PAP-562

PROTECTION AND PROVISION FOR THE SAFE OVERTOPPING OF DAMS AND FLOOD BANKS

BY

CIRIA

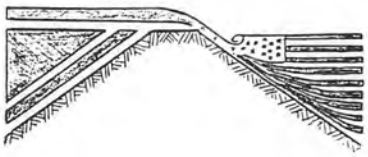
PAP-562

DEPARTMENT OF TRADE AND INDUSTRY

Protection and provision for safe overtopping of dams and flood banks

Report of Overseas Scientific & Technical Expert Mission

November 1987



CIRIA Project Report 2

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PROTECTION AND PROVISION FOR SAFE OVERTOPPING OF DAMS AND FLOOD BANKS

Report of Department of Trade and Industry
Overseas Scientific & Technical Expert Mission

Price £12

CIRIA
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FOREWORD

This report draws together information obtained in the United States by a team of UK engineers under the Department of Trade and Industry's Overseas Scientific and Technical Expert Mission (OSTEM) scheme.

The Government's OSTEM scheme covers all sectors of UK industry, although civil engineering missions are relatively rare. The broad objectives of the OSTEM scheme are:

- to increase the flow of industrially relevant civil scientific and technical information into the United Kingdom
- to make this information available as widely as possible within the United Kingdom
- to explore the potential for scientific and technical co-operation between UK and overseas organisations
- to promote a greater awareness amongst senior representatives of industry of advances taking place overseas through direct contact with key organisations and personalities in their subject areas.

The subject area covered by this report is
Protection, and provision for safe overtopping, of dams and river flood banks from erosion by flow in extreme flood events, together with related aspects of (a) dam/reservoir safety, and (b) protection of river banks.

As sponsor organisation, CIRIA was responsible for organising and leading the mission, and for dissemination of information to interested parties in the UK at the subsequent seminar on 24 November 1987 for which this report was prepared.

The mission team comprised:-

M E Bramley	CIRIA, Leader
M F Kennard	Rofe, Kennard & Lapworth
R Baker	University of Salford
C D Hall	Netlon Ltd
R W P May	Hydraulics Research Ltd
C Tuxford	Brooklyns Ltd to August 1987 (now with Ardon International Ltd)

and was selected to cover practitioner interests in the design/operation and manufacturing sectors, together with basic and applied research interests.

This report has been written by the mission team. Sections 2 through to 7 set out in turn the information presented by each of them at the seminar.

The mission visit took place over the period 16 to 24 July 1987, and was centred at:

- US Army Corps of Engineers, Waterways Experiment Station at Vicksburg, Mississippi
- US Bureau of Reclamation, Engineering and Research Center at Denver, Colorado.

Further information was obtained at the American Society of Civil Engineers (ASCE) National Hydraulics Conference at Williamsburg, Virginia from 3 to 6 August 1987.

Data presented in this report was gathered as a result of detailed meetings (see Appendix A) which were held with key US counterparts. CIRIA endorses the concept of the OSTEM scheme which provides an official framework for such meetings to take place at a national level. This view was shared by many of the team's US counterparts who remarked on the unique opportunity that the scheme provides for a formal exchange of information and know-how.

Acknowledgements

CIRIA and the mission team gratefully acknowledge the assistance of their many US counterparts who helped to arrange the mission and took part in the meetings. Particular thanks are due to Mr George Powledge, Chief Technical Review Staff at the USBR Engineering and Research Center, and to Mr Leroy McAnear, Chief Soils Division at the Waterways Experiment Station. The important role of the Department of Trade and Industry in funding the bulk of the mission costs and in establishing its official status as a UK delegation is also acknowledged.

Summary

This report presents information obtained by a DTI mission investigating research and practice in the US related to the protection and provision for safe overtopping of dams, road embankments and river flood banks subject to erosion by flow in extreme flood events. It emphasises items which are relevant either to UK industry or to similar problems in the UK, but which are not generally disseminated in the UK.

US engineers face a major task in upgrading standards of protection provided by many of their existing dams and flood embankments. Practice is being advanced both through an improved understanding of the mechanisms of erosion and failure of embankments subject to overtopping, and by research and trails on new methods of protection. Research studies and evaluation of past events have demonstrated the influence of soil type, flow pattern and local geometry of the embankment on the extent of erosion, and have confirmed that many embankments have survived limited overtopping. There is now an increasing acceptance among US dam engineers that designing for limited overtopping can provide a cost-effective and technically acceptable alternative to an enlarged service spillway for suitable low-hazard dams.

Methods of erosion protection currently favoured in the US are grass, rip-rap, roller compacted concrete and gabion mattresses. Some use has been made of geotextiles and concrete block systems, but these methods are still undergoing evaluation. Significant advances are reported in the development of design methods for grass and rip-rap protection. Little new information was obtained on the use of geotexile or cellular concrete reinforced grass. Full scale tests on the performance of these reinforcing systems used on ungrassed embankments are confirming the importance of soil type and grass root integration on the effectiveness of reinforced grass as developed in the UK.

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The protection, and provision for safe overtopping, of dams and flood banks against erosion by flow in extreme flood events has attracted increasing attention from UK engineers and researchers in recent years. The reasons for this include:-

- the implementation of the Reservoirs Act: 1975, and the sponsorship of a programme of research into reservoir safety by the Department of the Environment
- the need to increase the standard of safety provided by many existing structures, often as a result of re-estimation of the design flood
- interest in materials and structures which are environmentally attractive and acceptable
- the increasing range of new or novel materials which are available for consideration as low-cost protection against erosion.

1.1 Scope of UK applications

Normal engineering practice in the UK as regards overtopping of dams differs from that related to flood embankments. With dams which come within the scope of the Reservoirs Act, the inspecting engineer will normally only permit overtopping in cases where a breach of the dam would cause negligible risk to life and limited damage to property. The standard adopted for overtopping would be no less than a 150 year event. With UK dams, low-cost protection is more often used as lining for auxiliary spillways rather than as protection against general overtopping. With river flood banks, overtopping is more common since lower standards (25 year or less) are adopted for protection of agricultural land where life and property are not at risk. Most flood banks are grassed, and interest relates primarily to the effectiveness of grass alone in resisting erosion although an increasing number of schemes now incorporate purpose-lined overspill zones into controlled washlands or flood relief channels.

Other structures, such as farm reservoirs, amenity lakes and flood storage ponds, also retain flood water and may be designed to permit limited overtopping or incorporate a purpose-designed auxiliary spillway.

With rivers, canals and drains, the engineer is also interested in cost-effective and environmentally acceptable methods of bank protection. Many of the criteria for protection of embankments against erosion by overtopping also relate to protection of river, canal and drain banks in the occasionally-wetted zone above water level.

1.2 Present State-of-the-Art

Understanding of the physical processes of erosion of embankments and steep waterways, and of the performance under flow of many of the materials put forward to protect them, is still incomplete. Many of the design methods are empirically based, and experience of prototype performance is limited. There is therefore considerable benefit to be gained from examination of similar state-of-the-art data from the United States where similar problems and applications are found, particularly since these are both more numerous, and often on a larger scale.

1.3 Background to US applications

In contrast to the approximate figure of 2000 dams in Britain which come within the scope of UK dam safety legislation (reservoir capacity greater than 25 000m^3), over 63 000 dams were inventorised in the United States under the US government's recent national program of inspection (US Corps of Engineers, 1972). Less than 10% of these are classified as high hazard (requiring main spillway to pass a PMF inflow). Many of the remaining 57 000 dams require upgrading to enable the so-called "safety design flood" to be accomodated.

In addition to the above "jurisdictional" dams, there are many thousands of farm dams for irrigation and stock watering, soil conservation dams and urban flood detention ponds in the US which do not come within the remit of state dam safety programmes, but which require effective means of accomodating flood inflow. As in the UK, many low-lying areas of the United States are subject to risk of river flooding, and over 10 000km of river flood banks (levees) have been constructed to control this. These levees, together with road embankments built within the river flood plain, constitute a further and very significant risk to damage by overtopping in the United States.

In relation to their length, rivers in the United States probably require more protection against erosion than those in the UK. This is because many areas of the United States are young geologically, and rivers are more active. Because of the lower density of population, considerable interest exists in low-cost methods of protection.

1.4 Recent US initiatives

A number of recent, or ongoing, US initiatives in the subject area of this report should be noted:-

- **ASCE Task Committee on Mechanics of Overflow Erosion on Embankments**

  The objectives of the committee are to document data being collected through research and through review of present practice on:

  - mechanics of erosion rates of embankment soils
  - configuration of slopes, and methods of protection required to prevent erosion during overtopping.

  (The Committee's report is expected to be published in 1988 in the ASCE Journal of Hydraulic Engineering. Membership comprises experts from Bureau of Reclamation, Denver; Soil Conservation Service, Washington; Corps of Engineers' Waterways Experiment Station, Vicksburg; Agricultural Research Station, Stillwater; Federal Highway Administration, Virginia; and leading consultants and academics.)

- **Streambank Protection Programme**

  This programme, has recently been carried out by the US Army Corps of Engineers under the Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251 and is known as the Section 32 programme. It included an evaluation of the effectiveness of existing and new methods of bank protection. The programme was completed in 1982, except for the long-term evaluation of a number of trial lengths of protection.

- **Full-scale trials on performance of unprotected embankments, and different protection systems under overtopping conditions**

  Two major programmes have been implemented through consulting engineers Simons Li & Associates using a large test flume facility in Fort Collins, Colorado by the Federal Highways Adminstration, the Bureau of Reclamation and the Department of Agriculture's Forest Service.

- **Long-term programme of research on the failure processes and design of earth and grass-lined emergency spillways**

  The Agricultural Research Service of the Department of Agriculture has carried out pioneering work for more than 40 years at its Water Conservation Structures Laboratory at Stillwater, Oklahoma to develop predictive methods for the design of low-cost spillways and grass-lined channels.

In conjuction with these recent formal initiatives, practising engineers have developed experience in recent years in the design and construction of low-cost protection using the following materials:

• Concrete-based	Concrete blocks; roller-compacted; soil-cement stabilisation
• Geotextiles	Mats; meshes; woven and non-woven fabrics
• Rock	Rip-rap; gabions
• Vegetation	Grass

1.5 Coverage of the Report

Sections 2 to 7 present data perceived to be of interest to UK practitioners and researchers from the viewpoint of the design engineer, the product manufacturer/supplier, and the researcher. The subject, contents and author of each section are given in the Contents (pages iii and iv).

It is emphasised that the report has been written against the background of current understanding and the state-of-the-art in the UK. The report seeks to avoid presenting data which is already available or to report on practise which is generally accepted in the UK. For this reason, the report is necessarily a collection of data and views, rather than a comprehensive review. It is complimentary to data presented in CIRIA Report 116 on Design of Reinforced Grass Waterways and the forthcoming CIRIA report on protection of UK river and canal banks.

CIRIA wishes to emphasise that the viewpoint presented by the report is necessarily constrained by the limits of what the mission team could practically cover in the time available. In this respect, the report may well be incomplete in some areas, and the opinions expressed are only those of the authors.

1.6 Availability of references

References which are not readily obtainable in the UK may be viewed by appointment at CIRIA's office.

US Government publications can be obtained through their UK agents:-

Microinfo Ltd	Tel: 0420-84300
PO Box 3	Fax: 0420-89889
Alton	Telex: 858431
Hants	Contact: Mr Roy Selwyn
GU34 2PG	

EROSION AND PROTECTION OF EMBANKMENTS

M E Bramley - CIRIA

2.1 Introduction

This section reviews the mechanics of erosion of embankments under headings of soil types, flow pattern and prediction of erosion. Present design recommendations and developments for grass-lined channels are discussed in the context of grassed embankments. Mechanisms of embankment failure by erosion are reviewed and the significance of soil type and localised geometry of the embankment on the failure mode is emphasised. Current practice relating to the hydraulic design of various protection systems adopted in the US is reviewed with regard to the selection of protection and design details (where appropriate this cross-refers to other sections).

2.2 Mechanics of erosion of embankments and steep waterways

Present understanding has been obtained from observations of erosion processes on dams, cofferdams, road embankments and unlined spillways subject to overtopping or extreme flows, as well as from laboratory model studies. It is necessary to understand the mechanics of erosion in order to evaluate the susceptibility of the unprotected embankment/waterway to erosion, and to design protective measures. The following description draws together information provided by the ASCE Task Committee members with recent US references by Ralston (1987), Miller and Ralston (1987), and Chen and Anderson (1986).

Failure through erosion of surface material by overtopping flow must be distinguished from failures by piping, by liquefaction or by mass sliding or slumping. These might however act in conjunction with the erosive forces created by high surface flow velocity in causing a failure. Failures due to piping and liquefaction require saturation of the soil matrix and/or a relatively permeable soil. Mass sliding or slumping also require some significant degree of soil saturation, and occurs particularly as flood flow recedes leaving the saturated slope in an unstable condition.

The mechanics of erosion of unprotected embankments need to be considered in terms of both the soil characteristics, the overall flow pattern, and the effect of local discontinuities in surface geometry.

2.2.1 Soil types

A distinction is generally drawn between the erosion mechanisms for embankments or slopes formed of cohesive soils and those of non-cohesive or granular soils. Although the initial processes of erosion are similar, the ultimate failure mechanisms and the rates of erosion differ for each type. Seepage effects can be significant in embankments of granular material, and the presence of permeable and/or impermeable zones within the embankment can radically affect the failure mechanism.

Data drawn together by the ASCE Task Committee (see Section 3, Tables 3.4 and 3.5) confirms that low, well-compacted embankments of cohesive or well-graded granular material with fines have withstood limited over-topping depths for limited periods. Conversely seepage through relatively clean rockfill in conjunction with overtopping has been observed to lead to shallow sliding.

Difficulty in quantifying the erosion of soils and the effects of the flow pattern (see below) still constrains the ability of the engineer to predict accurately the response of the embankment to overtopping. In particular, the erodibility of a cohesive soil depends on several controlling parameters which act interdependently; the following parameters are generally considered useful for evaluating erodibility:

- shear stress due to flow
- critical shear stress of the soil (i.e. threshold of motion)
- percent clay
- plasticity index (PI)
- compaction
- percent organic matter
- cation exchange capacity (CEC)

PI is frequently quoted as a prime indicator for critical shear stress of a cohesive soil.

Chen and Anderson (1986) propose the relation

$$\tau c = 0.019\ (PI)^{0.58}$$

where τc is in lb, ft units

in their proposed methodology for estimating embankment damage due to flood overstopping. (Critical shear stress for noncohesive soils are calculated on conventional sediment transport theory - e.g. Shields).

2.2.2 Flow pattern

Figure 2.1 shows the different flow regimes associated with embankment overtopping or flow down a steep waterway.

A considerable amount of US overtopping experience relates to cofferdams, highway embankments and levees where overtopping is usually associated with a high tailwater level, and a terminal velocity condition is not reached on the downstream slope which is normally the case with dam overtopping or flow down a steep channel such as a spillway.

Figure 2.1 also shows the accepted classification of flow patterns relating to differing tailwater regimes which is based on work by Kindsvater (1964):-

- free plunging flow, terminal velocity on downstream face
- free plunging flow
- free surface flow } velocity on downstream face
- submerged surface flow } restricted by tailwater level

With submerged flow, critical depth conditions do not form on the crest and discharge is controlled by tailwater conditions. With surface flow, the flow separates from the embankment surface at the downstream shoulder and "rides" over the tailwater surface. With plunging flow, the flow remains in contact with the downstream slope and plunges under the tailwater surface producing a hydraulic jump. The maximum velocity attained on the downstream slope depends on the overtopping head (or discharge intensity, q), the hydraulic roughness of the slope and the head loss across the embankment. Experience has shown that surface flows are substantially less erosive than plunging flows.

A clear distinction is drawn between flow conditions before and after the occurrence of localised surface discontinuities which signify the onset of failure due to high localised erosion.

2.2.3 Prediction of erosion

For conditions prior to the occurrence of localised surface discontinuities, methods have been developed to estimate the rate of erosion of the embankment soil.

Resulting from the full-scale trials described in Section 4.3, Chen and Anderson (1986) have developed a predictive model EMBANK for determining the rate of erosion (Figure 2.2) with bare soil conditions. This utilises the general formula for erosion rate developed by the USDA, Agricultural Research Laboratory (Temple, 1987).

$$E = K (\tau - \tau_c)^a$$

where E is the detachment rate per unit area ($ft^3/sec\text{-}ft^2$)
K and a are empirical coefficients dependent on soil properties
τ is the local effective shear stress due to the flow

The choice of values for the empirical coefficients is clearly of critical importance. Figure 2.3 shows the erosion equations deduced by Chen and Anderson as a result of their full-size tests and review of other work. Lowest erosion rates occur with highly cohesive soils. Low-cohesive soils show highest erosion rates at low net shear stress (difference between actual shear stress and critical shear stress), with noncohesive sand/gravel soils showing highest erosion rates at high net shear stress.

A review of other available methods for estimating embankment erosion rates is given in Appendix 2.1. Details of a predictive numerical model for embankment erosion being developed by USBR are given in Appendix 2.2.

Based on their embankment erosion model, Chen and Anderson have developed a series of design charts for estimating bare soil embankment damage. Typical examples are given in Figure 2.4. Further curves demonstrate the effects of a hard embankment crest, and a vegetated slope.

2.3 Grass-lined channels

The US Department of Agriculture's Agricultural Research Service has a long history of research into the performance and flow characteristics of grass-lined channels through their Water Conservation Laboratory at Stillwater, Oklahoma.

The USDA design curves giving hydraulic roughness of different types of grass as a function of discharge intensity, q, are well known in UK. These curves are incorporated into the CIRIA design procedure (Whitehead and Nickelsons, 1976; Hewlett et al, 1987) for permissible velocity in grass-lined channels as a function of flow duration. The CIRIA design approach is based on general field experience and does not take into account variation in soil type.

Recent USDA research (Temple et al, 1987) has been aimed at developing a more rational grass/soil model, thus extending the USDA permissible velocity design procedure (Cox and Palmer, 1948), for the stability design of grassed waterways. This model can be further extended to enable the extent of erosion damage or time to failure to be computed for flow in steep grassed waterways or overtopping of grassed embankments (Temple, 1987).

USDA categorise erosion failure of grass-lined channels in three ways:-

1. Local failure associated with major discontinuities in cover or boundary geometry

2. Direct physical destruction of the vegetal cover (e.g. grass plant or soil/root mat uprooting)

3. Localised soil erosion leading to removal of the vegetal cover.

Use of the extended model would only relate to failure Category 3, however this would generally precede a Category 2 failure. As with the Chen and Anderson model, it must be emphasised that the extended model relates only to the initiation of failure, and not to subsequent high localised erosion rates. USDA field and laboratory experience suggests that 60mm (2½ inches) can be taken as a realistic limit to erosion depth for signifying complete failure of the protective grass lining. This applies to a broad range of soil and grass conditions.

The model can be used to simulate both the physical processes of protection and erosion. The vegetation initially protects the soil boundary as a result of both direct protection and boundary layer effects caused by interference between the flow and the vegetal elements. The stability design method takes account both of the protection, and of the critical shear stress of the soil. In the extended model, as locally weaker and/or shallow rooted elements are removed by the flow, more of the stress is transmitted directly to the soil and erosion rates progressively increase. This increased erosion in turn results in further elements being removed, allowing flow and stress concentrations to develop which further increases the local erosion rate. Finally, all grass cover is lost over an area sufficiently large to allow formation of a head-cut. This signifies the transition to failure Category 3, i.e. a major surface discontinuity, with an associated extreme localised erosion rate.

The model equations compute local effective shear stress, τ, in terms of

$$\tau = \gamma ds \left[1 - C_{Ff}\right] (n_s/n_f)^2$$

where γds is the general unit weight/depth/slope term of shear stress
C_{Ff} is a vegetal cover factor for the failure area
n_s is Manning's coefficient of hydraulic roughness associated with the soil
n_f is Manning's coefficient for the grassed surface, or for the failure area in the case of the extended model.

In the extended model, further equations express C_{Ff} as a function of initial vegetal cover and local erosion depth, and n_f as a function of initial hydraulic roughness and localised erosion depth. A key assumption in the extended model is that flow depth is determined by the more general grass cover conditions, and not by the localised conditions in the area of developing failure. (Similar arguments relate to the calculation of flow velocity, and localised failure conditions with reinforced grass; see Hewlett et al, 1987).

The erosion rate formula adopted in the extended model has been introduced in Section 2.1.3. As with the Chen and Anderson model, the principal uncertainty relates to the values adopted for the empirical coefficients K and a. However the new USDA procedure is able to provide answers which are reasonably consistent with observed failure times. Since the type of grass cover and soil are taken into account in the model, this method provides a useful indication of the sensitivity of erosion rate to variations in these parameters.

2.4 Mechanisms of failure by erosion

2.4.1 Failure sequence for cohesive-soil embankments

The following scenario has been developed by the Soil Conservation Service and Corps of Engineers based on field and laboratory observations. Figure 2.5 refers.

After the onset of high localised erosion, a small overfall occurs and a scour hole begins downstream of the overfall. The process of enlargement of the scour hole is again determined by the soil characteristics and flow pattern. The geometry of the overfall increases the local flow concentration by "capturing" flow from the sides as well as upstream. The scour hole is deepened and maintains a vertical face at the head until a critical height is reached, whereupon the vertical face collapses. Headcutting thus occurs due to the progressive upstream advance of the overfall through the embankment. Breaching of the crest therefore generally occurs not through direct erosion, but due to loss of support due to erosion of the downstream face.

This process of highly localised erosion, with the complex 3-dimensional flow pattern in the scour hole, cannot yet be modelled or predicted with accuracy.

The following observations can be made on scour hole progression:-

- the strength of the soil in the perimeter of the overfall determines the height which the overfall can attain before head collapse
- the half-rounded shape of the overfall provides vertical support to the soil mass through horizontal arching.
- attaining critical height for head collapse depends on the removal of material from the scour hole by the flow. In this respect, submergence of the scour hole by tailwater (e.g. rising tailwater) substantially reduces the rate of erosion.
- the rate of erosion can be accelerated by the presence in the embankment of any granular elements, such as toe drains or drainage blankets, which can undermine the more resistant cohesive material.

2.4.2 Failure sequence for non-cohesive or granular embankments

Non-cohesive soils do not allow localised scour to develop a vertical face, and surface-erosion progresses more uniformly across the downstream face with a gradually flattening gradient which eventually reaches the crest. This form of erosion can be modelled using a shear stress method, however it has been observed to occur in conjunction with piping and mass sliding (see Section 2.1).

The presence of cohesive zones (e.g. a core) in the fill has been observed to retard the rate of erosion, typically by restricting further erosion of granular material until the cohesive element has been undermined and broken away by the flow.

2.4.3 Local geometry and other factors affecting erosion

Local discontinuities or changes in slope of the downstream shoulder, such as a berm, ditch or power pole, provide a place for initiation of erosion

and will accelerate breaching. Erosion has often been observed to be initiated at the junction of the crest with the downstream slope, and at the toe of the downstream slope (Chen and Anderson, 1986)

An even distribution of flow across the embankment is also less erosive than flow which becomes concentrated due to local variations in slope or low points in the crest, or most significantly narrowing geometry in the fill-abutment contact zone.

2.5 Current practice related to hydraulic design of protection systems

2.5.1 Use of new or low-cost systems

Several agencies in the US, including the Bureau of Reclamation, are now utilising low-cost protection systems to prevent erosion of embankments due to overtopping, or as lining to auxiliary spillways. Use is generally limited to embankments less than 15m (50ft) high. As in the UK, design practice relies strongly on engineering judgement. There is no standard approach to hydraulic design; current practice for the different elements of the design is summarised in this section. Note that this report generally excludes aspects of geotechnical stability, which was regarded as an area where well established practice exists.

Different materials commonly used for embankment protection are summarised in Section 1.4. There was general agreement that each design was sufficiently site-specific for no single material to be considered the universal panacea. Principal interest appeared to be in roller compacted concrete, precast concrete blocks, and grass for embankment protection and auxiliary spillways, together with three-dimensional geotextile matting for less critical installations.

As in the UK, designers appreciate the advantages of the protection being flexible and thus being able to accomodate minor movements due to settlement or to give early warning, through its deformation, of loss of underlying fill. Some designers however considered that roller compacted concrete being monolithic had a clear advantage over other materials as being less susceptible to progressive failure caused by localised deterioration or damage to protection: e.g. a local bare patch of grass cover, or a broken concrete block.

Most designers stressed the importance of giving careful attention to the design details at the crest, sides and toe of the protection - recognising that these are key locations where failure can be initiated.

2.5.2 Description of flow and selection of protection

All designers regarded the description of flow over the embankment as the initial stage of the design process. Where dam overtopping was concerned, this included consideration as to whether flow can be confined to an auxiliary spillway. In a significant number of cases, it was considered that the length of crest would be insufficient to do this. In one case (Moler, 1987), a hydraulic model study had been carried out to investigate the effects of flow concentration on the downstream face caused by the narrowing abutments.

This initial stage of design would include consideration of the potential seepage flow through the embankment. There was general agreement that protection of permeable embankments requires more careful consideration of the stability of the protection than with relatively impermeable embankments, since measures need to be taken to accomodate seepage through

flow without affecting the integrity of the protective layer. Most US experience of embankment protection relates to relatively low permeability fills. With impermeable protective material such as roller-compacted concrete, it is usual practice to lay the material on a filter/drainage layer which is able to pick up any flow through the embankment from upstream.

There was general agreement that downslope seepage flow should be discouraged, and that the protection must allow drainage of the embankment to take place following discharge. This is to avoid undue build up of excess uplift pressure on the protection.

Differences of opinion existed among different designers regarding the key hydraulic parameters which determine the type of protection. It was agreed that these differences were principally due to the different design conditions pertaining (a) to different sites, and (b) to the potential failure mode of different materials.

Key parameters were considered to be:-

- design discharge intensity at crest ($m^3/s/m$)
- overtopping head and tailwater level in relation to crest (determining submergence effects and reduction in modular discharge)
- design maximum shear stress (or tractive force) on protection
- design maximum velocity of overtopping flow
- duration of flow event

With relatively flexible protection systems (such as grass, geotextiles, rip-rap or concrete blocks) which are able to exhibit a significant local variation in geometry, the potential exists in high velocity supercritical flow for the specific energy of the flow (mainly velocity head) to be mobilised locally and to cause localised forces (form drag and uplift) giving rise to failure. This failure scenario differs from the shear failure scenario which is associated with tractive force theory. This explains CIRIA's preference for design maximum velocity as a criterion for selection of grass or reinforced grass protection.

