, Typar. The soil cement was tested for 2 hours on a 2H:1V slope with the overtopping depth reaching 4 feet. No erosion was noted of either the embankment material or the soil cement. Although the test results were positive, a possible limitation of soil cement that was not analyzed in this test, is the failure mechanism associated with long term weathering. In Phase II the affects of weathering was addressed.

The soil cement mixture tested by SLA in Phase II [9], was mixed in a portable cement mixer and consisted of road base aggregate no larger than 3/4 inch, type II soil, no. 2 Portland cement, and water. The d_{50} for the aggregate was 0.95 mm and 8 percent (by dry weight) Portland cement was used, yielding a 28 day compressive strength of 880 psi. The mixture was placed in lifts which were 4 inches high and 3 feet wide by 4 feet long, overlapping such that the slope resembled a staircase. The top step had an anchor trench, 18 inches deep and 12 inches wide, which was embedded into the downstream edge of the embankment crest, leaving much of the embankment material on the crest exposed. Two different configurations were tested; one with soil cement placed on a 3H:1V slope and one with soil cement placed on a 2H:1V slope.

For tests on a slope of 3H:1V, the soil cement steps overlapped two feet and was compacted with a hand tamping device to 95 percent of standard Proctor density. In order to simulate weathering effects, the soil cement was subjected to freeze-thaw and wet-dry cycles throughout the 4 months that it was allowed to cure. Due to this exposure, there was surface flaking on the soil cement steps of less than 0.2 inches, but no disintegration or through-going cracks.

For tests on the 2H:1V slope, the soil cement was compacted with a vibrating plate compactor and allowed to cure for 7 days. At two different locations along the downstream slope an artificial cold joint (12 mil black plastic) was placed between two soil cement lifts.

Soil Cement was stable, under both testing conditions, with velocities reaching 16 fps under a discharge of 96 cfs and shear stresses up to 47 psf for a 10 hour duration.

The only noticed damage was a "rounding off" of the step edges and degradation of the exposed portions of the crest. Thus, the critical velocity of the soil cement tested by SLA was reported as greater than 16.0 fps and the critical shear stress as greater than 45 psf. SLA also report a cost range for an eight foot wide step configuration constructed out of soil cement as \$1.4 to \$6.0/sq.ft. (1986). The cost will vary dependant on the percentage of concrete applied to the design mix.

2.6.2 RCC

The literature concerning Roller-Compacted concrete is quite extensive, with most of the focus on case studies of rehabilitated dams. Over thirty dams in the united states have been modified using RCC and several have since been subjected to overtopping flows [24,25]. For example, a detention dam on Ringtown Dam No. 5, in Ringtown Pennsylvania, is a RCC structure that acts as a service spillway . The structure was constructed, in 1980, as an emergency effort to control the amount of debris transported downstream after Mt. St. Helens erupted. No outlet structure was constructed, therefore for 11 months the entire river was passed over the RCC. The velocities for the first six months were estimated to be 15 to 20 fps and during this time the sediment and rock filled the reservoir area up to the crest of the spillway. In the next 5 months, rocks up to 1 foot in diameter were being passed over the structure at high velocities. In 1982, the volcano erupted again and refilled the reservoir with debris. This time overtopping resulted in a breach of the embankment in two different places. The breached areas passed the river and left the spillway free to be analyzed. The surface of

the RCC was ground to a terrazzo-like finish and at one location the RCC had eroded about 6 inches deep. The limited damage under high velocity flows and much debris shows the relative strength of RCC as an overtopping protection alternative.

Another case where overtopping occurred on a RCC dam is that of Kerrville Ponding Dam near Kerrville, Texas [26]. Originally Kerrville Ponding Dam was constructed as a replacement dam for an earthen embankment that failed during overtopping. The RCC structure was overtopped by heads greater than 14 feet only thirty days after completion. For four days the flood waters passed over the entire dam. Two years later, the RCC structure was subjected to an overtopping head of about 16 feet. Neither of these events caused any noticeable erosion of the RCC, except for the washing away of some poorly compacted material on the downstream face.

An article in the October 1992 issue of Civil Engineering entitled "RCC at 10" [27], by John Prendergast, discusses the merits of RCC as a low cost and rapid construction method. Two basic methods of approach are identified for the mix design: soils-based and concrete-based. As would be expected the concrete-based method uses more cementitious materials but, when compressive strength is the main criterion, the soils-based method is noted as capable to provide the needed strength without added costs for cementitious materials. Portions of the article were based on material presented at the February 1992 specialty conference, Roller Compacted Concrete III, and many quotes were cited by Kenneth D. Hansen, senior water resources engineer with the Portland Cement Association (PCA) affiliate in Denver. Because guarding the upstream face against seepage is a major concern for RCC protection, Hansen listed four main

types of upstream facings: (1) membrane-faced concrete panels; (2) reinforced and jointed concrete facing; (3) unreinforced concrete facing with water-stopped joints; and (4) unreinforced concrete without water-stopped joints.

Hansen also wrote an article which was published in Civil Engineering magazine, in August of 1993, entitled "RCC: Rehab Results" [25]. Along with a description of the overtopping at Ringtown Dam, the article contained a discussion of design development for RCC overtopping protection. For example, mortar bonding of the top two to three layers is suggested by Hansen, "to increase the shear resistance of the RCC layers where there is little normal force above the lift line". With consideration to uplift pressures, drainage systems should be incorporated with adequate capacity to avoid saturation. The structural design of the RCC protection should be based on the same design principles used for conventional chute spillways, erosion cutoffs, underdrains, anchorages and stilling basins.

Testing has been done by the USBR on concrete stepped overlays to determine the hydraulics associated with overtopping flow on step surfaces both horizontal and sloping. More details are given about the USBR's testing in section 2.7.2

Table A.1 in Appendix A, summarizes some of the characteristics of 30 dams that have been modified using RCC as armor protection for the embankment or spillway, in the past 15 years [24]. From the data tabulated in Table A.1 a range of costs for RCC as overtopping protection can be estimated as 30 to 90 dollars per cubic yard. It was also noted by Hansen that the greater the quantity of RCC to be placed, the lower the unit cost because, the basic equipment costs will be distributed more widely.

2.7 Prefabricated Concrete Blocks

In recent years, a great deal of the research effort for armor protection has been focused on the behavior of precast concrete blocks under the conditions of high velocity flow. Two general types of blocks which are the basis for design are cellular blocks and wedge-shaped blocks. Cellular blocks are designed in a grid-like fashion with openings that can be filled with soil and seeded for grass growth. Different brands of cellular blocks have been tested, some of which are cable-tied and others which are non-tied or interlocking. The wedge-shaped blocks were originally designed in Russia in such a way that takes advantage of the hydrostatic forces exerted by overtopping flow. Since their original design in the mid 1970's, these Russian blocks have undergone several tests and experienced beneficial modifications as a result of testing. In this section, the discussion will be broken up into the two subgroups of cellular and wedge-shaped blocks.

2.7.1 Cellular Blocks

In much of the literature, cable-tied cellular blocks are referred to as Cellular Concrete Mats (CCM) or Articulated Concrete Blocks (ACB). These prefabricated mats of precast cellular concrete blocks, or grids, are tied together by cables and placed as a mattress on top of a geomembrane fabric or bedding material. The non-tied interlocking blocks are precast concrete blocks designed to interlock in the plane of the slope (plan-bonded), vertically (stack-bonded), or both. The interlocking blocks are typically placed by hand, block by block on a geomembrane fabric or bedding material. In the past, the USACE has used cable-tied, cellular concrete mats for the protection of vulnerable

coastlines [28]. Figures A.8-A.11 in Appendix A, depict sketches and photographs of cellular blocks and their application at Blue Ridge Parkway Dams.

CIRIA conducted tests on articulated concrete blocks along with the previously mentioned tests of plain grass and geotextiles at Jackhouse Dam (sections 2.2 and 2.3). Three different cable tied blocks, Armorflex, Petraflex, and Dycel were tested at Jackhouse Dam with an underlying geotextile and good grass growth. Non-tied, interlocking Dymex blocks without a geotextile mat and Grasscrete, an in-situ concrete surface with cells for grass growth were also tested. Dymex was the only block system that failed during the testing. The limiting velocity was recorded as 16 fps with a duration of 1 hour. Damage was noted in all of the systems near the following velocities and durations: Dycel -23 fps after 3 hours; Petraflex - 26 fps after 3 hours; Armorflex - 23 fps after 2.5 hours; and Grasscrete - 26 fps after 3 hours. "Damage" was categorized by CIRIA as loss of grass and soil from block cells, while "failure" was categorized as the uncontrollable erosion of the subsoil. Some of the characteristics of the concrete systems tested by CIRIA are listed in Table 2.8

The previously mentioned Phase II (section 2.3) tests by SLA also involved the testing of cable-tied concrete blocks with geotextile underlayers, without grass, using the same flume as other tests and an embankment constructed out of Type I soil. The block systems tested in this study were Armorflex, Petraflex-Vick and Dycel 100, with slightly different characteristics than those listed for CIRIA's tests in Table 2.8. The characteristics of concrete blocks tested by SLA are summarized in Table 2.9.

Table 2.8. Summary of Characteristics of CIRIA-Tested Cellular Block Systems [9]

SYSTEM	THICKNESS (in)	WEIGHT (psf)	CABLING
Dycel 100	3.94	29.4	Longitudinal
Petraflex H41212(.)	3.74	32.8	Lateral and Longitudinal
Armorflex 140	3.35	27.7	Longitudinal
Dymex	3.94	22.7	none
Grasscrete GO	3.94	30.7	none

Table 2.9. Summary of Characteristics of SLA-Tested Cellular Block Systems [9]

SYSTEM	THICKNESS (in)	WEIGHT (psf)	CABLING
Dycel 100	4	33	Longitudinal
Petraflex-Vick	4	42	Lateral and Longitudinal
Armorflex	4.75	36	Longitudinal

In SLA's tests, failure occurred with the Dycel 100 blocks on a 2H:1V embankment slope with an overtopping depth of 1 foot and a maximum velocity of 13 fps. This failure was attributed to "liquefaction of the embankment soil with a resulting shallow-seated slip surface", caused by uplift.

The Armorflex block system, tested by SLA, were stable on the 2H:1V slope with overtopping depths of 1 and 2 feet, under velocities of 19 fps. The Armorflex blocks eventually failed when overtopping depths reached 4 feet. Just as with the Dycel 100

system, negative pressure at the downstream end of the crest and block flutter, caused uplift of the system and allowed saturation of the embankment soil. Failure was due to liquefaction of the embankment soil.

The Petraflex-Vick system, tested by SLA, was the most stable cellular block system tested for overtopping, showing no signs of instability on the 2H:1V slope with overtopping depths from 1 to 4 feet and velocities up to 21 fps.

In comparison to the tests conducted at CIRIA, SLA's testing revealed lower limiting velocities. The lower limiting velocities reported for the concrete block systems tested by SLA were attributed to the conditions under which each were tested. CIRIA's block systems had an established grass growth which allowed the formation of a well developed root system to form, while SLA's systems had no grass growth. The embankment material for CIRIA's testing was clay and SLA used Type I soil as embankment material. The channel geometries were also different. CIRIA's channels were trapezoidal and SLA's flume was rectangular. Also the flow fields imposed on CIRIA's Jackhouse Dam did not truly overtop the embankment, as the discharge was released directly onto the embankment slope. SLA simulated actual overtopping flow, allowing the water to overtop the crest. Therefore, the limiting velocities reported by SLA, are more conservative and a better estimate of limiting velocities for actual overtopping flow conditions. Table 2.10 shows a comparison of critical velocities reported for each block system by both CIRIA and SLA. The critical shear stress was not available from CIRIA testing, but the critical shear stresses reported by SLA are included

for comparison with other tested alternative protection measures that may not have reported critical velocities (i.e. geotextiles and riprap).

Table 2.10. Comparison of CIRIA & SLA Reported Critical Velocities for Cellular Block Systems And SLA Reported Critical Shear Stresses [9]

BLOCK SYSTEM	CRITICAL VELOCITY (fps) CIRIA - with grass	CRITICAL VELOCITY (fps) SLA - without grass	CRITICAL SHEAR STRESS (psf) SLA - without grass
Dycel	23-26	< 7.3	< 7.0
Petraflex	>26	> 17	> 32
Armorflex	23-26	12-15	12-20
Dymex	15	Not Tested	Not Tested
Grasscrete	26	Not Tested	Not Tested

CIRIA - clay as underlying soil

SLA - Type I soil as underlying soil

In addition to critical velocities, SLA also reported critical shear stresses and maximum uplift pressures for each system tested. The uplift pressures were measured with pressure transducers located at the brink, midway down the downstream slope and just before the toe of the embankment. It was believed that the weight of the block and the cable tying of the blocks, could reduce the instability caused by uplift pressure.

The results from both of the agencies testing suggested that the weight of the block is a stability factor in the revetment system, because the heaviest block in both series of tests, Petraflex, resisted the highest velocities without instability. It is also

noted that grass growth is not essential for adequate protection at low to mid range flows, because in SLA's testing no grass growth was present.

Before installing any type of concrete block system it is essential to be sure that the blocks have been tested for overtopping flow conditions, because as testing results show, different blocks behave differently. Also, during installation an underlying geotextile mat and proper anchorage of the blocks are crucial. Good grass growth will enhance stability, and is recommended by CIRIA not only for stability but also to deter vandalism.

Because of the limited maximum discharge of the test facility at Jackhouse Dam, no failure was observed of the cable-tied concrete block systems that were tested. Therefore, R. Baker [29] of the University of Salford, under sponsorship of the Science and Energy Research Council (SERC), began laboratory testing, in 1987, to determine the factors that affect the stability of concrete blocks. A scaled test rig was constructed with a 1.5H:1V slope, with a maximum flow of about 14 cfs, generating peak velocities near 25 fps. Blocks used in this scale model were a quarter of the size tested in the full scale CIRIA tests. Using Froude number similarity, the scaled model's peak discharge is reported to represent a prototype flow of 40 cfs/ft and velocity of about 49 fps. In this series of tests, the pressure fluctuations were measured by the inclusion of two brass blocks equipped with pressure transducers. One block had transducers at the center of each face and the other had a row of three pressure transducers on the upper face only. The pressure transducers were connected to a data logger sampling at 100 Hz.

The results from SERC's tests revealed that failure is due to spatially varied, flow-induced pressure fluctuations. Thus, the weight of an individual block is not responsible for stability in a cellular block system, as once believed. The inter-block friction holding the system together is the main stability factor. Once the inter-block friction force is surpassed, failure will occur. During testing, gravel particles were brushed into the joints between the blocks in a non-tied system to increase the inter-block forces. The block systems with enhanced inter-block forces withstood velocities representing about 43 fps while those tested without enhanced inter-block forces failed at less than 33 fps.

For several test runs one block, in a non-tied system, was raised slightly higher than the others to observe how failure progresses. As expected, no matter what location it was in, this block was lifted first and as a result, progressive failure of the entire system was induced almost instantaneously. When testing cable-tied systems, the failure was similar, but the cable only allowed partial lifting of the block. Thus cable-tying can prevent progressive failure of the entire system unless failure is defined as the lifting of one block.

In 1991, the first United States application of cellular concrete mats (CCM), as armor protection to an existing dam, took place on three Blue Ridge Parkway dams, Bass Lake, Price Lake, and Trout Lake [30]. All three dams are classified as high hazard, with expected overtopping depths due to the PMF ranging from 1.9 feet at Bass Lake Dam to 4.1 feet at Price Lake Dam and maximum velocities ranging from 22 fps to 26 fps respectively. The CCMs extend from the crest to the downstream toe of each dam. For Price Lake Dam a reinforced concrete apron was provided, because the velocities may

exceed 26 fps (a limiting velocity according to CIRIA testing). However, at Bass and Trout Lake Dams the CCM was extended about 40 feet below the toe of each dam for an apron. The CCMs were either buried 5 feet deep and anchored with helical anchors or attached to a 10 feet deep concrete toe wall. On the downstream slopes and in the apron area, helical and duckbill anchors stabilize the mats.

For the parkway dams, the cellular blocks used had open areas of 15% to 35%, weighed 40 to 45 psf, and were tied laterally and longitudinally with polyester cables. An underlying woven geotextile was provided for filtration and the allowance of penetration of grass roots on the embankments. The blocks were filled and covered with seeded topsoil to provide a natural looking appearance.

The designers of the parkway dams' protection system suggested field testing of any anchors to be used in future designs. The need for full-scale model tests and field performance data to verify various anchors, anchor spacing, splices and connections spacing, and to explore the additional anchorage provided by well-established grass and the effect of turbulence at the embankment toe was also identified by the designers.

Since the modification of the three parkway dams, at least three other locations have also used cellular concrete mats for armor protection. Table A.3, in Appendix A, summarizes the characteristics of the parkway dams along with dams located on Wilkerson Lake, Strahl Lake, and Sandy Run [41]. Although no cost information for the cellular mats was reported in Table A.3, R. L. Wooten et. al [30] provided a range of costs from contractors bidding for the job of applying Cellular-concrete mats to the three Parkway dams. These bids ranged from 24 to 69 dollars per cubic yard.

2.7.2 Wedge-Shaped Blocks

Two different configurations of wedge-shaped blocks have been tested for use in spillway protection, and for overtopping protection on earthen embankments. The original design consists of butt-jointed, wedge-shape blocks with drainage holes and metal ties to hold the blocks together (see Figure A.12, Appendix A [34]). A variation to the original design is an overlapping wedge-shaped block with drainage vents and no tie restraints (see Figure A.13, Appendix A [41]). The overlapping blocks have an advantage in that they provide more inter-block restraint. Both types of blocks are laid out on the embankment such that the side joints of each successive row are staggered to discourage flow from concentrating at the joints. An underlayer must be provided in order to; assist the relief of excess hydraulic pressure through drainage vents, prevent movement of subsoil particles due to seepage, provide an even foundation, and to protect the subsoil from erosion by drainage flow. This underlayer can be one or two layers of granular material, a geotextile or a combination of geotextile and granular material.

Russian Testing

Yuri Pravdivets, director of the Mitishchi Branch of the Moscow Institute of Civil Engineers, invented and initiated the testing on a wedge-shaped block with drainage holes, placed in a step profile and designed to withstand the hydraulic forces of overtopping flow. The stability of the block is said to "increase as a result of an inertial hydrodynamic force on the upper surface of the block brought about by the curvilinear fluid motion at the facing" [31]. This additional hydrodynamic force results in a reduction of the required

thickness, compared to flat concrete blocks of the same size. Prototype testing has been continuously carried out for several years at the construction site of Deneister power scheme in Moscow [32]. The testing structure is an overflow cofferdam, 23 feet high and 820 feet long. The test section is 66 feet wide with approximately 3 by 3 by 1 foot, precast wedge-shaped blocks. The downstream face gradient of the blocks is 4.5H:1V on an embankment slope of 1.5H:1V. The blocks are underlain with a drainage layer of 8 to 12 inch diameter riprap. Several floods and ice flow have passed over the cofferdam with satisfactory results. Peak unit discharges reached 459 cfs/ft in 1978 and 1979. [31].

The testing for stability of a proposed spillway in Russia was done at the Dneiper power station in 1977 [33], with the following facility characteristics. A 46-foot wide 118-foot long test chute was constructed in the existing spillway dam on a 6.5H:1V slope and protected with wedge-shaped blocks which were 10 by 6 by 2.3 feet. The blocks were subjected to an overtopping head of 115 feet, with unit discharges of 2,100 cfs/ft and mean velocities of 56-75 fps. The wedge-shaped blocks proved to be stable under these conditions.

The underlying drainage filter and the dimensions and spacing of drain holes have a strong impact on the stability of the blocks. To prevent piping at the interfaces of the layers and provide free seepage under to protective facing it is recommended by Pravdivets that the filter base grain size relationship be $d_{15}/d_{85} = 4-5$ [31]. The filter material can be placed in 2 or 3 layers, depending on the embankment, with a thickness at least 8 inches or 3 to 4 times d_{85} . It is essential that the drain holes be smaller than the smallest grain size of the uppermost filter material so the filter material will not be sucked up through the

drain holes. Pravdivets also reported that the spacing of drain holes typically should be 4 to 5 diameters in the longitudinal direction and 5 to 6 diameters in the lateral direction, with the last row of holes in the flow direction no more than two step heights away from the next block. A non-submerged surface hydraulic jump is noted as the best regime for a stepped toe.

The slope of the embankment that wedge-shaped blocks can be applied to depends greatly on the underlying material. Pravdivets recommends a mild slope of 8H:1V for clays and silty clay soils, while higher strength granular materials have supported stepped block spillways on 2H:1V and 1.4H:1V slopes.

Great Britain testing

CIRIA proposed a evaluation of the performance of wedge-shaped block in 1987 [34] citing technical literature from Russia, [31,35,36] and listing the following attributes of the wedge-shaped blocks:

1. The flow pattern over each block is such as to create a curvature of flow which imposes a downward force on the block, thus increasing its stability.
2. A low-pressure separation zone forms downstream of each step thus enabling any seepage flow beneath the blocks to be evacuated with consequent relief of potentially disruptive uplift forces due to seepage pressures.
3. The large surface roughness helps to dissipate flow energy.

In their proposal, CIRIA also provided the following table (Table 2.11) of indicative size data for wedge-shaped concrete block protection in the terminal velocity zone on a typical 3H:1V slope.

Table 2.11 Characteristics of Wedge-Shaped Blocks [34]

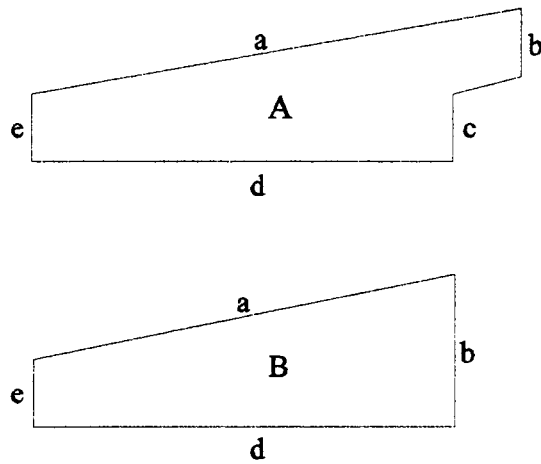
OVERTOPPING DEPTH, H	DISCHARGE INTENSITY	AVERAGE BLOCK THICKNESS	TERMINAL VELOCITY, V_t
6.7 ft	54 ft ² /s	9 inches	30 fps
19.7 ft	269 ft ² /s	14 inches	46 fps

Also from the Russian literature, CIRIA noted that the effectiveness of the blocks has been tested, in the field, at discharge intensities up to 640 cfs/ft with mean velocities of 66 fps.