Discussion on different types of protection follows:

Grass

Current practice generally follows USDA practice and involves (a) selection of grass type, (b) iterative calculation to determine design velocity/shear stress conditions utilising empirically-derived curves relating hydraulic roughness to discharge intensity for any given grass type, and (c) check that permissible upper limits of shear stress or velocity are not exceeded (Table 2.1). The limited erosion model described in Section 2.2 is not yet used in design practice.

Geotextiles and temporary protection

Geotextiles are discussed in detail in Section 5. Cotton and Chen (1987) quote values of permissible shear stress and hydraulic roughness for synthetic three-dimensional open mat (e.g. Enkamat) ungrassed lining along with other temporary protection (Table 2.2).

Rip-rap is discussed in detail in Section 6.

Concrete blocks are discussed in detail in Section 7

Roller compacted concrete and **soil cement stabilisation** are designed to the minimum practical thickness for placing, and no specific hydraulic design criteria are applied to determine thickness. Because the flow would be highly aerated and would occur rarely, no concern was expressed about cavitation damage. RCC is discussed further in Section 3.

Gabion mattresses

An upper limit design velocity of 20 ft/s (6.1 m/s) for a 225 mm thick mattress from has been established from full-scale tests, and the ASCE Task Committee Report will draw attention to the potential of gabions (used in conjunction with an underlying geotextile) to provide protection against overtopping.

2.5.3 Design details

(a) Crest

General practice is to take the protection well upstream of the location of critical depth control on the crest and usually upstream of the crest centreline. The ASCE Task Committee draw attention to the variation in location of the control depending on upstream head and crest geometry.

(b) Sides

With purpose-designed channels, sloping sides with an appropriate freeboard allowance are usually adopted to terminate of the lining. Major concern was expressed over the importance of effective edge details if the flow must, of necessity, be deflected or concentrated (e.g. by abutment geometry). It was agreed that deflection or concentration of flow should be avoided wherever possible.

(c) Toe and tailwater conditions

General practice is to consider the various tailwater conditions which will occur over a range of discharge from low overtopping depth through to design discharge, and to provide protection accordingly. Tailwater is perceived to reduce the risk of local damage where it either causes a submerged surface flow pattern (Section 2.1.3), or inhibits further acceleration of the flow. It was considered that the erosion potential may be increased at the front of the jump under free plunging flow, and that strong protection may be needed in this zone but no quantification of this was available.

The protection is generally continued downstream of the embankment (or slope) as an apron, the elevation of which may be determined by conjugate depth related to subcritical tailwater conditions where these exist. The need to make provision for energy dissipation or to anticipate scour downstream of the embankment or waterway was well understood. It was also recognised that these requirements can have major implications on the economics of the design, particularly with high discharge intensities and high flow velocities. In several cases, the protection has been terminated at a cut-off wall as a precaution against undercutting by scour.

(d) Abrupt changes in geometry are avoided wherever possible in regions of high velocity flow.

2.6 Future developments

The limited vegetation which can be relied upon in more arid regions of the United States means that use of reinforced grass - particularly utilising concrete blocks - is not feasible. Also, in some applications, the duration of flow or submergence may be such that vegetation would not survive. Interest therefore exists in developing non-vegetative methods of protection which are relatively inexpensive and easy to construct.

The US Federal Emergency Management Agency has identified in their report on Safety of Existing Dams (see Appendix 3.3) the potential benefits which might be provided by a novel wedge-shaped concrete block design developed in the USSR, but not utilised in the West. These blocks exhibit high stability in high velocity flow due to their shape and the related flow pattern, and also provide for good drainage of seepage flows (Novak (ed), 1983; see also Section 7.3.5).

Similar interest exists in the UK, and research into the performance of wedge-shaped blocks has been carried out at University of Southampton (Nouri, 1984). CIRIA is currently promoting a limited programme to evaluate and demonstrate the effectiveness of such blocks. This would be undertaken jointly by researchers and interested parties in the US and UK. The research pre-proposal which is currently under discussion with US agencies is included in Appendix 7.1.

Table 2.1 Maximum permissible velocities and shear stresses for vegetative l[illegible]gs

Permissible Velocities

Cover	Slope Range (%)	Permissible Velocity, ft/s: Erosion-Resistant Soils	Permissible Velocity, ft/s: Easily-Eroded Soils
Bermuda Grass	0-5	8	6
	5-10	7	5
	>10	6	4
Buffalo Grass, Kentucky Bluegrass, Smooth Brome, Blue Grama	0-5	7	5
	5-10	6	4
	>10	5	3
Grass Mixture	0-5	5	4
	5-10	4	3
	Do not use on slopes steeper than 10%.		
Lespedesza Sericea, Weeping Love Grass, Ischaemum (yellow bluestem), Kudzu, Alfalfa, Crabgrass	0-5	3.5	2.5
	Do not use on slopes steeper than 5% except for sideslopes in a combination channel.		
Annual-used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudan Grass	0.5	3.5	2.5
	Use on slopes steeper than 5% is not recommended.		

Permissible Shear Stress

Retardance Class, Vegetative Cover*	Permissible Unit Shear Stress (lb/ft^2)
Class A	3.70
Class B	2.10
Class C	1.00
Class D	0.60
Class E	0.35

*Classification of vegetal covers as defined by the USDA Soil Conservation Service

Source: ASCE Task Committee based on USDA dat

Table 2.2 Proposed maximum permissible shear stresses for "temporary" lining materials

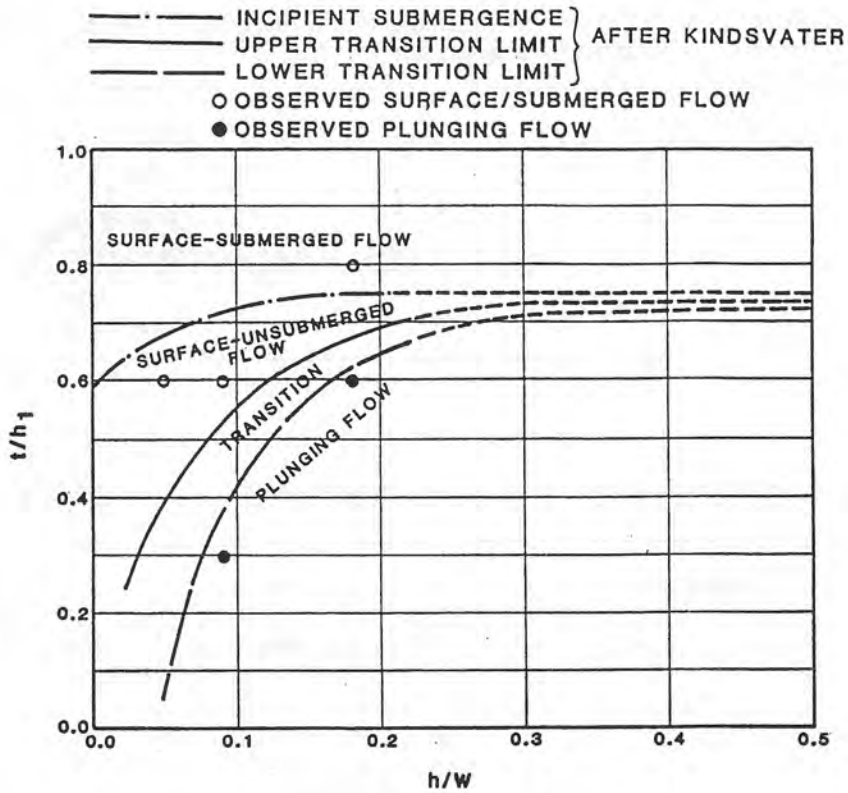
Permissible Shear Stresses for Lining Materials

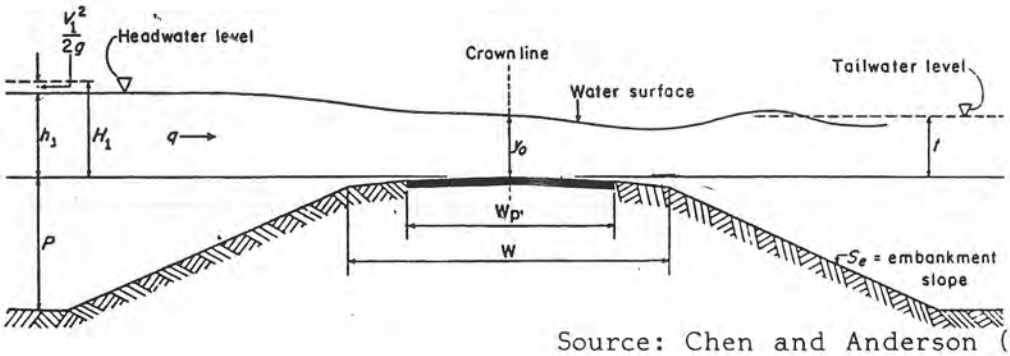
Lining Category	Lining Type	Permissible Unit Shear Stress (lb/ft^2)	(Kg/m^2)
Temporary	Woven Paper Net	0.15	0.73
	Jute Net	0.45	2.20
	Fiberglass Roving:		
	Single	0.60	2.93
	Double	0.85	4.15
	Straw with Net	1.45	7.08
	Curled Wood Mat	1.55	7.57
	Synthetic Mat	2.00	9.76

Manning's Roughness Coefficients

Lining Category	Lining Type	n - value: Depth Ranges 0-0.5 ft (0-15 cm)	0.5-2.0 ft (15-60 cm)	>2.0 ft (> 60 cm)
Temporary	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021

Source: Cotton and Chen (1987)





Source: Chen and Anderson (1986)

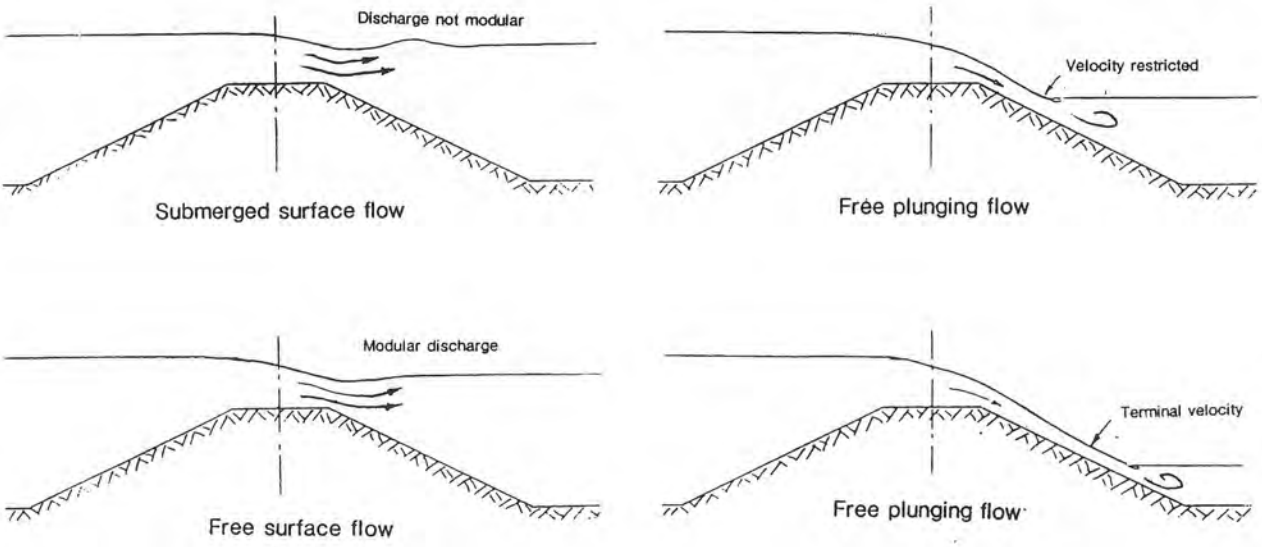
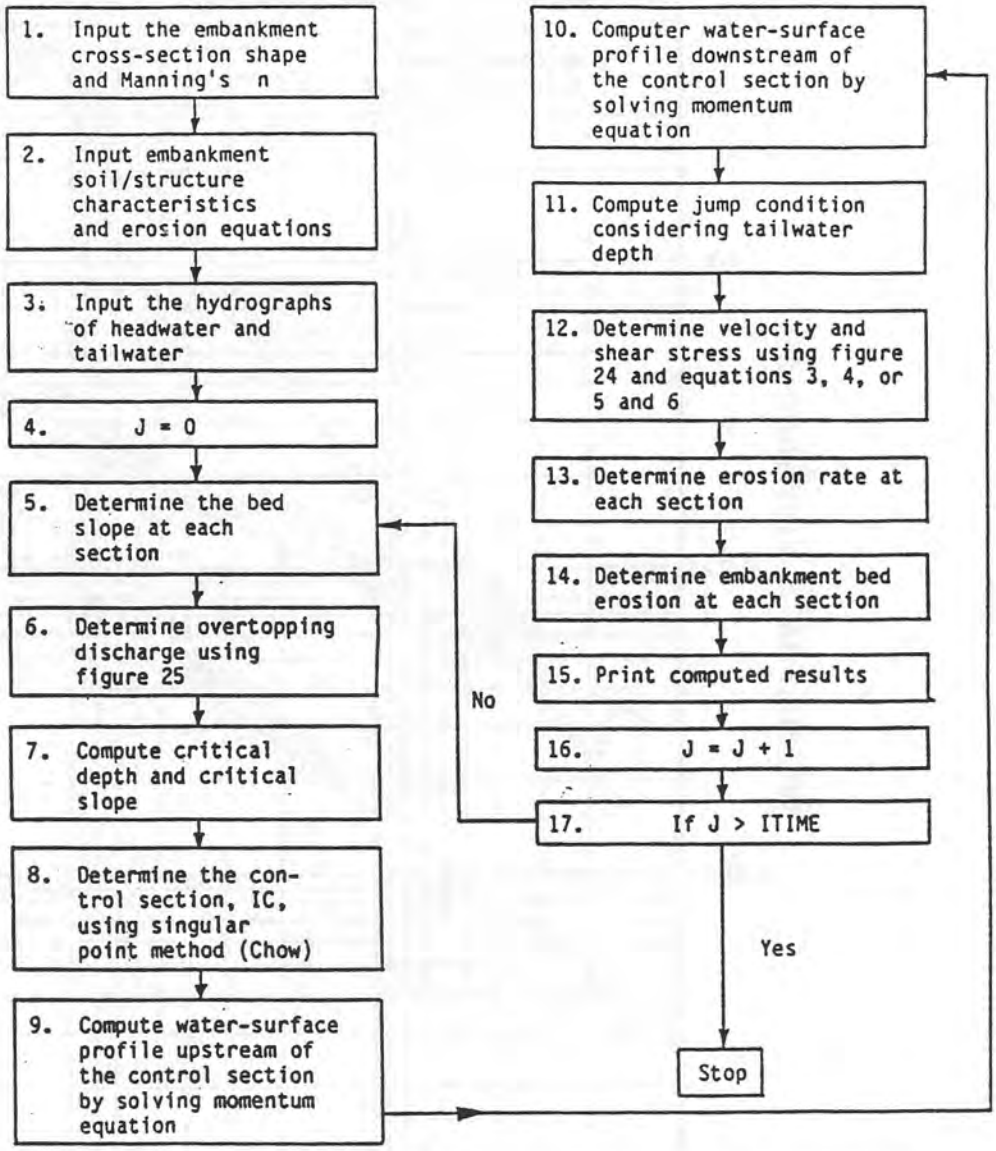
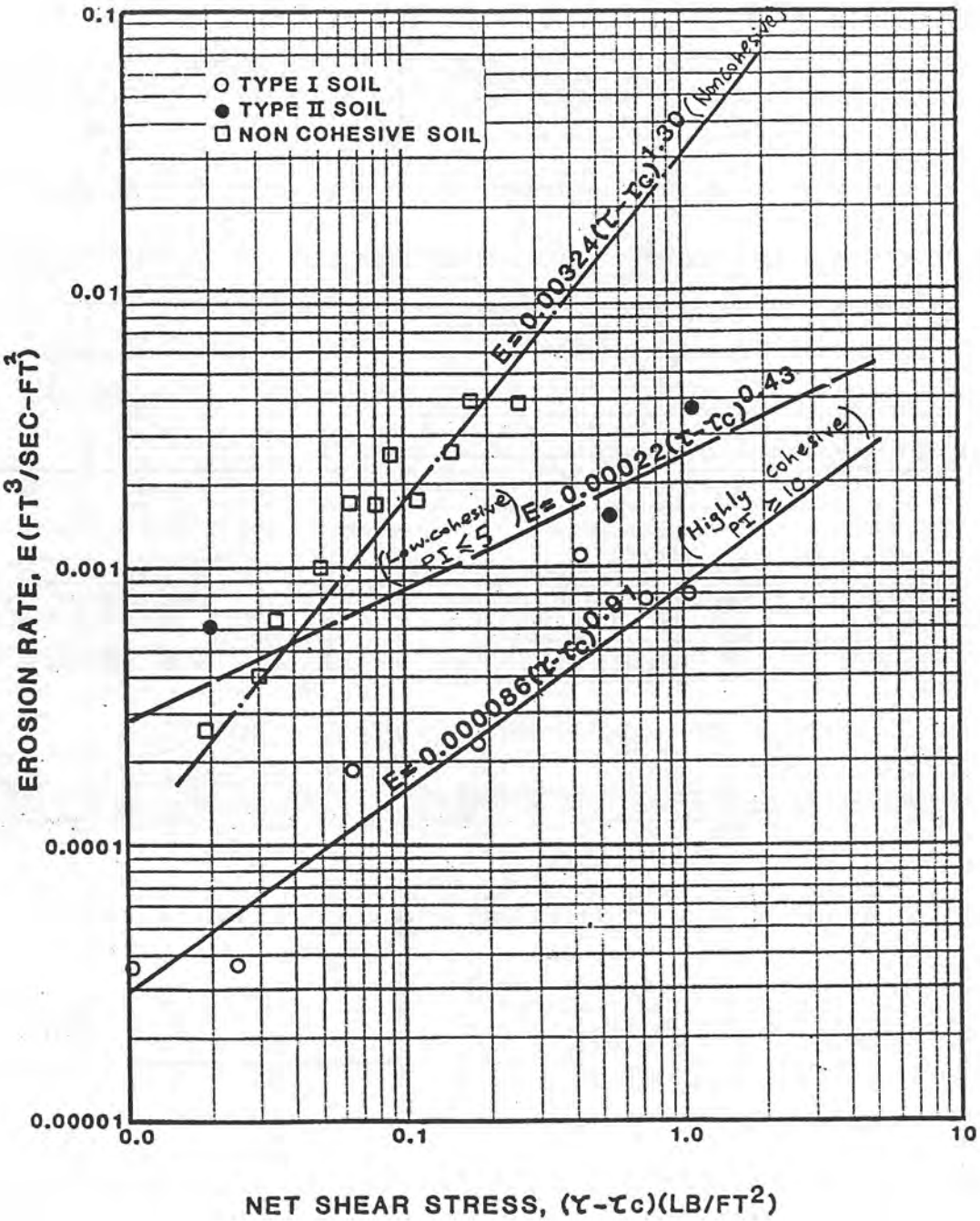


Figure 2.1 Classification of overtopping flow regimes



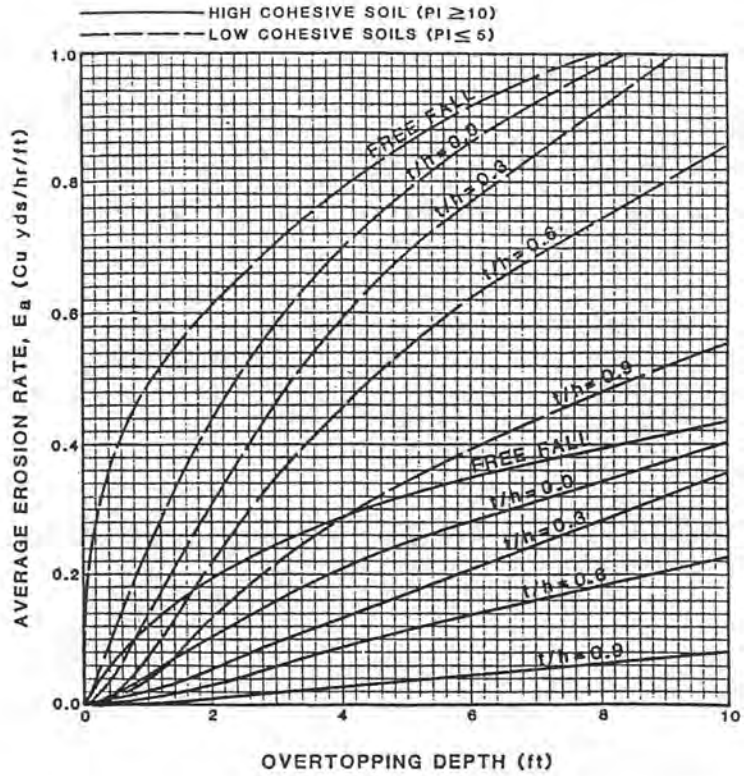
Source: Chen and Anderson (1986)

Figure 2.2 Logic diagram for EMBANK erosion model

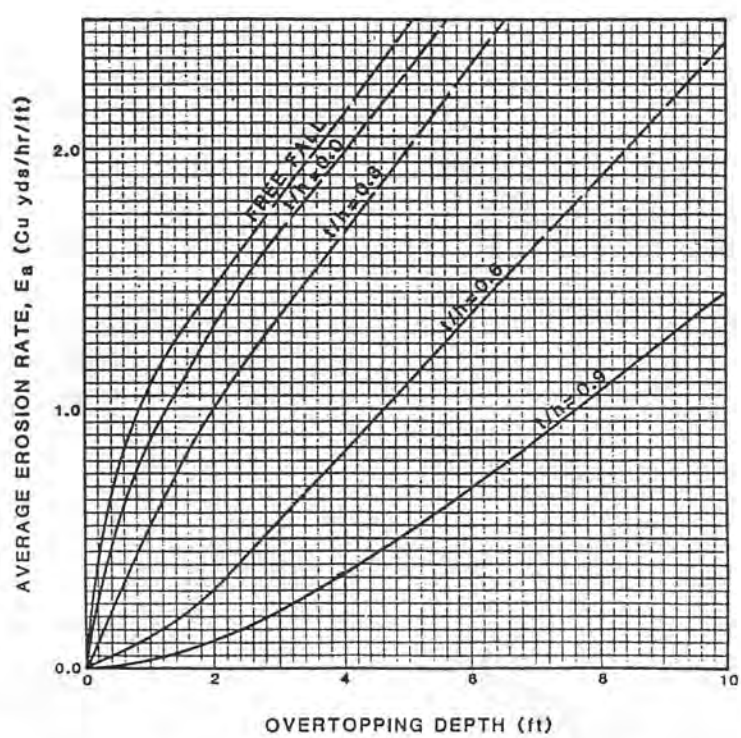


Sources: Chen and Anderson (1986), Temple (1987)

Figure 2.3 Typical equations for erosion rate, various soil types



4-hour flow; 5ft high embankment; cohesive bare soil



4-hour flow; 5ft high embankment; non-cohesive bare soil (d_{50}<8mm)

Source: Chen and Anderson (1986)

Figure 2.4 Typical graphs for average erosion rate of embankment under overtopping

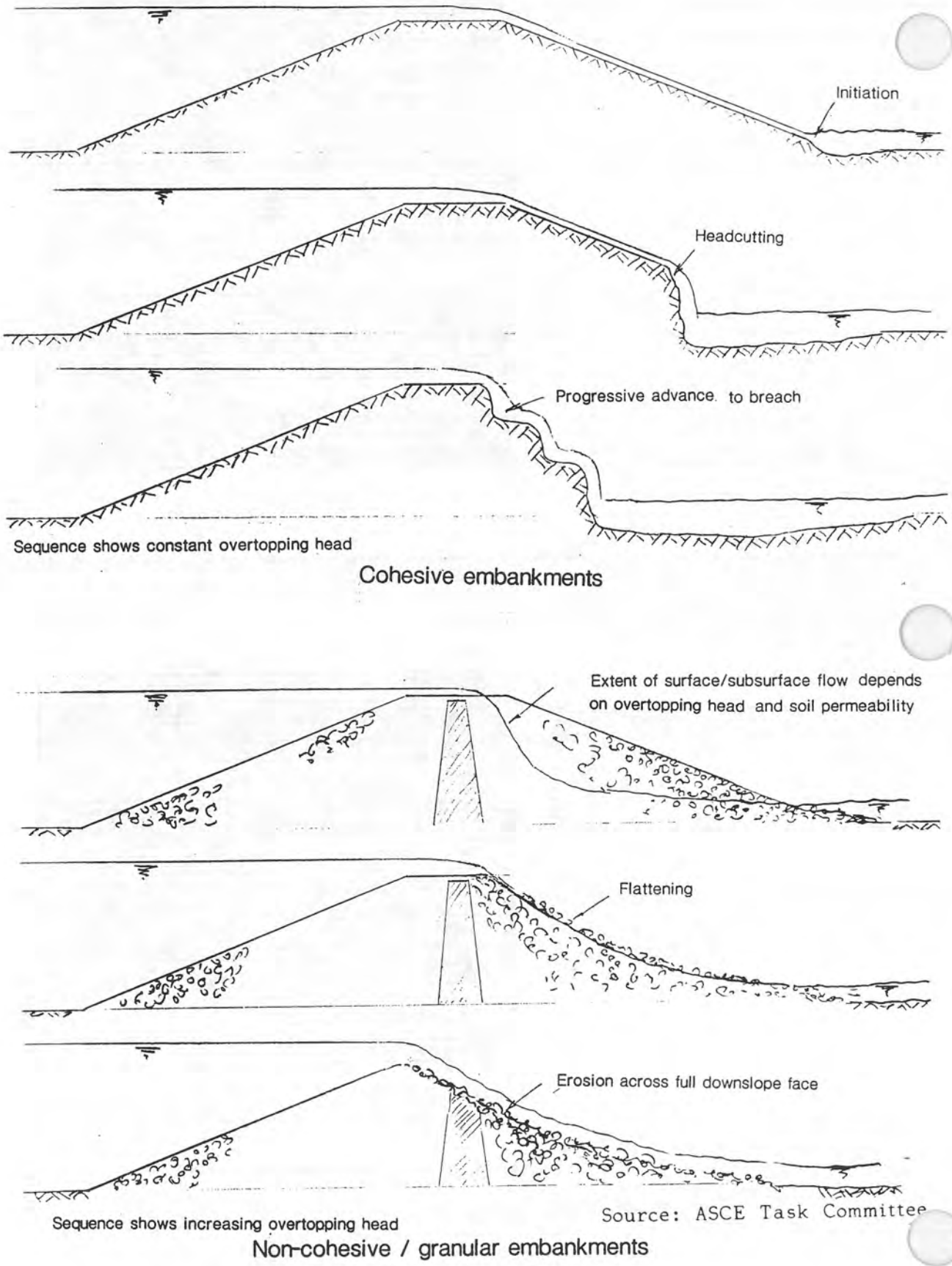


Figure 2.5 Typical mechanisms of embankment failure under overtopping

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SAFETY AND OVERTOPPING OF DAMS

M F Kennard - Rofe, Kennard & Lapworth

3.1 Introduction

This section discusses the general framework of dam safety in the US with particular reference to flood standards, overtopping of embankments, and methods of erosion protection which US engineers are currently utilising in order to permit safe overtopping under extreme flood events. (Further details of the different methods are given in other sections - see Section 2.4.2). Details of the use of roller compacted concrete as overtopping protection are given, and a recent development in the design of fuse plug spillways (which may be utilised as a part of a low-cost spillway) is discussed.

3.2 Dam Safety

3.2.1 General

There are about 80,000 "jurisdictional" dams in the USA, many of which have not been regularly inspected in the past. Each state is responsible for its own dam safety, and has a State Engineer and engineering staff responsible for inspections and ensuring compliance with State dam safety regulations. The Federal Government through USBR, Corps of Engineers, Forest Service, Tennessee Valley Authority and Soil Conservation Service, etc. are responsible for about 4000 dams, or about 5% of the total.

In 1972, the National Dam Inspection Act was enacted, and the Corps of Engineers had the responsibilities to prepare or develop:

- an inventory of dams;
- a survey of each State and Federal agency's capabilities, practices and regulations regarding the design, construction, operation and maintenance of dams;
- guidelines for inspection and evaluation of dam safety;
- recommendations for a comprehensive national programme.

A report on these activities and proposed legislation to implement a Federal dam safety programme was transmitted to Congress in November 1976 but lack of funding prevented the execution of the detailed dam inspections.

However the Teton dam failure in 1976, during the first filling, reactivated intense interest and governmental concern for dam safety. In 1977, a Presidential memorandum directed federal agencies to review their dam safety practices. An Inter-Agency Committee on Dam Safety (ICODs) was set up, and in turn sub-committees were established to cover particular topics. Federal Guidelines for Dam Safety was published in June 1979. In October 1979, a second Presidential Memorandum directed Federal Agencies to adopt and implement the Federal Guidelines.

In 1981, the Corps of Engineers completed inspections of nearly 9000 high and medium hazard non-Federal dams. Those inspected were those located where loss of life and major property damage would likely occur in the event of dam failure. Results of the inspection indicated that about one-third of the dams inspected were unsafe. Wherever a dam was designated as unsafe, the Corps of Engineers generally recommended the immediate preparation of an emergency action plan. As indicated in the Federal Guidelines, sound integrity of the dam is not a basis to avoid preparation of emergency plans.

The Federal Emergency Management Agency (FEMA) is responsible for coordinating dam safety and for encouraging States to implement programmes. Guidelines on selected topics are prepared by sub-committees of ICODS and published by FEMA. These guidelines include:

- Guidelines for selecting and accommodating inflow design floods for dams
- Guidelines for earthquake analyses and design of dams
- Emergency Action Planning guidelines for dams

ICODS is chaired by FEMA.

In 1970, the US Committee of ICOLD (USCOLD) produced and submitted to the governors and legislatures of all fifty states, a model law for State supervision of safety of dams and reservoirs, and this model still remains the ideal being promoted in FEMA. The model covers design, construction, operation, alteration, repair, enlargement and removal of dams.

An Association of State Dam Safety Officials (ASDSO) has been established to promote liaison and exchange of information between States. Activities include annual conferences. The theme of the 1987 conference in September was "Dam Safety in the States".

The Federal Energy Regulatory Commission (FERC) exercises nominal control over power dams as any dam with a power plant has to be licensed by FERC.

3.2.2 US Bureau of Reclamation

From the founding of USBR in 1902 until the 1960's, dam safety was dealt with a part of the Bureau's normal operations without any separate personnel or section. In the 1960's, the subject became more formalised with a Review of Maintenance Programme, which included dam safety. In the late 1970's after recent dam failures, including Teton, a high level Dam Safety Office under an Assistant Commissioner was established. This has now become a Division of Dam Safety, whose present Chief is James R Graham. This division receives all instrumentation and performance data relating to the Bureau's dams (about 300).

USBR has total responsibility for all Bureau dams most of which are large, strategic structures. It also has technical input to some other Agency dams, in its areas such as for the National Parks. It sometimes engages the assistance of specialists and boards of experts. In emergencies, after a call from the Region, it will set up a team from the Design and Dam Safety Divisions to deal with the emergency, with responsibility to the Regional Director.

The Dam Safety Division has responsibility for all dam safety inspections of its own dams and those of other Agencies for which it assumes responsibility. This involves about 100 inspections each year. If there are known deficiencies at a dam, inspections take place annually. If there are no problems, then they are inspected at 6 year intervals with a Review of Maintenance in between at 3 year intervals.

Some dams have local personnel involved who make periodic visits, whilst others do not.

3.2.3 State of Colorado

There are about 2600 dams in Colorado and the State Division of Water Resources is responsible for the inspection of all non-federal dams in the state. Since 1889, the State Engineer in Colorado has had responsibility for safety of dams. He has power to determine the safe water level of a reservoir, and in the event of work being required after an inspection he can lower the permitted water level and thus give him leverage to get the owner to carry out improvement work.

All dams over 10 ft high and impounding more than 100 acre-ft (27 million gallons; 123 000 m^3) have to have the designs approved. Any registered civil engineer can design a dam.

The division has 16 engineers to carry out inspections and a team for review of design.

Of the 2600 dams in the State, 238 are high hazard, 350 moderate hazard and over 2000 low hazard. All high and moderate hazard dams are inspected annually. Low hazard dams are inspected at 5 year intervals. For Federal dams, the State participates in the inspections with the Federal agency.

The dam safety programme, including the inspections, is funded by the State. They do however make a nominal charge to the owner for safety inspection of up to $125 per day, and for review of designs of $2 per $1000 of construction cost up to a maximum of $200. There is a time limit of 6 months on the review period for new design. The law provides for disagreement between the owner and the State Engineer to be settled in the courts, but no such cases are known.

A statistical analysis of embankment dam failures in Colorado has been published (Samed et al, 1987). 78 failures have been recorded of which 59 occurred between 1799 and 1950. Conclusions include that the rate of failure has decreased with time, and that the most common cause of failure (pre 1950) was overtopping, whilst from 1950 to 1985 seepage and/or piping along with slope failures were the most common causes.

3.3 Flood standards

In 1982, the Corps of Engineers issued "Recommended Guidelines for Safety Inspections of Dams" which contained guidelines for determining spillway capacity requirements for low, intermediate, and high dams with low, significant and high hazard classifications. Similar guidelines have been used by USBR and the Soil Conservation Service. State and local agencies, private companies and engineering design firms have generally adopted the guidelines of one of the federal agencies in determining spillway capacity requirements, although in many cases, particularly for low and intermediate height dams, no specific guidelines were followed.

The Corps of Engineers guidelines relating to Dam Size Classification, Hazard Potential Classification and Recommended Spillway Design Floods are given in Tables 3.1, 3.2 and 3.3 respectively (CoE, 1982).

A further task committee has been established under the American Society of Civil Engineers (ASCE) to suggest general standards for the hydrologic safety design of dams. Their final report is expected to be published in early 1988; the draft report is currently under peer review, (Newton 1987).

The ASCE committee report recommends procedures for selecting the safety design flood for a dam assuming that flood and dam failure probabilities have been defined. The report reviews the evolution of design flood selection procedures, recommends selection procedures, and discusses a number of technical issues including probable maximum determinations, defining flood probabilities of extreme events, dambreak flood routing, future projections, evaluating the economic and social consequences of failure, risk analysis, legal liability for dam failures, and computing indemnification costs. The recommended procedures utilize a risk-based analysis which compares the likelihood of dam failure to the social, economic and environmental consequences directly attributable to dam failure. The financial analysis proposed for incorporating into the design decision those failure consequences which can be quantified in monetary terms is based upon the estimated cost to fully indemnify possible victims for their financial loss. A separate assessment is used to include into the decision the social and environmental impacts not measurable in monetary terms.

3.4 Overtopping of embankments

In parallel with the inspection and reappraisal of the safety of many existing dams which has been undertaken in the US in recent years, interest of US engineers has also been directed towards obtaining better information on the performance of dams, flood embankments, road embankments and cofferdams which have been subject to overtopping.

Miller et al (1985), Miller and Ralston (1987), Chen and Anderson (1986), Powledge and Sveam (1987) describe the work done under different Federal organisations (Corps of Engineers, Soil Conservation Service, Federal Highways Administration, and USBR) in studying case histories of overtopping in order to understand the mechanics and causes of erosion damage, and developing methods to prevent erosion and failure.

As explained in Section 1.4 of this Report, these data are being drawn together by the ASCE Task Committee on Mechanics of Overtopping Erosion on Embankments. Tables 3.4 and 3.5 present selected data on overtopping events from the draft ASCE Task Committee report. The Task Committee's work confirms that many low embankments constructed of cohesive or well-graded granular material have withstood limited overtopping depths for limited periods, particularly when the crest and downsteam face is protected by vegetation.

A unique storm event occurred in Michegan in September 1986 when about 250mm rainfall was experienced in less than 48 hours. 20 dams were overtopped and 11 failed. The Corps of Engineers investigated these following the event, and details are given in Appendix 3.1. A video was obtained of one of the dams being breached. The conclusions of the CoE investigation was that unless specifically protected, most dams will fail if overtopped for long enough, particularly where surface discontinuities occur in the crest or downstream face. None of the dams was designed to accept overtopping, or was equipped with an auxiliary spillway.

Conclusions on the performance and mechanics of erosion of embankments subject to overtopping are reviewed in Section 2.1 of this Report.

Alternative methods of protection for the crest and downstream face of the embankment currently under consideration and/or being used in the US are summarised in Section 2.4.2 of this Report. Developments in the use of roller compacted concrete as a protection method are discussed overleaf.

3.4.1 Roller compacted concrete protection (RCC)

The use of RCC for dams has made considerable advances in recent years. The USBR have found RCC to be competitive for gravity dams and are just completing the Upper Stillwater Dam which is 290 ft high and contains 1.07 million yd^3 of RCC out of a total of 1.6 yd^3. In 1987, 708 000 yd^3 of RCC has been placed and in July construction was proceeding at a rate of 2 ft/day. A slip form paver has been used for face comcrete. The CIRIA development work (Dunstan, 1981) on use of slip form paver and high paste PFA concrete has been proved and developed extensively. Very detailed concrete testing has been undertaken (Crow et al, 1984).

An article by Hansen (1987) lists over 20 RCC dams over 20m high that have been built or are under construction in eight countries since 1980. Upper Stillwater is the largest RCC gravity dam so far built. Design methods adopted by USBR allow for tensile strength of the concrete, thus permitting steeper downstream slopes than previous practice.

At USBR there is some concern for the weathering properties of RCC on the exposed faces, if placed against shuttering or left unformed. Vibrated slip formed concrete is considered suitable, although further work is being done on air entrainment, but less well compacted outer concrete is not as satisfactory.

Use of RCC as downslope protection on embankments

Alongside the use of roller compacted concrete for new dams, considerable use is being made of RCC for auxiliary spillways and rehabilitation of old dams (Hansen and France, 1986).

A number of examples now exist of RCC being used for auxiliary spillways on embankment dams, and others are known to be under consideration. At the time of the visit to USBR, a hydraulic model test had been commenced of a RCC spillway for McClure Dam, near Santa Fe in New Mexico. The downstream face at a 2:1 slope of the 33m high embankment dam was being tested. The model test was concerned with investigating the dissipation of energy down the stepped slope and the stilling arrangements required at the toe. Recent tests on RCC gravity dams with stepped spillways showed that up to 80% of the energy was dissipated by flow over the steps, and that stilling arrangements could be quite modest for spillway heights up to 200 ft (61m) and discharge intensities up to 129 ft^2/s (11 $m^3/s/m$) (Housten, 1987).

The initial tests for the McClure Dam were with 300mm high steps, and 600mm steps and 1.2m high steps were to be tested. It was stated that the 1.2m steps were proposed to be constructed without formwork. Deterioration of the downstream slope due to weathering was not being considered by USBR as they were not responsible for the design.

Examples of this type of auxiliary spillway include:

Spring Creek Dam, Colorado

This 15m high embankment dam has been faced on the 2.5:1 downstream side with RCC placed in steps. The original spillway could pass only 26 m^3/s before the dam would overtop. The routed PMF is calculated to be 571 m^3/s. With the modified dam, the upstream head for the auxiliary spillway section would be about 2.5m and the discharge intensity about 7.5 $m^3/s/m$. The discharge is contained within RCC walls, and therefore the flow does not reach the mitres. The maximum duration of overflow was

computed to be only 5 hours. The design and construction is described by Moler (1987). The dam is shown in Figure 3.1 before, during and after modification.

Kerville Ponding Dam, Texas

The erosion resistance and overtopping potential was proved when the 6m high dam with a 2:1 RCC stepped downstream face (2m horizontal width) was overtopped by 4.4m for 4 to 5 days only 30 days after completion of construction. The dam had originally been built in 1980 as a concrete-capped clay embankment that could withstand flood overflow. A 61m length of the crest was depressed 0.3m to act a service spillway. In 1984, the dam had been overtopped by 3m and the embankment and concrete cap had been damaged. Rehabilitation was effected by removing the downstream section, rebuilding the shoulder and facing with RCC.

In the literature there are several other examples of RCC faced auxiliary spillways (France and Whiteside, 1987). Typical examples are shown in Figures 3.2 and 3.3.

Designs of auxiliary spillways on the downstream slopes have normally consisted of 225 to 300mm thick horizontal lifts, placed in a stairstep fashion. USBR consider the method to be cheaper and quicker, and as reliable as more traditional rehabilitation approaches.

This type of auxiliary spillway is obviously becoming an accepted and popular method in USA. It has good erosion resistance, as would be expected, and has been demonstrated as satisfactory in the Simons Li & Associates tests as described in Section 4.4.

In a study by Willing & Partners Pty Ltd, Consulting Engineers, Australia of additional spillway capacity for a large flood storage pond at Tongabbie Creek Retarding Basin 3, the relative merits of a tied precast concrete block revetment, precast cellular concrete Seabee units, rip-rap and RCC were evaluated. Information for the study was provided both by the USBR and CIRIA. The hydraulic design criteria were a discharge of 300 m^3/s with discharge intensity 4.0 $m^3/s/m$ and a peak velocity of 8.0 m/s.

Their recommendation was for a RCC facing, and their cost comparison showed this option to be the cheapest. The consultant's report on this study includes a table of the advantages and disadvantages of the four systems considered and this is reproduced in Table 3.6. As the Seabee unit has only recently been introduced to UK, details of this system is also given in the Appendix 3.2.

3.4.2 Fuse plug spillways

Interesting information was obtained on recent R&D related to Fuse Plug Spillways, which are defined in the book "Safety of Existing Dams" as:

> "A form of auxiliary or emergency spillway designed to be overtopped and eroded away during a very rare and exceptionally large flood".

Many have been built throughout the world, but the USBR (Pugh and Gray, 1984) state that there has not been a documented case of a fuse plug-controlled spillway actually operating, and that most of the information in the literature is associated with studies conducted in 1959 to design a fuse plug-controlled spillway for the Oxbow Project on the Snake River in Idaho and Oregon. Since this reference, a fuse plug spillway washed out in Swaziland in 1984, but as it occurred quickly during night-time, its performance was not observed (Engels and Sheerman-Chase, 1985). It is known

that a fuse plug spillway in Indonesia did not wash out due to the protective effect of vegetation which had been allowed to establish itself.

As dam safety concerns have renewed interest in the use of fuse plugs as an economical alternative to installing gates, or other designs, for auxiliary spillways, the USBR decided to study in a physical model the washout process and the rate of lateral erosion as the fuse plug erodes. The results were published by Pugh and Gray (1984) and Pugh (1985). The model was designed to simulate typical prototype fuse plugs from 3 to 9m high. The model embankments were from 0.15 to 0.38m high and 2.7m long at scales of 1:10 and 1:25. The model was 8m wide. A maximum model flow of 0.61 m^3/s was used.

A typical test used the following procedure:

1. With the adjustable control weir at a low level, the valves controlling the inlet flow were opened; the entire flow of 21.5 ft^3/s (0.61 m^3/s) entered the headbox, passed over the control weir and through the measurement weir

2. The test was started by raising the reservoir water surface with the control weir to a predetermined level where water began flowing through the pilot channel

3. As the fuse plug embankment washed away, more water passed through the breach. The water surface was kept at a constant level by gradually raising the control weir

4. The flow through the measurement weir, the level of the water surface in the reservoir and the time were recorded continually. Each test was videotaped and photographed

5. Flows through the breach were computed by subtracting the measurement weir readings from the total (initial) flow.

The model fuse plug embankment was designed with similar zones to these found in most zoned embankment dams. The arrangement of the zones, and the failure mode are shown in Figure 3.6 and 3.7. The main difference between a fuse plug and a typical dam is the arrangement of the impervious core. The fuse plug core is inclined so that when the downstream material is washed away, pieces of the core break off from bending under its own weight and under the water load. A sand filter prevents piping that may develop through cracks in the core, and the filter should be more permeable than the downstream shoulder fill which is normally compacted sand and gravel designed to be non-cohesive and highly erodible once the washout process begins.

The method to initiate breaching by the reservoir water preferred by USBR is a pilot channel. By placing highly erodible materials in the pilot channel section, breaching will occur rapidly, and the remainder of the fuse plug embankment will wash out laterally at a constant, predictable rate without overtopping of the fuse plug embankment.

The series of tests enabled the results to be compared with the Oxbow Dam fuse plug which was the subject of a ½ scale field test in the 1950's (Tinney and Hau, 1961). Good agreement with the previous results were obtained.

The conclusions of the USBR series of tests were summarised by Pugh as:

- A properly designed fuse plug embankment will wash out in an orderly and predictable manner, and can be used when additional flow capacity is needed to pass a large flood through a reservoir. The fuse plug will preclude the use of the auxiliary spillway during small floods

- The lateral erosion rate (after the initial breach) is primarily a function of the erosion rate of the embankment material and not a function of the strength of the impermeable core

- The erosion rates and discharge coefficients determined in this study can be used in flood-routing computer programs to help design fuse plug embankments

- Ratios of depth of water to embankment height, and depth of water to weir width have significant effects on erosion rates

- The sand filter, embankment material, and their gradation have significant effects on erosion rates

- A model design method is described that compensates for the fact that the Reynolds number is normally too low to properly simulate sediment transport in a Froude scale hydraulic model. This method uses settling velocity adjustments and dimensionless unit sediment discharges to adjust the model grain sizes and/or the model sediment density.

The USBR references contain a full discussion of the methods and techniques with results, charts of lateral erosion rates, descriptions and photographs.

A video has been obtained showing the erosion process of the model fuse plug embankment.

3.5 Miscellaneous

Details of two widely-circulated books in the US on dam safety, which were prepared by committees representing a number of federal agencies, are given in Appendix 3.3.

While in the US, the writer had the misfortune to be caught up in the aftermath of an extreme storm event. Details of the event, in which 250mm of rain fell in 10 hours, are given in Appendix 3.4.

Table 3.1 Dam Size Classification

Category	Impoundment Storage (acre-feet)	Impoundment Height (feet)
Small	< 1,000 and ≥ 50	< 40 and ≥ 25
Intermediate	≥ 1,000 and < 50,000	≥ 40 and < 100
Large	≥ 50,000	≥ 100

Table 3.2 Hazard Potential Classification

Category	Urban Development	Economic Loss
Low	No permanent structure for human habitation	Minimal (undeveloped to occasional structures or agriculture)
Significant	No urban development and no more than a small number of habitable structures[a]	Appreciable (notable agriculture, industry, or structures)
High	Urban development with more than a small number of habitable structures[a]	Excessive (extensive community, industry, or agriculture)

[a]Because this definition does not cite a specific number of lives that could be lost, difficulty was experienced in determining whether dams should be categorized as having "significant or high hazard potential." The issue was clarified by emphasizing that the hazard potential classification should be based on the density of downstream development containing habitable structures. For example, dams located upstream of isolated farmhouses would be classified as having significant hazard potential, and those located upstream of several houses or a residential development would be classified as having high hazard potential.

Table 3.3 Recommended Spillway Design Floods

Hazard	Size	Spillway Design Flood (SDF)[a]
Low	Small	50- to 100-year frequency
	Intermediate	100-year to 1/2 PMF
	Large	1/2 PMF to PMF
Significant	Small	100-year to 1/2 PMF
	Intermediate	1/2 PMF to PMF
	Large	PMF
High	Small	1/2 PMF to PMF
	Intermediate	PMF
	Large	PMF

[a]The recommended design floods in this column represent the magnitude of the spillway design flood (SDF), which is intended to represent the largest flood that need be considered in the evaluation of a given project, regardless of whether a spillway is provided; i.e., a given project should be capable of safely passing the appropriate SDF. Where a range of SDF is indicated, the magnitude that most closely relates to the involved risk should be selected.

100-year = *100-year exceedance interval.* The flood magnitude expected to be exceeded, on the average, of once in 100 years. It may also be expressed as an exceedance frequency with a 1% chance of being exceeded in any given year.

PMF = *probable maximum flood.* The flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The PMF is derived from the probable maximum precipitation (PMP), which information is generally available from the National Weather Service, NOAA. Most federal agencies apply reduction factors to the PMP when appropriate. Reductions may be applied because rainfall isohyetals are unlikely to conform to the exact shape of the drainage basin. In some cases local topography will cause changes from the generalized PMP values; therefore, it may be advisable to contact federal construction agencies to obtain the prevailing practice in specific areas.

Source: Corps of Engineers (1982)

Table 3.4 Selected overtopping events at dam and levee embankments

Embankment	Approximate Height, ft.	Material	Max. O. T. Depth, ft.	Duration, hrs	Extent of Damage(5)
R. D. Bailey cofferdam(3)	60	well graded sand and sandstone	6.5	4(1)	B
Bloomington cofferdam	30	clayey sand gravel	5	10(1)	B
Clarence Cannon cofferdam(4)	45	clay	5	20(1)	B
B. Evertt Jordon cofferdam	30	silty-clay	1	>24	0
Little Blue River Levee	15	clay (vegetated)	1	12(2)	0/B
Jacksonport Levee	15	clay	1-2	1 week	0
Rocky Run Dam	20	loess	1.5	5	MD
Rocky Run Protection Levee	9	loess	2	6	MD
Elm Fork	36	clay (vegetated)	0.4	3	MD
Colorado Retarding Structures(4) M-1	38	SC-CL (vegetated)	1.3	2-4	0
S-1	23		0.75	2-4	0
W-1	49		3	1-2	MD
Upper Elk River Nos. 37, 41, 42 and Big Caney No. 39(4) (4 dams)	39-47	clay and gravel	0.3-2.5	1-4	0
Rainbow	47	clay	2	12[1]	B
W. Fork Pt. Remove Cr. Site 2	35	lean clay, gravel, and silty sand	1.2	<2	0
W. Fork Pt. Remove Cr. Site 4	50	lean clay gravel, and silty sand	1	<2	0
East Willow Cr. W/S, Str. D-2	28	lean clay	1.5-2.0	1.0+	0
Bogue Cr., Dam No. Y-30-57	18	lean clay	0.6	<1	0
Deep Cr. W/S, Dam 30A	39	lean clay, silt, silty sand and sandy clay	0-0.6	<1	MD
Randall RC&D CA-191-BH	22	lean clay	1.0	<12	0
Klinkner (Westfork Kickappo W)	28	silt-lean clay	2.1	2	0
Oros [2, 3]	100	zoned earth and rockfill	2.7	12-18(1)	B
Armando de Salles [2, 3]	110	earth fill	4	0.3(1)	B
Euclides de Cumba [2, 3]	200	earth fill	4	7.5(1)	B
McCarty	54	sandy clay	unknown	"few"(1) <24	B

1 Elapsed overtopping time until breaching

2 During 1977 overtopping, no breaching; during 1982 overtopping, breaching after 12 hrs.

3 Breaching would have occurred much more quickly if the crest width had not been approximately 400 feet

4 High tailwater greatly increased the time to breach for a severe event

5 Damage Code: B - Breached
MD - Moderate Damage
0 - Minor or No Damage

Source: ASCE Task Committee

Table 3.5 Selected overtopping events at highway embankments

	Roadway	Embankment Height (ft)	Embankment Material	Peak Overtopping Conditions Maximum Depth (ft)	Peak Overtopping Conditions Duration (hours)	Extent of Damage
1.	Castor River at Zalma, State Highway 51, Bollinger County, MO	4	Sandy, Low-Cohesive	3.0	26	0
2.	Little Black River near Grandin County Highway K, Ripley County, MO	10	Sandy-Clay	3.6	9	0
3.	Illinois Bayou near Scottsville, AR, at Arkansas State Highway 164	10	Sandy-Silt, Noncohesive	4.0	12	0
4.	Earth Road in Granite Reservoir, WY	4	d_{50}=2.7 mm Noncohesive	1.0	10	B
5.	Wyoming State Highway 487 at Sand Creek near Shirley Basin	10	d_{50}=0.4 mm PI = 10	2.3	42	B
6.	Taft Hill Road at Cache la Poudre River in Fort Collins, CO	8	Sandy, Low-Cohesive	0.5	30	MD
7.	Gila River at U.S. Highway 70 (Bylas Bridge)	6	Sandy-Silt d_{50}=0.4 mm	3.4	38	B
8.	Gila River at State Highway 87 near Sacaton, AZ (milepost 148.0)	5	Sandy d_{50}=0.50 mm Low-Cohesive	3.1	60	B
9.	Peak Canyon at Interstate Highway 19 near Nogales, AZ (milepost 14)	5	Sandy d_{50}=0.30 mm Low-Cohesive	1.8	--	MD
10.	Prairie Ave., Cheyenne, WY	5	Sandy clay d_{50}=0.9 mm, PI = 4.3	2.5	3	MD
11.	Windmill Road, Cheyenne, WY	5	Sandy silt d_{50}=1.0 mm	3.0	3	B