CIRIA began their testing of wedge shape blocks using the same model facility at Salford University that was used in testing flat concrete blocks. Figure 2.1 represents the shapes and sizes of the two different wedge shapes tested, type 'A' and type 'B'. Three different sizes of type 'A' and one size of type 'B' were studied.

The smallest block tested, which was constructed out of epoxy resin, was reported to represent a block with a 4 inch average thickness and the maximum flow was representative of 215 cfs/ft. The other blocks were cast in concrete and one brass block was included in the testing of the larger size block with pressure tappings along the perimeter. The blocks were extremely stable in free flow conditions, with no signs of movement at the peak discharge. All of the blocks slid on the 2.5H:1V slope at low flows,

if no toe restraint was provided. The overlapping wedge block, type 'A', became more stable if the discharge was raised, because the down force on the overlap held the downstream block to the slope. The test was run with a complete row of blocks (type 'A') missing and they did not fail until the flow was turned off. A geotextile based under drain was used in each run.



	BLOCK A (in.)			BLOCK B (in.)
Size	Small	Medium	Large	Medium
a	1.20	2.40	7.2	1.96
b	0.18	0.36	1.08	0.91
c	0.22	0.44	1.32	N.A.
d	0.94	1.88	5.64	1.9
e	0.20	0.40	1.20	0.43
block width	1.0	2.0	5.88	2.0

Figure 2.1. CIRIA Model Wedge-Shaped Blocks [34]

From their testing of wedge-shaped blocks, CIRIA reported that the step height to tread ratio, and the angle of the top face relative to the bed slope, are important in the design of the block shape. It was also noted that instability occurs at the toe under high tailwater conditions and that an underdrain with an outlet at the toe will increase the stability of flat or wedge shape solid concrete blocks. CIRIA also concluded that "a wedge shaped block offers the most stable design with substantial savings in concrete thickness," [29] adding that the stability of the saturated embankment and the design for energy dissipation at the toe were probably the most critical design criteria. CIRIA is currently analyzing the results from this and other tests in order to create a design guide for stepped block spillways [37].

United States Testing

In 1990, the USBR began testing "stepped concrete overlays" using a 1.5-foot wide laboratory flume with the capability of being adjusted to any slope between 2H:1V and 4H:1V [38]. Model unit discharges up to 14 cfs/ft and up to 2.8 feet of overtopping head were simulated. Velocity profiles were measured using a laser doppler anemometer in order to determine the dissipation of kinetic energy. To determine the stability of the overlay and drain placement, surface pressure profiles were also measured. The three different geometries shown in Figure 2.2 were the subject of testing and they are described as horizontal steps, steps sloped 10 degrees below horizontal, and steps sloped 15 degrees below horizontal.

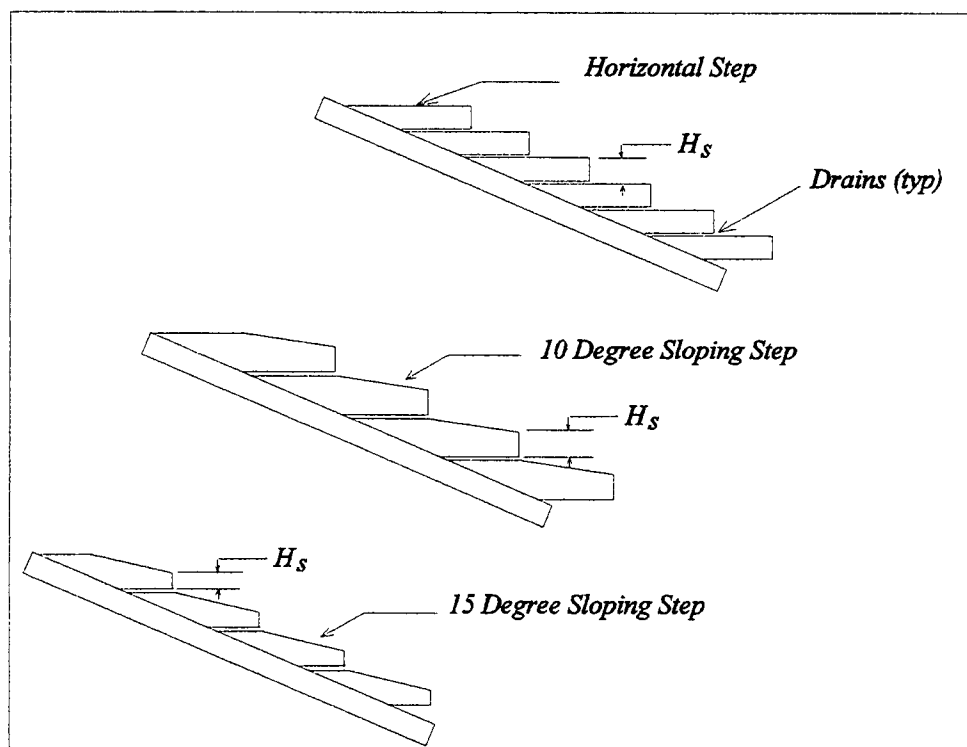


Figure 2.2. Step Geometries Tested by USBR [38]

Through the flume testing of these steps, design data for drain placement, stability, and energy dissipation were provided. It was determined that the pressures acting on the surface of each step depends on the geometry of the step, overtopping head, and the distance down the slope. The greater the ratio of overtopping head to step height, the less likely that a negative pressure will be generated. Without the generation of a negative

pressure zone, aspiration of the flow surface drains will not occur. For the horizontal step geometry tested, all of the measured pressures were positive, therefore no aspiration occurred. The 10 degree sloping step developed subatmospheric pressures, just a few steps down the embankment slope, under low overtopping flows. The steeper 15 degree sloping step was said to "further enhance the development of subatmospheric pressure on the step surface" [38]. The horizontal steps tested showed more energy dissipation than the 15 degree slopes.

According to the flume tests of the USBR, both horizontal and sloping step geometries have benefits and limitations. Sloping steps produce aspiration of flow surface drains and also cover more surface area of the embankment, but do not dissipate as much energy. On the other hand, the horizontal steps dissipate more energy, but provide no aspiration. With a greater dissipation of energy the need for downstream erosion protection is reduced, while drainage concerns are reduced with aspiration. Thus, the appropriate step geometry depends on the specific needs of the structure being overtopped.

The next phase of study for the USBR was to test the stepped concrete overlays on a prototype embankment. The facility used for testing was a full scale 50 foot high embankment dam in Ft. Collins, Colorado. For this series of testing, the wedge blocks studied were similar to those tested by CIRIA pictured in Figure 2.2. Two flat blocks with sloping faces of 10 and 15 degrees were tested along with an overlapping block. As mentioned previously in section 2.4 (riprap), the final report is not yet available but some of the results have been published in ASDSO conference proceedings and the results concur with the flume testing [39].

Because no known dams in the United States have used wedge-shaped blocks for overtopping protection cost information is not readily available. At this time, a wedge-shaped block cannot be ordered from a vendor like the cellular blocks can. The block shape would first have to be designed and then a mold must be made in order to cast the block. Through an informal survey of concrete manufacturers, the USBR received a wide range of approximate costs for the overlapping wedge shape block, spanning from \$5 to \$20 per block [40].

CHAPTER 3

ASDSO STATE SURVEY

3.1 General

Several dams throughout the United States, and other countries, have been modified, in situations where spillway capacity is inadequate, using one or more of the armoring alternatives described in Chapter 2. These dams have been armored in an effort to prevent erosion in the event of overtopping, which in effect has not been prevented by modification. The National Research Council, supported by the Federal Emergency Management Agency (FEMA), addressed the issue of embankment overtopping in a 1983 report entitled Safety of Existing Dams Evaluation and Improvement (SEDEI) [1]. The following quotation is taken directly from this publication to illustrate the thinking at that time about allowing overtopping.

Overtopping

In the engineering of permanent embankment dams, overtopping has been strictly avoided, for sound reasons. While the advantages of earthfills are well known, their vulnerability to erosion unless properly safeguarded is basic. There is also ample record of the dislodging of rockfills subjected to overspill. An almost inviolable rule in the design of these structures has been to keep them free of superimposed conveyance structures, whether they be spillways, fish ladders, or pipelines. Any such facility that would obscure the embankment from inspection or conceal underlying adverse conditions has been regarded as objectionable.

These rules are well founded, and any deviation from them should not be taken lightly. However, confronted with the current reality of numerous dams of this type that are inadequately protected from flood damage and considering the insufficiency of funds for immediate full-scale increase in spillway capacity by traditional methods, the engineer may be obligated to weigh the merits of passing

water over the embankment, at least as a temporary expedient.

Remedial Measures

Various ways have been proposed to protect embankments during overtopping. Principal among these have been either armoring or reinforcing the embankment. Also the use and/or contribution of parapets have been proposed to prevent the overtopping.

While the overtopping of earthen embankments is discouraged and is not recommended, if overtopping is justified by the engineer, provisions may be made to allow such action. Not only does SEDEI acknowledge the conditional allowance of overtopping as stated above, but in addition, armor protection is mentioned as a principal remedial measure in such cases.

In an effort to investigate the current thinking about the allowance of overtopping of non-federal dams at the state level, a survey was sent to each of the fifty ASDSO (American Society of Dam Safety Officials) state Representatives. A copy of the survey form can be found in Appendix A as Figure A.14. The responses received to the following three questions are summarized in Table 3.1.

1. Does your state have any procedures which allow embankment overtopping?
2. Does your state allow the option of any kinds of embankment protection or armoring for inadequate spillway capacity?
3. If protection is allowable, are there any conditions or limitations to what could be done?

3.2 Results

Of the 45 states that responded to the survey questionnaire, twenty-seven percent (12 states) reported that their state has procedures which allow embankment overtopping,

while seventy-three percent (33 states) do not. Although a large number of states do not have written procedures allowing embankment overtopping, many will consider allowance of overtopping on a case-by-case basis, if it is approved by the state and it can be proven that failure will not occur. Of the negative responses, twelve percent either consider overtopping as "failure" and expressly prohibit it, or strongly discourage the intentional overtopping of earthen embankments.

In response to Question 2, regarding the allowance of armor protection for inadequate spillway capacity, seventy-three percent reported that they would consider armor protection as a valid option for rehabilitation, while twenty-seven percent replied "no". Of the positive responses, the main stipulations mentioned were that the proposed design be in accordance with good engineering practice and that acceptance would be evaluated on a case-by-case basis. Of the negative responses, a few states claim they would consider an armoring option under unusual circumstances, but prefer not to allow armor protection as an option. For example, California rejected a armoring design which had been approved by the Federal Energy Regulatory Commission (FERC) due to high seismicity in the area. New York stated that overtopping was considered failure; however, the state said that they may allow overtopping in "marginal" cases. In fact, one case has been conceptually allowed in New York, but the owner has not proceeded. Utah prefers other methods for rehabilitation, but noted that they would consider armoring as an alternative for short duration storms with low overtopping depths (1 foot or less).

Some states submitted particular conditions or limitations to armor protection, in response to Question 3. While most states just require good engineering judgement or

proof of no failure, some were more specific. For example, Washington reported that the only alternatives the state allowed for armoring are gabions and RCC overlays. In Idaho, armor protection is allowed for embankment dams with "physical limitations", if flood routing the inflow design flood (IDF) is the only viable alternative to offset inadequate spillway capacity for emergency release. Both depth and duration of the peak flow would need to be considered to determine the embankment protection needed to resist erosion or displacement of the embankment or foundation and any proposed armoring protection must be studied by modeling. New Jersey requires flume tests of any proposed armoring protection to assure that it will withstand expected flows. Depending on the dam size and classification, New Jersey may still require, on a case by case basis, that the principal spillway pass a specific flood while allowing a larger event to overtop.

Along with the inquiry concerning the application of overtopping protection for the modification of earthen embankments, a fourth question was attached which asked for any names, and or contacts, for dams under each state's jurisdiction where overtopping protection was used as a means of remediation. From the responses to this question, Table 3.2 lists several non-federal dams which have employed armor protection, throughout the United States.

The growing use of armor protection and the responses from this survey indicate that with time and technology, the idea that *all* embankment overtopping should be assumed to be a "dam failure", is rapidly changing. Armor protection is actually being considered, by many states, as a viable option to rehabilitate dams having insufficient spillway capacities. For example, the state of Tennessee has general procedures which

allow overtopping, and considers armor protection an option for inadequate spillway capacity, as long as the method used will keep the dam from failing under the inflow design flood, as required in the regulations. The following excerpt is taken from the most recent update to the Tennessee Department of Health and Environment Division of Water Supply's rules and regulations applied to the Safe Dam Act of 1973 [2].

(3) Emergency Spillway

- (a) All dams shall have an emergency spillway system with capacity to pass a flow resulting from a 6 hour design storm indicated in (b) below for a hazard classification appropriate for the dam. However, if the applicant's engineer provides calculations, designs, and plans to show that the design flow can be stored, passed through, or passed over the dam without failure occurring, or if he can successfully demonstrate to the Commissioner by engineering analysis that the dam is a safe structure and can certify that the dam is sufficient to protect against probable loss of human life downstream, said dam design may be approved by the Commissioner.

(b) Minimum Freeboard Design Storms

<u>Hazard Potential Category</u>	<u>Size</u>	<u>Freeboard Design Storm (6 hr)</u>
Category 3 (Low)	Small	100 yr.
	Intermediate	1/3 PMP
	Large	1/2 PMP
Category 2 (Significant)	Small	1/3 PMP
	Intermediate	1/2 PMP
	Large	PMP
Category 3 (High)	Small	1/2 PMP
	Intermediate	PMP
	Large	PMP

Because the state of Tennessee considers armor protection a possible alternative for remediation, engineers representing owners of dams with inadequate spillway capacity

can begin evaluating these dams as possible candidates for armor protection. Lake in the Sky Dam, located in Blount County Tennessee (section 2.3), is an existing dam which has already incorporated a geotextile as overtopping protection. Currently, the Tennessee Valley Authority, a federal agency, is investigating the rehabilitation of it's Chickamauga Dam. Chapter 4 provides a statement of the current situation, a brief historical summary, results from hydrologic and erosion studies, and a summary of possible alternatives for remediation. The feasibility of armor protection for this specific site is being examined.

Table 3.1. Responses to ASDSO State Survey

STATE	PROCEDURES ALLOWING OVERTOPPING	ALLOWANCE OF ARMORING	CONDITIONS OR LIMITATIONS
Alaska	No	Not at this time	Not Applicable
Alabama	NO RESPONSE		
Arizona	No	No designs submitted for review. Would review with "healthy skepticism," but not reject out of hand.	N/A
Arkansas	Yes	Yes	From Rules Governing Design and Operation of Dams Section 707.1 A. Spillway design Flood. ... Overtopping during the SDF (spillway design flood) may be allowed if properly prepared analyses demonstrate that: (1) overtopping will have a return interval greater than 25 years; and (2) the dam will withstand the projected overtopping without failure
California	No such procedure	No such option is available	No designs have been approved (one considered) One design approved by FERC but CA rejected due to high seismicity in the area
Colorado	No	Yes	Design must be in accordance with Colorado Regulations
Connecticut	No Response		
Delaware	No	No	Not Applicable
Florida	No	Yes	Permits for construction must be based on flow from 25 yr 24 hr storm events without erosion, as a minimum. Design is the call of the Engineer of Record - subject to review
Georgia	Yes	Yes	The design of the protection has to be done by a registered professional engineer with design experience on dams. The design has to ensure that the embankment will not suffer any structural damage or be breached by the flow over it.

Table 3.1. (cont.) Responses to ASDSO State Survey

STATE	PROCEDURES ALLOWING OVERTOPPING	ALLOWANCE OF ARMORING	CONDITIONS OR LIMITATIONS
* Hawaii	No	Prefer not to - but will consider	Design must show adequate proof and documentation If one imperfect part of system fails the dam may fail.
Idaho	Yes (site specific)	Yes	Allowed for embankment dams with physical limitations if flood routing the IDF indicates overtopping is the only viable alternative (to offset inadequate spillway capacity) for emergency release, considering: depth/ duration of peak flow, which determines the embankment protection needed (by modeling) to resist erosion/displacement of embankment/foundation
Illinois	Yes	Yes	Design must show that protection will remain functional at design storm conditions
Indiana	Select situations	One design has been approved	The protection must meet minimum, current criteria for such design. Conditions and limitations appropriate to the design and site would be placed on the permit.
Kansas	No	Yes	Engineering judgment and practice
Kentucky	No	No - expressly prohibited by state regulation	N/A
Louisiana	No	case by case	Depth of overtopping, duration, and velocity determine the type of armoring that would be allowed, Owner's engineer must certify that proposed armoring will be adequate for the force anticipated.
Maine			
Maryland	Yes	Yes	Open to legitimate designs that can be clearly demon- strated to provide adequate protection
Massachusetts	None specific	Yes site-by -site	None specific, designer must submit a written request explaining design parameters

Table 3.1. (cont.) Responses to ASDSO State Survey

STATE	PROCEDURES ALLOWING OVERTOPPING	ALLOWANCE OF ARMORING	CONDITIONS OR LIMITATIONS
Michigan	none written may allow	case by case	Based on accepted state of the art engineering at the time the issue arises.
Minnesota	No	Yes	Embankment must be designed using sound engineering practice.
Missouri	Yes - if armored	Yes	Available options considered on a case by case basis
Mississippi	Not addressed	Not addressed by state reg.s	N/A
Montana	No	Yes	Limitations are those which might endanger the dam . All aspects of the installation of protection are considered hydraulic adequacy, stability, erosion, back cutting, surcharge, loading of the embankment, and others
Nebraska	None formal	Yes if needed	None needed thus far, so none established
Nevada	No	Yes	Protection must be demonstrated to perform adequately at overtopping conditions.
New Hampshire	No guidelines	Yes	Any sound engineering solution may be accepted. Any modified dam must have the capability to pass the design storm event associated with its hazard classification.
New Jersey	Yes	Yes	Depending on dam size and classification, may still require that the principal spillway be capable of passing a certain, more frequent flood event while, allowing a larger event to overtop. The selected protection must be tested to assure that it will withstand expected flows (flume test)
New Mexico	No	No	NA
New York	No-considered failure Marginal cases-maybe	Not explicitly - Very conservative	One case conceptually allowed - owner has not proceeded

Table 3.1. (cont.) Responses to ASDSO State Survey

STATE	PROCEDURES ALLOWING OVERTOPPING	ALLOWANCE OF ARMORING	CONDITIONS OR LIMITATIONS
North Carolina	Yes	Yes	State allows proven technologies which meet engineering standards and will be hydraulically stable and will not jeopardize the safety of the dam.
North Dakota	No	No	No
Ohio	No	Must be approved by state	For dams which require a spillway design flood of the full PMF, the existing spillway must be capable of passing the 50% PMF
Oklahoma	No	N/A	N/A
Oregon	No	Yes	Any design would require approval
Pennsylvania	Yes	Yes	For existing dams, spillway must be modified to handle the PMF. The engineer must demonstrate that the armoring will withstand the depth and duration of overtopping with an adequate margin of safety. On new dams armoring is only allowed where it will protect the embankment by overtopping of storms in excess of the spillway design flood.
Rhode Island	No - discourage	Could be allowed but no written regulations or provisions	Dependent on site conditions
South Carolina	Considered in rare cases	Considered as last resort	Dependent on site conditions
South Dakota	Yes	Yes	Case-by-case basis
Tennessee	Not specific general	Yes	Must be convinced that whatever method is used will actually keep the dam from failing under the design rain-fall required in the regulations.

Table 3.1. (cont.) Responses to ASDSO State Survey

STATE	PROCEDURES ALLOWING OVERTOPPING	ALLOWANCE OF ARMORING	CONDITIONS OR LIMITATIONS
Texas	Yes	Yes	Must be properly designed and have supporting documentation. The design must be a permanent solution.
* Utah	None written but will consider	Will consider - prefer other method	Not overtopping the entire embankment
Vermont		Yes	Short duration storms - low overtopping depth (i.e. 1ft) Provide project discharge without damage to the spillway or embankment. Design in accordance with good eng. practice
Virginia	No	Case by case basis	Needs to be based on competent engineering design. Needs certification by the owner's engineer and require approval of Virginia Soil and Water Conservation Board
Washington	Yes	Only Gabions and RCC overlays	Gabion overflow sections have been allowed on small dams (<30'), slopes no steeper than 2:1.
Wisconsin	No Response		
* West Virginia	None written	Yes	Must demonstrate that the design is adequate for the purpose intended. Some designs have been rejected.
Wyoming	Yes-(not prohibited)	Review (case-by-case)	Minimum design discharge (hazard class) and possibly up to 1.5 feet of freeboard if abutments subject to erosion

* Response by phone conversation

Table 3.2. Known Dams Under State Jurisdiction Employing Some Type of Armoring Protection Against Overtopping

Type of Armoring	State	Dam Name	Comments
Geotextile	Tennessee	Lake in the Sky Dam	Tensar - polyethylene matting covered with vegetation
Riprap	West Virginia	Lake Chewva	6H:1V slope with 4 ft. diameter riprap; 2 to 3 ft. overtopping head at PMP
		Laurel Lake	6H:1V to 10H:1V slope with 2.5 to 3 ft. diameter riprap; 2 to 3 ft. overtopping head at PMP
		Lake of Woods	4H:1V to 6H:1V slope with 3 ft. diameter riprap 2 to 3 ft. overtopping head at PMP
		Grandview	All sizes of riprap
Gabions	Idaho	Little Wood River Dam	Still under investigation; determined that riprap for construction erosion is sufficient for now
	Tennessee	Steele Creek Dam	riprap
	South Dakota	Stockade Lake Dam	Grouted riprap
	Texas	Bartholomew Park	5H:1V slope; State restricted velocities to 17 fps for gabions (originally 3H:1V with 23 fps) during .5 PMF; Height - 25 ft.
		Regional Detention Pond	Cost - \$125/cu.yd.
	Ohio	Provident Reservoir (1991)	4.6 ft. overtopping at design flood (PMF); 3H:1V to 7H:1V slope; Cost - \$47.50/sq.yd.
		South Dam (1994)	Gabions and RCC protection against 3.0 ft. of overtopping head at design flood (PMF); Embankment also raised 4 ft.
	West Virginia	Twin Branch Dam (1991-2)	Considered a historical structure; maximum velocity : 20 fps; Design storm 500 yr. 2.5H:1V to 3H:1V slopes; 18 inch gabions - keyed in

Table 3.2. (cont.) Known Dams Under State Jurisdiction Employing Some Type of Armoring Protection Against Overtopping

Type of Armoring	State	Dam Name	Comments
Gabions (cont.)	Maryland	Upper Lake Royer (in progress)	15 ft. high dam - gabions less expensive than riprap because only 1/2 of the thickness required
Concrete	South Carolina	Forrest Lake Dam	Pumped concrete into fabric bags and anchored them down on all sides.
	New Jersey	West Branch	Reinforced concrete 20 ft. high dam; .5 PMF; 12 inch thick concrete with weep holes and riprap lining
* RCC	Maryland	Lower Lake Royer (in progress)	25 ft. high dam; 6.5 ft. overtopping head at .5PMF Project will increase 40 ft. spillway to 250 ft. Embankment will also be raised to contain flood
	Michigan	White Cloud Dam	
		Boney Falls Dam	
	Montana	Lima Dam	
	New Jersey	Lake Lenape Dam	
	South Dakota	Ponca Dam	
		He-Dog Dam	
		Rosebud Dam	
		Horsetheif Dam	
	Wyoming	Kemmerer Reservoir	
* Cellular-Concrete Blocks	Texas	Tensleep Reservoir	
	North Carolina	3 Blue Ridge Parkway Dams	Several dams armored with RCC

* See Appendix A: Table A.2 for additional information on RCC protected dams, and Table A.3 for additional information on Cellular-Concrete Block protected dams

CHAPTER 4

TVA'S CHICKAMAUGA DAM

4.1 Statement of Problem

Due to the urban location and height of the Tennessee Valley Authority's (TVA) Chickamauga Dam, the failure of this structure would result in large economic loss and possible loss of human life. Therefore, it has been placed in a high hazard category (consistent with the Federal Guidelines for Dam Safety), and thus should be able to safely pass the probable maximum flood (PMF). At present, Chickamauga Dam's spillway capacity is only about 77 percent of the PMF, therefore, some type of modification is necessary [1].