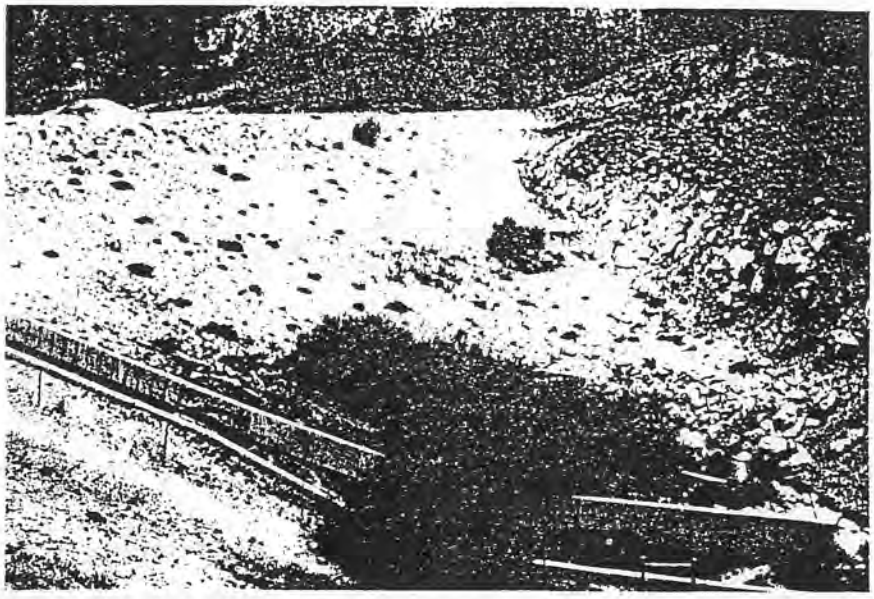
Damage Code: B - Breached
MD - Moderate Damage
0 - Minor or No Damage

Source: ASCE Task Committee

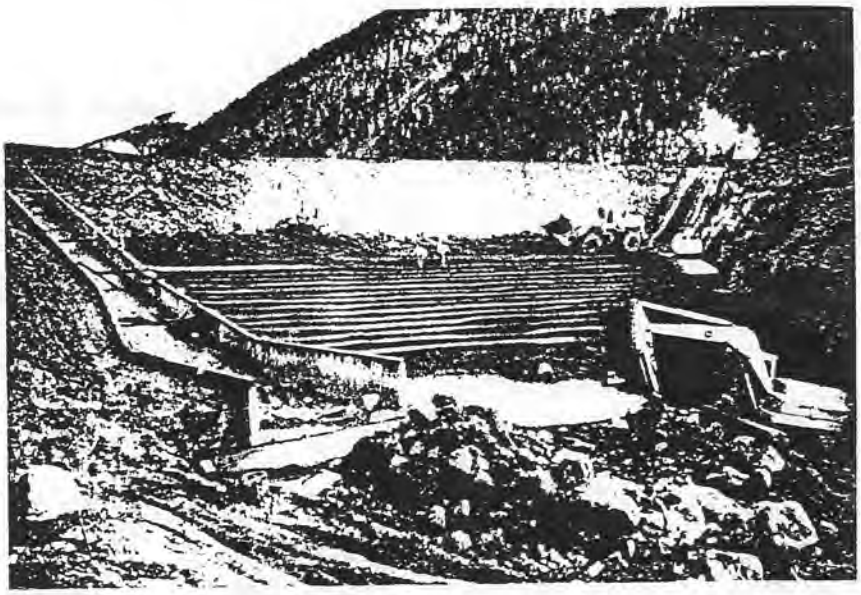
Table 3.6 Advantages and disadvantages of embankment protection systems

Systems	Advantages	Disadvantages
Flexmat (concrete block mat)	• Size of Flexmats allow swift installation and coverage • Flexmats are able to follow crest curvature • Concrete blocks are tied to geotextile fabric • Drain layer is not required	• No previous use on overtopping embankments • No model test to verify performance under high velocitie flows • Flexmats not currently manufactured in N.S.W. • Difficult to tie flexmats into exposed rock face
Seabee units	• Vitrified clay resistant to weathering • Seabee units are manufactured locally thereby reducing delivery costs • Drain layer is not required	• No previous use of Seabee units on overtopping embankments • Interlock of Seabee units is reduced on curved crest transition • Susceptible to vandalism • No model test to verify performance under high velocity flows • Units are individually hand placed • Armour layer is in jeopardy if a single unit is plucked • Difficult to tie seabees into exposed rock face
Rip-rap	• Rock armour is a well-proven protection measure • Rock armour can not be vandalised	• Size of rock armour will creat installation difficulties • Difficult to tie rock armour into exposed rock face • Difficult to armour abutments • Depth of protection layer is a significant proportion of total embankment
Roller-compacted concrete	• RCC is a proven embankment protection system • Abutment treatment is a simple extension of face protection • Simple to tie RCC into exposed rock surface	• Experienced supervision is required • Care is needed to ensure the surface of the RCC is clean of dirt, mud and debris throughout RCC placement • Quality control is important

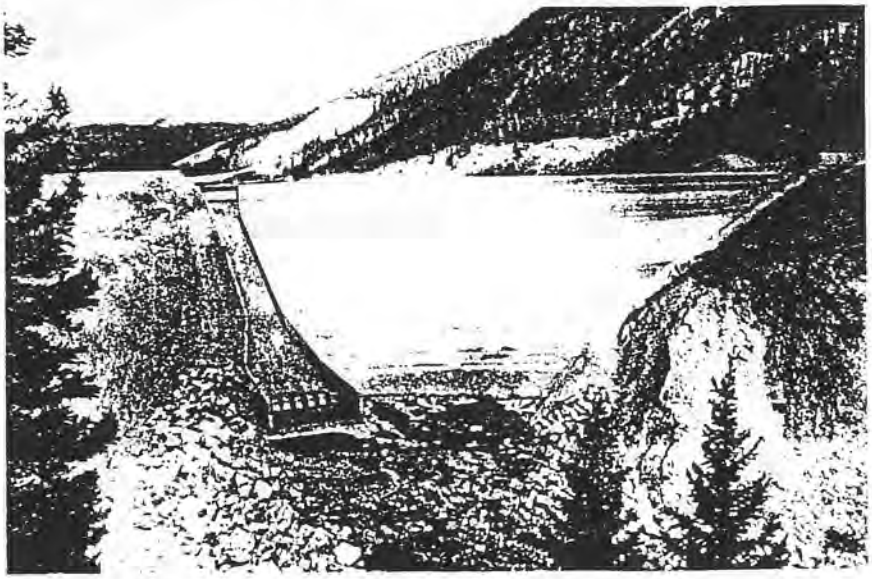
Source: Willing & Partners Pty Ltd (1987)



Before facing



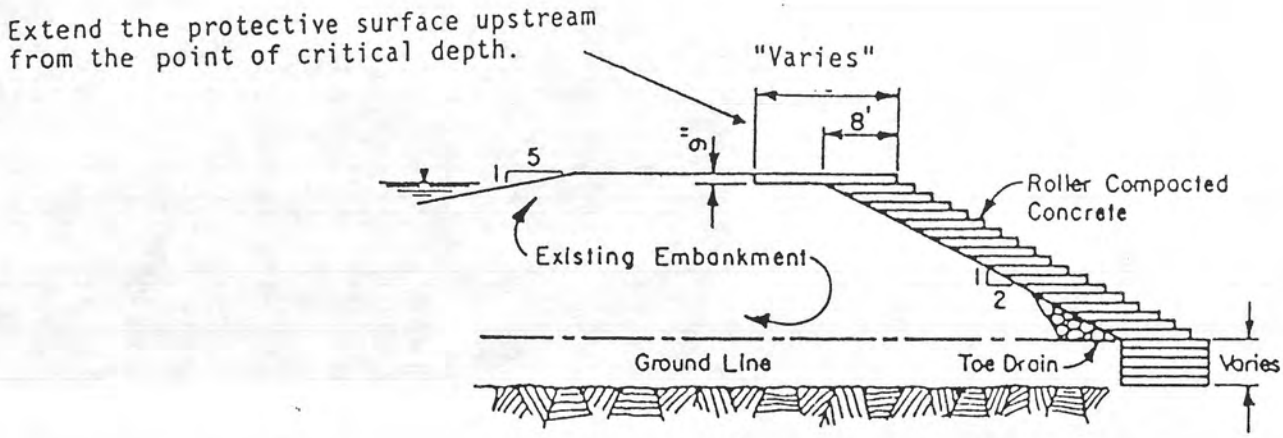
During construction



After completion

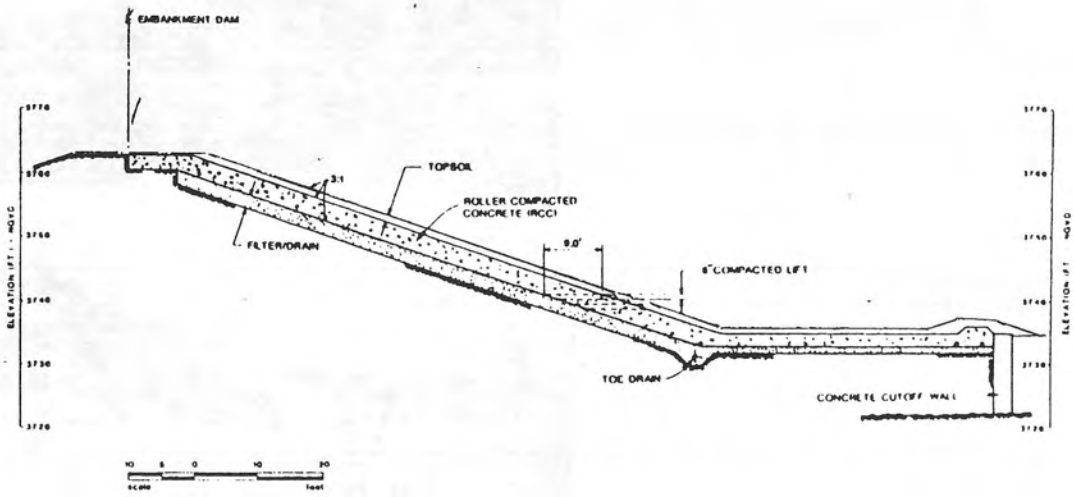
Courtesy: Morrison-Knudsen Engineers Inc.

Figure 3.1 RCC facing on Spring Creek Dam



Source: ASCE Task Committee

Figure 3.2 Example of RCC-faced embankment



Courtesy: Geotechnical Engineers Inc.

Figure 3.3 Example of RCC-faced spillway

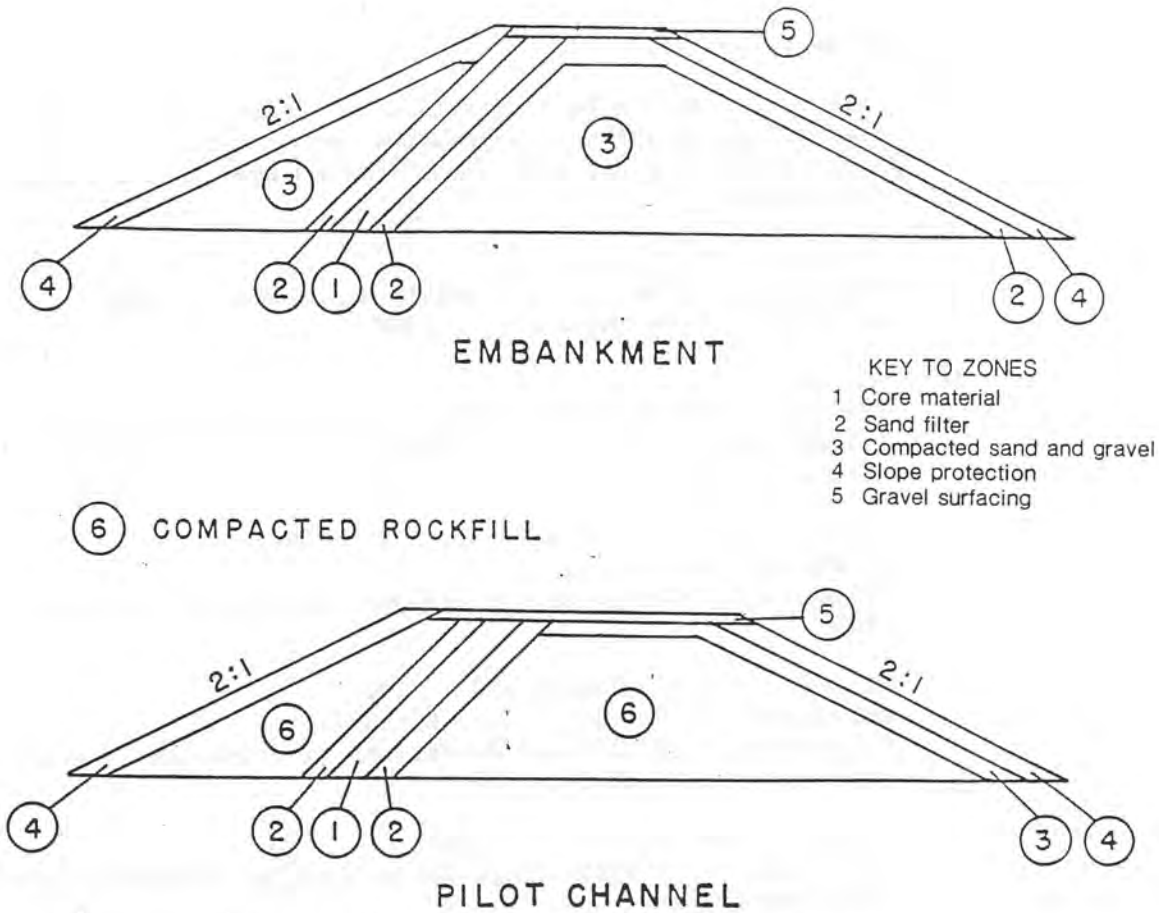


Figure 3.4 Fuse plug embankment and pilot channel

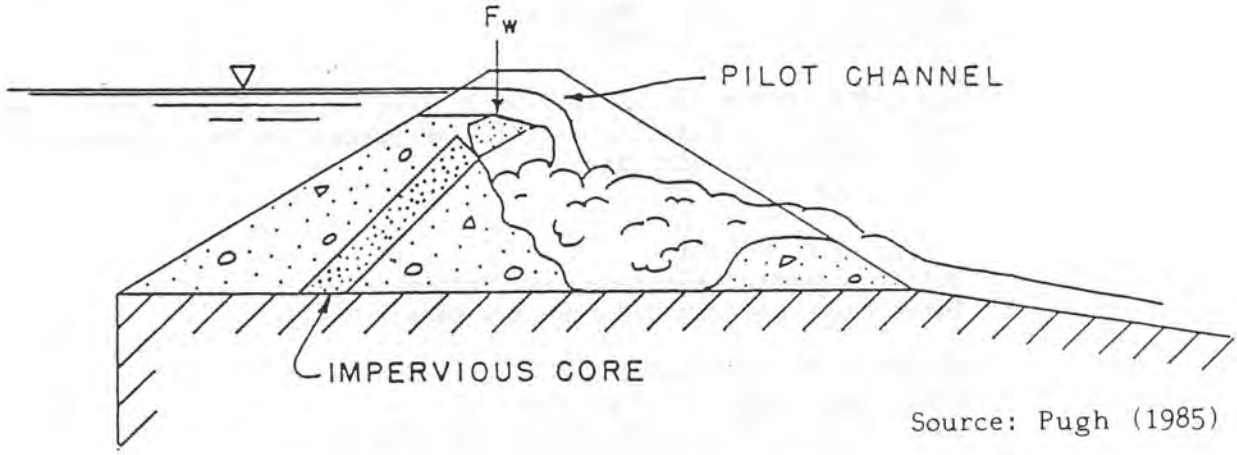


Figure 3.5 Failure mode of impervious core on fuse plug spillway

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RESEARCH ON PERFORMANCE OF EMBANKMENT PROTECTION SYSTEMS IN OVERTOPPING FLOW

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4.1 Introduction

The bulk of the funding for recent research on embankment protection systems in the USA has been made available by the Federal Highway Administration. Their primary concern is for satisfactory surface water run off from the highway, either in highway ditches (Chen and Cotton, 1986), or as flood water flowing down the face of the road embankment (Chen and Anderson, 1986).

Since highway embankments are usually relatively low in relation to tail water level during flood conditions, the FHA have put emphasis on performance at high degrees of downstream submergence, and much of the literature on overtopping refers to this condition. The test facilities have been designed so that they are capable of overtopping the embankment with large volumes of water, but the height of embankment has been insufficient for the water to accelerate under gravity to its"normal" depth (terminal velocity) conditions. Some of the results of the tests thus need careful interpretation if they are to be used as safe conditions on a dam embankment with only nominal tail water level.

4.2 Model Studies

Model studies simulating embankment dam overtopping conditions have been carried out at the USBR (Powledge and Dodge, 1985). A flume 915mm wide, 1220mm high and 9150mm long was constructed in the hydraulics laboratory at the USBR Engineering and Research Center in Denver, Colorado. Within the flume, a 650mm high embankment was constructed using local clayey sand with a 49% fines content ($<$ 0.074mm) and 6% gravel ($>$ 4.76mm), a liquid limit of 25% and plasticity index 9%. The soil was compacted in 75mm layers to 95% standard Proctor density for all but test run 8 where it was overcompacted to 102%. The first tests were carried out with embankment slopes of 6:1 but this was found to be too stable, and the slopes were increased in later tests to 4:1. The model scale was taken as 1:15 so the embankment represented a 9.8m high dam.

Water was supplied to the model using a recirculating pump, with discharge measured by a 200mm venturi meter. Hydraulic factors were scaled using Froude number similarity with the typical simulated discharge intensity being $3.7m^3/s/m$ ($40ft^3/s/ft$), although some tests were run as high as $8.1m^3/s/m$ ($87ft^3/s/ft$). The chosen design criterion was that the simulated dam should be capable of withstanding an overtopping head of 1.2m for 4 to 6 hours.

The embankment had a hard crest and the upper part of the downstream face was lined for different tests with a variety of materials including rip-rap, gabions and rock mattresses. The bare embankment condition was also tested. Results of the tests are given in Table 4.1.

Powledge and Dodge concluded:

1. Scour is initiated by localised surface features such as boulders, trees and low points on the crest.

2. Scour also occurs when flow over a fixed bed makes a transition to a soil bed or vice versa. The smoother the fixed surface, the more the scour.

3. Flow on the eroding surface rapidly transforms into a 'chute and pool' mode. This flow condition has a lesser eroding power than the initial smooth flow.

4. The model gabions and rock mattresses were stable at all times but erosion occurred at or near to their transition with the unprotected embankment. (A change in position of the erosion in various tests was attributed to sagging and bulging of the gabion tops).

5. Rip-rap "fluidized" and washed away very early in the test, and was considered by the USBR to be unsuitable for protection of the downstream face of a dam (but see Section 6.4 of this Report).

6. Overcompacting the embankment fill reduced the erosion to half that observed for standard compaction with similar protective treatments.

Concern is expressed in the report over the ability to scale up the results to absolute prototype values. The data is thus presented as a qualitative comparison between the performance of different protection systems.

4.3 Full Scale Tests - Phase I

These tests were carried out by Simons Li and Associates under contract to the Federal Highway Adminstration and the US Department of Agricultures' Forest Service (Chen and Anderson, 1986).

Prototype sections of road embankment were constructed at the Engineering Research Centre of Colorado State University. The embankments had varying cross-section, with a height of 1.8m, crest width from 3.0 to 6.7m, and downstream slopes varying from 2:1 to 3:1 (horizontal to vertical). In addition, two different embankment fill materials were used - a clay of low plasticity (unified soil classification CL; AASHTO classification A6) with 40% sand and 60% clay/silt, and a specially made up soil (unified soil classification SM-SC; AASHTO classification A-4(0)) with about 60% sand and 40% clay/silt. The effect of paving the embankment crest with two surfaces 300mm gravel and 100mm bitumen was also investigated. Parts of the downstream slope were seeded with a grass mixture, whilst others were protected with erosion control products.

A 900mm wide movable flume, with headbox, inlet diffuser and tailwater control was positioned across the embankment and was progressively moved along the embankment to test different configurations. Figure 4.1 shows a cross-section through the test area. Water was recirculated through the flume using diesel-powered mixed flow pumps with a maximum capacity of $2.9m^3/s$. The overtopping depths tested ranged from 0.15 to 1.22m, and the tail water condition was varied from 10% water surface drop to free fall. The tests were run for durations ranging from 1 to 20 hours.

Variables measured in the tests included:

- Discharge - using a calibrated elbow meter installed in the supply pipeline
- Velocity of flow - using a Marsh-McBirney 201 electromagnetic current meter
- Depth of flow - measured at 0.6m intervals downslope
- Water surface profile
- Embankment profile

The original proposal was to move the flume along the embankments which were to have been 91.4m long, thus testing the effect on erosion resistance of increased soil cohesion due to ageing, and of areas of special construction within the embankment. However, to seal the flume, two 450mm wide trenches had to be cut through the embankment. On the first tests, this resulted in severe erosion along the flume walls. The technique was subsequently abandoned for all cases except those with grass. Instead the flume was left stationary and the required embankment profiles and linings were constructed within the flume. This operation needed special care because large plant could not be used, the material was placed by hand in 150mm layers and mechanically compacted with a vibrating plate and rammer. As far as possible construction was carried out to the FHA specification.

An important part of the investigation was to confirm the hydraulics of flow over an embankment with various tail water conditions, and to this end further pilot tests were carried out with a rigid bed embankment. The conclusions reached broadly confirmed those of Kindswater (1964) and can be simplified into two cases - (a) the free flow condition with no tail water, and (b) the submerged case. In the former case, critical depth formed on the crest and the discharge was controlled simply by the upstream head. Two types of flow pattern were observed, plunging flow when the supercritical flow drove under the tail water and, with rising tail water, surface flow when the flow jet rode over the tail water leading to a reverse rolling flow at the toe. When flow over the embankment crest was submerged, the flow pattern was always surface flow (Figure 2.1 in Section 2 refers). Erosion of the embankment surface is related to the shear stress on the bed and hence the velocity of the flow. Erosion rates for the plunging flow case, with a higher velocity near the bed, were thus greater than those for the surface flow case. This led to the conclusion that the higher the tail water, the more stable the embankment against surface erosion.

As well as plain grass, the other lining systems under test were: (cross-sections of the construction are given in Figure 4.2)

1. Gabion mattresses - 0.9m wide, 2.4m long and 152mm deep made from 19 gauge wire and filled with 75 to 150mm rock. A Dupont Typar 3401 non-woven geotextile was placed under the mattresses.

2. Soil cement - 300mm thick layer placed normal to the crest and downstream side slopes. The material was provided by a local ready mix plant with 11% cement by weight and at 10% moisture content. Plaster sand composed the remaining additive to the mixture.

3. Geoweb - The embankment was lined with a Dupont Typar 3401 geotextile, standard Geoweb 200mm deep with 1.2mm thick walls was placed on top and held in place with wooden stakes. The cells were filled with 25 to 50mm stone.

4. Enkamat - 9mm thick Enkamat was used as a turf reinforcement on one of the grassed slopes. The material was pinned at 1m centres and buried with 25 to 50mm of topsoil before seeding. In addition a further area of Enkamat was set aside, topsoiled and seeded, then subsequently lifted as a turf for use in the flume. In this case the 'turf' was held down at 1m centres with metal stakes. The extent of root development into the subsoil was not clear.

The schedule of tests is given in Table 4.2.

Results of the tests as reported by Chen and Anderson were as follows:

1. Unprotected - The erosion of the bare soil embankments started immediately, and in general the erosion rates correlated well with the method outlined in Section 2.1.1 of this Report.

2. Paved crest - For FHA tests III to V, the embankment crest was paved as for standard road construction. At the lower overtopping heads (0.15, 0.3 and 0.6m), the embankment and gravel shoulders remained intact but at the high overtopping head of 12m pavement was broken, lifted off the embankment and after four hours of flow, completely lost.

3. Grassed slopes - Failure was attributed to pockets of grass being plucked out by the flow, leading to a point of scour that gradually enlarged. It was assumed that such erosion started in areas where the grass roots were poorly established. In addition, serious toe erosion occurred at the higher velocities. At low velocities ($<$1m/s), the grass-lined embankments performed well but, contrary to other researchers' findings, there was no evidence from the tests to show that grass enhanced stability at higher velocities.

4. Gabion mattresses - The failure mechanism for gabion mattresses was defined as the movement of rocks within the mattress which ultimately led to exposure of the geotextile in the upstream end of the mattress diaphragm. Under the most severe test (1.2m overtopping head, free fall, peak velocity 6m/s), only 10-20% of the rocks had migrated downstream after 2 hours and the embankment was in no danger of being eroded. However, attention was drawn to the fact that the gabion basket is vulnerable to vandalism and could deteriorate with time. The stability of the system as tested depends to a large extent upon the integrity of the baskets.

5. Soil cement - Two potential failure mechanisms were postulated, undermining of the construction at the toes and the presence of surface cracks. Neither failure type occurred and the construction was completely stable at the end of the test. However, surface cracking would normally be associated with the material undergoing a freeze/thaw cycle, which did not happen on the tests and the solid steel flume bed prevented erosion at the toe. Consequently the material may not have been tested in its worst service condition.

6. Geoweb - The failure of the Geoweb was caused by the stone particles being removed from the cells by the flow. As a result, the exposed wall of the Geoweb was subjected to drag forces from the water, causing the material to elongate in the direction of flow and expose the embankment soil at the flume edges. Failure of the embankment occurred soon after. Various attempts were made to anchor the Geoweb with more pins and to lay it in different configurations, but no arrangement lasted more than one hour.

7. Enkamat - The failure of the Enkamat was associated with the tearing of the material itself and the erosion of the embankment below the matting. Areas of noticeable scour occurred around the retaining pins that anchored the mat to the embankment. In addition the vegetation in the mat was quickly removed. However, grass growth in the Enkamat was relatively poor and the one year growing season had not allowed a strong root system to develop. The conclusions reached were that in this condition Enkamat assisted in bank stability at low flow rates (0.3m overtopping head, velocity up to 2m/s) but that it encouraged embankment erosion at higher flow rates.

The critical hydraulic conditions for the various protection systems are given in Table 4.3.

The following critical velocities are recommended by Chen and Anderson for the modes of construction described.

Geoweb	1.8 m/s
Gabion	4.6 m/s
Soil cement	6.0 m/s
Enkamat	3.0 m/s

Based upon the results of the tests, the report by Chen and Anderson describes the hydraulics of flow over the embankment, and develops the mathematical model for the hydraulic design and prediction of erosion of an earth embankment subject to overtopping described in Section 2.1.3 of this Report. A computer program written in FORTRAN is listed in an appendix to the report by Chen and Anderson to help the reader put the model on a computer.

4.4 Full Scale Tests - Phase II

These tests are currently being carried out by Simons Li and Associates (SLA) under contract to the Federal Highway Administration and the US Bureau of Reclamation. A report on the results is expected in 1988.

A steel flume 27.5m long, 3.4 high and 1.2m wide, complete with inlet head box, flow deflectors and adjustable exit gate, has been constructed by SLA adjacent to a disused gravel pit near Fort Collins, Colorado. Water can be recirculated from the gravel pit through the flume using a diesel-powered mixed flow pump capable of a maximum discharge of $2.4m^3/s$. In addition a further $0.8m^3/s$ can be provided by extra temporary pumps. A cross-section through the test facility is shown in Figure 4.3.

1.8m high embankments, lined with various protection, are being constructed within the flume and tested with overtopping heads up to 1.2m and tail water depths from free fall to submerged. Under free fall conditions, peak velocity at the embankment toe reaches 7.5m/s with maximum overtopping head of 1.2m. The same measurements are being recorded as those described in the Phase 1 summary.

The standard embankment material is a sandy clay to sandy silt, to AASHTO classification A-4(0), with plasticity index 4.9 and liquid limit 21.4. 42.2% of the material is clay and the remainder sand. Fine low-plasticity clay has been selected by USBR/FHA for its low resistance to erosion. Other fill materials are also being tested. The basic embankment shape has a 7.3m crest length with downstream slopes varying from 2:1 to 4:1 depending on the test.

The schedule of testing, detailed in Table 4.4, comprises further bare soil tests with various downstream slopes, plus a variety of embankment protection systems including:

1. 150mm gabions filled with 75 to 150mm stone laid on a geotextile
2. 100mm deep Geoweb, laid on a geotextile, with the cells filled with 12mm stone and covered with a net
3. 100mm deep Geoweb filled with soil cement
4. Soil cement placed in 100mm lifts and 1m long slabs
5. Bare Enkamat placed on top of a Dupont Typar 402 geotextile

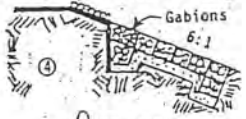
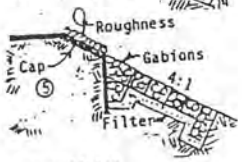
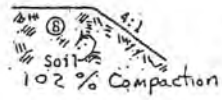
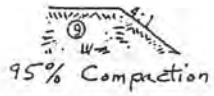
6. Asphalt-filled Enkamat, the asphalt will be placed on site
7. Three different cable-tied concrete block systems laid on geotextiles (Petraflex, Armorflex and Dycel)

The main differences between these and the Phase I tests are:

- A change of embankment material
- Geoweb has a netting cover to restrict the loss of stone
- The soil cement is placed in horizontal layers, one test will be constructed and left to assess the effect of weathering
- The Enkamat has a geotextile underlay
- The testing of additional materials like cable-tied concrete blocks.

Available results of the tests to date will be given at the seminar (see Appendix B).

Table 4.1 Summary of results of overtopping flow model

Schematic sketch and run numbers	Unit discharge ($ft^3/s/ft$)	Time (hr)	Erosion of available volume of material (%)	Maximum scour relative 2.5:1 slope (ft) Location from crest (ft)	Deepest scour hole (ft) Location from crest (ft)	Run No.
	40	1	15.8	-7.5* 15.2	10.0 15.2	1
	40	1	7.2	0.0 10.1	8.8 106.2	2
	40	1	13.4	-5.9 10.1	8.0 10.1	3
	40	1	2.4	8.8 40.6	1.9 83.7	4
		5	4.6	8.1 40.6	3.8 95.1	
	40	1	11.7	-1.3 52.8	3.4 59.2	5
	40	1	3.8	3.8 59.2	6.3 104.3	6
		5	8.4	2.5 83.7	10.0 83.7	
	40	1	9.1	-0.3 0.0	5.0 77.3	7
		5	14.1	-2.5 0.0	8.8 59.2	
	87	1	6.5	-0.3 0.0	3.8 toe	8
		5	6.5	-0.9 0.0	5.6 toe	
	87	1	12.7	-1.2 0.0	3.4 toe	9

* Negative indicates scour below 2.5:1 slope line.

Source: Powledge and Dodge (1985)

Table 4.2 Schedule of Phase I full scale tests

Series	Description of Test	Soil Type	Slideslope	Overtopping Depth, ft (D_{ot})	Water Surface Drop Over Embankment (percent of D_{ot})	Testing Duration (hr)
FHWA I	Bare-Soil Surface; No Protection	I	3:1	0.5, 1, 2, 4	20, 70, Free Fall (FF)	1, 4, 10, 20
FHWA II	Bare-Soil Surface; No Protection	II	3:1	0.5, 1, 2, 4	70	1, 4, 10, 20
FHWA III	Paved Surface/ Gravel Shoulder; No Protection	II	3:1	0.5, 1, 2, 4	70	1, 4, 10, 20
FHWA IV	Paved Surface/ Gravel Shoulder; Grass	I	3:1	0.5, 2, 4	FF	1, 4, 10
FHWA V	Paved Surface/ Gravel Shoulder Grass	I	3:1	0.5, 2	70	1, 4, 10
USFS I	Bare-Soil Surface; Enkamat	II	3:1	0.5, 2	FF	1, 4, 10
USFS II	Bare-Soil Surface; Geoweb	II	3:1	1, 2, 4	FF	2
USFS III	Bare-Soil Surface; Enkamat/Grass	II	3:1	0.5, 1, 2, 4	FF	2
USFS IV	Bare-Soil Surface; Gabion Mattress	II	2:1	1, 2, 4	FF	2
USFS V	Bare-Soil Surface; Soil Cement	II	2:1	1, 2, 4	FF	2

Source: Chen and Anderson (1

Table 4.3 Critical results of Phase I full scale tests

Protection Measure	Overtopping Depth (ft)	Discharge (ft^3/s-ft)	Average Flow Depth (ft)	Average Velocity (ft/s)	Maximum Velocity (ft/s)	Energy Slope	Manning's n	Shear* Stress (lb^2/ft^2)	Remarks
Geoweb	1.0	3.0	0.38	7.9	8.3	0.27	0.051	1.0	Significant toe erosion occurred after 9 hours of test.
Gabion	1.0	3.0	0.42	7.1	7.9	0.34	0.068	1.0	Stable
Gabion	2.0	8.4	0.82	10.2	10.9	0.27	0.066	2.0	Stable
Gabion	4.0	25.0	1.59	15.7	17.2	0.22	0.060	5.0	Some rock migrated, but gabion remained stable.
Soil Cement	1.0	3.0	0.32	9.4	11.5	0.21	0.034	0.6	Stable
Soil Cement	2.0	8.4	0.55	15.3	18.0	0.11	0.022	1.6	Stable
Soil Cement	4.0	25.0	1.48	16.9	20.0	0.022	0.017	1.9	Stable
Enkamat	1.0	3.0	0.38	7.9	8.0	0.28	0.051	1.0	Stable
Enkamat	2.0	8.4	0.80	10.5	12.0	0.15	0.047	2.5	Some erosion
Grass	0.5	3.0	0.17	5.9	6.1	0.33	0.044	0.4	Stable

*Note: Shear stress $\tau = \frac{1}{8}\rho f V^2$, where ρ is the water density, f is Darcy-Weisbach coefficient and V is the velocity.

Based on information by Chow, f = 0.02 (soil cement), 0.04 (grass), 0.06 (geoweb), 0.07 (enkamat and gabion).

Source: Chen and Anderson (1986)

Table 4.4 Schedule of Phase II full scale tests

Series	Description	Side Slopes	Water-Surface Drop	Depth of Flow	Comments
I	Bare soil (erodible) no pavement, no protection	3:1	FF 1',FF 1',2,FF	1' 2' 4'	
		4:1	FF	1',2',4'	
II	Bare soil (non-erodible) no pavement, no protection	3:1	FF 1',FF 1',2',FF	1' 2' 4'	
		4:1	FF	1',2',4'	
III	Gabion mattresses 6" thick, 3"-6" rock (similar to installations used in New Mexico)	3:1 2:1	2',FF 2',FF	2',4' 2',4'	Use geotextile filter and more erosive soil
IV	Geoweb, 4" thick filled with 1/2" rock and covered with netting	2:1	FF	1',2',4'	10-hour duration
V	Soil cement placed in 4" layers, 3' wide lifts	3:1 2:1	FF FF	1',2',4' 1',2',4'	10-hour duration
VI	Enkamat with geotextile liner	2:1	2',FF	2',4'	
VII	Enkamat filled with asphalt	3:1 2:1	2',FF 2',FF	2',4' 2',4'	
VIII	Cable-tied conc. blocks filled w/ compacted soil	3:1 2:1	2',FF 2',FF	2',4' 2',4'	10-hour duration
IX	Geoweb, 4" thick filled with soil cement	2:1	FF	4'	10-hour duration

The last geoweb test (series IX) is a lower priority test to be conducted at the end of the study if time and money are available.

Source: USBR

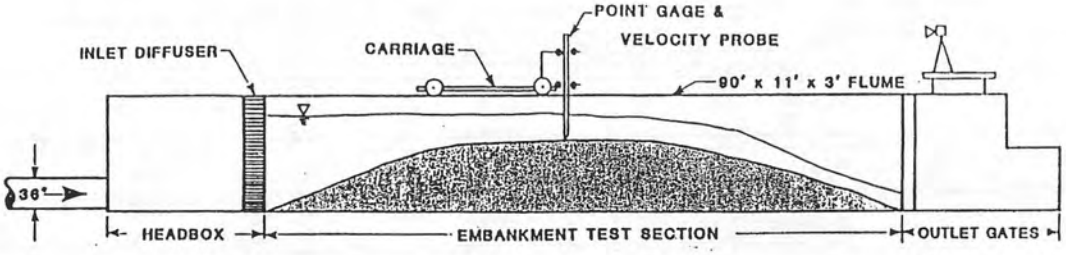


Figure 4.1 General arrangement for FHA/USDA full scale tests (Phase 1)

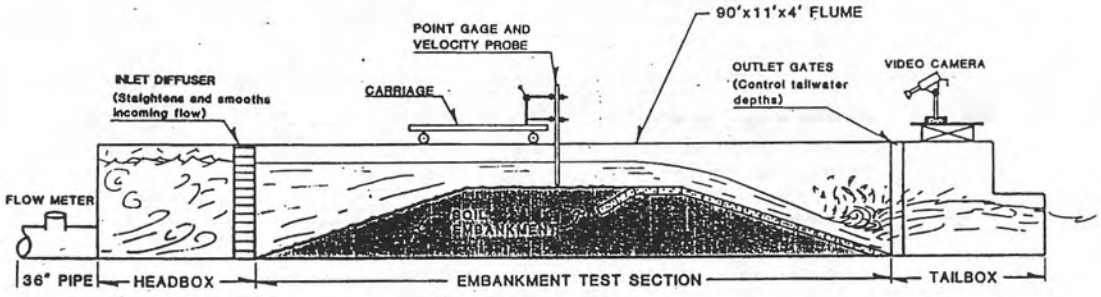


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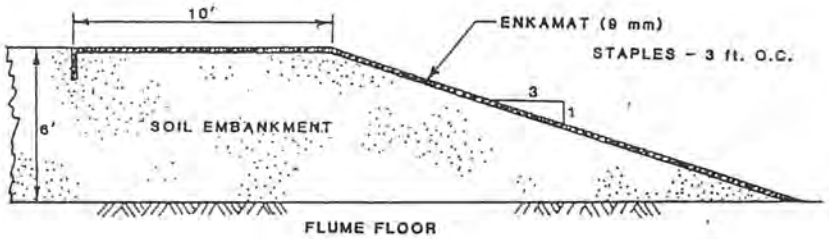
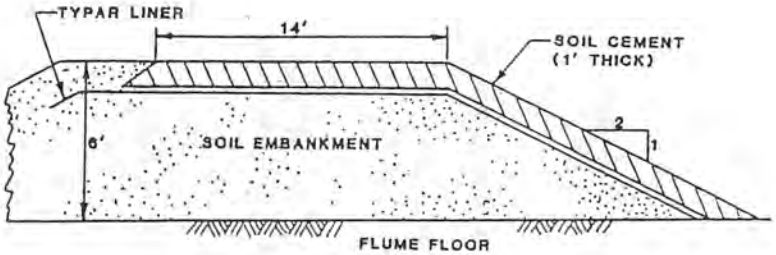
Figure 4.3 General arrangement for FHA/USBR full scale tests (Phase II)

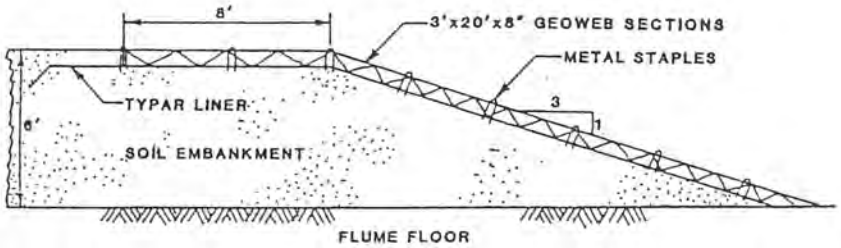


Cross-sectional view of enkamat protection measure.

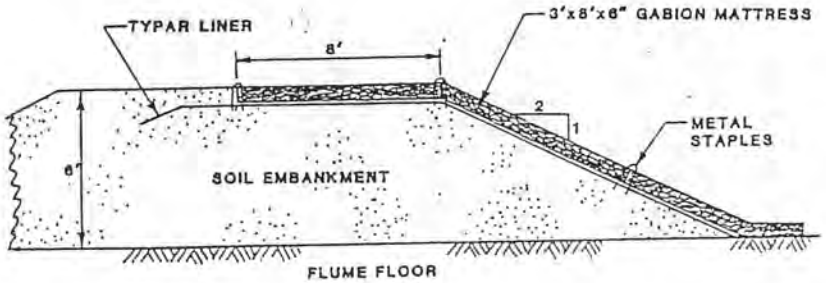


Cross-sectional view of soil cement protection measure.

Source: Chen and Anderson (1986)



Cross-sectional view of geoweb protection measure.



Cross- sectional view of gabion protection measure.

Figure 4.2 Cross-sections of protection systems tested under Phase I full scale tests

References to Section 4

1. Chen, Y H and Cotton, G K 1986
   Design of roadside channels with flexible linings
   US Federal Highway Administration Report No FHWA-IP-86-5, HEC No 15
   Simons Li & Associates, Fort Collins, Colorado, 1986

2. Chen, Y H and Anderson, B A 1986
   Development of a methodology for estimating embankment damage due to flood overtopping
   US Federal Highway Administration Report No FHWA/RD-86/126
   Simons Li & Associates, Fort Collins, Colorado, 1986

3. Powledge, G R and Dodge, R A 1985
   Overtopping of small dams - an alternative for dam safety
   Proceedings ASCE Hydraulics division Speciality Conference, Florida, 1985

4. Kindsvater, C E 1964
   Discharge characteristics of embankment-shaped weirs
   US Geological Survey Water Supply paper 1617-A, 1964

THE ROLE OF GEOSYNTHETIC MATERIALS

C D Hall - Netlon Ltd

5.1 The scale of potential US interest

"Many existing small embankment dams in the United States cannot safely pass design floods without overtopping" (1).

"Thousands of dams built in the United States (recently assessed to be over 57,000 Federal and non-Federal) have a potential to be overtopped by flood flows" (2).

"17% of all incidents involving embankment dams were due to overtopping related to inadequate spillway capacity" (2).

"From a sample of 1900 embankment dams, 81 had failed". "The most common cause of failures..... was the overtopping of the dam crest" (3).

For a mission whose subject area included the provision for safe overtopping of dams, it was apparent from these recently-made statements that both the topic and planned visits to the US Army Corps of Engineers and Bureau of Reclamation were apposite.

"When an embankment dam is expected to overtop .... most of the engineering profession assume the structure will fail" (4).

This gives the impression that overtopping, and the likelihood of resulting dam failure, is a way of life which, at least, enables field observations to be made on breach mechanisms and failure modes of embankments. Accordingly, all the above quotations were made by engineers dedicated to improving dam safety through such initiatives as the ASCE Task Committee on the Mechanics of Overflow Erosion on Embankments (4), where a compelling need is identified to consider the use of appropriate lower cost erosion protection.

On this basis, the prospects for using geosynthetic materials for erosion control appear to be good and one might expect to find the use of geosynthetics to have reached an advanced state in this application.

The following chart clarifies the potential applications for geosynthetic materials in erosion control systems:

Structure	Provision for Safe Overtopping	Method of Erosion Control	Scope for Geosynthetics
Dam	Spillway	r.c.	-
		R C C	-
		gabions	Fabrics
		rip-rap	Fabrics
		conc. blocks	Fabrics
		Flexible linings	Geomembranes
Levee	Crest and back slope	Soil cement	-
		Vegetation	Mats
Highway Embankment	Back slope	Soil cement	-
		Vegetation	Mats
		Gabions	Fabrics

The last column indicates that there are three areas of interest: filter fabrics as underlayers to armouring; the use of geomembranes as an impermeable channel lining; and mats in reinforced grass.

In some of these applications, a difference in attitude was highlighted between UK and US engineers towards erosion and consequent breaching of hydraulic structures. The UK engineer generally attempts to eliminate erosion completely on the basis that total prevention is the only point of control. The US engineer appears more prepared to define tolerable levels of erosion damage. A further point which differentiated UK and US engineers arose from a question of scale - US catchments and floods are frequently much larger, and the design flood event is often large by UK standards in terms of both discharge and duration.

5.2 The role of geosynthetics

5.2.1 Fabrics

The Bureau of Reclamation provided the most up to date reference on the use of 'geotextiles and related products', with a draft copy of USBR Design Standard No 13, Embankment Dams (5). In Chapter 20, a logical approach is set out on geotextile selection and indicates areas of acceptable uses. Among these, it recognises the filtration function of fabrics in 'slope protection' and the required qualities of soil retention and permeability. Although the document is mainly a compendium of published sources of reference and conventional geotextile wisdom, it nevertheless provides positive guidance to the dam engineer on appropriate use of geotextiles. Relevant tables on selection criteria and recommendations are reproduced in Appendix 5.1.

A key point to infer from the existence of this design standard is that geotextiles are acceptable in selected areas of dam construction. UK dam engineers have perhaps not been so prepared to take advantage of synthetic materials in dam construction and modification. When examining the use of geotextiles in a reinforcement function, for example, schemes have emerged where critical uses of geotextiles are made. The meaning of 'critical' here is that failure would impair the function or stability of the structure. The reinforced soil principle has been employed in dam crest raising and the steepening of the side slope of the embankment dam.

The Corps of Engineers have a similar design standard (6) but it did not appear so innovative in this respect, partly due to their extensive experience in the use of rip-rap and graded filters. Factors which were stated to affect whether a synthetic fabric could be incorporated in any structure were:

1. is it inspectable?
2. how great would the cost saving be?
3. is it critical? (in the USBR sense)
4. what is the existing State-of-the-art?

It is concluded that the US standards and design guidelines on the use of fabrics as filters and separators, for the applications in question, are at a no more advanced state than that which exists in the UK. A number of new publications (7,8,9,10) will be published shortly in the UK and will cover material specification through to design. These publications will assume the most recent research and design philosophies, and their scope will cover the separation and filtration functions of underlayers in erosion control systems.

5.2.2 Geomembranes

There is widespread use of geomembranes in the United States. Legislation on land fill and waste disposal, for example, is far in advance of the UK and so is the attendant technology for control and collection of leachate through the geosynthetic functions of separation, filtration and in-plane drainage. (Such a topic would be a good study area for a future OSTEM visit if one were to anticipate UK legislation being brought into line with the US, continental Europe and Japan).

Use of liners

Considerable US experience in the use of geomembranes as liners has developed from the provision and upkeep of canal systems for irrigation and conveyance of potable water, and this has led to consideration of the use of geomembranes in emergency spillways. In contrast, canals in the UK have largely been constructed for navigation and land drainage, and there is less experience in the use of geomembranes as liners. In the US, the use of geomembranes in canals dates back to 1969 with full scale installations under the Rehabilitation and Betterment Program commencing in 1973 with, for example, the Riverton Irrigation Project where 770 000m^2 of PVC liner were incorporated (11). This project illustrates the familiar change from traditional uses of concrete and bitumen-based materials towards synthetic materials. The reasons are common to many other cases where a change to geosynthetics has been made:

- easy to place
- no specialised requirements for mechanical equipment (such as pavers)
- less stringent requirements for site quality control
- more tolerance to movement

On the other hand, there are risks which need to be considered with the use of geosynthetics:

- uncertainty on durability
- vulnerability to installation damage and vandalism

A typical lined canal cross-section is shown in Figure 5.1. The general requirements of geomembrane liners are: inertness, durability, tensile strength, installability and repairability (12,13,14). These qualities are generally thought to be available from pvc (polyvinyl chloride) and PE (polyethylene); pvc tends to dominate the US market.

In all cases, the liner is covered with soil. The desirability and advantages of an uncovered canal liner have been recognised (13), and three study sites incorporating four different synthetic materials are presently being monitored by the USBR.

A USBR report (15) describes a pumped storage facility at Mt Elbert Forebay Reservoir and how the world's largest 'pond liner' installation was carried out in 1980 using 1.2 million m^2 of 1mm thick CPER (reinforced chlorinated polyethylene). Geological mapping had identified a portion of the site as an ancient landslide and there was concern that seepage would reactivate the slide. The invert was lined with 1.5m of earth fill but pore water monitoring indicated that additional measures were advisable. A membrane liner was chosen in preference to more traditional options of concrete or asphalt as it:

- avoided the need to claim large amounts of aggregate
- minimised installation time and enabled operation to commence as planned
- was considered better able to accomodate local strains which may initiate cracks in more rigid invert constructions.

Application to emergency spillways

This background experience gives the engineering confidence to allow the technology to evolve into more critical uses related to emergency spillways for low head situations. A study commenced in 1981 at the USBR with field testing being carried out in 1985/86 at Cottonwood Dam No 5 (16). In this attempt to develop a low cost application for medium-size dams, the geomembrane is again soil covered for protection but this layer is now regarded as sacrificial during flow. The installation and some of the design considerations are shown in Figure 5.2. This system performed satisfactorily at velocities in the range 6-8m/s, the covering soil having been removed at an earlier stage of the flow. One of the interesting conclusions in the study report is that vegetation is identified as a means of preventing the soil wash-out, and thus provides the possibility of reducing the maintenance requirement of replacing the soil after each operation of the spillway.

The development of a composite protective layer, incorporating with the geomembrane liner the UK practice of reinforcing the grass with geosynthetic mats, was identified as an area of potential future collaboration.

5.2.3 Mats

A mat is a three-dimensional flexible synthetic composition typically greater than 10mm thick. In the use of mats, the state of development in US and UK is roughly equal. However, major differences do exist in the concept and purpose of a mat. In the areas of interest of the OSTEM visit, UK engineers have a more developed concept of the engineering use of vegetation in conjunction with mats for the provision for safe overtopping of structures or of an emergency spillway (17, 18).

Geotextile-reinforced grass is creating interest in the US by being reported in US conferences (19), the technical press (20) and by policy making bodies such as the ASCE Task Committee (4). Up-to-date reviews and reference listings of the engineering use of vegetation in conjunction with geotextiles are contained in yet to be published authors' scripts which were supplied to OSTEM members (4,23,24). This degree of access to information is gratefully acknowledged.

The mats that are available in the US are similar in their specification to those used in the UK. Enkamat and Miramat are manufactured in the US, Tensar Mat and Erolan are European imports. A market-sectoring exercise shows that a majority of these materials are used in non-critical, relatively low stress environments such as highway drainage channel linings (25) and surface stabilisation of steep slopes subject to run-off. Various trials and demonstrations have taken place (26) with the result that a number of State Departments of Transport have produced guidelines on these applications. An example from the State of Georgia, based on a soil erodibility index, is given in Appendix 5.2

In an examination of these application areas, some conceptual differences in the use of mats became evident, i.e.

- the mat is not normally soil filled and is itself required to protect the soil of the channel (although natural accretion of sediment may take place within the mat).

- the mat is regarded more as a temporary establishment aid for vegetation rather than a means of long term enhancement of vegetation and its engineering function.

These two observations perhaps explain why the reinforced grass armour layer is not a well-established concept in the US. Contact with the Corps of Engineers Waterways Experiment Station also provided further insights as to why they had not provided the impetus to adopt geotextile-reinforced grass in erosion control. Reasons suggested by the Corps of Engineers for its limited use of in waterway erosion control included:-

- uncertainty on whether the right climate and soil types exist

- under the Section 32 program (see Section 1), the limited experience gained with vegetation in stream bank erosion control had ranged from 'satisfactory' to 'disastrous'

- as there is little experience of reinforced grass then little guidance can be offered on design, installation, supervision and inspection

- WES research tends to be concerned with verification rather than innovation

- administrative constraints on procurement of materials and aftercare of vegetation.

The use of mats in the US as protection against embankment overtopping is still very much at the research stage - partly because development is not co-ordinated centrally. R&D on the use of mats has been carried out under the USBR and FHA, while guidance on the engineering function of vegetation is spearheaded by the Department of Agriculture's (USDA) Agricultural Research Service [21,22] and the Soil Conservation Service (SCS). With its involvement in the more modest earth embankment dams, the SCS reflects the UK perspective much more than the Corps or the USBR with their responsibilities for large dams.