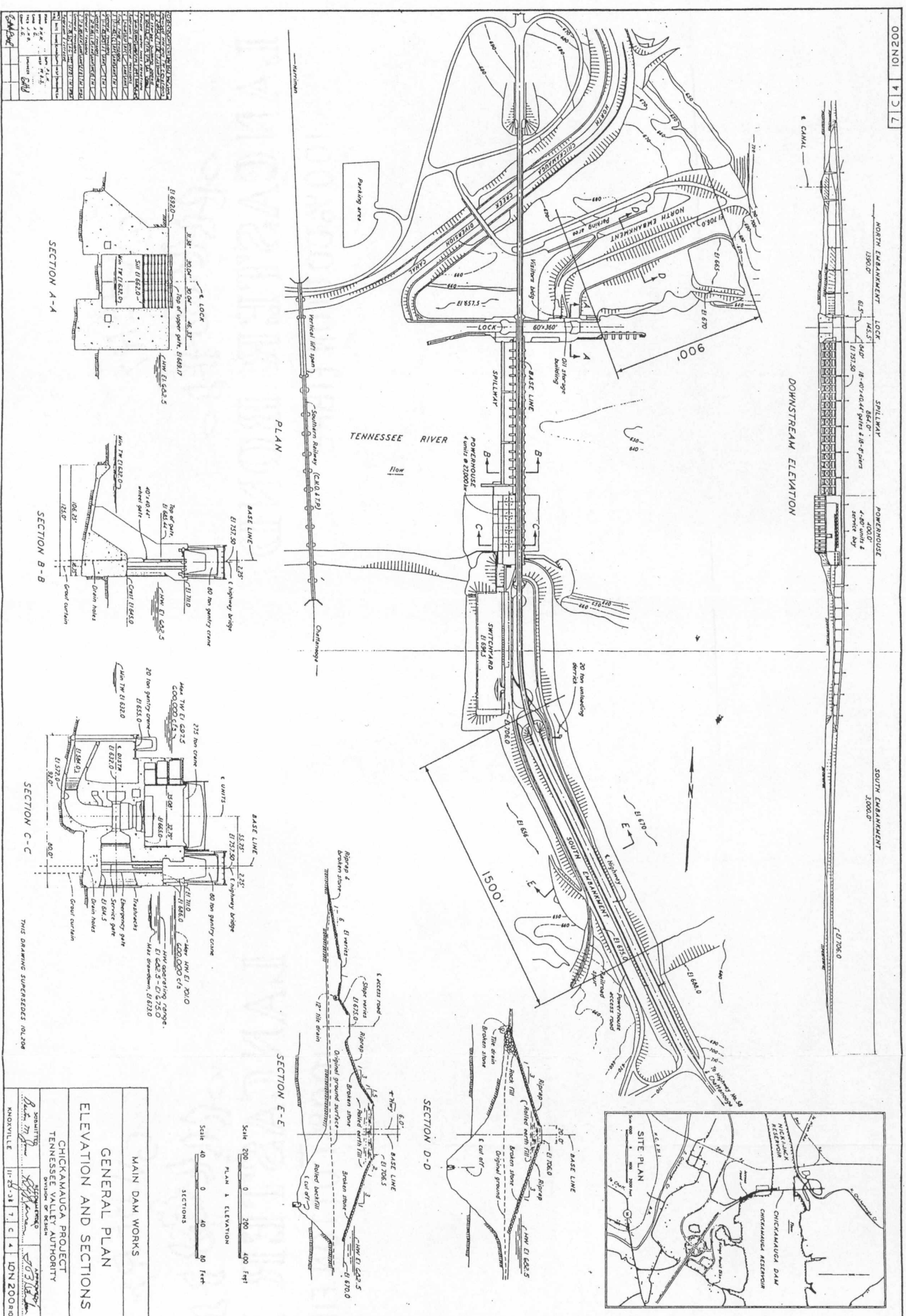
4.2 Project History and Description

The Chickamauga project was constructed by Tennessee Valley Authority in the late 1930's; and the dam was closed January 15, 1940. It is located on the Tennessee River at mile 471.0, in Hamilton County, just upstream from Chattanooga, Tennessee. The city of Chattanooga is built on highly flood prone land along the river and Chickamauga Dam helps to regulate flow past this city. Since operations began, a reported \$1.7 billion in damages have been prevented by Chickamauga Dam, in tandem with other dams located on the Tennessee and tributary rivers [2]. The dam is equipped with a powerhouse containing four generating units of 30,000 kilowatt rated capacity each, resulting in the

generation of 860 million kilowatt-hours of electricity in an average year [2]. Not only is Chickamauga Dam an important asset in flood control and power generation, through the use of a navigational lock, it is also an integral part of the inter-connected inland waterway system from the Great Lakes to the Gulf of Mexico. In addition to all of the dam's flood control and power benefits, the reservoir at Chickamauga provides 35,400 acres of water surface and 810 miles of shoreline at normal maximum pool, which attracts nearly 7 million recreation visits a year [2]. Clearly, as a multipurpose dam, Chickamauga is an essential advantage to Chattanooga and the surrounding region.

The project consists of a concrete gravity dam structure with a maximum structural height of 129 ft and 2 rolled earthfill embankments up to 50 feet high, with crest elevations of 706.0. The length of the north embankment is 1,390 feet and that of the south embankment 3,000 feet. A major highway runs along the crest of the south embankment and bridges over the concrete portion of the dam. A railroad track also runs along the length of the south embankment, and both the north and south embankments are encroached upon by access roads. At normal maximum pool level, (elevation 682.5) reservoir storage is 628,000 acre-feet. The concrete structure includes a 4-unit powerhouse and 18-bay, lift-gated spillway, and a 60-foot by 360-foot navigation lock [1]. Figure 4.1 shows the general plan, elevation, and cross sections of Chickamauga Dam.

The earthfill embankments were constructed on alluvial deposits about 50 ft thick, and consist primarily of relatively impervious clays and silts with riprap on the slopes. The earthfill was obtained from borrow pits on site and spread in layers about 8 in. thick by bulldozers. Tractor-drawn sheepfoot rollers were used to compact the embankments.



Neither of the embankments have interior drains, but, both are equipped with toe drains constructed of 12 in. open-joint, tile pipe laid in gravel along the downstream toe of the embankments [3].

The original design flood used was estimated in 1936 from studies made of maximum observed flood discharge rates. The hypothetical flood adopted for the original design had a peak inflow (unregulated) of 730,000 cfs and was determined from $Q = 5,000\sqrt{A}$ (where A is the unregulated drainage area above Chattanooga in square miles). Upstream storage in Norris reservoir was estimated to reduce the peak inflow to 685,000 cfs, which was adopted for the design of Chickamauga Dam [4].

4.2.1 Hydrologic Evaluation

In 1988, TVA evaluated the hydrologic safety of Chickamauga Dam. Because the social, economic, and environmental impacts would be large if the dam failed, the full PMF was used. At present, a flood equivalent to 77 percent of the PMF, or 847,000 cfs can be passed, without overtopping the dam. The estimated probability of exceeding a .77 PMF is estimated to be in the order of 1/200,000 (or 1/2,000 over a 100-year period) [1].

As defined by the National Weather Service, the PMF would be produced by a storm centered in the Valley and Ridge province to the north of Chattanooga and west of the Great Smoky Mountains. This storm would produce maximum precipitation (PMP) on the 20,790-square-mile watershed above the dam. The storm would most likely occur in March and produce 16.5 inches of rainfall in 3 days. The PMP storm would be preceded by a 3-day antecedent storm producing 6.4 inches of rainfall, which would end 3 days prior

to the PMP storm. The resulting flood would produce a peak discharge of 1,100,000 cfs (1.6 times the original design flood). TVA routed this flood through the existing reservoir system and calculated a spillway discharge of 931,000 cfs. Therefore, the remaining 169,000 cfs will flow over the embankments and lock. By allowing Chickamauga Dam to overtop and assuming no erosion or failure, the maximum water surface elevation was determined to be 716.2, which is 10.2 feet above the top of the earthen embankments and 5.2 feet above the concrete deck. Overtopping is predicted to occur for about 4 days. The concrete structure is considered capable of withstanding the hydraulic loading from the overtopping PMF, however, the earthen embankments would suffer damage due to erosion [1].

4.2.2 Erosion Analysis Summary

Along with the hydrologic and hydraulic analysis, TVA conducted an overtopping and erosion analysis on the earthen embankments of Chickamauga Dam, using the Federal Highway Administration's (FHWA) EMBANK model [5]. EMBANK is a program which was developed by the FHWA to estimate the extent of erosion that would occur to existing embankments. The hydraulic conditions at Nickajack Dam, located 46.3 river miles downstream, have an impact on Chickamauga Dam's tailwater elevations, which in turn affect the amount of erosion predicted. Because future modifications to Nickajack Dam were expected at the time of the Chickamauga study, erosion analysis was conducted for two different conditions. First, the model was used assuming no modifications to Nickajack Dam. The model was run a second time, taking in to consideration the

anticipated future modifications to Nickajack Dam. Subsequent to these analyses, Nickajack Dam has been modified but not exactly like the assumed modifications used in the EMBANK study. Therefore, the velocities and elevations determined using EMBANK are assumed to be the upper and lower limits for the possible range of velocities and elevations associated with the actual modifications which have been made to Nickajack Dam.

Routing the PMF through Chickamauga Dam results in a maximum tailwater elevation between 711.3 and 713.0, and maximum overtopping velocities between 36 fps and 39 fps. Figures 4.2 and 4.3 show the predicted velocity and erosion profiles for both assumptions concerning Nickajack Dam, at the north embankment and south embankment respectively. Within the first 12 hours, the overtopping flow begins to undercut the roadway on the north embankment and after 38 hours, begins to undercut the highway on the south embankment. Submergence would occur after 24 hours of overtopping, thus reducing the velocities and erosion rates. After 4 days, the velocities would be non-erosive due to a headwater/tailwater difference of only 2 ft. Using EMBANK, the predicted erosion would occur above the normal minimum pool level (elevation 675.0), with final eroded crest elevations between 692.0 and 696.3 for the south embankment and between 680.5 and 687.6 for the north embankment [6].

4.2.3 Downstream Consequences Of Erosion During PMF

From the resulting hydraulic analysis and water surface profile generated by EMBANK, it is judged that the Chickamauga reservoir water would not be completely

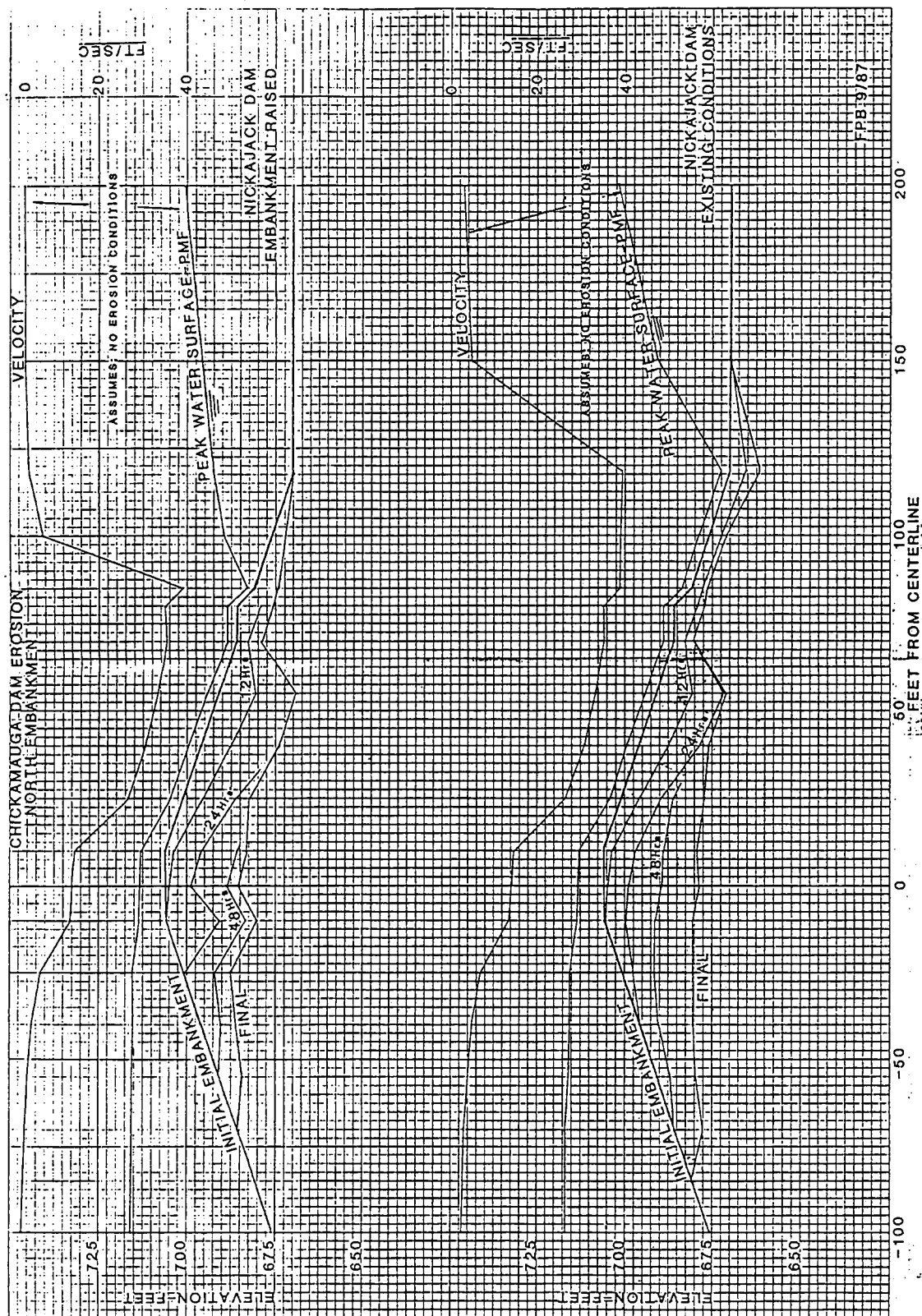


Figure 4.2. Chickamauga Dam North Embankment Erosion, Velocity and Water Surface Profile

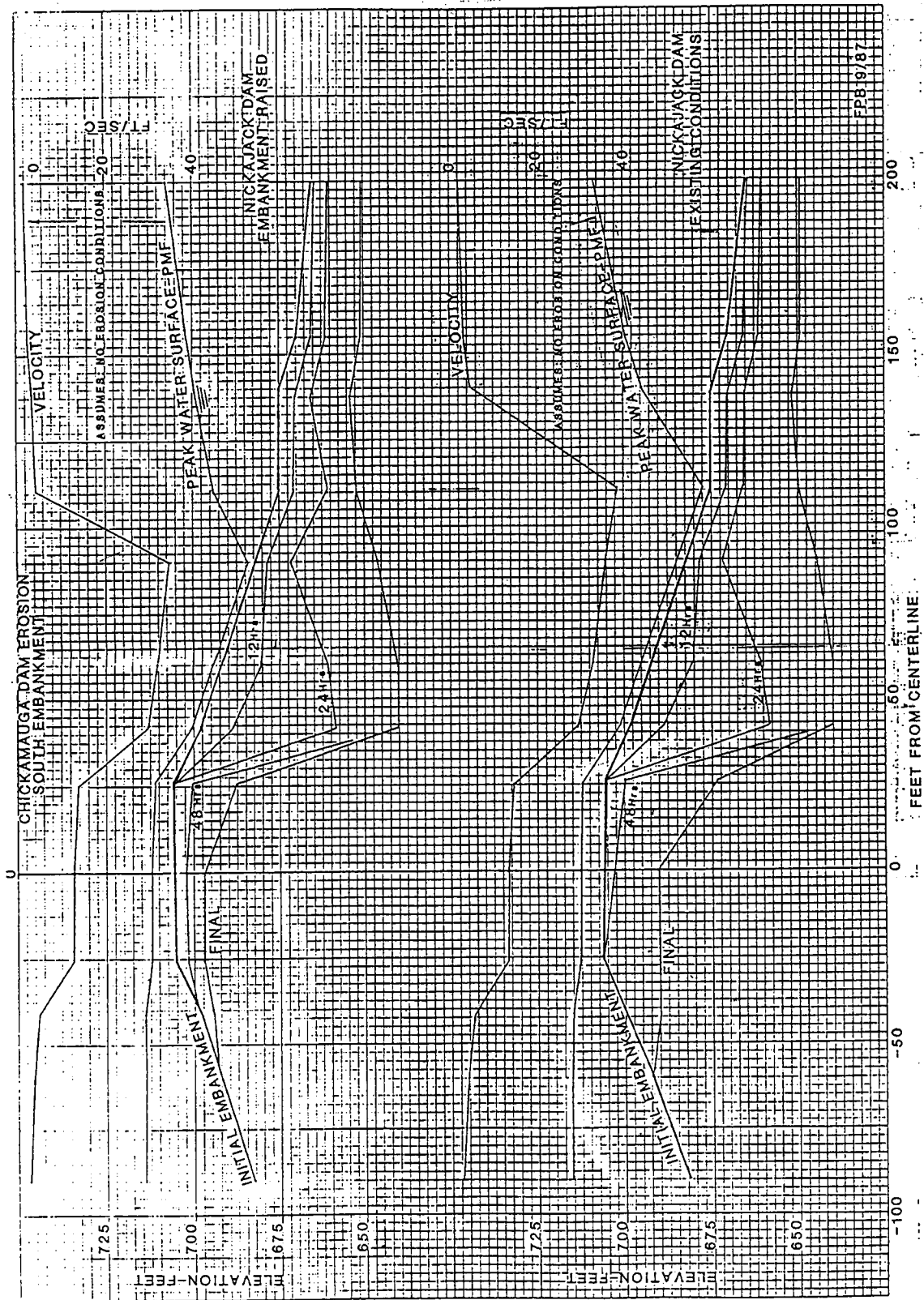


Figure 4.3. Chickamauga Dam South Embankment Erosion, Velocity and Water Surface Profile

released because of embankment erosion; therefore, the only incremental damages would be the damage to the embankments themselves and any increase in flood levels downstream due to the predicted erosion. The incremental damages associated with the PMF at Chickamauga Dam can be defined as the additional damage, above and beyond that which will occur if no erosion takes place. The flood levels downstream of Chickamauga Dam would be increased if the embankments eroded to the aforementioned levels, as compared to the flood levels downstream if there were no erosion to the embankments. The maximum increase in flood level due to erosion is 0.7 feet immediately downstream, reducing to no increase 30 miles downstream in Nickajack reservoir [6]. An estimation of costs associated with these damages can be found in Chapter 5.

4.3 Remediation Alternatives

Because Chickamauga Dam cannot pass the PMF without extensive erosion to the embankment sections, several ideas have been expressed as alternatives for remediation by former TVA engineer, Louis Buck [6]. Among these are the following three: (1) increase the spillway capacity and/or height of the earthen embankments, (2) do nothing and allow the earthen embankments to erode, or (3) apply armoring protection to the earthen embankments. Some of the positive and negative impacts of these options, are discussed in the following sections.

4.3.1 Increase Spillway Capacity Or Embankment Height

In an effort to prevent overtopping by passing or containing the flood waters, the traditional methods of increasing the spillway capacity or raising the height of the dam were considered. Through studies, TVA determined it to be impractical to increase the spillway capacity to pass the full PMF, due to high costs and physical constraints. The water surface elevation was determined to be 712 when routing the PMF without Chickamauga Dam. This elevation is 6 feet above the existing embankments, therefore this alternative has not been further investigated [1].

In order to contain the PMF, the earthen embankments would need to be raised 12.5 feet. In this case the negative impacts far outweigh the positive. The only positive impact will be the minimization of downstream flood levels and consequential damage, while the negative impacts include high construction costs and increased upstream flood levels. The construction costs would include substantial earth moving and the appropriation of property, since the north embankment would have to be extended 1.5 miles. The ramifications of an increase in upstream flood levels will be extensive damage to the upstream towns of Dallas Bay, Dayton, and Soddy Daisy. Each would be faced with emergency costs and benefit and income losses in addition to the social, political, and environmental impacts of the flood level increase. TVA does not consider raising the dam an alternative because raising the upstream flood levels will also impact a nuclear power plant that is located just upstream from Chickamauga Dam and prevention of damage to that structure is a primary concern.

4.3.2 Do Nothing

If nothing is done to remediate Chickamauga Dam, the earthen embankments will overtop in floods exceeding 77 percent of the PMF, and begin to erode. There would be incremental damages due to erosion, and the earthen embankments will have to be rebuilt. All the impacts of this alternative are not negative when comparing this alternative to alternative one. Included in the positive impacts are, no increase in upstream flood levels or consequences, a minimal interruption in benefits (the highway on the south embankment would be inundated and eroded during overtopping), and no project costs. However, doing nothing to modify Chickamauga Dam to safely pass the PMF, would not be consistent with federal guidelines.

4.3.3 Armoring Protection

Because the alternative of raising the height of the earthen embankments will impact the upstream nuclear power plant, and increasing the spillway size has been deemed impractical, armoring protection is being considered as an alternative option which may be cost effective. The earthen embankments would be allowed to overtop as in the "do nothing" alternative; however, they would be protected from erosion with some type of armor, thus eliminating the cost of rebuilding the embankments, and eliminating incremental damage due to erosion. As in the "do nothing" alternative, the positive impacts include: no increase in upstream flood levels or consequences and a minimal interruption in benefits (the highway on the south embankment). The initial construction

cost and possible maintenance costs of armoring, along with no reduction in downstream impacts are the only negative impacts identified.

In order to determine if armoring would be an acceptable alternative, it must be shown to be hydraulically and economically feasible. By considering the limitations of each of the types of armor discussed in Chapter 2, and comparing them with the predicted hydrologic analysis for Chickamauga Dam during the PMF, it can be determined which methods of armoring would be most effective. The economic feasibility is determined by estimating the costs associated with the addition of armor to the earthen embankments, and comparing those to the estimated benefits to be gained by armoring, as opposed to no modifications. Also by comparing the estimated costs of different hydraulically feasible methods the most cost effective armoring alternative can be established. The results of these analyses are presented in Chapter 5.

CHAPTER 5

STUDY RESULTS

5.1 Hydraulic Feasibility

After reviewing the limitations of the various armoring alternatives discussed in chapter two, each option is compared with the predicted hydraulic conditions at Chickamauga Dam. Plain vegetation has been reported by CIRIA (Table 2.2) to have a maximum limiting velocity of about 15 fps while the velocities are predicted during the PMF, at Chickamauga Dam, to reach 39 fps. In the same respect, geotextile/soil/root mats can be eliminated from consideration because of their reported limiting velocities of 8 to 20 fps (Table 2.5). However, the use of geotextiles as an underlying filter for other armoring alternatives which *are* hydraulically feasible for Chickamauga Dam is still a possibility.

Because riprap has not been proven to withstand the predicted design velocities during USBR testing, and most applications in the field have moderate slopes of 4H:1V to 10H:1V (Table 3.2), this option is not encouraged. Another negative factor against riprap would be the high tailwater conditions at Chickamauga which may cause displacement of the rocks due to saturated foundation material. However, riprap or gravel could be used as an underlying filter material for other armoring alternatives.

Gabions under overtopping flow conditions have only been tested up to velocities of 24 fps on a 4H:1V slope (section 2.5). No failure to the embankment was reported, but the tests showed evidence of wire deformation, displacement of rock fill and scour

downstream of the gabions. The duration of testing was a small fraction of the expected overtopping duration at Chickamauga, and with more time, more damage may occur. More tests are necessary to determine a more quantitative failure mode for gabions. Of relevance is the fact that the wire baskets may deteriorate with time. The low probability of the PMF event occurring at Chickamauga suggests that it may be an extremely long time before protection is needed, and there is no assurance that the baskets will not have deteriorated before that time.

Gabions are not being considered for armoring protection at Chickamauga Dam, because of their limited use on steep slopes and the unknown results that may occur with a long duration of overtopping, steep slopes, and submergence. Additional testing with slopes and flows more characteristic of Chickamauga's situation, may prove that gabions are a possible solution. However, because of the extreme conditions at Chickamauga Dam and the high hazard classification, TVA prefers that a more rigid form of protection be applied. Therefore, the focus of this study leads to the alternatives of cements and prefabricated concrete blocks.

Of the two cement applications discussed (roller-compacted concrete (RCC) and soil cement), RCC has been more widely used in applications for armoring protection, with over 30 dams in the country armored with RCC [1] (Appendix A, Table A.2). Also, since RCC typically has a higher cement content than soil cement, RCC will provide greater stability. Many of the existing RCC protected dams have expected overtopping characteristics similar to, or exceeding, those expected at Chickamauga Dam during the PMF. For example, Ashton Dam in Idaho is a 60-foot structure requiring protection from

the PMF and overtopping is predicted to last 10 days with a maximum overtopping depth of 12 feet. In comparison, Chickamauga Dam's predicted overtopping duration and depth are 4 days and 10.2 feet respectively. A few RCC protected dams have been tested in the field, (i.e. Ringtown 5 Diversion Dam and Kerrville Ponding Dam discussed in section 2.6.2), and the damages to the RCC, if any, were limited. Since RCC can be placed on a slope as steep as 1H:1V, no modifications to Chickamauga Dam's existing slopes of 1.5H:1V to 6H:1V would be necessary except perhaps the removal of existing riprap.

With respect to the use of prefabricated concrete blocks, both cellular concrete blocks and wedge-shaped (or stepped concrete overlays) performed well in tests. At least 1 of 3 cellular block systems tested by the USBR (Petriflex) was stable at velocities greater than 17 fps (Table 2.10), but determination of the limiting velocity was not possible due to limitations of the testing facilities. CIRIA reported damage to the soil fill and vegetation of the same brand of block systems at 26 fps. Chickamauga Dam's maximum predicted velocity during the PMF is 39 fps, therefore concrete cellular blocks are not considered hydraulically feasible at this time. The blocks which have been tested are blocks that were available from manufacturers at the time of testing. Since the time of testing, vendors (Armorflex) have begun manufacturing a larger block. Tests on new and improved designs of cellular concrete blocks may prove that it is possible to use cellular concrete block systems for overtopping protection under higher velocities. Unfortunately, no tests have been performed (1995).

Wedge-shaped blocks or stepped concrete overlays have been reported to withstand discharges greater than those expected at Chickamauga Dam during the PMF. However, it

may be necessary to modify the embankment slopes, depending on the saturated shear strength of the underlying soil material. Wedge-shaped blocks have been constructed successfully in Russia on slopes of 2H:1V and 1.4H:1V on higher strength granular materials (section 2.7.2). Because the tailwater will be high during the PMF at Chickamauga Dam, preventive measures must be taken to insure that the blocks do not become unstable due to turbulence or backwater. Of particular concern is the location of the hydraulic jump and the turbulence it induces. The blocks which would be impacted by the hydraulic jump would probably require an anchor system for stability.

An underlying filter of either geotextiles and/or granular material must be provided beneath the blocks to protect the subsurface of the embankment from erosion due to seepage beneath the blocks and aspiration.

5.2 Economic Feasibility

In order to establish the economic feasibility of armoring Chickamauga Dam, an analysis of the estimated costs associated with the application of RCC and prefabricated concrete blocks will be compared with the benefits of armor protection. The estimated costs associated with the application of RCC and prefabricated concrete blocks will also be compared to each other to determine which alternative is the most cost effective.

5.2.1 Armoring Benefits

The benefits associated with the armor protection of Chickamauga Dam will be defined as the aversion of incremental damages that would occur due to erosion. As

previously mentioned in Section 4.2.3, the flood levels downstream of Chickamauga will be increased, if the embankments erode, as compared to no occurrence of erosion to the embankments. The maximum increase in flood levels due to erosion is 0.7 feet immediately downstream, reducing to no increase 30 miles downstream [2]. This small increase in flood levels was reported by TVA, in 1988, to cause \$80 million worth of damages to Chattanooga property (residential, commercial, industrial, and public). Other damages, which include the previously mentioned erosion damage to Chickamauga Dam and the total loss of project benefits suffered while the dam is reconstructed, have been estimated at \$150 million and \$25 million respectively. The benefit losses assume a 2-year reconstruction period for the dam and the loss of 230,500 acre-feet of flood control storage for those 2 years. Included in the total project benefit losses are loss of power (based on replacement costs for energy and capacity), navigation (due to the need for more expensive transportation for the 2-year period that barge traffic would not be available), recreation and fishing. It is anticipated that recreational visits and reservoir fishing would return to current levels in 3 to 5 years.

Because these estimates were made in 1988, the estimates were updated to 1995 costs. A TVA rating curve was used to determine the index for the year 1988 as 1360 and given an index of 1650 for the current year (1995), the ratio for cost increase from 1988 to 1995 was taken as $1650/1360$ or 1.2. Therefore, indexing to current dollars, the cost savings that armoring would provide are as follows:

Averted damage to Chattanooga Property -----\$96 million

Averted damage to Chickamauga Dam -----\$180 million

Averted Benefit Losses -----\$30 million

This results in a total of \$306 million worth of damages averted by the armor protection of Chickamauga Dam's earthen embankments.

The \$306 million does not include clean-up costs, environmental impacts, or social impacts. Social impacts would include emergency evacuation, reoccupation, temporary quarters, relief and care of flood victims, and the sum of individual losses. The representative figures for these costs are not available at this time, and would require an in-depth analysis, which is beyond the scope of this report. With this in mind, using only the estimated \$306 million as damage reduction benefits attributable to armoring, results in a very conservative figure for cost-benefit comparison.

Because the probability of the PMF is reasonably assumed by TVA as 1/1,000,000, the risk must be factored into the cost-benefit comparison through the use of a damage-frequency relationship. Using the limited available data, a simplistic relationship between the damages that will be averted by armoring Chickamauga Dam's earthen embankments and the frequency or probability of event is shown in Figure 5.1.

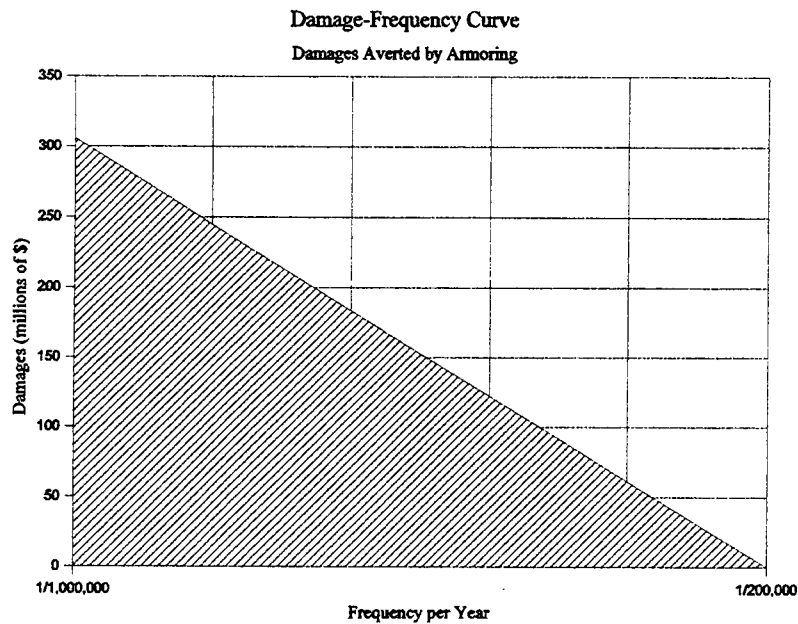


Figure 5.1. Averted Damage-Frequency Diagram

As defined in the National Economic Development Benefits Procedures Manual for Urban Flood Damage Reduction [3] the expected annual damages can be calculated by computing the area under the damage-frequency curve. In this study of Chickamauga Dam, the damages of interest are those averted by the application of armor to prevent erosion during overtopping. It is assumed that no damages will be averted by the addition of armor, if the embankments are not overtopped. Because Chickamauga Dam can safely pass 77 percent of the PMF without overtopping and the probability of the .77 PMF being exceeded is $1/200,000$, this probability was assigned the value of zero averted damages.

The 306 million dollars of averted damages are associated with the PMF which has a probability of 1/1,000,000. Typical damage-frequency curves are convex in shape and the area under the curve is calculated by integrating the function that represents the curve. Because the only existing damage data for Chickamauga Dam attributed to the erosion of the earthen embankments is that reported for the PMF (306 million dollars) and .77 PMF (zero dollars), the damage-frequency curve is represented by a straight line. Therefore, the calculated area under the curve of \$612/year is a rough approximation of the average annual averted damages due to armoring.

5.2.2 General Armoring Costs

Many site specific details must be taken into consideration in order to estimate the cost of armoring Chickamauga Dam. However, some details may be excluded from the evaluation because they will suffer the same impacts whether the earthen embankments are armored or not. Although these factors do not directly enter into the cost analysis, allowances must be made for the switchyard, existing buildings, and railroad tracks that are specific to Chickamauga Dam.

The switchyard, located at the downstream toe of the south embankment, will be impacted whether or not the embankments at Chickamauga are armored. Relocation would be difficult, because there is no available land and the switchyard is partially owned by Chattanooga Electric Power Board. Also, a new structure would have to be built before the existing structure could be removed. In an event as remote as the PMF, TVA considers the loss of the switchyard an acceptable risk. Because the switchyard will be

destroyed whether erosion to the embankments occurs or not, the cost of replacement is not a factor in this evaluation. However, consideration during design is necessary because the switchyard is near the toe of the embankment and much scour and erosion may occur in that area.

The railroad tracks located on the south embankment belong to TVA and are no longer in service. At one time, equipment for the power station was transported by rail, but today, all supplies are delivered by truck. Therefore, if the embankments were armored, the railroad tracks would most likely be removed. The railroad removal cost is not considered in this evaluation, because, the railroad tracks would have to be removed before reconstruction of the embankments began.

The existing buildings at the dam site include the powerhouse, the observation building (located just north of the lock on the north embankment), and the lock operations building (located on top of the south wall of the lock). The powerhouse is constructed of reinforced concrete and is expected to withstand the flood waters; the buildings near the lock however, will most likely be destroyed, but the foundations are expected to remain. These super structures are considered expendable by TVA for an event as remote as the PMF. The cost to replace these buildings are mutual for the erosion and non-erosion case and are therefore not a factor in this evaluation. However, provisions should be made during the design of armor to attach the slope protection to the slab on which the observation building is constructed.

Another consideration is the existence of approximately five acres of wetlands immediately downstream from the south embankment. The area will be flooded

regardless if the embankments are armored or not, but special care must be taken if armor protection is placed in that general area. Special permitting may be required, or other additional fees incurred, because of the existing wetlands.

The remainder of this section is broken up into subsections which identify different cost estimates for the two hydraulically feasible armoring alternatives of RCC and wedge-shaped blocks at Chickamauga Dam. Before summarizing the unique costs associated with each alternative, a brief discussion of common costs is presented.

5.2.3 Common Costs

Some of the costs associated with armoring will be roughly the same for either alternative, such as flume testing, initial slope preparation, and underlying filter material. This section will address the estimation of mutual costs and the estimated costs associated with each specific alternative will be addressed in following sections.

Any armoring design should be tested under simulated conditions representing those expected during the PMF at Chickamauga to assure stability. Therefore, RCC and/or wedge-shaped blocks should be flume tested or tested on a prototype embankment before installation. According to TVA estimates, the cost for testing would be about \$200,000. For the purpose of a conservative estimate, and allowing for more than one geometric configuration to be tested a cost of \$400,000 will be allotted for the testing ¹.

For any chosen method of armor protection the slopes would have to be prepared,

¹ Estimates were made by TVA engineers with previous exposure to cost information in similar applications.

first of all by the removal of the existing riprap. The slopes of the north embankment are protected both on the upstream and downstream sides by riprap placed on a gravel blanket. The gravel blanket is between 8 and 12 inches thick and the riprap is 1-3 feet thick on the upstream slope and 1-1.5 feet thick on downstream slope. An estimated cost of \$6/cu.yd² for the removal of 108,100 cu.yd of riprap and gravel blanket [4] yields \$648,600. Not included in this cost, is the acquisition of a source for a spoil area and associated costs. With approximately \$0.65 million in transportation costs alone, the complete cost of removal of riprap will be estimated at \$1 million with the assumption that the spoil can be reused or sold.

Filter material will need to be placed beneath the armoring regardless of the alternative chosen. If a gravel mattress can be used some cost may be saved in the removal of riprap and gravel. If a geotextile mat is used the cost will be on the order of \$4/sq.ft.³. The underlying geotextile should be anchored on the upstream slope of the dam, just below the reservoir's normal pool level, and extend up to and over the crest, then down the downstream side and should be anchored at the toe. Estimating the surface area noted for coverage to be 800,000 sq.ft. results in an approximate cost of \$3.2 million for the installation of a geotextile underlayer.

The initial costs of embankment preparation and flume testing, for armoring Chickamauga Dam with RCC or wedge-shape blocks, are estimated near \$3.7 million.

^{2,3} Estimates were made by TVA engineers with previous exposure to cost information in similar applications.

The unique costs associated with each particular alternative will now be estimated to establish the most economical option. The initial costs will then be added to the cost of each alternative. These figures will then be compared with the estimated benefits to determine whether or not the expense of armoring Chickamauga Dam is warranted.

5.2.4 Roller-Compacted Concrete Costs

In the case of RCC, no modifications need to be made to the slope of the embankments, because application has been successful on slopes as steep as 1:1 with no form work necessary. Assuming a conservative design step that is 2 feet high by 12 feet wide spanning the entire length of the embankments, and a maximum height of the dam taken as 50 ft, 25 steps would be necessary on the downstream slope. The combined length of the north and south embankments is 4,390 ft, resulting in 100,000 cu.yd of concrete material needed. The estimated cost for RCC construction ranges from \$30/cu.yd to \$90/cu.yd depending on the contractor, the aggregate used and the quantity of RCC to be placed. Unit cost for equipment may be reduced when spread out over a greater quantity of material. Using the high end of the cost range or \$90/cu.yd yields roughly \$9 million for RCC construction costs and \$3 million for the \$30/cu.yd construction cost.

Other Considerations

Because roadways must pass along the crest of both embankments at Chickamauga, the crest may need to be protected using reinforced concrete, because of the

live vehicular loads expected. Consideration of protection for the upstream slope and toe area must also be made. Also of concern are the manner in which the RCC overtopping protection will be tied into the abutments of the embankments and the design of any necessary drainage systems. The design of such detail is beyond the scope of this thesis. However, TVA estimates the cost of such transitional pieces to be around \$5 million.

5.2.5 Wedge-Shaped Blocks Costs

For a rough cost estimate a block size similar to the block used for testing as mentioned in the paper by the USBR's Kathleen H. Frizell entitled "Hydraulic Design of Stepped Concrete Overtopping Protection For Embankment Dams" [5] will be assumed. Blocks which are 2 feet wide with step heights of 0.2 feet applied to a structure with a height of 50 ft and a length of 4,390 ft will require less than 600,000 blocks. Estimating a manufacturing cost of \$5-\$20/block, results in a material cost between \$2.7 million and \$11 million. The number of blocks is greatly exaggerated because the size of block designed for the flows at Chickamauga Dam will most likely be larger; this may in turn increase the unit cost of the block therefore, making up for the discrepancy in size. Also, the slope of the face of the block was not considered when determining the number of rows needed, it was assumed that the step will be horizontal, which will give an overestimate of the number of blocks needed.

Other Considerations

In order to apply wedge-shaped blocks to the downstream slope, portions of the

slope may need to be modified, as addressed in section 5.1, which will increase the cost.

Because wedge-shaped blocks have been determined to be unstable under the hydraulic jump, and Chickamauga will be submerged, consideration must be given to the possible necessity of anchoring the wedge-shaped blocks to the embankments. This type of anchoring will increase the cost of the materials as well as the installation of blocks. TVA estimates that the cost of anchoring a block adds an additional cost of \$50 to \$70 per block. Assuming all of the blocks must be anchored, this additional cost ranges from \$30 million to \$42 million. However, all of the blocks may not have to be anchored. A more in-depth analysis is needed to determine this factor. Assuming the requirement of anchorage to all of the blocks results in a possibly conservative estimate of costs.

Particular attention will have to be given to the design of crest and toe blocks associated with the wedge-shaped block alternative. The crest of both embankments have roadways which will have to be tied in to any crest block designed for use with the wedge-shaped blocks. Thus, it may be necessary to apply reinforced concrete to the crest area. The toe block must be designed large enough to encompass any surface that the hydraulic jump may come into contact with.

Also of concern are the manner in which the wedge-shaped block overtopping protection will be tied into the abutments of the embankments and the design of any necessary drainage systems. The design of such detail is beyond the scope of this thesis. For lack of other information, the TVA estimate of \$5 million for transitional pieces is also assumed for the application of wedge-shaped blocks.

Table 5.1 shows a summary of the estimated costs for armoring Chickamauga Dam's Earthen Embankments with either RCC or wedge-shaped blocks.

Table 5.1. Summary of Estimated Costs for Armoring Chickamauga Dam's Earthen Embankments

FACTORS	RCC (millions of dollars)	Wedge-Shaped Blocks (millions of dollars)
General Construction costs	3-9	2.7-11
Underlying geotextile filter	3.2	3.2
Riprap Removal	1	1
Flume Tests	0.4	0.4
Anchorage	N/A	30-42
*Other Details	5	5
TOTAL	13 - 19	42 - 63

* Other details include drains, toe block, crest block, upstream protection to normal pool, and abutment detailing.

Both RCC and wedge-shaped blocks, if properly designed, have the capability of providing the necessary overtopping protection for Chickamauga Dam's earthen embankments. Further tests must be conducted to insure that any proposed designs are appropriate. Economically, RCC is a more favorable alternative than wedge-shaped blocks, at one third of the cost. The uncertainty of cost associated with wedge-shaped blocks is mainly due to lack of information, as wedge-shaped blocks have not yet been implemented as overtopping protection in this country (April 1995). More studies may show that anchorage of all of the blocks is not necessary which may significantly reduce

the costs. RCC has been used throughout the country and much information is available to assist in the design of functional overtopping protection which also suggests that RCC is a more positive alternative than wedge-shaped blocks.

In order to compare the cost of armoring with the average annual benefits of averted damages due to armoring the cost must be multiplied by a capital recovery factor or discount rate. Assuming the life of the armoring project to be 100 years, and given TVA's present discount rate of 8.75%, the average annual cost range for armoring will be \$1.14- \$5.51 million/year.