The research on geosynthetics in erosion control was initiated in 1983 by a study sponsored jointly by the Federal Highway Administration (FHA) and the USDA Forest Service (27) to investigate embankment damage due to flood overtopping (see Section 4.3 for general report). As a major part of the study, full scale flume tests were conducted on commonly used protection measures including plain grass, mats, gabion mattresses, rip-rap and soil cement. The materials were arranged on a 1:3 downstream face of a 2m high embankment where the crest was overtopped by flow. The proprietary synthetic mat tested was Enkamat which was installed 'peak side down' and covered with up to 50mm of soil and seeded with a dominantly (60%) wheatgrass mixture. It was tested both in the bare soil condition without vegetation established, and in a grassed-over condition with vegetation established. In the latter case, it was tested after having been transplanted on to the embankment slope within the flume. The results of the tests and the recommendations for critical design velocities are reviewed in Section 4.3 of this Report. In the evaluation of the tests, it was reported that 'the failure mechanism associated with Enkamat was related

to rippling or stretching, or to noticeable erosion of the embankment beneath. The presence of grass had a very little effect'. It was concluded that:

- Enkamat performed well in overtopping heads less that 0.3m (critical velocity of 3.0m/s)

- Enkamat effectiveness depends greatly on the type and pattern of retaining pins (usually wedge-shaped timber pegs in the US)

- Enkamat has the potential to be effective if properly installed.

These conclusions agree with the guidance being given by CIRIA (28) following similar full-scale testing in the UK, although substantially higher critical velocities are recommended for vegetated mats (4-6m/s depending on flood duration). Broad agreement exists on the critical installation factors, which are:

- intimate contact with the substrate,

- avoidance of turbulence and local scour set up by pegs, pins, irregularities and laps.

Critical aspects of installation also emerged at a meeting arranged at USBR offices in Denver with manufacturers of erosion control systems ranging from concrete block revetment products to fibrous and synthetic geotextile mats. Key topics which were discussed included:

- the enhanced ability of a mat, when used in conjunction with grass as a reinforced grass armour layer, to counter or ameliorate the various recognised modes of failure of plain grass protection,

- the need for a range of installation specifications which can be matched to the severity of hydrualic loading in any particular application,

- forms of retaining pins suitable for high velocity flows,

- the need to distinguish between geosynthetics used as short-term establishment aids and those used for permanent reinforcement,

- conditions in which either limiting velocity or limiting tractive force is the more appropriate basis for design of the protection system,

- the need to consider the effect of flow on the subsoil or underlayer below the protection, and the generation of uplift forces,

- appropriate parameters for specifying a mat such as tensile strength, thickness, tensile and flexural stiffness.

In the main, these topics appear to have been considered more extensively in the UK, and lead to the conclusion that the critical uses of mats are more advanced in the UK.

There was a tendency for manufacturers and suppliers of protection materials in the US to be more on the periphery of research, and some of the field testing relating to the application of their materials, than their counterparts in the UK. This is partly understandable as there appears to be no forum in the US, such as CIRIA, for example, where it is possible for

different sectors of the industry to collaborate in a research project. It was also noted that US legislation requires engineers in government service to remain strictly neutral in all dealings with the commercial sector, and this may well restrain collaborative developments of product applications. In this respect, no focal point for US manufacturers or suppliers was identified for future collaboration with the UK.

5.3 Conclusions

The two key objectives of the OSTEM visit were (a) to increase the flow of industrially relevant scientific and technical information into the UK, and (b) to explore the potential for scientific and technical cooperation between UK and US organisations. This section of the Report has sought to contribute to these objectives from the UK manufacturing industry's point of view.

Conclusion 1: Erosion control using reinforced grass

Principal experience in the US of the application of geosynthetic fabrics in erosion control relates to their use as short-term establishment aids as opposed to permanent reinforcement. It was established that the practices evolved in UK and Europe for the use of geosynthetics, in particular mats, as permanent reinforcement are probably more advanced than in the US where few special methods of specification and installation have been developed for such critical applications.

Future collaboration

When considering further UK/US co-operation, it should be noted that there is no UK trade association for manufacturers and suppliers of geosynthetic materials that would compare with, for example, the Flexible Revetment Association. UK manufacturers and suppliers would have to seek co-operation either through their membership of a research and information body such as CIRIA, or through international societies such as the International Geotextile Society, or directly through their US representation and contacts. Clearly, the former is preferable.

The principle US organisations that are presently directly involved in gaining knowledge relevant to development of the use of geosynthetics are the USBR, the Federal Highway Administration and the USDA's Soil Conservative Service and its research arm, the Agriculture Research Service. Currently, the USBR and FDA are evaluating erosion control systems (see Section 4 of Report) and the ARS is continuing its investigations into vegetal measures (see Section 2.2 of Report) which should eventually take on the improved performance provided by reinforcement. Additionally, the USBR and Corps of Engineers both expressed interest in developing collaborative research arrangements in the US along the lines utilised by CIRIA. The CIRIA model arouses great interest in the US and its publications are held in high esteem.

It is forseeable that, resulting from present US initiatives, some prototype evaluation of erosion protection systems similar to that conducted in the UK at Jackhouse Reservoir, will be undertaken in the US. Such an 'evaluation program' might mirror previous US programmes in coast protection and stream bank erosion control.

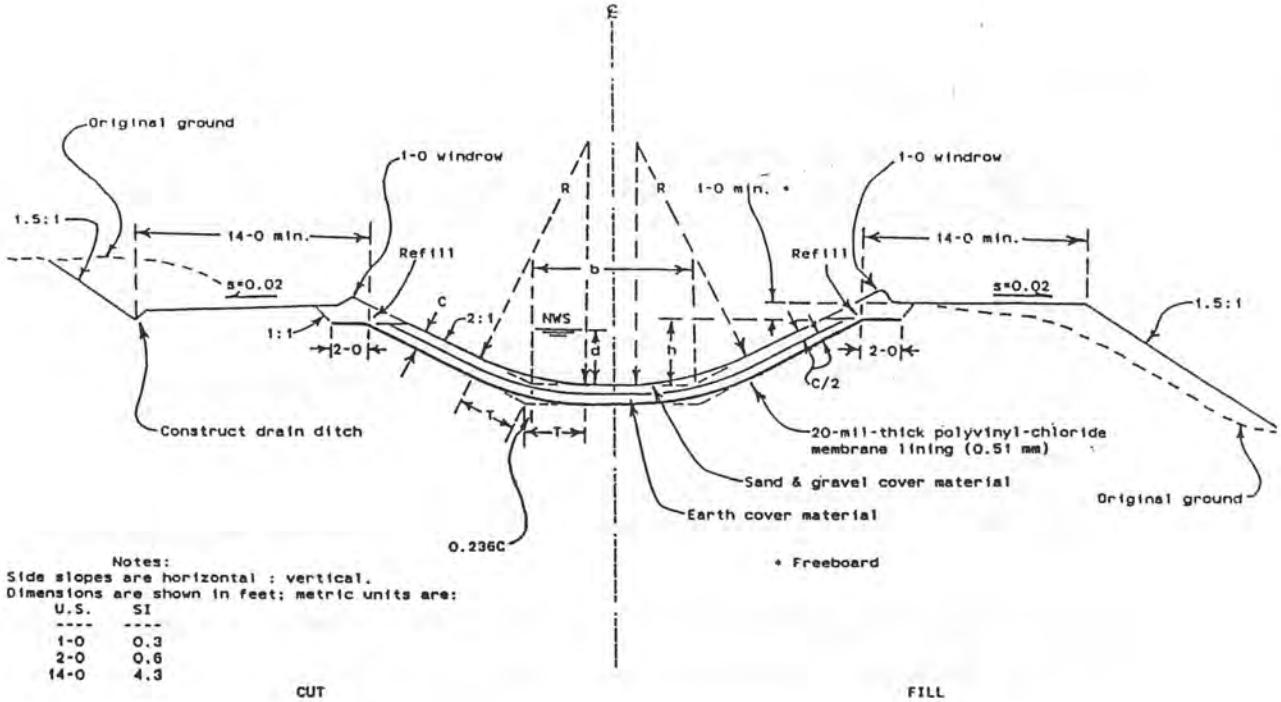
Such field trials, which could conceivably be set up in about two years time, would appear to present the best opportunity for future collaboration with the US, and could also enable materials of UK origin to be evaluated in critical US applications.

Conclusion 2: Geomembrane linings

The use of geomembranes represented the aspect in which most new knowledge was acquired in relation to the study area of the mission. Their inclusion in US erosion control applications probably came about because geomembranes are in common use in other civil engineering practices, such as land fill and the control of refuse and toxic wastes.

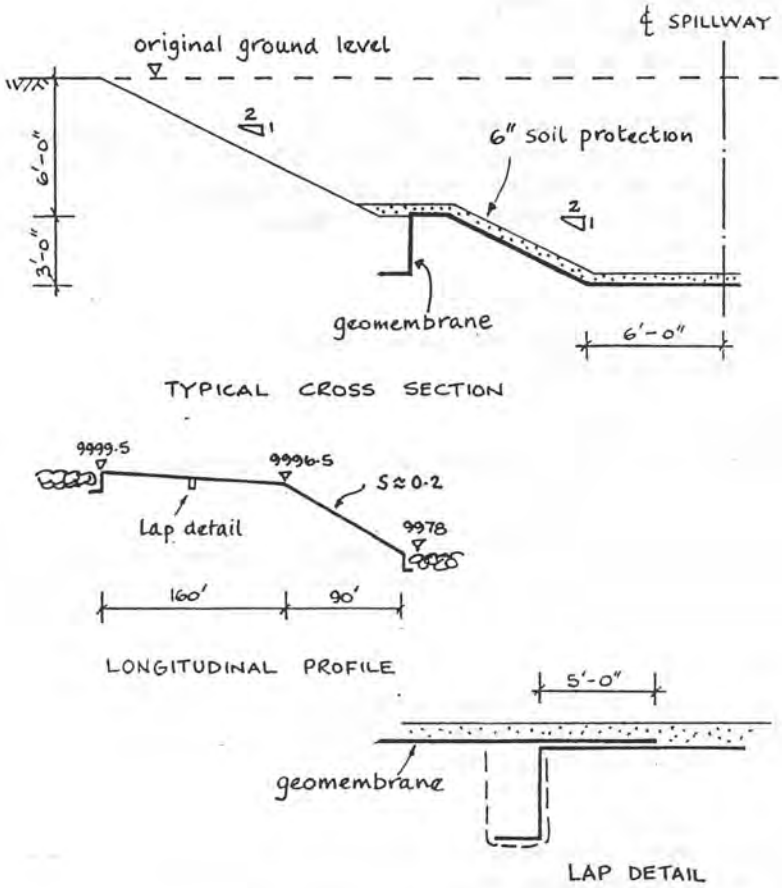
Conclusion 3: Fabric separators and filters

No areas of new knowledge were discovered on the visit.



Source: USBR

Figure 5.1 Typical geomembrane-lined canal



Source: Timblin and Morrison (1987)

Figure 5.2 Detail of geomembrane in emergency spillway

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DESIGN OF RIP-RAP PROTECTION AGAINST CURRENT ATTACK

R.W.P. May - Hydraulics Research Ltd

6.1 Introduction

Rip-rap is extremely widely used in the US as riverbank protection and as protection downstream of hydraulic structures. It is much less widely used as protection to dams or flood embankments against overtopping flow. However useful information on recent experimental studies and on present design practice was obtained for all applications during the OSTEM visit. The three areas are to some extent complimentary and are therefore all covered in this section, which has a wider coverage than other sections of the Report.

The flow of water over a layer of rip-rap produces steady and fluctuating forces of lift and drag which, if they exceed the restoring force due to gravity, will cause movement of individual particles and possible failure of the protective layer. Additional destabilising forces can be caused by flow within the rip-rap layer, either parallel or normal to its surface. The gravity force is proportional to the immersed weight of a particle, but its contribution to stability depends upon the slope of the rip-rap layer and its orientation relative to the flow. It is, therefore, necessary to distinguish between:

(a) flow over a near-horizontal bed, where the gravity force is normal to the bed;

(b) flow along a streambank, where the gravity force has a destabilising component which acts down the bank and thus normal to the direction of flow;

(c) flow in a steep channel, where the destabilising component of gravity acts down the slope and in the same direction as the flow.

Flow overtopping an embankment will usually correspond to case (c); in such situations the ratio of flow depth to stone size is often much smaller than in cases (a) and (b). An additional factor which affects stability is the degree of turbulence in the flow; this will normally be higher in steep channels and downstream of structures (such as stilling basins and gates) than in lengths of river or canal.

6.2 Streambank protection

6.2.1 Present design recommendation

Stone size

The method currently recommended by the US Army Corps of Engineers (CoE) for the design of rip-rap protection for streambanks is described in Engineer Manual EM 1110-2-1601 (1970) and Engineer Technical Letter ETL 1110-2-120 (1971). This method is suitable for sizing the material required on the bed and sloping sides of channels carrying flood flows with a normal degree of turbulence caused by the rip-rap roughness. The shear stress τ_o acting on a section of the channel boundary is determined from:

$$\tau_o = \frac{\rho_w V^2}{[(32.6 \log_{10}(12.2y/D_{50})]^2} \qquad \text{------ (1)}$$

where ρ_w is the desity of water, V is the depth-averaged velocity at the section, y is the depth of flow and D_{50} is the median size of the stone. For the rip-rap to be stable, this shear stress should not exceed the critical shear stress τ_c given by the Shields-type relation:

$$\tau_c = 0.04\ \Omega(\gamma_s - \gamma_w)\ D_{50} \qquad \text{------ (2)}$$

where γ_s is the specific weight (in air) of the rip-rap and Ω is the slope function

$$\Omega = \left[1 - (\sin\theta/\sin\phi)^2\right]^{1/2} \qquad \text{------ (3)}$$

in which θ is the angle of the bank to the horizontal and ϕ is the angle of internal friction (or repose) of the rip-rap.

Grading

ETL 1110-2-120 (1971) recommends the following grading curve for rip-rap on streambanks:-

n(%)	D_n/D_{50}	W_n/W_{50}
100	1.34	2.40
85	1.26	2.00
60	1.08	1.25
50	1.00	1.00
40	0.89	0.70
15	0.58	0.20
10	0.52	0.14
5	0.46	0.10

where D_n (or W_n) is the size (or weight) of particle which is not exceeded by n% of the material by weight. EM 1110-2-1601 (1970) recommends that the stones should be predominantly angular in shape, that no more than 25% of the stones should have a length greater than 2.5 times its breadth or thickness, and no stone a length exceeding 3.0 times its breadth or thickness.

Other methods

An alternative approach to the design of rip-rap protection is the Safety Factors Method (SFM) described by Stevens et al (1976). This is based on shear stress like the CoE method, but differs in that it allows a safety factor against movement to be incorporated. Another difference is that the applied shear stress is estimated from the slope of the channel and the depth of flow, instead of from the stone size (as in Equation (1)).

6.2.2 Present research and development

The US Army Corps of Engineers is currently funding a major programme of research on rip-rap which involves experimental work at the Waterways Experiment Station (WES) at Vicksburg and Colorado State University (CSU) at Fort Collins. A new design method for rip-rap based on results from these studies will be given in Engineer Manual EM 110-2-1605. This document is still in draft form, so the following information is provisional and is based on discussions at WES and CSU, and on a research report by Ruff et al (1987) and a paper by Maynord and Ruff (1987).

Laboratory tests on rip-rap have been carried out at CSU using a large tilting flume measuring 61m long x 2.4m wide x 1.2m deep and with flows up to about $1.4m^3/s$. The stability of rip-rap has been studied for the case of a flat bed and for a trapezoidal channel with a side-slope of 2H:1V; the sizes of rip-rap tested in the latter case were D_{50}= 12.7mm and 25.4mm. Further tests are planned with side-slopes of 1.5H:1V and 3H:1V. The criterion for failure of the protective layer was taken to be not the start of particle movement but the development of sufficient damage to expose the underlying material.

Significant findings from the tests with the 2H:1V side-slope were:

(a) more failures occurred on the horizontal bed of the trapezoidal channel than on the sloping side;

(b) some stones moved considerable distances along the channel prior to failure (e.g. up to 15m during tests lasting 2-3 hours);

(c) at failure, values of the Shields parameter (calculated for the whole cross-section) were all below the Shields curve;

(d) increasing the thickness of the rip-rap layer from $1.5D_{50}$ to $2D_{50}$ increased its stability.

Analysis of the CSU data by the Corps of Engineers (CoE) has led to a new design method based not on shear stress (as in EM 1110-2-1601) but on flow velocity. The results also indicated that the slope function Ω in Equation (3), with a normally-used value of $\phi = 40°$, over-estimated the destabilising effect of the side-slope. The resulting design equation does not therefore use Ω, and has the form:

$$\frac{D_{30}}{y} = C \left[\frac{V}{\{g(s-1)y\}^{1/2}} \right]^{2.5} \qquad (4)$$

where V is the depth-averaged velocity at the toe of the slope, y is the corresponding depth and s is the specific gravity of the stone ($= \gamma_s / \gamma_w$). Noteworthy features of the equation are that the D_{30} size and not the D_{50} is used, and that the exponent is 2.5 and not 3.0 as in previous equations of this type such as Stephenson (1979) and Pilarczyk (1984). Provisional values of the coefficient C are as follows:

Criterion	Application	C
Incipient failure	Channel bottom	0.30
	1:2 sides, straight channel	0.24
	1:2 sides, curved channel	0.30
Safe design	Channel bottom	0.36
	1:2 sides, straight channel	0.30
	1:2 sides, curved channel	0.36

These figures are for a rip-rap layer of thickness $T = D_{100}$ or $1.5D_{50}$, whichever is the greater. Increasing the thickness to $T = 1.33D_{100}$ reduces the value of C for incipient failure on a 1:2 slope from C = 0.24 to 0.19.

When applying Equation (4), the CoE suggests that the velocity V can be estimated from Manning's equation using a roughness value of:

$$n = 0.045\, D_{90}^{1/6}, \text{ if } y/D_{90} \quad 10 \qquad \text{------ (5)}$$

or

$$n = \frac{R^{1/6}}{14.1 + 18.0 \log_{10} (R/D_{90})}, \text{ if } y/D_{90} \quad 10 \qquad \text{------ (6)}$$

where D_{90} and R are in m, and R is the hydraulic radius of the flow.

In order to continue its research on rip-rap, the CoE has recently commissioned a major new outdoor test facility at WES (see Figures 6.1 and 6.2). The facility features a trapezoidal channel with the following dimensions: maximum depth 0.91m; bottom width 3.66m; side slopes 1V:2H; overall length 238m; gradient 0.2 per cent. The maximum flow rate is $5.7m^3/s$, and the facility is suitable for testing rip-rap with a maximum size of about 63mm. Topics to be studied in the research programme include: effects of channel bends; effects on stability of gradation and rip-rap thickness; flow resistance of rip-rap; velocity distributions; composite channels with sand beds and protected sides.

6.3 Protection downstream of structures

6.3.1 Present design recommendations

Stone size

The design methods described in Section 2 are not appropriate for channels downstream of structures, such as weirs and stilling basins, which produce a high degree of flow turbulence. Recommendations on the sizing of rip-rap protection for these situations are given by the CoE in the document Hydraulic Design Criteria HDC 712-1 (1970). The minimum weight of stone, W_{50}, needed for stability is calculated from the Izbash (1970) formula

$$V = E\left[2g\,(s-1)\right]^{\frac{1}{2}} D_{50}^{\frac{1}{2}} \qquad \text{------ (7)}$$

where D_{50} is the diameter of the equivalent sphere calculated from

$$D_{50} = \left[6W_{50}/(\pi\gamma_s)\right]^{1/3} \qquad \text{------ (8)}$$

The following values of the coefficient E are used:

E = 0.86, for high turbulence (e.g. at stilling basins)
E = 1.20, for low turbulence (e.g. flood control channels)

The thickness of the rip-rap layer is recommended to be a minimum of 1.0 D_{100} for cases of low turbulence and 1.5 D_{100} for high turbulence. If the material is placed under water, the thickness is increased by 50%.

The United States Bureau of Reclamation (USBR) provides a graphical method in Engineering Monograph EM 25 (1963) for sizing rip-rap downstream of stilling bases. This Monograph is used worldwide. The design curve corresponds to the equation

$$V_b = 4.92\, D^{\frac{1}{2}} \qquad \text{------ (9)}$$

where V_b is the bottom velocity in m/s and D is the particle diameter in m; the specific gravity of the stone is assumed to be s = 2.65. Comparison with Equation (7) shows that Equation (9) corresponds to a value of E = 0.86, as used by the CoE for cases of high turbulence. Although Equation (9) is based on the bottom velocity, EM 25 suggests that it be used with the average velocity at the end sill of the stilling basin (i.e. discharge/flow area at end sill). The particle size D is not precisely defined, but EM 25 recommends that most of the rip-rap should consist of stones whose size conform to the design curve given by Equation (9); the thickness of the rip-rap layer should be at least 1.5D.

Grading

In the CoE method, the grading of the rip-rap is required to fall between upper and lower bands based on the minimum W_{50} weight as determined from Equation (7). These bands are:

Upper	Lower
W_{50}, based on economic size and requirements for layer thickness	$W_{50} \geqslant W_{50,min}$
$W_{100} \leqslant 5\ W_{50,min}$	$W_{100} \geqslant 2\ W_{50,min}$
$W_{15} \leqslant W_{100}$ for filter	$W_{15} \geqslant$ upper $W_{100}/16$

Also the bulk volume of stone lighter than the W_{15} size should not exceed the volume of voids in the material without this lighter stone.

In the Lower Mississippi Valley Division (LMVD) of the CoE, it was found that use of these guidelines led to a large number of rip-rap gradings being specified. These increased the cost of production at the quarries because the screens had to be changed in order to produce each grading, even though the differences between them were sometimes relatively small. By making minor variations to the normal grading requirements, LMVD engineers (1982) found that it was possible to produce a set of standardised grading bands for rip-rap used at structures. These bands are given in Table 6.1, and the corresponding flow velocities for high and low turbulence are calculated from the Izbash formula, Equation (7); the stone weights were calculated assuming a minimum specific gravity of s = 2.49.

6.3.2 Prototype experience

Filter

Apparent failures of rip-rap have occurred at some CoE projects. At Steele Bayou Outlet Channel, for example, 12m of scour unexpectedly developed downstream of a fairly low-head gated structure. This was found to be due to a sink hole in the rip-rap which was possibly caused by bed material being drawn out through the filter layer. Filters beneath rip-rap are normally designed using the well-known Terzaghi filter criteria. (The CoE seldom uses grouted rip-rap because this has been found to fail by undermining).

USBR experience with filters and gradings

The USBR, like the CoE, has experienced problems with rip-rap protection which may have been due to inadequacies in the underlying filter rather than in the size of stone in the surface armour layer. A non-standard type of filter has been successfully used in conjunction with rip-rap for a major protection scheme on the Columbia River designed by the Canals and Diversion Structures Division of USBR. Approximately 10km of the river downstream of Grand Coulee Dam required protection from high velocity turbulent flows, and from the effects of rapid changes in water level associated with power rejections at the hydro-electric plant. The river bed is sandy with some clay, and the banks are up to 23m high with side-slopes varying from 4H:1V to 6H:1V. The maximum discharge is about 11 300m^3/s and flow velocities are in the range 1.8 to 3.3m/s. After comparing various guidelines, it was decided to use a rip-rap with a minimum D_{50} size of 0.61m ($W_{50,min}$ = 318kg) and the following grading bands:

Upper	Lower
$W_{50} = 2.57\ W_{50,min}$	$W_{50} = W_{50,min}$
$W_{100} = 6.07\ W_{50,min}$	$W_{100} = 2.57\ W_{50,min}$
$W_{15} = 0.50\ W_{50,min}$	$W_{15} = 0.13\ W_{50,min}$

The thickness of the rip-rap layer was 1.22m (i.e. $2D_{50}$). Beneath it was placed a filter layer with a minimum thickness of 0.46m, and consisting only of material coarser than 76 mm; sand was omitted because it was considered to have been a cause of problems in previous schemes. The protection system has performed satisfactorily since being installed in 1985.

6.4 Protection of steep channels

Most of the design methods for rip-rap described in Sections 6.2 and 6.3 are not suitable for determining the size of material needed to protect the downstream face of an overtopped embankment. This is because, in the latter case, the component of gravity acting down the slope has a significant effect on the stability of the rip-rap.

6.4.1 Presently available design methods

Formulae for determining sizes of stable stone in steep channels or rock weirs have been developed by Olivier (1973), Hartung and Scheuerlein (1970), Stephenson (1979) and Smith (1986); the Safety Factors Method (SFM) mentioned in Section 6.2 is general in its scope, and so can be applied to steep channels. Air entrainment occurs in high-velocity flows down steep channels, but may not be reproduced correctly in model tests at reduced scales. The only formula of those cited which takes account of air entrainment is the one due to Hartung and Scheuerlein (1970); Knauss [1979] compared it with Olivier's equation, and showed that aeration has the effect of increasing the stability of the stones.

6.4.2 Present research

Materials for protecting the downstrean face of an overtopped embankment have been compared qualitatively in a model study carried out by the USBR at a scale of 1:15 (see Section 4.2; also Powledge and Dodge, 1985). The tests were performed in a 0.91m wide flume using embankment slopes of 6H:1V or 4H:1V and corresponding prototype unit overtopping discharges of 3.7 or $8.1 m^3/s/m$. One test was performed using model rip-rap simulating prototype material with sizes between 152mm and 610mm. On a 6H:1V slope at a unit discharge of $3.7 m^3/s/m$ (corresponding to a prototype velocity of about 9m/s), the rip-rap became fluidised and was eroded immediately. Other materials such as gabions and rock mattresses provided considerably better protection under similar or more adverse conditions.

A laboratory study has been carried out by Colorado State University (CSU) for the US Nuclear Regulatory Commission to determine the size of rip-rap needed to stabilise and protect waste mounds from shallow overtopping flows. Results of Phase I of the study are given by Abt et al (1987 a, b) and preliminary findings from Phase II were described during a visit by members of the OSTEM team to CSU.

The tests were carried out at full scale in two flumes 2.4 m and 6.1 m wide. The embankments were formed from compacted sand and protected by a geofabric, followed by a 152mm thick sand/gravel filter beneath the rip-rap cover layer. The types of rip-rap tested had D_{50} sizes of 26, 56, 104, 130

and 157mm, and embankment slopes were varied from 1% to 20%. Discharge was gradually increased until failure of the rip-rap occurred, this being defined as exposure of the underlying filter layer or geofabric. It was found that the unit discharge at which stones first started to move was 73%-79% of the unit discharge at failure, q_f.

Comparison of the results with Stephenson's (1979) formula showed that the latter did not give safe estimates of rip-rap stability for channel slopes less than 10%; the Safety Factors Method gave conservative predictions for all the slopes tested. A provisional best-fit equation to the CSU data for the unit discharge at failure is

$$D_{50} = 0.503\ S^{0.43} q_f^{\ 0.56} \qquad \text{------ (10)}$$

where S is the channel slope, and D_{50} is in m and q_f in $m^3/s/m$.