The average annual benefits gained by reducing the damages from overtopping during the PMF from \$306 million to zero by armoring the earthen embankments, was determined to be \$612/year. In comparison with average annual costs for armoring ranging between \$1-\$5 million/year, neither armoring alternative appears to be cost effective. However, because TVA is a federal agency it is obligated to meet the guidelines set forth by the federal government, concerning dam safety. Because Chickamauga Dam can safely pass only 77 percent of the PMF and the federal guidelines require that 100 percent of the PMF must be able to safely pass, some type of remediation must be done. In comparison with the other suggested fixes of raising the height of the dam or increasing the spillway capacity, armoring proves to be the most attractive alternative at this time, with RCC being the most cost effective of the hydraulically feasible armoring materials discussed.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Literature Review

Through an extensive literature review, it was determined that much interest lies in the application of armor protection. Various United States agencies and at least two other countries have participated in the testing and analysis of different alternatives for armor protection. A good data base for future overtopping protection designs has being generated and is continuing to grow, with tests conducted using scaled laboratory models and several prototype structures. Several dams have been modified using one or more of the alternatives discussed in Chapter 2: vegetation, geotextiles, riprap, gabions, cements, and prefabricated concrete blocks. All of these armoring alternatives are noted as acceptable with varying degrees of limitations, as shown in the summary table located in Appendix A, Table A.1

If overtopping protection is being considered as a rehabilitation method, the method to apply is largely dependent on the expected overtopping velocities and duration of overtopping. The designer must consider the site conditions of the dam, as well as the limitations and costs of application when choosing an armoring alternative. As recommended by CIRIA [1], planning should include but, should not be limited to, the following considerations:

- Overall concept of the waterway, and frequency and duration of flow
- Risk (acceptability of failure)
- Design discharge and hydraulic loading

Properties of subsoil
'Dry' use in normal no-flow conditions (agricultural use, risk of vandalism)
Maintenance ability and requirements of the owner
Appearance
Capital and maintenance costs
Access to site and method of construction
Climate
Strategy for design, specification, construction, and future maintenance.

Additional information has been gained through construction methods and design details of existing dams with armor protection, and investigations of existing dams with armor protection that have been overtopped. In many cases, armor protection has proven to be a cost efficient means for rehabilitating earthen dams with inadequate spillway capacity.

6.2 ASDSO Survey

From the ASDSO survey it was determined that many states have readily accepted armor protection as a remediation alternative for dams with inadequate spillway capacity and some, although with reservations, will consider the allowance in marginal cases. The responses to this survey showed that over two-thirds of the states in the nation will consider armoring protection. Many dams under jurisdiction of varying states have been armor protected using one or more of the alternatives mentioned in Chapter 2. Table 3.1 summarizes the responses received from each state, while Table 3.2 lists existing dams under state jurisdiction that have employed some type of armoring protection (as compiled from the survey responses). With the increased acceptance of armoring protection and its widespread use, engineers will have cost effective alternative for rehabilitation measures

to existing dam structures, and the benefits of lessons learned by engineers involved in the construction of existing armor protection. The growing use of armor protection and the responses from this survey indicate that with time and technology, the idea that all embankment overtopping should be assumed as "dam failure", is rapidly changing.

6.3 Chickamauga Dam

Because Chickamauga is considered a high hazard dam it should be able to safely pass the full PMF according to federal and state guidelines, but at this time it is only capable of passing 77 percent of the PMF. From hydrologic, hydraulic, and erosion analysis performed on Chickamauga Dam, by TVA, it was determined that during a flood exceeding 0.77 PMF the earthen embankments will overtop by a maximum of 10.2 ft and subsequently erode. Although the erosion is not expected to completely release the water stored in the reservoir, a significant increase in damages will occur downstream as a result of this erosion. Protecting the earthen embankments from erosion due to overtopping flow will prevent the incremental damages that would occur downstream if the embankments overtop and erode.

6.3.1 Hydraulic Feasibility

Because the overtopping velocities at Chickamauga Dam during the PMF are expected to reach between 36 and 39 fps, the armor alternatives, discussed in Chapter 2, which had limiting permissible velocities were deemed hydraulically infeasible (i.e. plain vegetation and filled geotextiles). Furthermore, gabions and riprap were eliminated

because not enough quantitative data is available for their performance under overtopping flow at high velocities, and due to Chickamauga's classification as a high hazard dam, more rigid alternatives are desirable. RCC, and wedge-shaped blocks were the most durable types of protection discussed and research has proven them both to be feasible alternatives for overtopping protection at Chickamauga Dam. Costs were estimated to determine the economic practicality of both of these alternatives for overtopping protection.

6.3.2 Economic Feasibility

Although the cost estimates made in Chapter 5 are approximate, they indicate that the total costs to armor Chickamauga Dam, could range from \$13- \$65 million. Based on a very simplistic analysis of average annual benefits gained by averting the damages from overtopping during the PMF by using armoring protection, neither armoring alternative appears to be cost effective. However, something must be done to remediate Chickamauga Dam in order to comply with federal and state guidelines and armoring is the most appealing of the alternatives discussed. Estimates show that RCC can be applied for armor protection at one-third of the cost of the application of wedge-shaped blocks. This is partly due to the fact that no application of the wedge-shaped blocks for overtopping protection has been reported in the United States, so design criteria and costs are speculative.

Further investigations involving the wedge-shaped blocks should include designing a wedge-shaped block specifically for the flows expected at Chickamauga, then testing the

block to assure its performance. Also a determination of the location of the hydraulic jump would be necessary to properly design an anchorage system. After a definite block is defined and required anchorage is determined, accurate cost information will be more readily available. Because more information is available on RCC for armoring protection than wedge-shaped blocks, and because the costs are significantly lower, RCC is the most attractive armoring alternative at this time.

6.4 Conclusions and Recommendations

Much effort has gone into the research of the stability of various types of armoring protection placed on earthen embankments to guard against erosion during overtopping flow. Many studies are still going on and others are planned for the future. There is an adequate amount of information available on several of the armoring alternatives discussed in this thesis for engineers to begin applying armor protection to small earthen dams with low velocity flows. However, there is still a need for future testing of armoring protection.

In the literature, the testing results of different agency are not presented in consistent units which makes comparison of different armoring alternatives difficult. Future studies should incorporate consistent measurements and standard testing procedures to provide a consistent basis for comparison of the performance of different materials.

Future studies should also be targeted towards the establishment of design standards for each of the possible armoring alternatives for erosion protection against

overtopping flow. Such standards should address the circumstances under which armoring protection should or should not be allowed and any limiting conditions of each alternative. The limiting conditions associated with a particular armoring alternative may be hydraulic and/or economic. There is also a need for more testing which focuses on design details such as proper anchorage, and drainage systems. The effects of high tailwater on the armoring protection should be modeled in scaled or prototype tests, in order to provide a better understanding of how to deal with the application of armoring protection under this condition.

The organization of a central data base where information from the analysis of existing armor protected dams which have experienced overtopping flows can be compiled would be of great service to the engineer considering armor protection as a remediation alternative. Because experience is the best teacher, the failures probably more so than the successes will provide a wealth of information concerning the strengths and weaknesses of designs. By learning from others mistakes, an engineer can avoid making the same errors.

Because armoring protection of earthen embankments has been proven as a cost effective and hydraulically feasible alternative to traditional more expensive measures of remediation for dams with insufficient spillway capacity, many more dams in the future may be considered for armoring protection. However, until a more structured format for design standards are developed it is up to the design engineer to use good judgement and care in choosing a site specific armoring protection scheme based on the available data and experience of others.

CITED REFERENCES

CITED REFERENCES

Chapter 1

1. Association of Dam Safety Officials, "1994 State Dam Inventory Data", March 1995.
2. Federal Guidelines for Dam Safety, Federal Emergency Management Agency, June, 1979.
3. Tschantz, B.A., "1989 Report on Review of State Non-federal Dam Safety Programs", August 1989. Association of Dam Safety Officials and Federal Emergency Management Agency.
4. Glossary of Terms for Dam Safety, Federal Emergency Management Agency, FEMA 145, February, 1988.
5. McLean, F.G. And Hansen, K.D., "RCC: Rehab Results", Civil Engineering, August 1993.
6. Wooten, R.L., Powledge, G.R., Whiteside, S.L., "Dams Going Safely Over the Top", Civil Engineering January 1992.
7. Denning, J., "Small-Dam Rehabs", Civil Engineering, January 1993.
8. McLean, F.G. And Hansen, K.D., "Roller Compacted Concrete For Embankment Overtopping Protection", Geotechnical Practice In Dam Rehabilitation, Pp. 188-209, 1993.
9. Kindsvater, C.E. , "Discharge Characteristics of Embankment-Shaped Weirs" Studies of Flow of Water Over Weirs and Dams, USGS Water Supply Paper 1617-A, 1964.
10. Clopper, P.E., Chen Y.H., "Minimizing Embankment Damage During Overtopping Flow", Federal Highway Administration Report RD 88-181, Nov. 1988.

Chapter 2

1. Soil Conservation Service. 1954. Handbook of channel design for soil and water conservation. SCS-TP-61, rev., 34 pp.
2. Soil Conservation Service. 1977. Design of open channels. SCS-TR-25, 225 pp.
3. Temple, D.M., K.M. Robinson, R.M. Ahring, and A.G. Davis. 1987. Stability Design of Grass-Lined Open Channels. U.S. Department of Agriculture, Agriculture Handbook 667, 175 p., illus.
4. Riley, R.C., "A Procedure For Evaluating Permissible Limits of Overtopping of Vegetated Earthen Embankments", ASDSO Annual Meeting, Austin Texas, 1986.
5. Soil Conservation Service, DAMS2, Draft of Technical Release 48, Structure Site Analysis Computer Program Technical Reference and Users Manual, Soil Conservation Service U. S. Department of Agriculture, Nov. 1994.
6. Temple, D.M., J.A. Brevard, J.S. Moore, G.J. Hanson, E.H. Grissinger, and J. M. Bradford. 1993. "Analysis of Vegetated Earth Spillways", ASDSO 10th annual conference, Kansas City, MO, Sept. 1993.
7. Clopper, P.E., Chen Y.H., "Development of Methodology for Estimating Embankment Damage During Overtopping Flow", Federal Highway Administration Report RD 86-126, March 1987.
8. Hewlett, H.W.M., Boorman, L.A., Bramley, M.E., "Design of Reinforced Grass Waterways", Construction Industry Research and Information Association (CIRIA), Report 116, 1987.
9. Clopper, P.E., Chen Y.H., "Minimizing Embankment Damage During Overtopping Flow", Federal Highway Administration Report RD 88-181, Nov. 1988.
10. Giroud, J. P., "Functions and Application of Geosynthetics in Dams", Water Power & Dam Construction, June 1990.
11. McWhorter, J.C., T.G. Carpenter, and R.N. Clark, Erosion Control Criteria for Drainage Channels. Prepared for the Mississippi State Highway Department in cooperation with U.S. Department of Transportation, Federal Highway Administration, by Agricultural Experiment Station, Mississippi State University, State College, Mississippi, March 1968.

12. Cox, A.L., R.C. Adams, and T.B. Lawson, "Erosion-Control Study, Final Report Part II, Roadside Channels." Louisiana Department of Highways Research and Development Section in cooperation with U.S. Department of Transportation, Federal Highway Administration, 1971.
13. U. S. Geological Survey, "Performance of Temporary Ditch Linings." Prepared for the Federal Highway Administration at the U. S. Geological Survey Gulf Coast Hydrosience Center, Bay St. Louis, Mississippi, 1985.
14. Chen, Y. H., and G. K. Cotton, "Design of Roadside Channels with Flexible Linings." Hydraulic Engineering Circular No. 15, Report No. FHWA-IP-86-5, Washington, D. C., Federal Highway Administration, 1986.
15. Bureau of Reclamations, *Design of Small Dams*, Third Edition, Denver, Colorado, 1987.
16. Normann, J.M. (1975). Design of stable channels with flexible linings, Highway engineering circular No. 15. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.
17. Soil Conservation Service (1969). "Engineering Field Manual for Conservation Practices." Soil Conservation Service, U.S. Department of Agriculture, Washington, D.C.
18. Simons, D.B., and Senturk, F. (1977). "Sediment Transport Technology." Water Resources Publications, Ft. Collins, CO.
19. Simons, D.B., and Senturk, F. (1992). "Sediment Transport Technology." (2nd ed.). Water Resources Publications, Ft. Collins, CO.
20. Powledge, George R.; Sveum, D. L., "Overtopping Of Small Dams - An Alternative For Dam Safety, Hydraulics and Hydrology In The Small Computer Age", American Society Of Civil Engineers Specialty Conference, Lake Beuna Vista, FL., Vol.2, Pp.1071-1076, 1985 Aug.
21. Frizell, Kathleen H., Phone Conversation, October 1994.
22. Department of the Interior, Bureau of Reclamation, "Mix Design Investigation of Asphaltic Concrete for Dam Facing - Glen Elder Dam Missouri River Basin Project." Kansas, Report No. REC-ERC-7616, 1976.
23. Akky, M. R., "The Erodibility of Cement Stabilized Soil." Ph.D. Dissertation, University of California at Davis, Davis California, 1974.

24. McLean, F.G. And Hansen, K.D., "Roller Compacted Concrete For Embankment Overtopping Protection", *Geotechnical Practice In Dam Rehabilitation*, Pp. 188-209, 1993.
25. McLean, F.G. And Hansen, K.D., "RCC: Rehab Results", *Civil Engineering*, August 1993.
26. Jackson, H.E., "Overtopping Protection Using Roller-Compacted Concrete", *ASCE Hydraulic Engineering*, 1992
27. Prendergast, J., "RCC at 10", *Civil Engineering*, October 1992.
28. Wooten, R.L., Powledge, G.R., Whiteside, S.L., "Dams Going Safely Over the Top", *Civil Engineering* January 1992.
29. Baker, R., "Pre-cast Concrete Blocks for Dam Spillways", *Water Power & Dam Construction*, July 1989.
30. Wooten, R.L., Whiteside, S.L., Welsh, R. And Wirkus, K.E., "Construction of CCM Overtopping Protection On Three Parkway Dams", *Geotechnical Practice in Dam Rehabilitation*, Pp. 210-224, 1993.
31. Pravdivets, Y.P., Slissky, S.M., "Passing Floodwaters Over Embankment Dams", *Water Power and Dam Construction*, Pp. 30-32, July 1981.
32. Pravdivets, Y.P., "Flood Passage Over the Cofferdam of the Dneister Power Developments", (in Russian), *Energeticheskoe Stroitelstvo* No.11, 1978.
33. Grinchuk, A.S., Pravdivets, Y.P., Schekhtman, "Tests of a Protective Facing Allowing the Overflow of Floods of Large Unit Discharges", (in Russian). *Energeticheskoe Stroitelstvo* No.4; 1977.
34. CIRIA, "Evaluation of Performance of Wedge-Shaped Block Protection Systems in High Velocity Flow on Spillways and Embankment Overtopping", Proposal for Discussion, Dec. 1987.
35. Grinchuk, A.S. and Pravdivets, Y.P., "Precast Reinforced-Concrete Revetment of Earth Slopes Used for Discharging Water", (in Russian), *Hidrotekhnicheskoe Stroite'stvo*, No. 7, July 1977, 25-28.

36. Grinchuk, A.S., Pravdivets, Y.P., Schekhtman, N.V. "Tests of Earth Slope Revetments Permitting Flow of Water at Large Specific Discharges", (in Russian), *Hidrotekhnicheskoe Stroite'stvo*, No.4, April 1977, 22-26.
37. Rofe, Kennard & Lapworth, "Design of Stepped Block Spillways", CIRIA Research Project 407, Feb. 1994.
38. Vermeyen T., D. Mares, "Alternatives for Enhancing Spillway Capacity Currently Being Pursued by the U. S. Bureau of Reclamations", United States Committee on Large Dams, Twelfth Annual USCOLD Lecture Series, Ft. Worth Texas, April 1992.
39. Frizell, K.H.; Ruff, James F., "Large-scale Embankment Overtopping Protection Tests", ASCE National Conference on Hydraulic Engineering, San Francisco, Pp. 1951-1956, 1993.
40. Powledge, G.R., Ralston, D.C., Miller, P., Chen, Y.H., Temple, D.M., "Mechanics of Overflow Erosion on Embankments. I: Research Activities", ASCE Journal of Hydraulic Engineering, Pp. 1040-1055, Vol. 115, No. 8, Aug, 1989.
41. Koutsourais, M. "A Study of Articulated Concrete Blocks Designed to Protect Embankment Dams", Association of Dam Safety Conference Proceedings, Boston Massachusetts, 1994.

Chapter 3

1. Committee on the Safety of Existing Dams, Safety of Existing Dams Evaluation and Improvement, National Academy Press, Washington , D.C., 1983 (pp 245-6).
2. Tennessee Department of Health and Environment Division of Water Supply, *Rules and Regulations Applied to the Safe Dam Act of 1973.*, Revised Oct. 1987.

Chapter 4

1. Tennessee Valley Authority, Chen / Hydro Review, file 108-96.03B
2. Tennessee Valley Authority, Chickamauga pamphlet.
3. Tennessee Valley Authority, The Chickamauga Project Technical report No.6, TVA.

4. Tennessee Valley Authority, Chickamauga Safety Analysis EAP and Economic Analysis, file 108-96.01.
5. Clopper, P.E., Chen Y.H., "Development of Methodology for Estimating Embankment Damage During Overtopping Flow", Federal Highway Administration Report RD 86-126, March 1987.
6. Tennessee Valley Authority, Chickamauga Erosion Studies, file 108-96.03A

Chapter 5

1. McLean, F.G. And Hansen, K.D., "Roller Compacted Concrete For Embankment Overtopping Protection", Geotechnical Practice In Dam Rehabilitation, Pp. 188-209, 1993.
2. Tennessee Valley Authority, Chickamauga Erosion Studies, file 108-96.03A
3. Urban Flood Damage Reduction National Economic Development Benefits Calculation Procedures Manual.
4. Tennessee Valley Authority, The Chickamauga Project Technical report No.6
5. Frizell, K.H., "Hydraulic Design Of Stepped Concrete Overtopping Protection For Embankment Dams", Association Of State Dam Safety Officials - Eighth Annual Conference, 1991.

Chapter 6

1. Hewlett, H.W.M., Boorman, L.A., Bramley, M.E., "Design of Reinforced Grass Waterways", Construction Industry Research and Information Association (CIRIA), Report 116, 1987.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. Akky, M. R., "The Erodibility of Cement Stabilized Soil." Ph.D. Dissertation, University of California at Davis, Davis California, 1974.
2. Association of Dam Safety Officials, "Supplement to the 1989 Report on Review of State Non-federal Dam Safety Programs", March 1991.
3. Association of Dam Safety Officials, "1994 State Dam Inventory Data", March 1995.
4. Baker, R., "Pre-cast Concrete Blocks for Dam Spillways", Water Power & Dam Construction, July 1989.
5. Birchall, Jonathan And Pinyan, Charles, "Grass Roots Approach to Spillways", Engineering News Record, Pp. 40-41, Aug 28, 1986.
6. Buck, L.E., "Evaluation of Embankment Erosion at TVA's Chickamauga Project", ASCE Hydraulic Engineering, Pp.115-117; 1991.
7. Buck, L.E., "Structural and Non-Structural Alternatives for Accommodating Larger Floods at Dams" ASCE Hydraulic Engineering, Pp.1228-1233; 1992.
8. Bureau of Reclamations, *Design of Small Dams*, Third Edition, Denver, Colorado, 1987.
9. Caciulescu, S; Stematiu, D., "Repairs And Heightening A Dam Built At the Beginning of the 20th Century", International Commission On Large Dams - Seventeenth Congress - Vienna, Vol.2, Q.65, R.032, 1991.
10. Chugh, Ashok K., "Considerations For Throughflow And Overflow Rockfill Dams", Association Of State Dam Safety Officials - Sixth Annual Conference, 1989.
11. CIRIA, "Evaluation of Performance of Wedge-Shaped Block Protection Systems in High Velocity Flow on Spillways and Embankment Overtopping", Proposal for Discussion, Dec. 1987.
12. Clopper, P.E., Chen Y.H., "Development of Methodology for Estimating Embankment Damage During Overtopping Flow", Federal Highway Administration Report RD 86-126, March 1987.

13. Clopper, P.E., Chen Y.H., "Hydraulic Stability of Articulated Concrete Blocks", Federal Highway Administration Report RD 89-199, .
14. Clopper, P.E., Chen Y.H., "Minimizing Embankment Damage During Overtopping Flow", Federal Highway Administration Report RD 88-181, Nov. 1988.
15. Clopper, Paul E., Chen, Uang Hai, "Overtopping Damage Minimization", American Society Of Civil Engineers - Proceedings of the 1987 National Conference on Hydraulic Engineering - Williamsburg Va.; Pp.751, Aug. 3-7, 1987.
16. Clopper, P.E., "Protecting Embankment Dams with Concrete Block Systems", Hydro Review, April, 1991.
17. Committee on the Safety of Existing Dams, Safety of Existing Dams Evaluation and Improvement, National Academy Press, Washington , D.C., 1983 (pp 245-6).
18. Cox, A.L., R.C. Adams, and T.B. Lawson, "Erosion-Control Study, Final Report Part II, Roadside Channels." Louisiana Department of Highways Research and Development Section in cooperation with U.S. Department of Transportation, Federal Highway Administration, 1971.
19. Denning, J., "Small-Dam Rehabs", Civil Engineering, January 1993.
20. Department of the Interior, Bureau of Reclamation, "Mix Design Investigation of Asphaltic Concrete for Dam Facing - Glen Elder Dam Missouri River Basin Project." Kansas, Report No. REC-ERC-7616, 1976.
21. Dewey, R.L., Gillette, D.R., "Prediction of Embankment Dam Breaching For Hazard Assessment", Geotechnical Practice In Dam Rehabilitation, Pp. 131-144, 1993.
22. Edwards, Wayne D., "Stability Assessment Of Concrete Dams for Flood Loading", Association of State Dam Safety Officials.
23. Engineering News Record, "Precast Concrete Prevents Erosion", Engineering News Record, Pp. 19-20, Nov 16,1989.
24. Federal Guidelines for Dam Safety, Federal Emergency Management Agency, June, 1979.

25. Fieldler, W.R., Tierney, T., Wilson, J., "Overtopping and Piping Concerns At Black Rock Dam", United States Committee On Large Dams, Fourteenth Annual Lecture Series, 1994.
26. Forrest, M.P.; Wong, N.C.; MacDonald, T.C.; Hughes, D.K., "Abutment Protecting Against Major Overtopping At Railroad Canyon Dam", United States Committee On Large Dams, Twelfth Annual Lecture Series, 1992.
27. Forrest, M.P.; Wong, N.C.; and Summers, P.C., "Logan 'First' Dam Mitigating Against Fault Rupture and Overtopping", Association Of State Dam Safety Officials - Ninth Annual Conference, 1992.
28. Forrest, M.P.; Wong, N.C.; MacDonald, T.C.; Hughes, D.K., "Abutment Protection Against Major Overtopping at Railroad Canyon Dam", United States Committee on Large Dams - Twelfth Annual Lecture Series, Fort Worth, TX, April 1992.
29. Frawley, D.D. And Von Thun, J.L., "Economical Solution For Flood Overtopping Potential At Senator Wash Dam", Geotechnical Practice In Dam Rehabilitation, Pp. 634-642, 1993.
30. Frizell, K.H., B.W. Mefford, R.A. Dodge, T.B. Vermeyen, "Embankment Dams: Methods of Protection During Overtopping", Hydro Review, April 1991.
31. Frizell, Kathleen H., Phone Conversation, October 1994.
32. Frizell, K.H., "Hydraulic Design Of Stepped Concrete Overtopping Protection For Embankment Dams", Association Of State Dam Safety Officials - Eighth Annual Conference, 1991.
33. Frizell, K.H.; Ruff, James F., "Large-scale Embankment Overtopping Protection Tests", ASCE National Conference on Hydraulic Engineering, San Francisco, Pp. 1951-1956, 1993.
34. Frizell, K.H., "Stepped Spillway Design for Flow Over Embankments", Hydraulic Engineering, 1991.
35. Giroud, J.P., "Functions and Application of Geosynthetics in Dams", Water Power & Dam Construction, June 1990.
36. Giroud, J.P., Courc, J.P., Bally, P., et Delmas, P., "Comportement d'un testile non-tisse dans un barrage en terre", Proceedings of the International Conference on the use of Fabrics in Geotechnics, Volume II, Paris, April 1977. pp. 213-218. (in French).

37. Giroud, J.P., "Are Geosynthetics Durable Enough to Be Used in Dams?", Water Power and Dam Construction, Vol.41, No. 2, Feb 1989, pp.12-13.
38. Giroud, J.P., "Functions and Applications of Geosynthetics in Dams", Water Power and Dam Construction, Vol. 42, No.6, June 1990, pp.16-23.
39. Glossary of Terms for Dam Safety, Federal Emergency Management Agency, FEMA 145, February, 1988.
40. Gray, E.W., "Cellular Concrete Blocks for Overtopping Protection of Earth Dams", International Commission on Large Dams - Seventeenth Congress- Vienna, Vol.4, Q.67, R.025, 1991.
41. Grinchuk, A.S., Pravdivets, Y.P., Schekhtman, "Tests of a Protective Facing Allowing the Overflow of Floods of Large Unit Discharges", (in Russian). Energeticheskoe Stroitelstvo No.4; 1977.
42. Grinchuk, A.S. and Pravdivets, Y.P., "Precast Reinforced-Concrete Revetment of Earth Slopes Used for Discharging Water", (in Russian), Hidrotekhnicheskoe Stroite'stvo, No. 7, July 1977, 25-28.
43. Grinchuk, A.S., Pravdivets, Y.P., Schekhtman, N.V. "Tests of Earth Slope Revetments Permitting Flow of Water at Large Specific Discharges", (in Russian), Hidrotekhnicheskoe Stroite'stvo, No.4, April 1977, 22-26.
44. Hansen, Kenneth D., "Roller-Compacted Concrete For Overtopping Protection Of Embankment Dams", United States Committee On Large Dams-Twelfth Annual Lecture Series, 1992.
45. Hensley, Perry J.; Hennig, Charles C., "Overtopping Protection For A.R. Bowman Dam", Association Of State Dam Safety Officials - Eighth Annual Conference Proceedings, 1991.
46. Hensley, P.J., "Bureau of Reclamation Experience with Innovative Solutions to Increased Design Floods", Geotechnical Practice In Dam Rehabilitation, Pp. 672-687, 1993.
47. Hensley, P.J., Hennig C. C., "Overtopping Protection for A. R. Bowman Dam", ASCE Hydraulic Engineering, 1991.

48. Hewlett, H.W.M., Boorman, L.A., Bramley, M.E., "Design of Reinforced Grass Waterways", Construction Industry Research and Information Association (CIRIA), Report 116, 1987.
49. Jackson, H.E., "Overtopping Protection Using Roller-Compacted Concrete", ASCE Hydraulic Engineering, 1992.
50. Kindsvater, C.E. , "Discharge Characteristics of Embankment-Shaped Weirs." Studies of Flow of Water Over Weirs and Dams, USGS Water Supply Paper 1617-A, 1964.
51. Koutsourais, M. "A Study of Articulated Concrete Blocks Designed to Protect Embankment Dams", Association of Dam Safety Conference Proceedings, Boston Massachusetts, 1994.
52. Kybast, I., Parizek, J., "Safety of Existing Earth-Fill Dams Against Floods", International Committee On Large Dams - Fourteenth Congress - Rio De Janeiro, Vol.1, Q.52, R.037, 1982.
53. Lemons, Ronnie M.; King, John R.; and Rutledge, John L., Association Of State Dam Safety Officials - Seventh Annual Conference, 1990.
54. Lied, K.; Palmstrom, A.; Schieldrop, B.; Troblass, I., "Dam Subjected To Waves Generated By Avalanches, And To Extreme Floods From A Glacier Lake", International Commission On Large Dams - Twelfth Congress, Mexico, Vol.3, Q.47, R.009, 1976.
55. McGovern, Robert K.; Frizell, Kathleen H., "Design of A Continuously Reinforced Concrete Slab to Protect A.R. Bowman Dam During Overtopping", United States Committee On Large Dams - Twelfth Annual Lecture Series, 1992.
56. McLean, F.G. And Hansen, K.D., "Roller Compacted Concrete For Embankment Overtopping Protection", Geotechnical Practice In Dam Rehabilitation, Pp. 188-209, 1993.
57. McLean, F.G. And Hansen, K.D., "RCC: Rehab Results", Civil Engineering, August 1993.
58. McWhorter, J.C., T.G. Carpenter, and R.N. CLARK, Erosion Control Criteria for Drainage Channels. Prepared for the Mississippi State Highway Department in cooperation with U.S. Department of Transportation, Federal Highway Administration, by Agricultural Experiment Station, Mississippi State University, State College, Mississippi, March 1968.

59. Miller S. Paul; Ko, Hon-Yim; Cunn, Jeffery, "Embankment Overtopping, Hydraulics and Hydrology In The Small Computer Age', American Society Of Civil Engineers Specialty Conference, Lake Beuna Vista, FL., Vol.2, Pp.1082-1087, 1985 Aug.
60. Moore, J.S., D.M. Temple, H.A.D. Kirsten. Headcut Advance Threshold in Earth Spillways, Technical Notes, 1993 SCS/ARS.
61. Morse, D., "Geosynthetic Shields for Dams", Civil Engineering Jan. 1989.
62. Murthy, Y.K.; Mane, P.M.; Pant, B., "Tensile Failures In Some Stone Masonry Gravity Dams In India", International Commission On Large Dams - Thirteenth Congress - New Delhi, Vol.2, Q.49, R.031, 1979.
63. Normann, J.M. (1975). "Design of stable channels with flexible linings", Highway engineering circular No. 15. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.
64. Oswalt, N.R., "Overtopping Flow Spillway for Baldhill Dam Sheyene River, North Dakota", ASCE Hydraulic Engineering, Pp. 124-129; 1991.
65. Oswalt, N. R., "Overtopping Protection Alternatives for Dams", ASCE Hydraulic Engineering, Pp.1210-1215; 1992.
66. Powledge, G.R., Ralston, D.C., Miller, P., Chen, Y.H., Temple, D.M., "Mechanics of Overflow Erosion on Embankments. I: Research Activities", ASCE Journal of Hydraulic Engineering, Pp. 1040-1055, Vol. 115, No. 8, Aug, 1989.
67. Powledge, G.R.; Sveum, D.L., "Overtopping Embankment Dams - An Alternative In Accommodating Rare Floods", International Commission On Large Dams - Sixteenth Congress - San Francisco, Vol. 4, Q. 63, R.035, 1988.
68. Powledge, George R.; Sveum, D. L., "Overtopping Of Small Dams - An Alternative For Dam Safety, Hydraulics and Hydrology In The Small Computer Age", American Society Of Civil Engineers Specialty Conference, Lake Beuna Vista, FL., Vol.2, Pp.1071-1076, 1985 Aug.
69. Powledge, George R., P.E.; Pravdivets, Yuri P., D. SC., "Overtopping Of Embankments To Accommodate Large Flood Events, An Overview", United States Committee On Large Dams - Twelfth Annual Lecture Series, 1992.

70. Powledge, George R., P.E.; Wooten, R. Lee, P.E.; Whiteside, Stephen L., P.E., "Overtopping Protection Of Three Blue Ridge Parkway Dams", Association Of State Dam Safety Officials - Eighth Annual Conference Proceedings, 1991.
71. Pravdivets, Y.P., Slissky, S.M., "Passing Floodwaters Over Embankment Dams", Water Power and Dam Construction, Pp. 30-32, July 1981.
72. Pravdivets, Y.P., Bramley, M.E., "Stepped Protection Blocks for Dam Spillways", Water Power and Dam Construction, Vol.41, No. 7, July 1989.
73. Pravdivets, Y.P., "Flood Passage Over the Cofferdam of the Dneister Power Developments", (in Russian), *Energeticheskoe Stroitelstvo* No.11, 1978.
74. Prendergast, J., "RCC at 10", Civil Engineering, October 1992.
75. Prendergast, J., "Historic Dam Rehab: Handle with Care", Civil Engineering, October 1991.
76. Riley, R.C., "A Procedure For Evaluating Permissible Limits of Overtopping of Vegetated Earthen Embankments", ASDSO Annual Meeting, Austin Texas, 1986.
77. Rofe, Kennard & Lapworth, "Design of Stepped Block Spillways", CIRIA Research Project 407, Feb. 1994.
78. Shand, N.; Pells, P.J.N., "Experience In the Design And Construction Of Reinforced Rockfill Dams", International Commission On Large Dams - Tenth Congress - Montreal, Vol. aa, Q.36, R.019, 1970.
79. Simons, Li & Associates, "Full-Scale Hydraulic Studies of Cable-Tied Concrete Revetment Systems for Minimizing Embankment Damage During Overtopping Flows".
80. Simons, D.B., and Senturk, F. (1977). "Sediment Transport Technology." Water Resources Publications, Ft. Collins, CO.
81. Simons, D.B., and Senturk, F. (1992). "Sediment Transport Technology." (2nd ed.). Water Resources Publications, Ft. Collins, CO.
82. Soil Conservation Service. 1954. "Handbook of channel design for soil and water conservation". SCS-TP-61, rev., 34 pp.

83. Soil Conservation Service (1969). "Engineering Field Manual for Conservation Practices." Soil Conservation Service, U.S. Department of Agriculture, Washington, D.C.
84. Soil Conservation Service. 1977. "Design of Open Channels". SCS-TR-25, pp. 225 .
85. Soil Conservation Service, (1994) DAMS2, Draft of Technical Release 48, Structure Site Analysis Computer Program Technical Reference and Users Manual, Soil Conservation Service U. S. Department of Agriculture, Nov. 1994.
86. Temple, D.M., J.A. Brevard, J.S. Moore, G.J. Hanson, E.H. Grissinger, and J.M. Bradford. 1993. "Analysis of Vegetated Earth Spillways", ASDSO 10th annual conference, Kansas City, MO, Sept. 1993.
87. Temple, D.M., K.M. Robinson, R.M. Ahring, and A.G. Davis. 1987. *Stability Design of Grass-Lined Open Channels*. U.S. Department of Agriculture, Agriculture Handbook 667, 175 p., illus.
88. Tschantz, B.A., "1989 Report on Review of State Non-federal Dam Safety Programs", August 1989. Association of Dam Safety Officials and Federal Emergency Management Agency.
89. Tennessee Valley Authority, Chen / Hydro Review, file 108-96.03B
90. Tennessee Valley Authority, Chickamauga Erosion Studies, file 108-96.03A
91. Tennessee Valley Authority, Chickamauga pamphlet.
92. Tennessee Valley Authority, The Chickamauga Project Technical report No.6.
93. Tennessee Valley Authority, Chickamauga Safety Analysis EAP and Economic Analysis, file 108-96.01.
94. Tennessee Valley Authority, Russian Wedge-Shape Blocks Studies Proposal from CIRIA.
95. U.S. Army Corps Of Engineers, "Flow Over Non-Level Dam Crest: Supplement to HEC-1", User's Manual For Dam Safety Investigations, 1979.
96. USDA, SCS, Headcut Erodibility Index Photo Reference, September, 1993.

97. U. S. Geological Survey, "Performance of Temporary Ditch Linings." Prepared for the Federal Highway Administration at the U. S. Geological Survey Gulf Coast Hydroscience Center, Bay St. Louis, Mississippi, 1985.
98. Vermeyen T., D. Mares, "Alternatives for Enhancing Spillway Capacity Currently Being Pursued by the U. S. Bureau of Reclamations", United States Committee on Large Dams, Twelfth Annual USCOLD Lecture Series, Ft. Worth Texas, April 1992.
99. Vischer, D.; Funk, M.; Miller, D., "Interaction Between A Reservoir And A Partially Flooded Glacier: Problems During the Design Stage", International Commission On Large Dams - Seventeenth Congress - Vienna, Vol.1, Q.64, R.008, 1991.
100. Wooten, R.L., Whiteside, S.L., Welsh, R. And Wirkus, K.E., "Construction of CCM Overtopping Protection On Three Parkway Dams", Geotechnical Practice in Dam Rehabilitation, Pp. 210-224, 1993.
101. Wooten, R.L., Powledge, G.R., Whiteside, S.L., "Dams Going Safely Over the Top", Civil Engineering January 1992.
102. Wooten, Lee R.; Powledge, George R.; Whiteside, Stephen L., "Cellular Concrete Mat Overtopping Protection Systems On Three Blue Ridge Parkway Dams", Proceedings: 1990 National Conference of Hydraulic Engineering, 1990.
103. Wooten, Lee R.; Powledge, George R.; Whiteside, Stephen L., "CCM Overtopping Protection On Three Parkway Dams", American Society Of Civil Engineers - Hydraulic Engineering, Pp.1152-1157, 1990.

APPENDIX A

Table A.1. Summary of Armoring Alternatives

ARMOR TYPE	DESCRIPTION	APPLICATION	LIMITATIONS	COST	WHO TESTED
Vegetation	Selected vegetation on embankment	* Plant seeds and provide maintenance * Consider soil erodibility and type of vegetation * Establish root structure	* Poor cover until good growth established * Minimal critical velocity * Minimal permissible shear stress * 0.7 sq. ft./sec - 3 hours * 2.0 sq. ft./sec - 25 min.	Dependent on type of vegetation and maintenance required	CIRIA - Construction Industry Research and Information Assoc. ARS - Agriculture Research Service
Geotextiles	Synthetic fabrics, mats or large-scale cells	* Embed geotextile in soil * Cover entire surface area with vegetation * Roots will bind around geotextile and form a geotextile /soil/root mat	* Proper application and anchoring is essential * Typically used along with another method * Max. velocity - 22 ft/sec * Max. overtopping head - 4ft	Dependent on type of geotextile used \$0.75 - \$4 / sq. ft.	CIRIA (Great Britain) SLA - Simons, Li and Associates FHWA - Federal Highway Administration USBR
Riprap	Stone or rock fragments on embankment	Hand placed or dumped stone with specified diameter, grade and thickness, determined by engineering hydraulics	* Current design methods do not account for steep embankments, high velocity, and shallow flow	Dependent on availability of stone required. Transportation - \$6 / cu. yd.	USBR - U.S. Bureau of Reclamations
Gabions	Uniformly graded stone encased in wire baskets, placed on embankment	* Dimensions of stone and baskets determined by hydraulics. * Pack stone tightly * Anchoring of baskets may be necessary	* Deformation of wire basket * Displacement of rock fill * Scour downstream from gabions * Have not been tested for velocities > 24 ft/sec	\$30 - \$60 /sq. yd.	USBR SLA
Cements:					
The use of cement to form either soil cement or roller-compacted concrete					
Soil Cement	Mixing cement to soil	* Soils can be stabilized using 1-10% cement * 1-3% erodibility is reduced * >4% essentially no erosion	* Found to be most stable method for protection during overtopping (SLA) * No failure - to 25 cfs/ft with a head of 4 ft * Potential toe erosion and weathering	\$1.4-\$6 / sq. ft.	SLA

NOTE: Inconsistencies in units and parameters due to inconsistent testing and reported results in literature

Table A.1 (cont.) Summary of Armoring Alternatives

ARMOR TYPE	DESCRIPTION	APPLICATION	LIMITATIONS	COST	WHO TESTED
CEMENTS (continued):					
Roller-Compacted Concrete (RCC)	Similar to soil cement but typically stronger.	* Rolling concrete in layers causing a step formation	* Conceptual and intuitive designs, design criteria not well developed * Must provide adequate drainage * Concern for freeze-thaw and wet/dry conditions	1:1 slope 6-10\$/sq ft 3:1 slope 3-6\$/sq ft Case studies - \$30-\$90 / cu.yd.	Not laboratory tested In the field - detention dam on the North Fork of the Tottle River and the Ringtown Dam No.5 (frequent and continual flow)
Pre-fabricated Concrete Block Cable-tied	Precast concrete blocks placed on top of geotextile fabric or bedding material				
	Prefabricated mats of cellular concrete blocks or grids	* Blocks are tied together with cables forming a mat * The revetment system is placed as a mattress	* Must assure that the block to be used has been tested under overtopping conditions * Consider maintenance	\$3-\$6 / sq ft	CIRIA SLA
Interlocking	Precast cellular concrete blocks designed to interlock in the plane of the slope (plan-bonded), vertically (stack-bonded), or both	* Blocks are placed by hand block by block	* Good grass growth essential for greater cellular blocks stability * CCM's not tested for velocities > 26 ft/sec duration of overtopping is a limiting factor		
Wedge-Shaped Developed by Yuri Pravdivets of the Moscow Institute of C. E.	Blocks designed so that the flow pattern over each block creates a curvature of flow. This increases the stability by imposing a downward force on the block. A low-pressure zone is formed downstream of each block, therefore the seepage flow is "sucked up" by the uplift pressure. The large surface roughness helps to dissipate energy.	* The blocks have an overlapping edge and are placed on the downstream slope from toe to crest. * Consider crest blocks and toe blocks * Blocks may be cable-tied without overlap (overlap is more stable).	* Wedge tests show positive results up to their max. unit discharge * Mean velocities - 66 ft/sec Pravdivits * Underlying filter material should be considered (if gravel may wash away)	\$5-\$20 / per block estimate seems high (USBR)	Moscow - Pravdivets SLA CIRIA USBR

NOTE: Inconsistencies in units and parameters due to inconsistent testing and reported results in literature

Table A.2. Roller Compacted Concrete Overtopping Protection Projects (USA) [8]

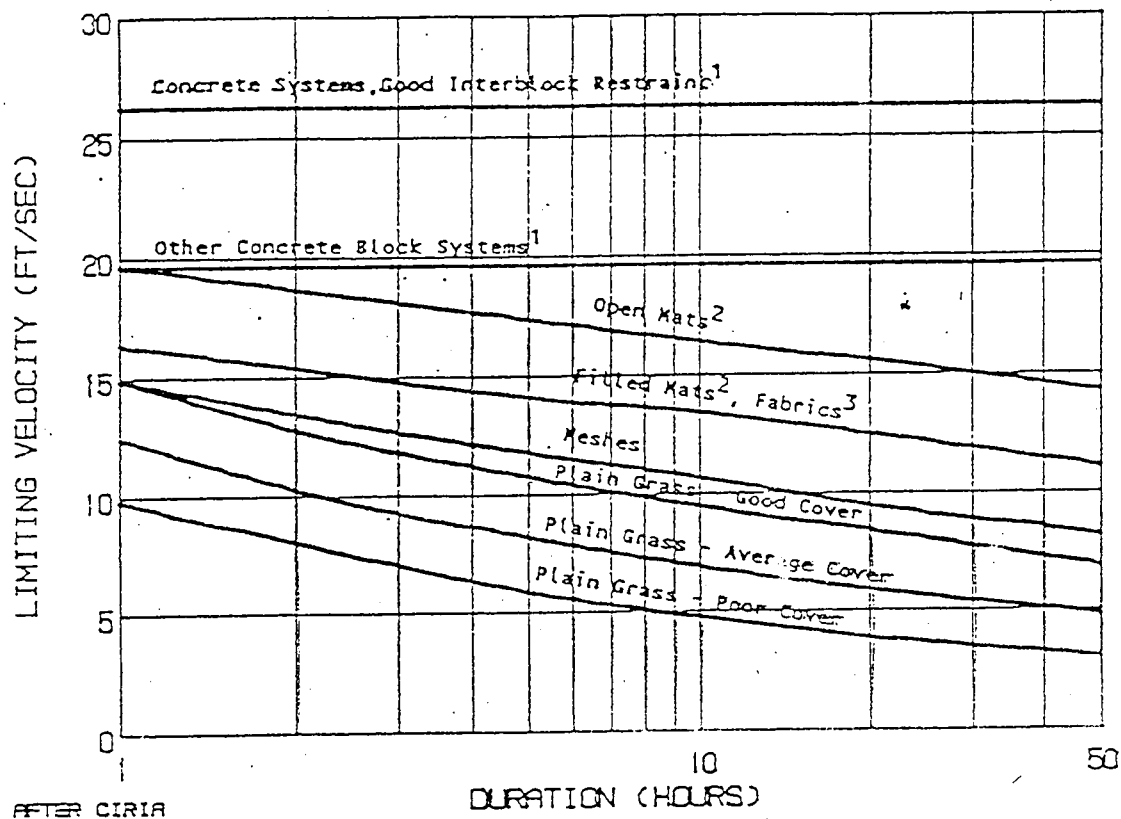
Name (Year Constructed)	City/State	Owner/Engineer	Max. Height (ft)	Max. Unit Discharge (cfs/ft)	Max. Over-flow Height (ft)	Protection Level (% PMF)	Total Duration Overflow (Max.)	RCC Cost (\$/cu.yd.)
North Fork Toutle River (1980) Replacement Service Spillway	Castle Dale, Washington	US Army Corps of Engineers Portland OR District	38	----	----	----	Continuous 11 months	37
Brownwood Country Club (1984)	Brownwood, Texas	Brownwood Country Club/Freese & Nichols	19	24.7	5.5	100	6 hours	42
Spring Creek (1986)	Gunnison, Colorado	Colorado Div. of Wildlife/Morrison- Knudsen Engineers	53	44.4	4.5	100	5 hours	36.93
Harris Park No.1 (1986)	Bailey, Colorado	Harris Park Water & San. Dist./ Edward Shaw	18	91(a)	10	100	----	47.25
Conanche Trail (1988)	Big Spring, Texas	City of Big Spring/ Freese & Nichols	20	60	6	100	5.5 hours	38.91
Addicks (1988)	Houston, Texas	US Army Corps of Engineers, Galveston District	48.5	7.1	2.2	100	28 hours	78.99
Barker (1988)	Houston, Texas	US Army Corps of Engineers, Galveston District	36.5	10.7	2.2	100	33 hours	78.99
Bishop Creek No. 2 (1989) New Emergency Spillway	Bishop, California	Southern California Edison/So. Cal Edison & J.M. Montgomery	41	24	3	70	18 hours	87.5
Goose Lake (1989)	Nederland, Colorado	City of Boulder/ Harza Engineering	35	9.1	2.1	7.5	5 hours	51.16
Boney Falls (1989)	Escanaba, Michigan	Mead Paper Company/ Harza Engineering	25	100	6.0 embk. 10.0 RCC	68	38 hours	59.51
Comanche (1990) New Emergency Spillway	Estes Park, Colorado	City of Greeley/ Morrison Knudsen Engineers	46	101	10	75	4 hours	67
Kemmerer City (1990)	Kemmerer, Wyoming	City of Kemmerer/ Woodward-Clyde	31	24	3.6	50	4 hours	75.06
Thompson Park No.3 (1990)	Amarillo, Texas	City of Amarillo/ HDR Engineering	30	30 (a)	4.3	50	12 hours	52.21
White Cloud (1990)	White Cloud, Michigan	City of White Cloud/ OMM Engineering	15	----	1.5	----	----	83