Most of the Phase I tests were carried out with the rip-rap layer having a thickness of $3D_{50}$ (except for the 130mm and 157mm materials for which the thickness was 305mm); increasing the layer thickness was found to increase the stability, particularly in the case of the smaller materials. Tests with types of rip-rap having the same D_{50} size but different grading curves showed that the failure discharge decreased as the coefficient of uniformity (D_{60}/D_{10}) was increased.

The results were also analysed to determine values of the Manning resistance coefficient for the rip-rap. It was found that the value of n varied with the slope of the channel, and the provisional best fit equation to the data is:

$$n = 0.0818\ (D_{50}\ S)^{0.159} \qquad \text{------ (11)}$$

where D_{50} is in m.

Another useful parameter measured in the CSU tests was the interstitial velocity V_i of the water flowing within the rip-rap. This is important when considering possible scour of the underlying material. All the sizes of rip-rap tested had values of porosity within the range 0.44-0.46. The provisional best-fit equation for V_i is:

$$V_i = 0.232\ (g\ D_{10} S)^{\frac{1}{2}} \qquad \text{------ (12)}$$

where g is the acceleration due to gravity. The interstitial velocity was found to be related better to the D_{10} size of the material than to the D_{50} size.

Model tests on materials for protecting earth dams were reported by El-Khashab et al (1987) at the ASCE conference in Williamsburg. Results for dumped stone having D_{50} sizes of 12mm to 35mm on slopes varying from 1/5 to 1/30 were used to calculate values of the empirical coefficient in Olivier's stability equation. The thickness of the rock layer was expressed in terms of the number N defined by

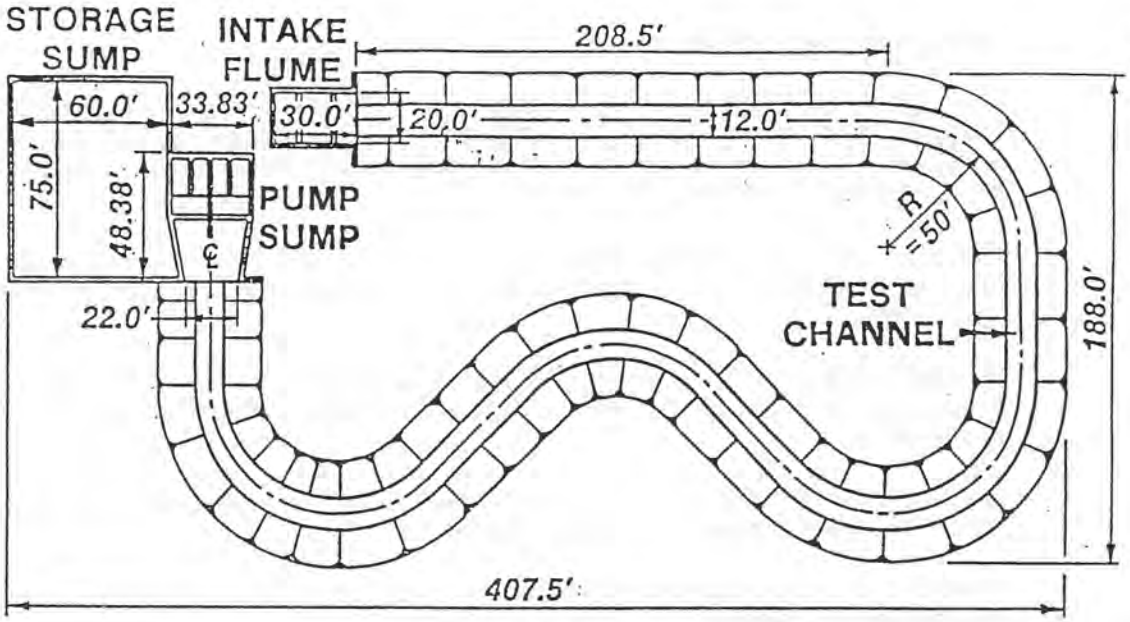
$$N = 1.33\ w\ (\gamma_s^2 W)^{-1/3} \qquad \text{------ (13)}$$

where W is the weight of a stone and w is the weight/unit area of the protective layer. It was found that the unit discharge at failure, q_f, increased with N until becoming constant for values of $N > 2.5$. The results also confirmed the relationship $q_f \propto S^{-7/6}$ given by Olivier's equation.

Table 6.1 Standard rip-rap gradations

	Gradation normally produced manually								Gradations normally requiring special handling				
Layer thickness in high turbulent flow (inches)	12	15	18	21	24	30	36	42	48	54	63	72	81
Layer thickness in low turbulent flow (inches)			12	14	16	20	24	28	32	36	42	48	54
Percent lighter by weight													
100	25 10	50 20	90 40	140 60	200 80	400 160	650 260	1000 400	1500 600	2200 900	3500 1400	5000 2000	7400 3000
50	10 5	20 10	40 20	60 30	80 40	160 80	280 130	430 200	650 300	930 440	1500 700	2200 1000	3100 1500
15	5 2	10 5	20 5	30 10	40 10	80 30	130 40	210 60	330 100	460 130	700 200	1100 300	1500 500
Flow velocity (ft/s)													
High turbulence	5.3	5.9	6.7	7.1	7.5	8.4	9.1	9.8	10.5	11.2	12.1	12.8	13.7
Low turbulence			9.3	10.0	10.4	11.7	12.7	13.7	14.6	15.6	16.8	17.9	19.1

Notes:
1. Data provided by Lower Mississippi Valley Division, Corps of Engineers; Section 6.3 refers
2. Specific weight of stones 155 lb/ft^3
3. * Weights tabulated in pounds



Source: US Army Corps of Engineers, Vicksburg

Figure 6.1 Plan of rip-rap test facility at Waterways Experiment Station

Figure 6.2 General view of rip-rap test facility

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THE ROLE OF CONCRETE BLOCK PROTECTION

C Tuxford - Ardon International Ltd
representing the UK concrete block revetment manufacturers

7.1 Introduction

This section reviews the present state of design practice and development of concrete block systems in the US for protection against erosion by current action, principally as a result of overtopping.

Although significant use has been made of concrete blocks in low-cost coast protection works (Corps of Engineers, 1981a) and in river bank protection, relatively little use has been made in the US of concrete blocks as protection against embankment overtopping or in spillways and other high velocity applications.

In river bank protection, use has been made of both proprietary cellular pre-cast concrete blocks (such as Armorflex and Petraflex) and site-manufactured solid concrete blocks. Use of these systems was reviewed by the Corps of Engineers under the Streambank Protection Programme (Corps of Engineers, 1981b) but neither type of system was considered sufficiently new or low-cost for any significant field evaluation to be carried out under the later phases of the programme (see Section 1.4).

The use of solid concrete blocks as river bank protection has been developed on a massive scale on the lower Mississippi River where the so-called Articulated Concrete Mattress (ACM) is used.

Proprietary systems of concrete block protection have been used in spillways on urban flood detention basins and in flood channels. The potential use of concrete blocks as low-cost protection against overtopping on existing dams was identified by the Federal Emergency Management Agency in 1983 (see Section 2.5). It is possible that poor performance of concrete block systems in coastal and river bank applications due to misapplication and poor design/construction has deterred the US dam engineer from using or adopting such systems, however probably the principal constraint on the use of concrete blocks has been lack of information on prototype performance.

Both the technical press (Birchall and Pinyan, 1986) and the ASCE Task Committee took particular interest in the full-scale field trials of reinforced grass systems carried out at Jackhouse reservoir by CIRIA in 1986. (These trials included systems utilising proprietary cellular blocks and cast in-situ concrete). The USBR subsequently decided to include full-scale tests on concrete block systems in their joint study with the Federal Highways Administration (see Section 4.4 of this Report), and one reason for the Mission was to continue the exchange of information on these parallel R&D programmes in the US and UK.

Clearly this USBR/FHA research is spearheading the development of a design philosophy in the US, and this should help provide the impetus in the US to generate a wider acceptance amongst engineers of the potential value of concrete blocks as protection against overtopping of dam and flood embankments. Considerable interest exists in the US among all sectors of the construction industry in exploring the use of concrete blocks in erosion control engineering, but it is interesting to note that at present there is no forum for manufacturers, engineers and clients to work together in such research and development.

Available systems

Proprietary revetment block systems most commonly used in the US are Armorflex and Petraflex (both cable-tied) and Tri-lock (interlocking), all of which are also available in the UK. Several forms of cast in-situ cellular concrete (similar to UK Grasscrete) and grassed paving blocks are also available, but these tend to be principally used for accessways, parking areas etc. The Terrafix revetment system which was developed in Germany has also been marketed in the US. Methods of manufacture are not considered to differ significantly from these adopted in the UK.

7.2 Mission objectives related to concrete block protection

The particular objectives of the author within the framework of the mission were as follows:

1. to identify the interaction between manufacturers and the design engineer during the development of a design incorporating concrete block revetment

2. to establish the extent of the manufacturers' codes and established practice relating to the design of concrete block revetments used in protection of dams and river flood banks against erosion by overtopping flow

3. to identify the construction procedures adopted by contractors for installing concrete block protection systems.

Information relating to these objectives was obtained by means of (a) meetings involving the full mission team and our hosts (see Appendix A); (b) additional meetings with smaller groups and individuals; (c) a half-day seminar in Denver to which US concrete block manufacturers, consultants and contractors had been invited by the mission team; (d) feedback through the ASCE National Hydraulics Conference at Williamsburg. It must by emphasised that with the diversity of civil engineering practice throughout the different States, and in the public and private sectors, it was not possible to obtain a comprehensive overview within the time available.

7.3 Concrete block protection against overtopping

7.3.1 UK background

Before reviewing present US practice and development, it is worth noting how the concrete block revetment industry operates within the construction industry in the UK.

(i) In general, the manufacturing industry developed the market itself initially. The general concepts of good civil engineering practice which were adopted have subsequently been refined taking into account the results of research programmes such as the CIRIA project on reinforced grass.

(ii) Because the majority of civil engineering applications for concrete block revetments are related to public-sector clients, good liaison has developed between the manufacturer and the UK consultant, the latter being frequently responsible for design.

(iii) Within the UK, one concrete block revetment manufacturer has, as part of its company, an Installation Team; others employ or have close links with specialist installers.

The above means that the UK manufacturer can be closely involved in a project from inception of design through to installation and completion of construction. In other words, involvement of manufacturers does not cease after winning the order for providing the concrete blocks.

(iv) A number of manufacturers have supported or commissioned research with UK research organisations. Data obtained from these research programmes is made available for the design engineer to appraise.

(v) Manufacturers also have developed their own design procedures for particular applications of revetment systems, and the design engineer may analyse and assess such procedures during the design process.

(vi) The formation of the Flexible Revetment Association among major manufacturers of concrete block and asphalt-based products has helped to provide a focal point for the promotion of manufacturers' views, in addition to providing a mechanism whereby research can be commissioned to promote the use of concrete block protection generally.

One major reason for this strong involvement by manufacturers is their recognition of the fact that the adequacy of the composite protection system depends on the design engineer taking due consideration of the interaction of the different component parts (e.g. concrete blocks, underlayer material, subsoil, anchorage, cabling, gravel blinding, grass etc.). It is in manufacturers' interests generally to ensure that their product is properly used, and that failures through incorrect use are minimised.

7.3.2 US practice

At a national level, Government agencies charged with the design of public projects have been cautious in their response to using proprietary concrete block systems. This is partly because the scale and strategic importance of much of the work undertaken by major agencies such as the USBR and CoE precludes the use of these systems. Other reasons mentioned for slow take-up of the product were:-

- installed cost, particularly at remote sites, can be high in relation to alternative forms of protection (see Section 2.4.2 of Report)

- difficulties in forming close liaison between the public sector and industry for developing appropriate applications and practice.

- lack of design guidance and prototype performance data in relation to alternative forms of protection (reinforced concrete, roller compacted concrete and rip-rap)

- lack of basic research on factors affecting the performance of concrete block protection systems.

The problem of lack of information will be helped by the report of the ASCE Task Committee which will include an assessment of the USBR/FHA full-scale tests (Section 7.3.3) and other available data.

From discussions with manufacturers and suppliers, it is understood that concrete block protection has been used in small spillway and flood channel installations, but that use is only localised in particular States and areas where manufacturing outlets exist. It was clear that one factor which has limited utilisation nationally in the US were the greater distances between

the site and the production centre as compared with the UK. Only one design engineer whom the Mission met had recent experience of design of concrete block protection systems for dam overtopping (France, 1987), however several delegates at the ASCE National Hydraulics Conference were currently considering use of concrete blocks for overtopping protection.

It was apparent from discussions held during the Mission that no design codes or established practices had been developed specifically for concrete block protection systems. Development of the design of the composite protection system for any specific application was left to the design engineer, and manufacturers saw their responsibility principally as suppliers of a product. No details of design practice were obtained other than the general principles described in Section 2.4 of this Report.

7.3.3 Research into performance of concrete block systems

As explained in Section 7.1, evaluation of the performance of cable-tied concrete block protection systems has been included in the full-scale test programme currently being undertaken by the USBR and FHA (see Section 4.4 of this Report). The test programme has examined the performance of concrete blocks as protection to an embankment constructed of low-plasticity material having a low resistance to erosion. Suppliers of the concrete block systems under test were invited by the USBR to submit details of any requirements for the underlayer or anchorage, and the systems have been installed accordingly.

Testing of the concrete block systems was not carried out until September 1987 and consequently the Mission team was unable to obtain information at first hand. The report on the tests is expected in 1988, however initial results indicate that within the range of concrete block systems tested, one has performed significantly better than the others. Analysis of the data is currently underway, and available results of the tests to date will be given at the seminar (see Appendix B).

A description of both phases of the test programme has been presented by the current researchers (Clopper and Chen, 1987). They appear to be continuing along the same analytical route developed by Chen and Anderson (1986) for the first phase of the study (Section 4.3 of this Report). This identified critical velocity and critical shear stress for the different types of protection system. It therefore seems likely that the researchers will seek to identify similar critical parameters for concrete blocks in their report to the USBR and FHA.

7.3.4 Differences between UK and US approaches

Conclusions of the ASCE Task Committee and the Phase I full scale test programme have been reported in Sections 2 and 4. Key conclusions having a bearing on the performance of concrete block systems are:-

(a) Cohesive embankment materials are much more resistant to erosion than non-cohesive soil.
(b) Exposure of a non-cohesive soil in an eroding surface within a cohesive soil embankment will accelerate the erosion rate.
(c) Initiation of erosion is encouraged by surface features that cause a disturbance in the flow.
(d) Seepage exiting on the surface of a permeable granular embankment accelerates the erosion rate.

It must be recognised that the key difference between the present full-scale tests in the US and the CIRIA work on reinforced grass relates to the fact that the USBR/FHA tests are looking at systems (and latterly concrete blocks) used as protection on ungrassed slopes and a highly erodible soil. The CIRIA work relates specifically to the use of concrete block protection with low permeability clay soils, and where the concrete blocks, underlayer and subsoil have become totally integrated by the action of the grass roots.

Differences between the approaches developed in the UK and US may well highlight the desirability or otherwise of incorporating a drainage medium below the concrete blocks and the effect of the subsoil material on the performance of the protection system. In the CIRIA field trial, although a regulating layer was laid underneath the block systems, this comprised a low permeability (14mm to dust) limestone aggregate. The minimum amount of material was used commensurate with forming the correct profiles. It was also concluded that the inclusion of a permanent drainage medium underneath the blocks would not have benefited the objective of obtaining deep root penetration which has been clearly demonstrated to provide additional shear and pull-out resistance over and above that expected from a concrete block system on its own (Hewlett et al, 1987).

Conclusion (d) above relates to embankments with moderate to high permeability. The difficulty facing the researchers analysing the results of the Phase II tests may well be in drawing appropriate conclusions for different types of subsoil. The current tests in the US on concrete blocks should nevertheless enable some assessment of the effect of incorporating a drainage medium into the protection design. The drainage medium used is a 3-dimensional mesh which is able to intercept seepage flow through the embankment and transmit flow underneath the blocks.

The US full-scale tests on concrete blocks will also provide further information on the comparative performance of single cable and dual cable concrete block systems, and the filtering/erosion protection performance of the geotextile underneath the concrete blocks.

It is inappropriate to speculate further on the conclusions of the researchers until they have had an opportunity to analyse and present their results in a considered manner. It is likely that future assessment of the CIRIA field trials data, the US full-scale test data, and the SERC-funded work being carried out by Baker at University of Salford on factors affecting the stability of block systems will confirm that different approaches are appropriate to grassed and ungrassed protection systems, and to different subsoil types. This would be somewhat similar to the different design approaches identified for blockwork protection systems against wave attack (PIANC, 1986).

7.3.5 Future development

Interest was identified among members of the ASCE Task Committee in carrying out an evaluation of the Russian wedge-shaped block system which appears to have merits for the protection of ungrassed slopes, particularly as regards evacuation of seepage flow to control potential uplift problems. Details are provided in Appendix 7.1 and Section 2.5 of this Report.

It was however felt that this future evaluation should not detract from the more immediate objectives of clarifying good practice for the use of conventional proprietary block systems.

7.4 Articulated Concrete Mattress - Lower Mississippi River

A site visit was made to the Mississippi River to inspect the major use being made by the Corps of Engineers of concrete blocks for bank protection. This was not central to the objectives of the Mission, but the scale of the operation makes it worth reporting.

The Articulated Concrete Mattress (ACM) is but one method adopted in the overall strategy of maintaining channels within the river which are able to carry flood flows efficiently and remain suitable for navigation. In the 1930s and early 1940s new channels were formed between long loops of river to carry floodwaters out of the flood plain faster and at lower flood heights. Other methods have included dredging, and the construction of dykes to help the river hold the proper alignment. The river banks are stabilised with the ACM.

The ACM consists of individual concrete blocks 1.5' x 4' x 3" which are wet cast in rows of 13 blocks, producing a single panel 25'x 4' weighing 900 kg and giving a surcharge of 120 kg/m^2. Wire of diameter 3/16" is embedded in the concrete (25 000 psi at 28 days) and this holds the panel together. This wire is not used to lift the panels as they are lifted by special grabs in panel form which keep the panels in a flat condition. The system is lifted flat and laid flat.

The system is manufactured and stored at strategic locations along the river, and barged from the casting areas to the installation sites. The scale of the operation is enormous. Each year, the Corps of Engineers invite manufacturers to tender for the production of the panels. The casting yard visited by the Mission team contained hundreds of panels stored in 12 panel stacks.

The sequence of operations is as follows:

- draglines are used to profile the bank to a stable slope from the top of the existing bank to well below the water surface. This profile usually has a 1:4 or 1:5 slope
- the panels are barged onto a massive floating launching barge (the mat barge) some 140' wide, on which the panels are connected together to form a continuous protection system, 125' wide by 200'-500' long
- the mattress is anchored on the river bank using 12" x 12" steel plate anchors driven 5' to 6' into the slope. The launching barge moves out into the river alongside a mooring barge, and the whole mattress slides over the edge onto the slope and bed of the river. One anchor is used per 8' mattress width for mattress lengths 75' or less; for mattress lenghts over 75', one anchor is used per 4' mattress width (1 block width)
- when the required protection has been laid the launching cables are cut and the whole operation is moved upstream to restart the sequence.

Within the Mississippi, the water levels vary by between 8' and 10' per annum, there is no wave action, and current speeds are the order of 8'/sec. Where necessary the mattress is lapped to provide continuity of protection, and the overlap can be from 5' to 15' of mattress. The upper banks of the river are usually protected with suitable sized rip-rap.

The lower Mississippi ACM must be , without question, the most extensively used revetment system anywhere, tens of thousands of square metres being produced and laid each year.

It was not possible to establish any particular design procedure for the ACM application on the banks of the Mississippi. The site which the Mission team visited had severe distortion of the ACM from the original profile, but this did not give cause for concern. In general, local failures of this type were left until it was considered necessary to place new panels on top of the existing ones. This approach would no doubt be considered in a different light in the UK, where the majority of projects by comparison are miniscule!

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EXISTING EMBANKMENT EROSION EQUATIONS

Developer	Equation	Comments
1. Wiggert & Contractor[21]	$E = \alpha V^{\beta}$	This equation was derived specifically for embankment erosion due to flood overtopping, where E = the erosion rate in tons/day/ft of the roadway and V = mean flow velocity on the downstream slope in ft/s. The given values of $\alpha = 0.25$ and $\beta = 3.8$ represent a compromise between cohesive and noncohesive soils.
2. Cristofano[22]	$\frac{Q_s}{Q_w} = K e^{-x}$	This equation computes rate of erosion for earth dam failures due to overtopping, where Q_s = erosion rate, Q_w = overtopping flow discharge, K = constant, $x = (b/H) \tan \phi_d$, b = base length of the breach, H = hydraulic head and ϕ_d = angle of friction.
3. Ariathurai and Arulanandan[23]	$E = M \left(\frac{\tau}{\tau_c} - 1\right)$	This equation computes erosion of cohesive soil, where M = erosion rate constant, ranging from 0.00012 to 0.0012 lb/ft^2/s; τ = shear shear stress, and τ_c = critical shear stress.
4. Chee[24]	$\frac{q_s}{q_c} = \left(\frac{1}{60}\right) K_1 K_2 K_3 K_4 \left(g \frac{D-0.5H}{q_c^2}\right)^{1/20} [1/(S_s-1)]^2 (D/d)^{1/8}$	This equation computes erosion rates for erodible fuse-plug dams, where q_s = erosion rate per unit width; D = water depth upstream of the dam; q_c = critical water discharge per unit width for D, height of dam, d = mean grain size, S_s = specific gravity of grain, and K = coefficients.
5. Agricultural Research Lab	$E = K (\tau - \tau_c)^a$	This equation computes detachment rate for erosion of cohesive soils.

(21) J. M. Wiggert, and D. N. Contractor, "A Methodology for Estimating Embankment Failure," an unpublished paper presented to Water Resources Engineers, Inc., Springfield, VA, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA 24060, no date, written around 1969.

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(23) R. Ariathurai, and K. Arulanandan, "Erosion Rates of Cohesive Soils," Journal of Hydraulics Division, ASCE, Vol. 104, No. HY2, February 1978, pp. 279-283.

(24) S. P. Chee, "Design of Erodible Dams," in Proceedings of International Conference of Water Resources Engineerings, Asian Institute of Technology, Bankok AIT, V. 1, 105-113, 1978.

Source: Chen and Anderson (1986)

DEVELOPMENT OF USBR NUMERICAL MODEL FOR EMBANKMENT EROSION

A computer model for simulating or predicting erosion of alluvial materials is being developed by Dr T Yang of the USBR. The original aim was to determine the rate and extent of erosion on the downstream face of an overtopped embankment, but the model is general in concept and is capable of being applied to a wide range of problems, such as for example local scour around a cofferdam in a river.

The model is called GSTAR (Generalised Stream Tube computer model for Alluvial River simulation), and is based on the theory that a natural channel will tend to adjust its width, depth and slope so as to minimise the rate of energy dissipation. Linking suitable equations for flow resistance and sediment transport and applying this principle, leads to a morphological model that is capable of predicting how the breaching of an overtopped embankment dam might develop with time.

Significant features of GSTAR are that it can take account of variations in grading of the alluvial material with position and depth, and that (unlike previous morphological models) it allows the channel width to vary as well as the depth and the slope. Three-dimensional situations are dealt with by using a set of two-dimensional stream tubes to determine the fluid dynamic behaviour of the flow; changes in channel shape from one time step to the next can then be found by applying the quasi-one-dimensional morphological model along each stream tube. The way in which the channel evolves is found by identifying, after each time step, the new combination of width and depth which will result in the minimum rate of energy dissipation.

Independent trials of GSTAR are currently being carried out for the USBR by the University of Minnesota to compare predicted and measured amounts of erosion at a dam in New Mexico that was overtopped. In order to provide an unbiased test of the model, the measured data have not been made available to the team at the University of Minnesota.

R.W.P.May

FLOODS AND DAM FAILURES IN MICHIGAN, SEPTEMBER 1986

The Corps of Engineers has investigated several dam failures in Michigan, when a severe storm of 8 - 13 in (200 - 300mm) rainfall in less than 48 hours occurred. It was stated that 5in/24 hrs represented a 1 in 100 year frequency. 20 dams were overtopped and 11 failed. They were mainly low dams for water supply, small hydro-electric power, and recreation. Details on some of the failures are as follows: (Imperial units have been used).

Hart Dam R = 10 in in 48 hrs
Capacity 3600 acre ft; 31 ft. high and 600 ft. long: 1927. No auxiliary spillway. On 11 Sept. a decision was made to breach the dam to release water and save the power house. Manual gates controlled the service spillway. A cut 50 ft. wide x 4 ft. deep was made. In 45 minutes there was a 200 ft. breach and after 8½ hrs there was a total breach.

White Cloud Dam - recreation reservoir. R = 10 in in 48 hrs
Capacity 475 acre ft; 1880 original timber crib design. Asphalt road on crest. No auxiliary spillway. Service spillway - wooden sluice gates. Sandbags used to raise crest of dam
2.5 ft. x 4 ft. wide breach cut - in 4½ hrs widened to 100 ft.; then 125 ft. wide by 20 ft. deep in sandy gravelly silty material.

Denham Lake Dam - recreation reservoir. 25/30 yrs old.
Capacity 200 acre ft. Built over old timber crib structure. Seepage had always been reported emerging at 1/3rd up the downstream face. Failed at 1.30 a.m. unseen. Assumed failed by excessive seepage rather than overtopping. Sand fill with no core.

Luther Dam
Capacity 65 acre ft, originally built in 1880 - washed out in 1910. 10 - 12 ft. high. Downstream slope 1 in 1. 350 ft. long - sandy clay material. Failed at 2 - 4 a.m. unseen.
80 ft. long length washed out.

Berrington Dam R = 10 in in 48 hrs.
Originally built in 1880 - formerly a power dam.
Damaged in 1976 and repaired.
Service spillway 4 x 3 ft. dia. hand controlled gates (one jammed).
Dam 6 - 8 ft. high; 500 ft. long. Downstream slope 1 in 1
Overtopped for 30 hours with 9 + in. and then deliberately breached - 30 ft. wide breach but not completely washed away.