Table A.2. (cont.) Roller Compacted Concrete Overtopping Protection Projects (USA) [8]

Name (Year Constructed)	City/State	Owner/Engineer	Max. Height (ft)	Max. Discharge (cfs/ft)	Max. Over-flow Height (ft)	Protection Level (% PMF)	Total Duration Overflow (Max.)	RCC Cost (\$/cu.yd.)
Ringtown No.5 (1991) Combined Principal and Emergency Spillway	Ringtown, Pennsylvania	Borough of Shenandoah/ Gannett-Fleming	60	56	7	100	frequent flow	46.16
Sallieck (1991) Two Emergency Spillways	Johnstown, Pennsylvania	Johnstown Water Authority/Gannett- Fleming	110	54	6.6	100	19 hours	79.41
Ashton (1990)	Ashton, Idaho	Pacific Power-Utah Power/Black & Veatch	60	122	12	100	10 days	----
Lake Lenape (1991)	Mays Landing, New Jersey	Atlantic County/ O'Brien & Gere	17	----	3	100	55 hours	70
Goose Pasture (1991)	Breckenridge, Colorado	Town of Breckenridge/ Tipton & Kalmbach	65	95	10	75	24 hours	47.88
Holmes Lake Dam (1991)	Marshall, Texas	T & P Lake, Inc./East Texas Engineering	31	----	5	26	5 hours	----
White Meadow Lake (1990)	Rockaway, New Jersey	White Meadow Lake Assn./O'Brien & Gere	20	----	1.4	50	12 hours	----
Butler Reservoir (1992)	Camp Gordon, Georgia	US Army Corp of Engineers, Savannah District	43	137	13.2	100	8 hours	----
Lake Diversion (1993) New Emergency Spillway	Wichita Falls, Texas	City of Wichita Falls/ Biggs & Mathews	85	316	20.4	100	6 days	30.18
Horsechief (1992)	Rapid City, South Dakota	US Forest Service - Denver	65	17	4.24	100	4.7 hours	50
Meadowlark Lake (1992)	Ten Sleep, Wyoming	US Forest Service - Denver	28	118	10.25	100	12.7 hours	66.25
Phillipsburg Dam No.3 (1992)	Phillipsburg, Pennsylvania	PA-American Water Co./O'Brien & Gere	20	14	6.9	100	15 hours	----
North Potato Diversion (1992) New Spillway	Copperhill, Tennessee	Federal Bankruptcy Court Dames & Moore	35	340	20	100	10 hours	65.51
Lima (1993)	Dell, Montana	Beaverhead Co. Red Rock River W&S Distric HKM Assoc.	54	61	9.3	100	11 days	51.57
Rosebud (1993)	Rosebud, South Dakota	Rosebud Sioux Tribe/ Harza Engineering	33	55	7	16.4	8 hours	77.77
Umbarger (1993)	Canyon, Texas	US Fish & Wildlife Service/GIS Consultants	40	216	17.5	100	24 hours	46.54

Table A.3. Dams with Cellular-Concrete Block Overtopping Protection [41]

Project Description			Hydraulic Characteristics			Dam Characteristics			Block Char.	
Dam (Date)	Engineer	Location	Hazard Class	Overtopping Depth (ft)	Velocity (fps)	Dam Height (ft)	Spillway Width (ft)	Downstream Slope (H:V)	Block Weight (lbs)	Block Area (sq.ft)
Bass Lake (1988/90) Price Lake (1988/90) Trout Lake (1988/90)	GEI Consultants	Blue Ridge Parkway, NC	High	2-4.3	22-26	28-40	262-525	3:1	59.5	1.3
Wilkerson Lake (1991/92)	USCOE-Savannah	Augusta, Georgia	Low	1.3	14.0	15.0	197.0	3:1	48.5	1.1
Strahl Lake (1992/93)	Fink, Roberts, & Petrie	Indianapolis, Indiana	High	2.3	16.0	24.0	262.0	3:1	103.5	1.8
Sandy Run (1992/93) (3 dams)	USCOE-Savannah	Augusta, Georgia	Low	1.6	16.0	13-16	197.0	3:1	48.5	1.1



- Notes:
1. Minimum superficial mass 28 lb/ft² (135 kg/m²).
 2. Minimum nominal thickness 0.8 inch (20 mm).
 3. Installed within 0.8 inch (20 mm) of soil surface, or in conjunction with a surface mesh.
 4. These graphs should only be used for erosion resistance to unidirectional flow. Values are based on available experience and information at the date of this report.
 5. All reinforced grass values assume well established, good grass cover.
 6. Other criteria (such as short-term protection, ease of installation and management, susceptibility to vandalism, etc...) must be considered in choice of reinforcement.

Figure A.1. CIRIA Velocity Duration Curve [8]

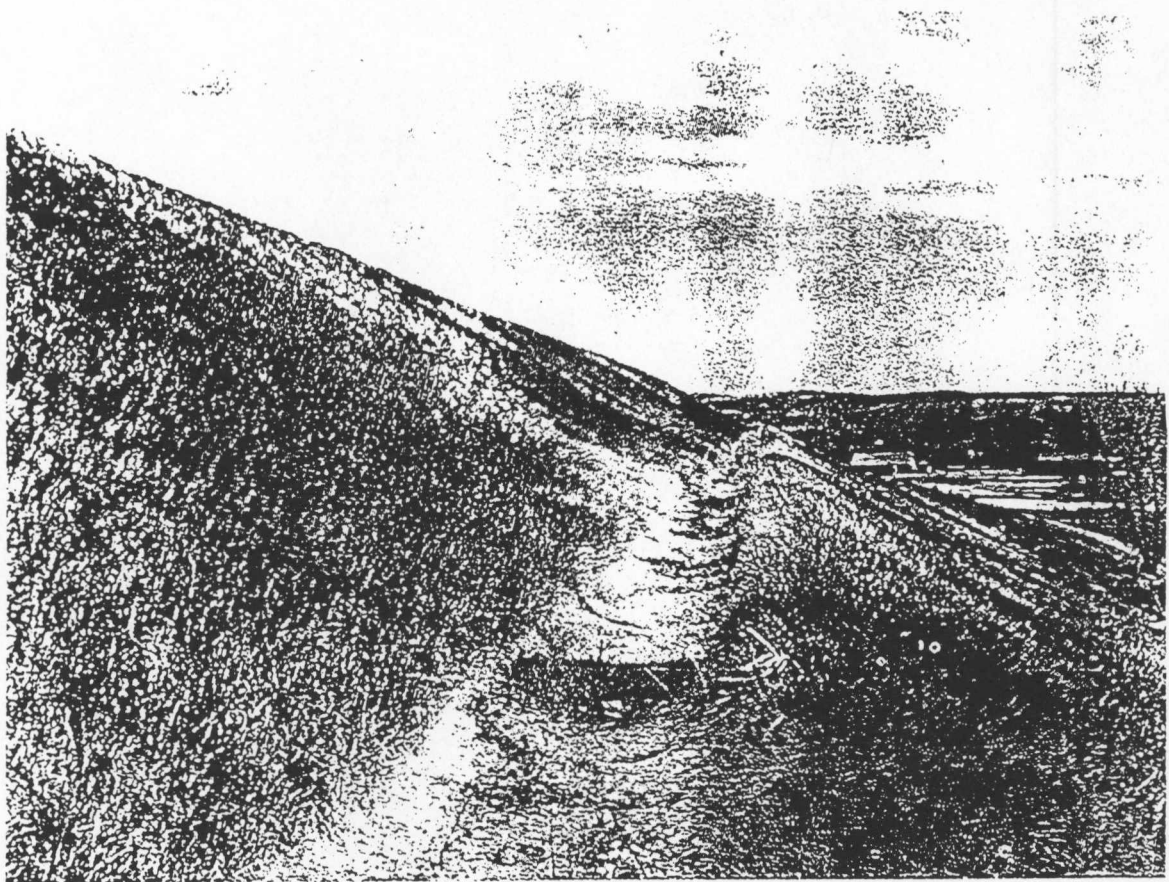


Figure A.2. Downstream slope of Belle Furche Dam protected by grass [15]

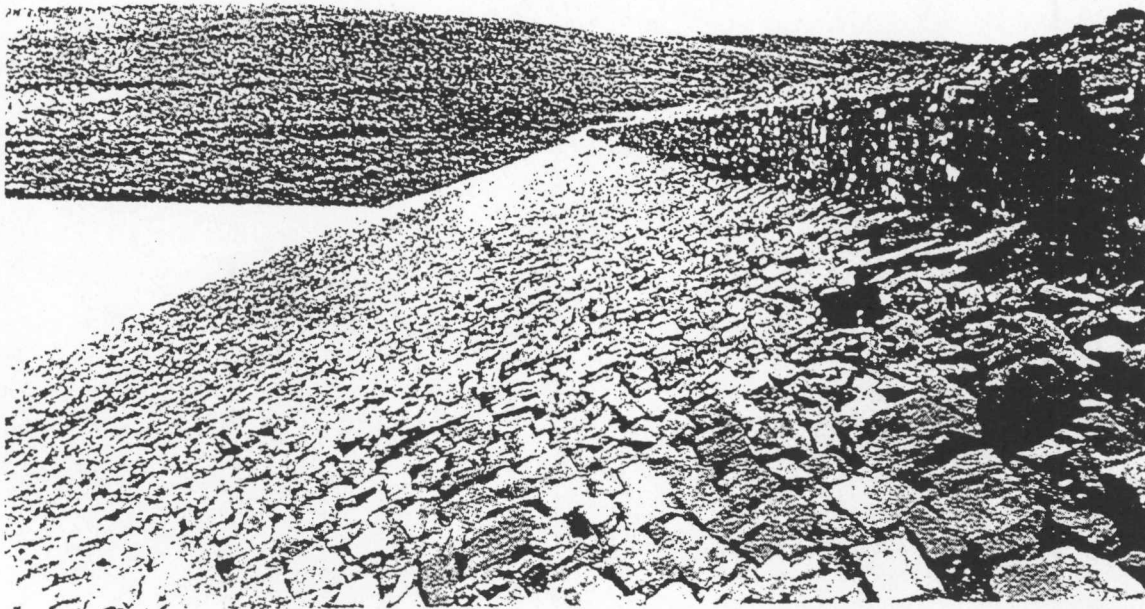


Figure A.3. Hand-placed riprap on Indian Creek Dike, Utah [15]

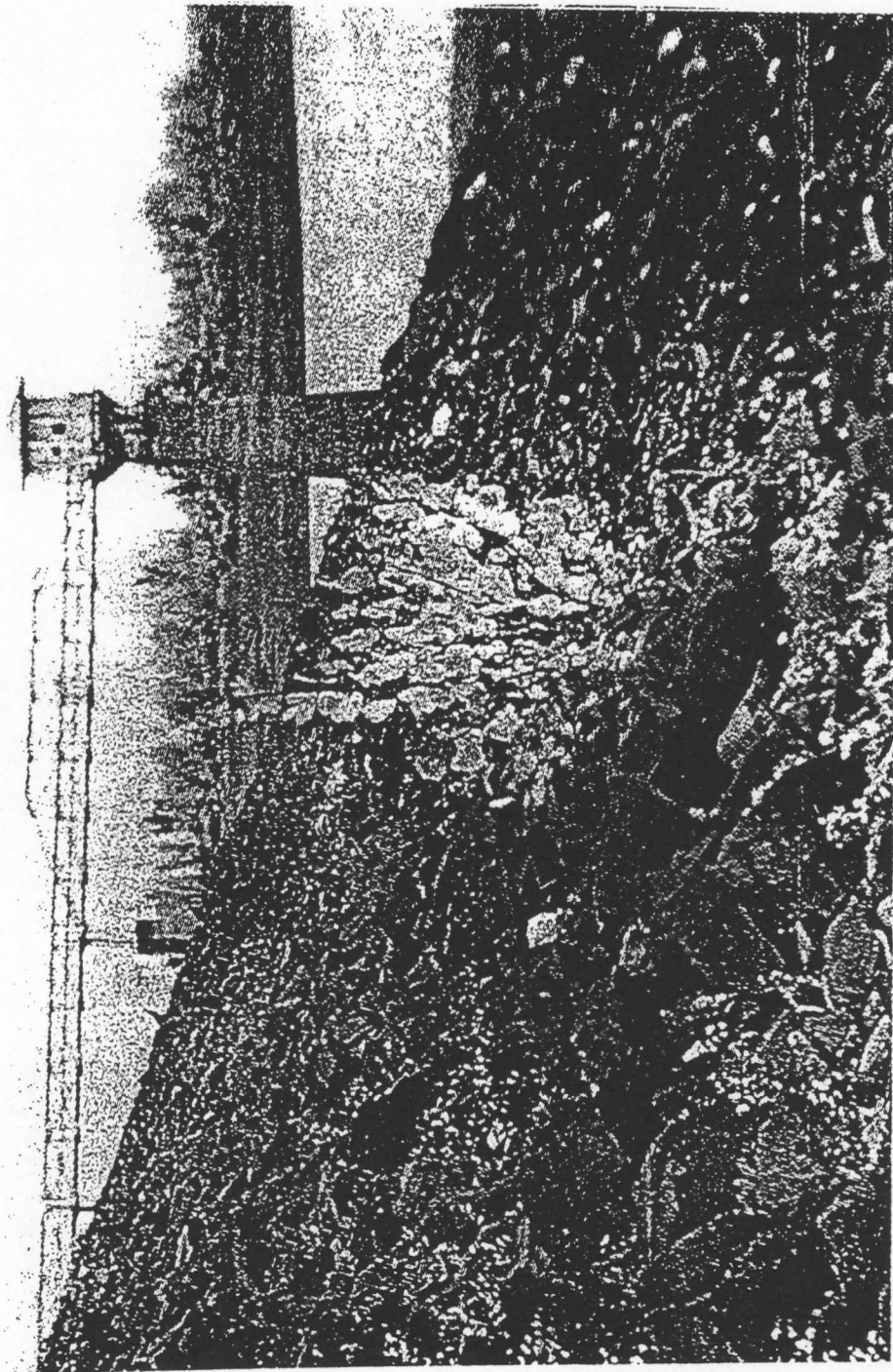
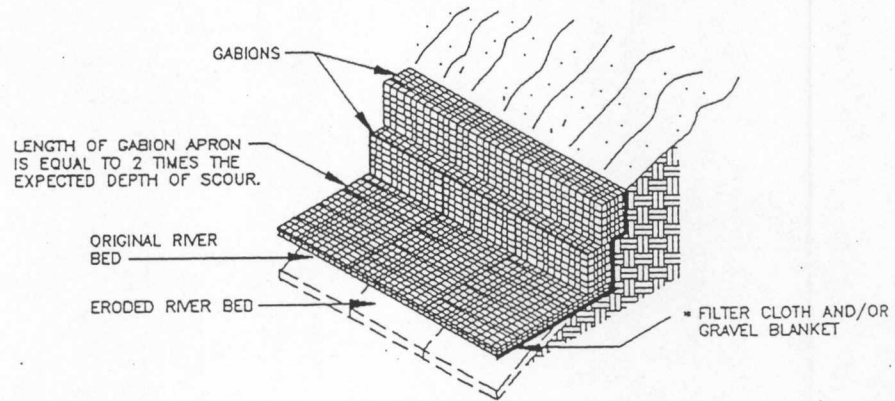
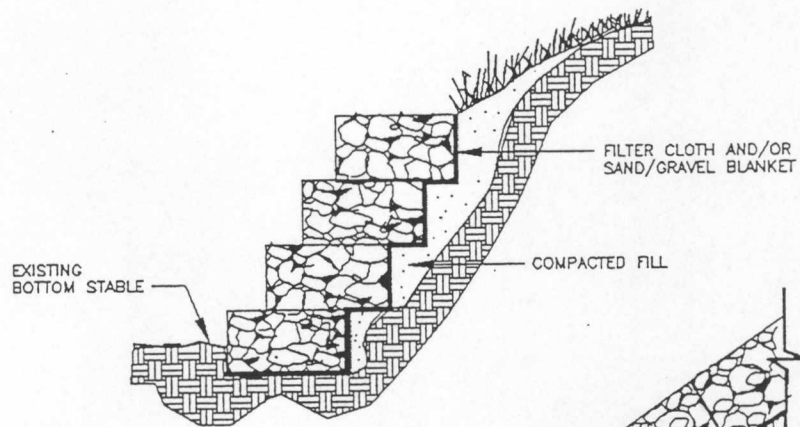


Figure A.4. Riprap on upstream slope of Cold Springs Dam, Oregon [15]

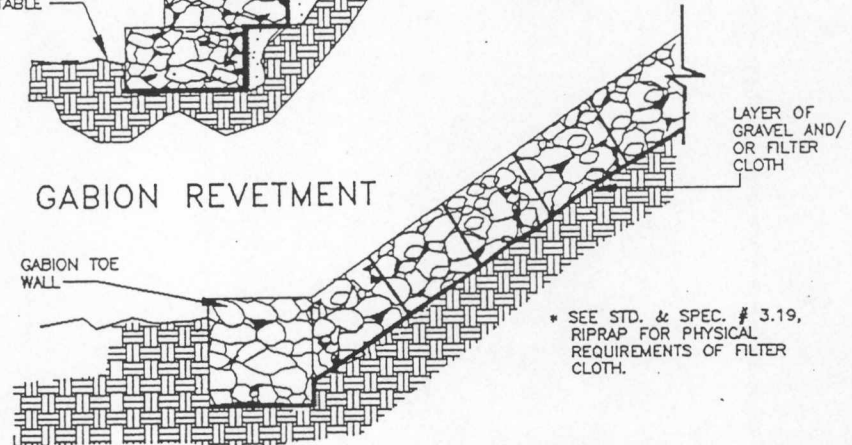
GABIONS



GABION TOE WALL



GABION REVETMENT



* SEE STD. & SPEC. # 3.19, RIPRAP FOR PHYSICAL REQUIREMENTS OF FILTER CLOTH.

REVMATTRESS / RENOMATTRESS

Figure A.5. Gabions

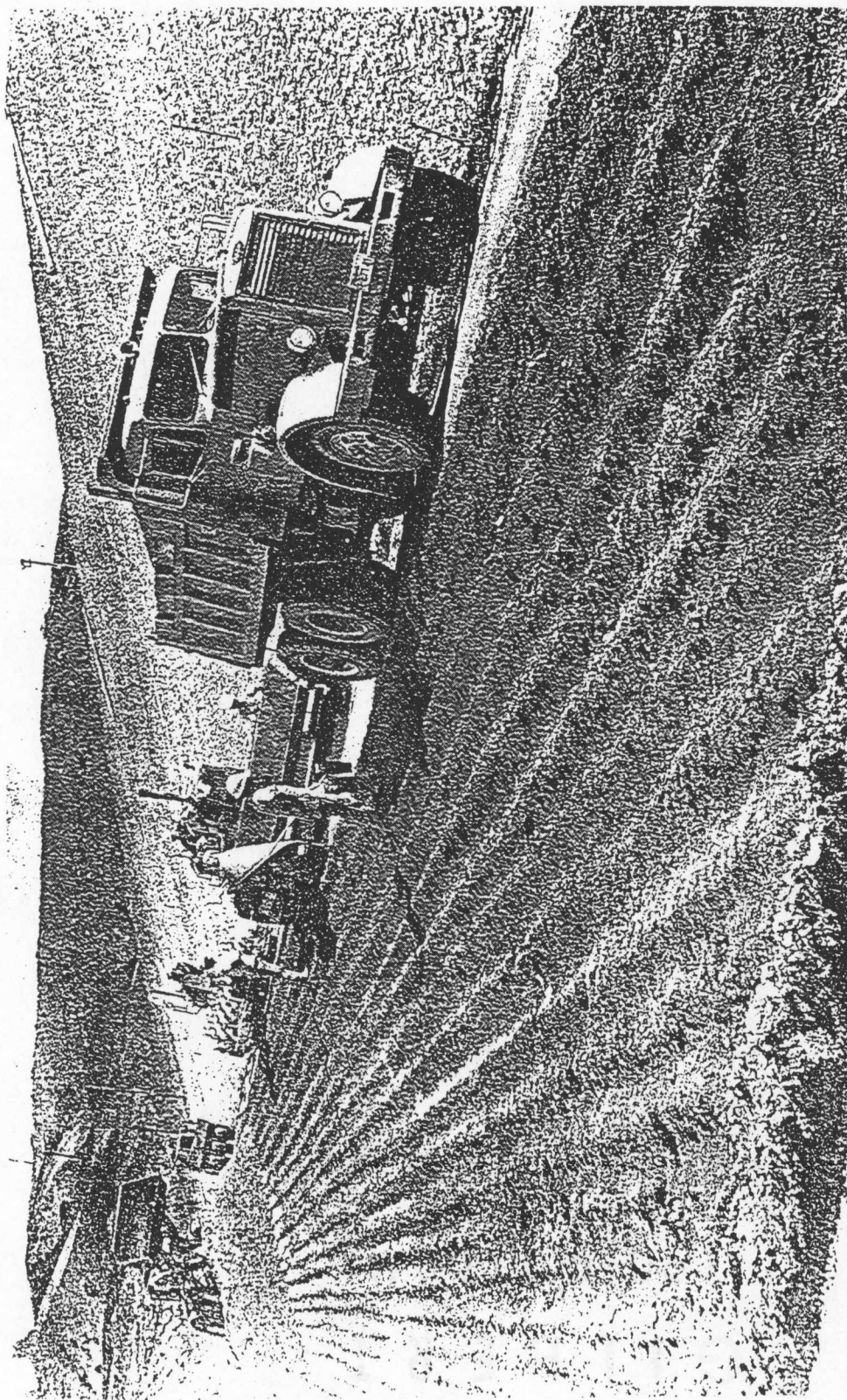


Figure A.6. Placement of Soil Cement Protection on Starvation Dam, Utah [15]

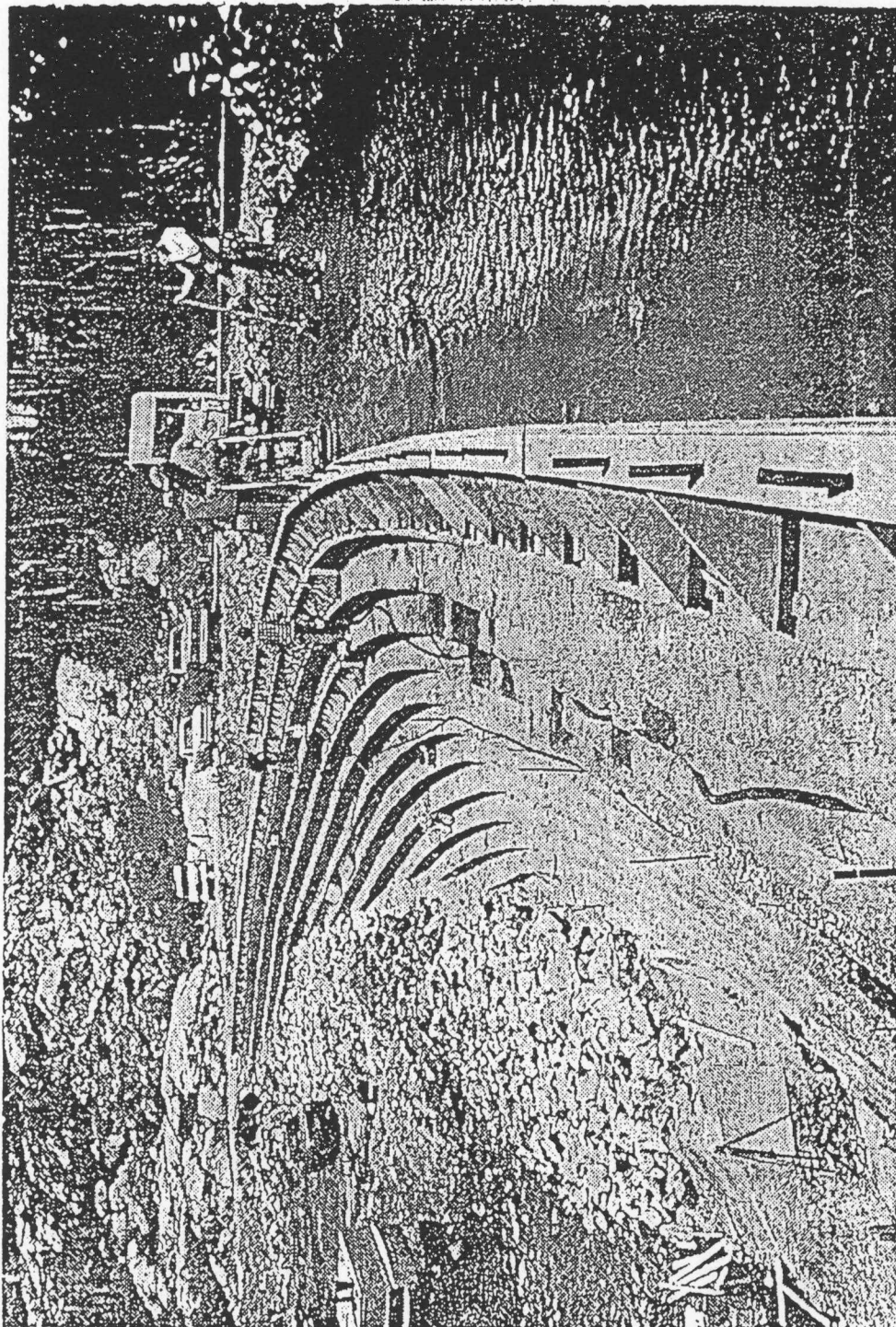


Figure A.7. Roller-Compacted Concrete Placement on Goose Lake Dam, Colorado [24]

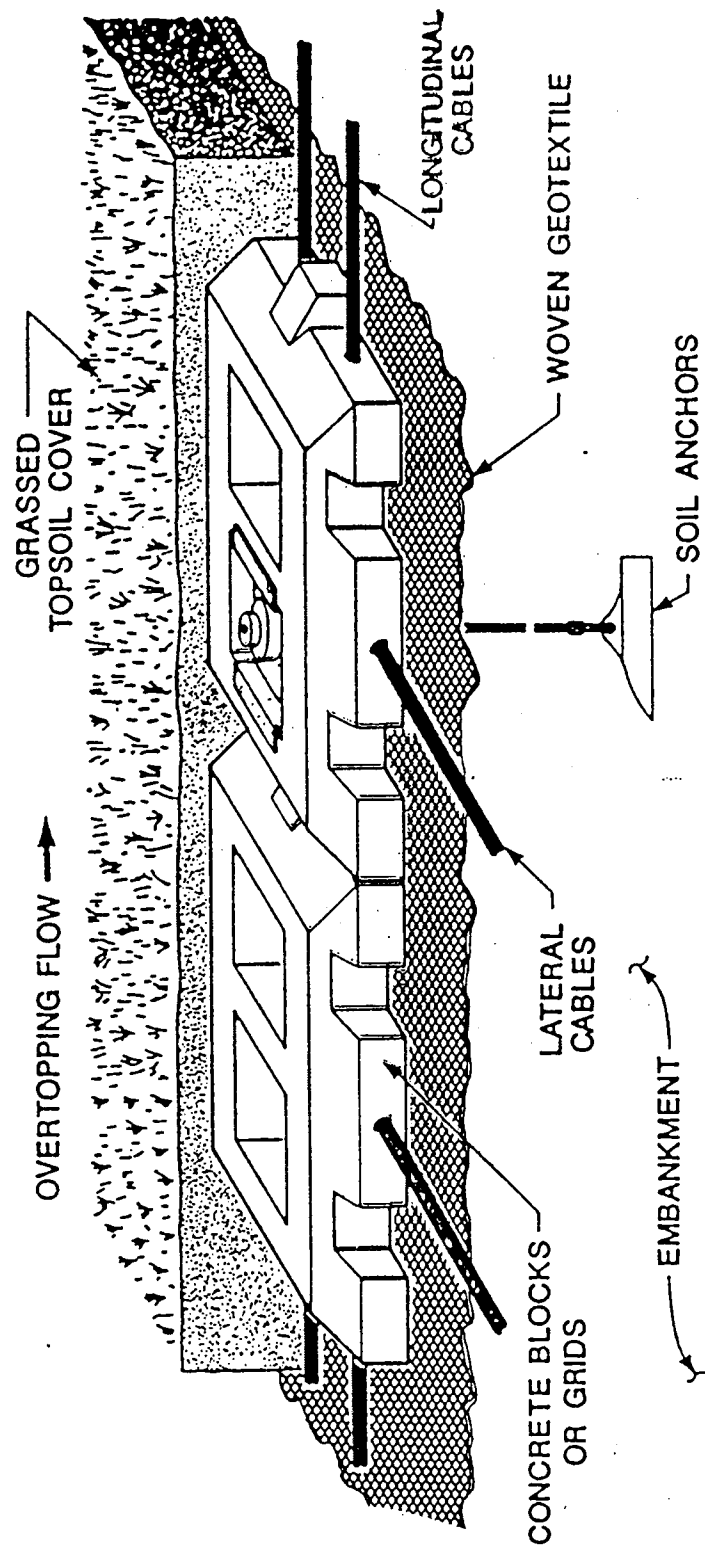


Figure A.8. Cellular Concrete Block Sketch [30]

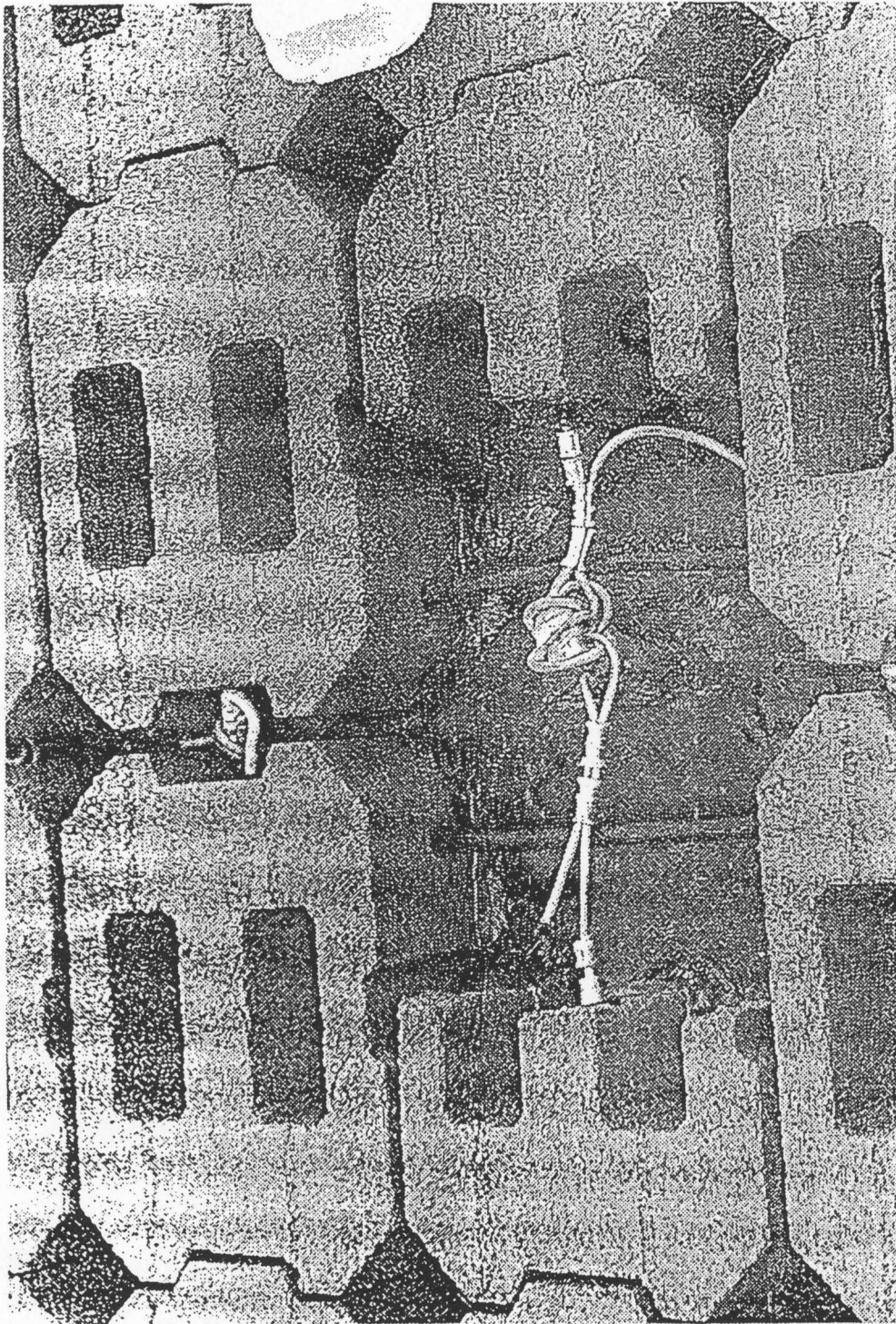


Figure A.9. Helical Anchor Attachment for Cellular Concrete Blocks [30]



Figure A.10. Placing CCMs at Bass Lake Dam [30]

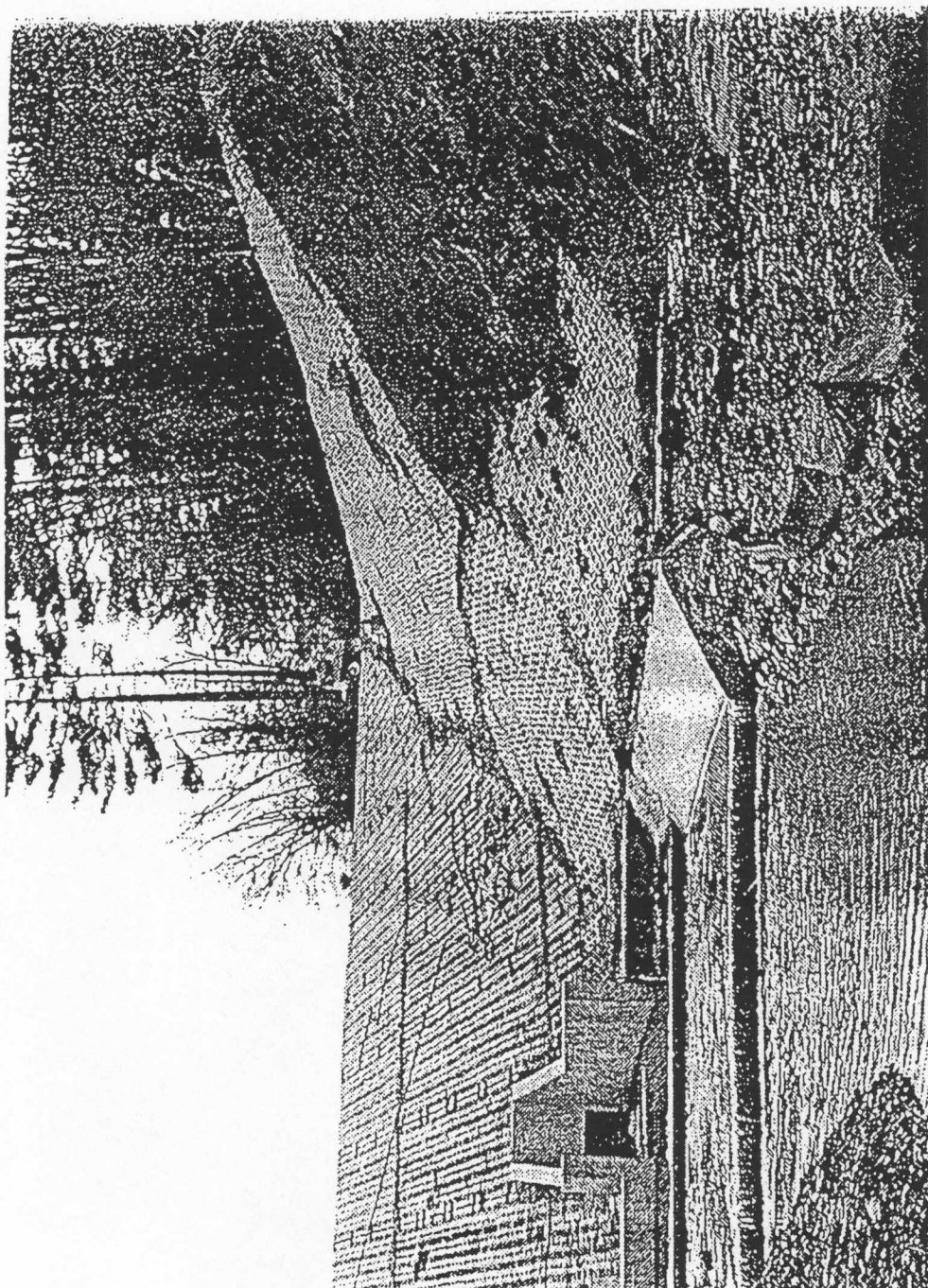


Figure A.11. CCMs on Trout Lake Dam [30]

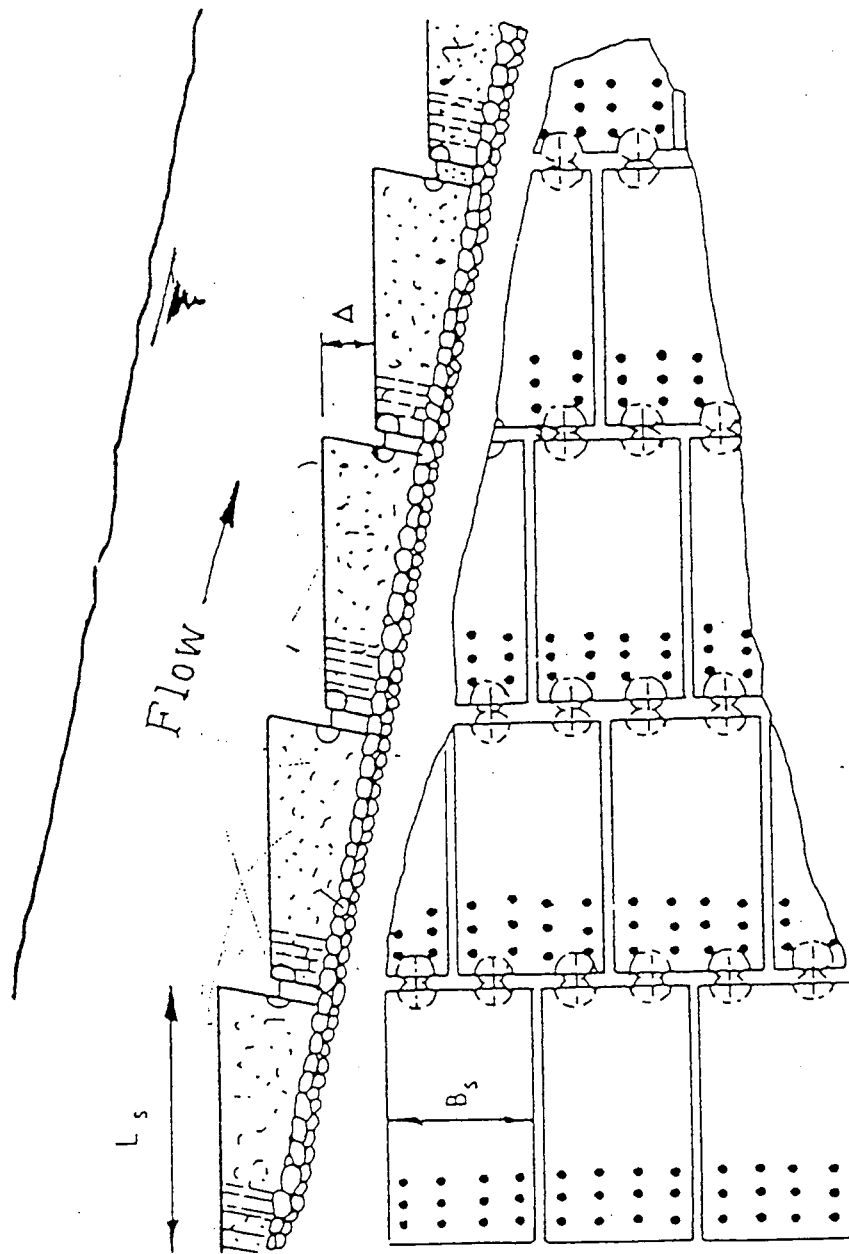


Figure A.12. Original Wedge-Shaped Blocks [34]

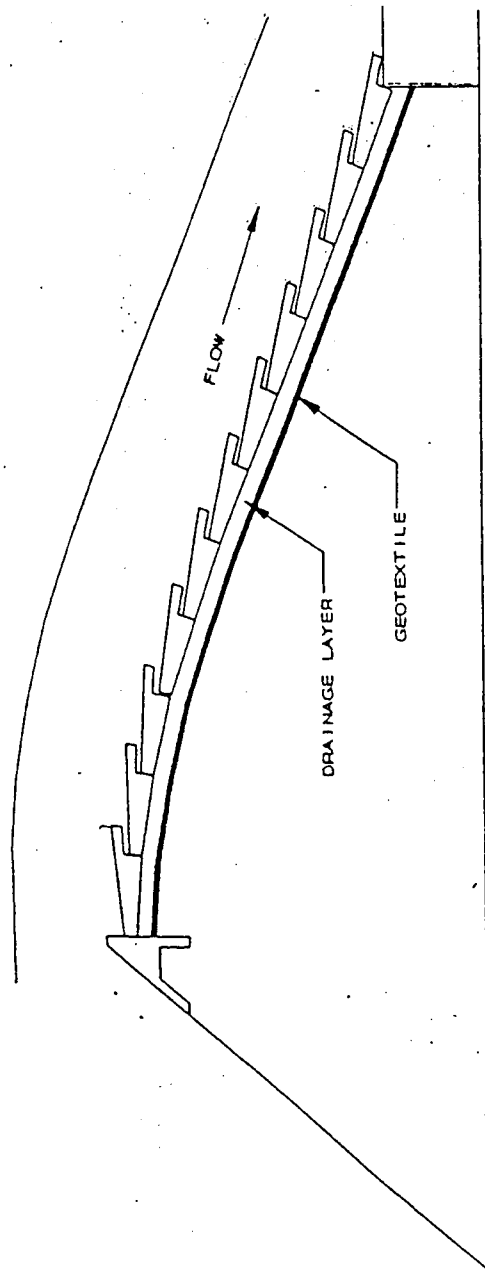


Figure A.13. Overlapping Wedge-Shaped Blocks [41]

Dear Sir,

February 28, 1995

The University of Tennessee, in partnership with The Tennessee Valley Authority, is conducting a study concerning the application of overtopping protection for the modification of earthen embankments, in situations where spillway capacity is incapable of meeting federal guidelines. We are also interested in the thinking at the state level, and would appreciate your taking the time to answer the following questions as they apply to your state. Please send your response to the address below, or fax to Susan Henderson, c/o Sam Shipman at (615) 632-3188.

1. Does your state have any procedures which allow embankment overtopping?

2. Does your state allow the option of any kinds of embankment protection or armoring for inadequate spillway capacity?

3. If protection is allowable, are there any conditions or limitations to what could be done?

4. Would you please list any names, and or contacts, for dams under your state's jurisdiction where overtopping protection was used as a means of remediation?

Please return this form to:

Ms. Susan Henderson / Mr. Sam Shipman
Tennessee Valley Authority
400 West Summit Hill Dr.
WT 10-K
Knoxville, TN 37902

Figure A.14. ASDSO Survey Form

VITA

Susan Renee Henderson was born in Clinton, Tennessee on June 22, 1967. She graduated from Clinton Senior High School in 1985. She began studying Engineering at the University of Tennessee, Knoxville in May of 1989, and in December 1993 received a Bachelor of Science Degree in Civil Engineering. Throughout her undergraduate career, she co-oped at Tampa Electric Company in Tampa, Florida. In January of 1994 , she entered the Master's program at the University of Tennessee, Knoxville in Environmental Engineering with an emphasis in water resources. During her graduate studies she served as a graduate teaching assistant.