Rainbow Lake Dam - privately owned recreation reservoir - 25 years old. Capacity 5400 acre ft.; 46 ft. high; 1 in 4 slopes. No auxiliary spillway; flood gate capacity 300 cusecs. Longest and most well constructed dam of those inspected by CoE.
Overtopped for 12 - 13 hours with head of 1.5 to 2.0 ft.
Water channelled down a service road leading to erosion and eventually breached after 5 hrs.

Charles Dam - Industrial water supply for a paper mill. 20 ft. high with 1 in 2 downstream slope. 80% of structure lost. There was an auxiliary spillway which was overtopped and water entered fill against concrete wing wall. In 20 minutes failure occurred due to piping and erosion and some overtopping.

The CoE conclusions from these studies include:

1. with no auxiliary spillway or with an inadequate spillway, an earth embankment will fail if overtopped long enough.

2. A discontinuity increases the risk of erosion and therefore failure.

A video is available of the cutting of the breach and failure of Hart Dam.

A paper is to be written on these failures by P.A. Gilbert (Corps of Engineers) in the near future.

LITERATURE ON SEABEES

SPECIFY PGH

SEABEES

WINNER OF THE A.B.C. INVENTORS AWARD
(Australian patent pending)

FOR
- COASTAL REVETMENTS
- SEA WALLS
- RETAINING WALLS
- WAVE ABATEMENT
- CURRENT ABATEMENT
- STABILISING EARTH BANKS AGAINST RUN-OFF
- FARM DAMS AND SPILLWAYS

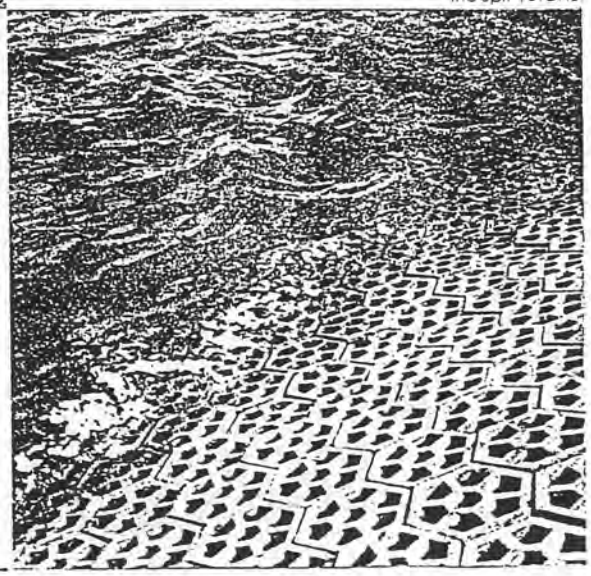
"North Haven", ADELAIDE

SAVE MONEY:

- PGH Seabees save labour and reduce costs because they are self-locating and quick to lay.
- PGH Seabees cost less because they use less material than other available products.
- PGH Seabees require very little surface preparation before laying.

PGH Seabees are made from the same material as vitrified clay sewer pipes which have a recognised service life of at least 150 years.

"The Spit", SYDNEY



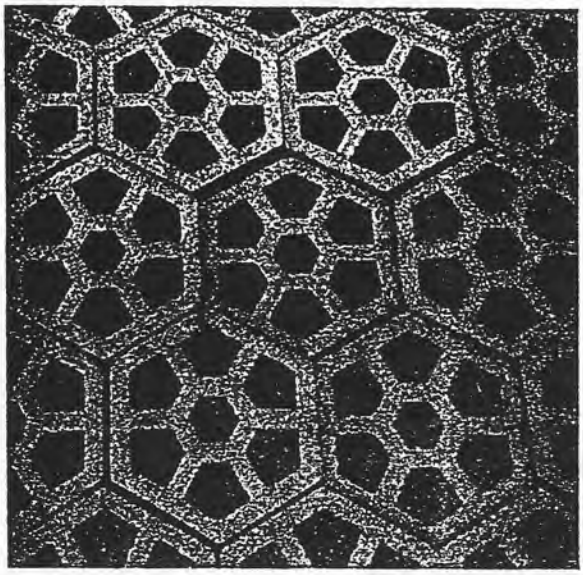
MAINTENANCE FREE:

The inherent qualities of vitrified clay make PGH Seabees highly resistant to:
- Salt crystal attack.
- Chemical attack.
- Weathering.

PGH CERAMICS PIPES N.S.W.

SEABEES

CI/SfB	
	Ng3



SAFE AND HEALTHY:

- PGH Seabees are vermin-proof, thus reducing health hazards.
- PGH Seabees are safe to walk on.
- PGH Seabees are more appealing to the eye than other products.

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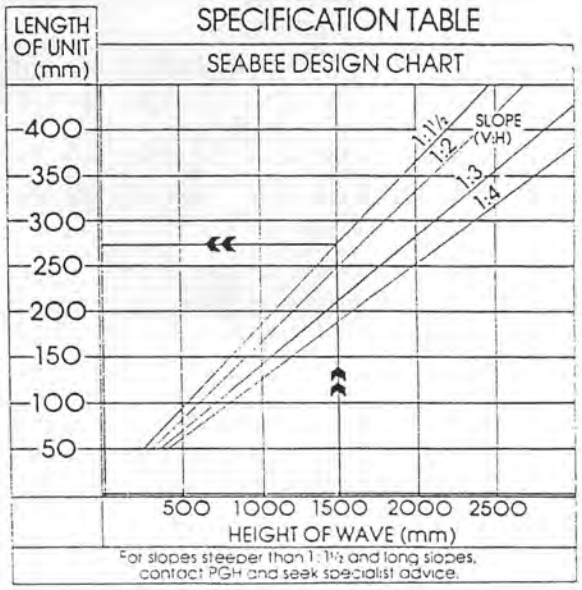

Extract from Hansard: — COASTAL PROTECTION BILL, 20th February, 1979.
Second reading

The Right Hon. Mr. L. S. Ferguson, Deputy Premier, Minister for Public Works and Minister for Ports.

"... The honourable member for Gosford will not need me to remind him of the recent heavy seas at Wamberal, where houses fell into the ocean ... the coastline is receding at a rate of 1.5 metres a year at Byron Bay and 2 metres a year at New Brighton ... The study suggest that $14 million worth of public and private assets on current values could be lost to the ocean over the next 50 years as a result of erosion ... and can be expected to continue."

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SAFETY OF EXISTING DAMS - EVALUATION AND IMPROVEMENT
book published by National Academy Press, Washington, D.C., 1983.

This 354 page book was prepared by a large number of civil engineers, representatives of State agencies responsible for dam safety, private corporate dam owners, geologists, hydraulic engineers, risk analysts and others knowledgeable about Federal and State dam safety programmes, including the Federal Emergency Management Agency.

The committee panels of eminent engineers and others covered Risk Assessment; Hydraulic & Hydrologic considerations; Concrete Dams; Embankment Dams; Instrumentation and Geological & Seismological considerations.

The list of chapter headings shows the range of this extremely valuable and interesting book:

The safety of dams
Risk-based decision analysis
Hydrologic and hydraulic considertions
Geologic and seismological considerations
Concrete and masonry dams
Embankment dams
Appertenant structures
Reservoir problems
Instrumentation
Glossary

Each chapter includes many references and a further list for recommended reading.

Numerous figures and tables are included. Amongst many very useful tables are ones on summary of selected dam break model capabilities; earth dam failures covering form, general characteristics, causes and preventative or corrective measures; required factors of safety for embankment dams; causes of deficient behaviour of instrumentation.

It is not practicable to adequately review this book other than to recommend it to all those concerned with dam safety. The problems, approaches and solutions in USA are naturally very similar to the same consideration of the subject in other countries. There is a considerable amount of interesting and valuable information and ideas contained within its covers.

SAFETY OF DAMS - FLOOD AND EARTHQUAKE CRITERIA

In 1985 a further 176 page book was published, in the same style, which covers flood and earthquake topics in greater detail.

The chapter headings are

- Extreme floods and earthquakes - the nature of the problem
- Summary of present practices on dam safety standards
- History of development of present practices
- Design Flood estimates: methods and critique
- Design earthquake estimates, methods and critique
- Consideration of risk in dam safety evaluations
- Risk and the calculus of legal liability in dam failures
- Proposed hydrologic criteria
- Proposed earthquake criteria
- Continuing development of hydrologic and earthquake engineering technologies

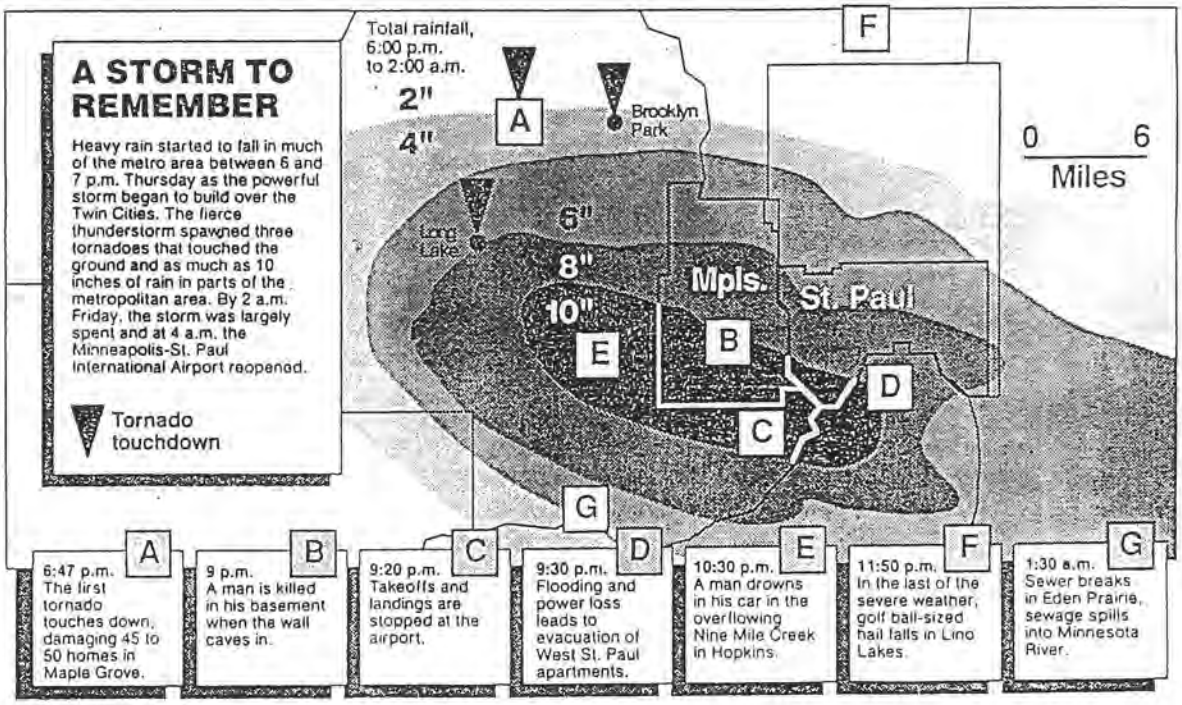
FLOODING IN MINNEAPOLIS, 23 JULY, 1987

During the period of the OSTEM visit, severe storms occurred over the urban areas of Minneapolis and St. Paul, Minnesota, when over 250mm (10 ins) of rain fell in under 10 hours. Only 2-3 days earlier over 100mm (4 ins) fell over substantially the same area. The ground was therefore saturated and run-off was rapid. Severe flooding occurred. It is not known if any reservoirs of flood detention ponds were overtopped or damaged.

Anisohetyl map, from the Minneapolis Star and Tribune of 25 July, is reproduced below.

The Minneapolis paper referred to the daily rainfall on 23 July of 232mm (9.15 ins) being a new daily record for that date surpassing the previous record for that date of 28mm (1.1 in) of 1918.

This storm appears to be about ¼ of the intensity of the maximum world rainfall (comparable to UK largest rainfalls) and serves to remind the dam engineer of the likelihood of extreme rainfall and severe flooding that can occur, especially in summer.



USBR DRAFT DESIGN STANDARD NO. 13, CHAPTER 20

GEOTEXTILES FOR EMBANKMENT DAMS

DRAFT

DESIGN STANDARDS

EMBANKMENT DAMS

NO. 13

GEOTEXTILES FOR EMBANKMENT DAMS CHAPTER 20

UNITED STATES
DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION
ENGINEERING AND RESEARCH CENTER
DENVER, COLORADO

Table 3. - Important criteria and properties - filtration and drainage applications

Criteria	Properties
Constructability	Thickness Weight Absorption (wet weight) Flexibility Tensile strength Puncture resistance Cutting resistance Seam strength Flammability Tear strength Ultraviolet stability
Durability	Chemical stability Biological stability Thermal stability Clogging resistance
Hydraulic	Thickness Permeability Piping resistance

All may not be important in every application.

Bell, J. R., R. G. Hicks, et al., "Evaluation of Test Methods and Use Criteria for Geotechnical Fabrics in Highway Applications," Oregon State University, Corvallis, Report No. FHWA/RD-80-021, 1980.

Table 4. - Important criteria and properties - erosion control applications

Criteria	Properties
Constructability	Thickness Weight Absorption (wet weight) Flexibility Tensile strength Puncture resistance Cutting resistance Seam strength Ultraviolet stability Tear strength Flammability
Permeability	Permeability Particle clogging resistance
Durability	Chemical stability Biological stability Thermal stability Abrasion resistance Animal, vegetable, and insect resistance Clogging resistance
Hydraulic	Permeability Piping resistance

All may not be important in every application.

Bell, J. R., R. G. Hicks, et al., "Evaluation of Test Methods and Use Criteria for Geotechnical Fabrics in Highway Applications," Oregon State University, Corvallis, Report No. FHWA/RD-80-021, 1980.

Table 8. - Retention criteria for geotextiles - one-direction flow conditions

Reference	Type of geotextile filter	Type of base soil	Retention criterion
Calhoun (1972)	Wovens	Granular soils	$O_f \leq D_{85}$
	Wovens	Cohesive soils	$O_f \leq 210$ m
Ragutzki (1973) Zitscher (1975)	Wovens	Sands	$O_f \leq 2.7 \cdot D_{50}$
Ogink (1975)	Wovens	Sands	$O_f \leq D_{90}$
	Nonwovens	Sands	$O_f \leq 1.8 \cdot D_{90}$
Cedergren (1977)	Wovens and nonwovens	Sands	$O_f \leq D_{85}$
Schober and Teindl (1979	Wovens Thin nonwovens	Sands	$O_f \leq B_1 (U) \cdot D_{50}$
	Thick nonwovens	Sands	$O_f \leq B_2 (U) \cdot D_{50}$
Giroud (1982)	Needle-punched nonwovens	Loose soils ($1 < U' < 3$)	$O_f < U \cdot D_{50}$
		Loose soils ($U' > 3$)	$O_f < \frac{9}{U'} \cdot D_{50}$
		Medium dense soils ($1 < U' < 3$)	$O_f < 1.5 \cdot U' \cdot D_{50}$
		Medium dense soils ($U' > 3$)	$O_f < \frac{1.35}{U'} \cdot D_{50}$
		Dense soils ($1 < U' < 3$)	$O_f < 2.U'.D_{50}$
		Dense soils ($U' > 3$)	$O_f < \frac{18}{U'} \cdot D_{50}$

Table 8. - Retention criteria for geotextiles - one-direction flow conditions - Continued

Reference	Type of geotextile filter	Type of base soil	Retention criterion
	Wovens and heat-bonded nonwovens	Soils with: $1 < U < 3$	$O_f < U' \cdot D_{50}$
		Soils with: $U > 3$	$O_f < \frac{9}{U'} \cdot D_{50}$
Heerten (1981) Heerten (1982)	Wovens and nonwovens	Granular soils ($U \geq 5$)	$O_f < 10 \cdot D_{50}$ $O_f \leq D_{90}$
	Wovens and nonwovens	Granular soils ($U < 5$)	$O_f < 2,5 \cdot D_{50}$ $O_f \leq D_{90}$
	Wovens and nonwovens	Cohesive soils	$O_f < 10 \cdot D_{50}$ $O_f \leq D_{90}$ $O_f \leq 100$ m
Loudiere et al. (1982) Loudiere et al. (1983)	Wovens and nonwovens	Granular soils ($U > 4$)	$O_f < D_{95}$
	Wovens and nonwovens	Granular soils ($U < 4$)	$O_f < 0.8 \cdot D_{50}$
	Wovens and nonwovens	Cohesive soils ($U > 4$)	$O_f < D_{85}$ $O_f \geq 50$ m
	Wovens and nonwovens	Cohesive soils ($U < 4$)	$O_f < 0.8 \cdot D_{50}$ $O_f \geq 50$ m

Table 8. - Retention criteria for geotextiles - one-direction flow conditions - Continued

Reference	Type of geotextile filter	Type of base soil	Retention criterion
C.F.G.G. (1983)	Wovens and nonwovens	Loose soils (U > 4)	$O_f < 0.8\ \gamma \cdot \delta\ D_{85}$
		Loose soils (U < 4)	$O_f < 0.6 \cdot \gamma \cdot \delta \cdot D_{85}$
		Dense soils (U > 4)	$O_f < 1.25 \cdot \gamma \cdot \delta \cdot D_{85}$
		Dense soils (U < 4)	$O_f < \gamma \cdot \delta \cdot D_{85}$
		Cohesive soils	Also: $O_f \geq 50$ m

Definitions:

O_f : Filtration opening size of the geotextile (in μm)
D_{50}, D_{85}, D_{90}: Particle size of the base soil (in μm)
U : Uniformity coefficient of the soil (D_{60}/D_{10})
U' : Linear uniformity coefficient of the soil (D_{60}/D'_{10})
$B_1(U)$, $B_2(U)$: Functions depending on uniformity coefficient of the soil (U)
γ : Parameter depending from the hydraulic gradient
δ : Parameter depending from the geotextile function

Table 9. - Permeability criterion for geotextiles - one-direction flow conditions

Reference	Geotextile characteristics	Earth structure	Permeability criterion
Schober and Teindl (1979) Heerten (1981)	• Compressed • After clogging occurred	Not specified	$k_g > k_s$
Giroud (1982)	• Compressed • After clogging occurred	Not specified	$k_g > 0{,}1 \cdot k_s$
Loudiere et al. (1983)	• Uncompressed • New	Earth dam	$k_g > 10^2 \cdot k_s$
C.F.G.G. (1983)	• Uncompressed • New	Earth dam	$= \frac{k_g}{T_g} > 10^5 \cdot k_s$

Definitions:

k_g : Normal permeability of the geotextile (in m/s)
T_g : Thickness of the geotextile (in m)
Ψ : Permittivity of the geotextile (in s^{-1})
k_s : Permeability of the soil (in m/s)

Table 10. - Summary of geotextile design and selection criteria for drainage, filtration, and erosion control applications

I. Soil Retention (Piping Resistance Criteria) 1/

Soils	Steady-state flow	Dynamic, pulsating, and cyclic flow
$\leq$50 percent passing 2/ U.S. No. 200 sieve	AOS $O_{95} \leq B\ D_{85}$	$O_{95} \leq D_{15}$ or $O_{50} \leq 0.5\ D_{85}$
	$C_u \leq 2$ or ≥ 8: B=1	
	$2 \leq C_u \leq 4$: $B=0.5\ C_u$	
	$4 < C_u < 8$: $B=\frac{8}{C_u}$	
$\geq$50 percent passing U.S. No. 200 sieve	Woven: $O_{95} \leq D_{85}$	$O_{50} \leq 0.5\ D_{85}$
	Nonwoven: $O_{95} \leq 1.8\ D_{85}$	
	AOS No. (fabric) $\geq$No. 50 sieve	

1. When the protected soil contains particles from 1 inch size to those passing the U.S. No. 200 sieve, use only the gradation of soil passing the U.S. No. 4 sieve in selecting the fabric.

2. Select fabric on the basis of largest opening value required (smallest AOS).

II. Permeability Criteria 1/

A. Critical/severe applications

k (fabric) $\geq$10 k (soil)

B. Less critical/less severe and (with clean medium to coarse sands and gravels)

k (fabric) $\geq$k (soil)

C. Additional qualifier (optional): $O_{95} > 2\ D_{15}$

1. Permeability should be based on the actual fabric open area available for flow. For example, if 50 percent of fabric area is to be covered by flat concrete blocks, the effective flow area is reduced by 50 percent.

Table 10. - Summary of geotextile design and selection criteria for drainage, filtration, and erosion control applications - Continued

III. Clogging Criteria

A. Critical/severe applications 1/

Select fabrics meeting I, II, IIIB, and perform soil/fabric filtration tests before specifications, prequalifying the fabric, or after selection before bid closing. Alternative: use approved list specifications for filtration applications. Suggested performance test method: Gradient ratio $\leq$3.

B. Less critical/nonsevere applications

1. Whenever possible, fabric with maximum opening size possible (lowest AOS No.) from retention criteria should be specified.

2. Effective open area qualifiers. 2/

Woven fabrics: Percent open area: $\geq$4 percent
Nonwoven fabrics: Porosity 2/ $\geq$30 percent

NOTE: 1. Filtration tests are performance tests and cannot be performed by the manufacturer as they depend on specific soil and design conditions. Tests to be performed by specifying agency or his representative. Note: Experience required to obtain reproducible results in gradient ratio test.

2. Qualifiers in potential clogging condition situations (e.g,, gap-graded soils and silty type soils) where filtration is of concern.

3. Porosity requirement based on graded granular filter porosity.

IV. Chemical Composition Requirements/Considerations

A. Fibers used in the manufacture of civil engineering fabrics shall consist of long chain synthetic polymers, composed of at least 85 percent by weight of polyolephins, polyesters, or polyamides. These fabrics shall resist deterioration from ultraviolet exposure.

Table 10. - Summary of geotextile design and selection criteria for drainage, filtration, and erosion control applications - Continued

B. The engineering fabric shall be exposed to ultraviolet radiation (sunlight) for no more than 30 days total in the period of time following manufacture until the fabric is covered with soil, rock, concrete, etc.

V. Physical Property Requirements (all fabrics)

	Fabric unprotected	Fabric protected 5/
Grab strength (ASTM: D-1682) (minimum in either principal direction)	200 lb	90 lb
Puncture strength (ASTM: D-751-68) 3/	80 lb	40 lb
Burst strength (ASTM: D-751-68) 4/	320 lb/in^2	145 lb/in^2
Trapezoid tear (ASTM: D-1117) (any direction)	50 lb	30 lb
Elongation at failure (ASTM: D-1682) (any direction)	20 percent	20 percent

1/ All numerical values represent minimum average roll values (i.e., any roll in a lot should meet or exceed the minimum values in the table). Note: These values are nominally 20 percent less than manufacturers typically reported values.
2/ Permeability should be based on the actual fabric open area available for flow. For example, if 50 percent of the fabric area is to be covered by flat concrete blocks, the effective flow area is reduced by 50 percent.
3/ Tension testing machine with ring clamp, steel ball replaced with a 5/16-inch-diameter solid steel cylinder with hemispherical tip centered within the ring clamp.
4/ Diaphragm test method.
5/ Fabric is said to be protected when cushioned from rock placement by a layer of sand or by zero height placement. All other conditions are said to be unprotected.
6/ In abrasive environments such as many erosion control applications, abrasion resistance must be considered. It is recommended that either soil-fabric abrasion evaluation tests be performed by the agency or his representative, or a standard abrasion test specified and the tested specimens visually examined to identify holes or other characteristics such as significant strength loss that may influence the performance of the fabric in these applications (i.e., run Tabor abrasion and submit specimens to agency).

Table 12. - Test for geotextile index properties

Property	Test method	Units of measurement
Mechanical strength - uniaxial loading		
a. Tensile strength and elongation		
1. Grab strength	ASTM D-1682, method 16 at 12 in/min (Fed. Std. 191, method 5100/5.9)	lb
2. Strip tensile strength	ASTM D-1682, methods 18 and 20 at 12 in/min	lb/in
3. Wide width strength	ASTM proposed method 11.3.84	lb/in
b. Poisson's ratio	No test	
c. Stress-strain characteristics (tensile modulus)	ASTM proposed method 11.3.84	lb/in
d. Dynamic loading	No standard	
e. Creep resistance	ASTM proposed	lb at rupture
f. Friction/adhesion (slick, rough, smooth)	Modified Corps of Engineers EM 1110 using Ottawa 20-30 sand	degrees
g. Seam strength	a-1, a-2, or a-3, above (depends on requirements)	lb or lb/in (as required)

Table 12. - Test for geotextile index properties - Continued

Property	Test method	Units of measurement
h. Tear strength	ASTM D-1117, method 14 (Fed. Std. 191, method 5136)	lb or lb/in (as required)
Mechanical strength - rupture resistance		
a. Burst strength	Mullen burst - ASTM D-3736, method 4 (Fed. Std. 191, method 5122)	lb/in^2
b. Puncture resistance	Modified ASTM D-751 using 5/16-inch flat-tipped pod	lb
c. Penetration resistance (dimensional stability)	No standard	----
d. Fabric cutting resistance	No standard	----
e. Flexibility (stiffness)	Modified ASTM D-1388, method 5 using 2- x 12-inch sample (Fed. Std. 191, method 5206)	mg/cm^2
Endurance properties		
a. Abrasion resistance	Modified ASTM D 1175 using Calibrase wheels, 1,000 cycles and 2.2 pound load (Fed. Std. 191, method 5304)	% strength retained in weakest direction (2-in strip test)
b. UV (Ultraviolet) radiation stability	ASTM D-4355	% strength retained (2-in strip test)

Table 12. - Test for geotextile index properties - Continued

Property	Test method	Units of measurement
c. Chemical and biological resistance	No standard for geotextiles (for textiles: Fed. Std. 191, methods 5760, 5762, 2015, 2016, and 2053)	
e. Wet and dry stability	No standard	
f. Temperature stability	No standard	
Hydraulic		
a. Opening characteristics		
1. AOS (apparent opening (size)	ASTM proposed method 11.12.84	U.S. sieve equivalent
2. Porometry (pore size distribution)	Use AOS for O_{95}, O_{85}, O_{50}, O_{15}, and O_5	----
3. POA (percent open area)	U.S. Army Engineers Waterways Experiment Station AD-745-085	percent
4. Porosity	No standard	
b. Permeability and permittivity	ASTM proposed method 11.14.84	k in cm/s in cm^{-1}
c. Soil retention ability	Empirical relations to opening characteristics	----
d. Clogging resistance	No standard - see soil-fabric tests	
e. In-plane flow capacity (transmissivity)	Koerner and Bove, 1983	gal/min/ft width

EXAMPLE OF STATE DEPT. OF TRANSPORT GUIDELINE ON EROSION PROTECTION

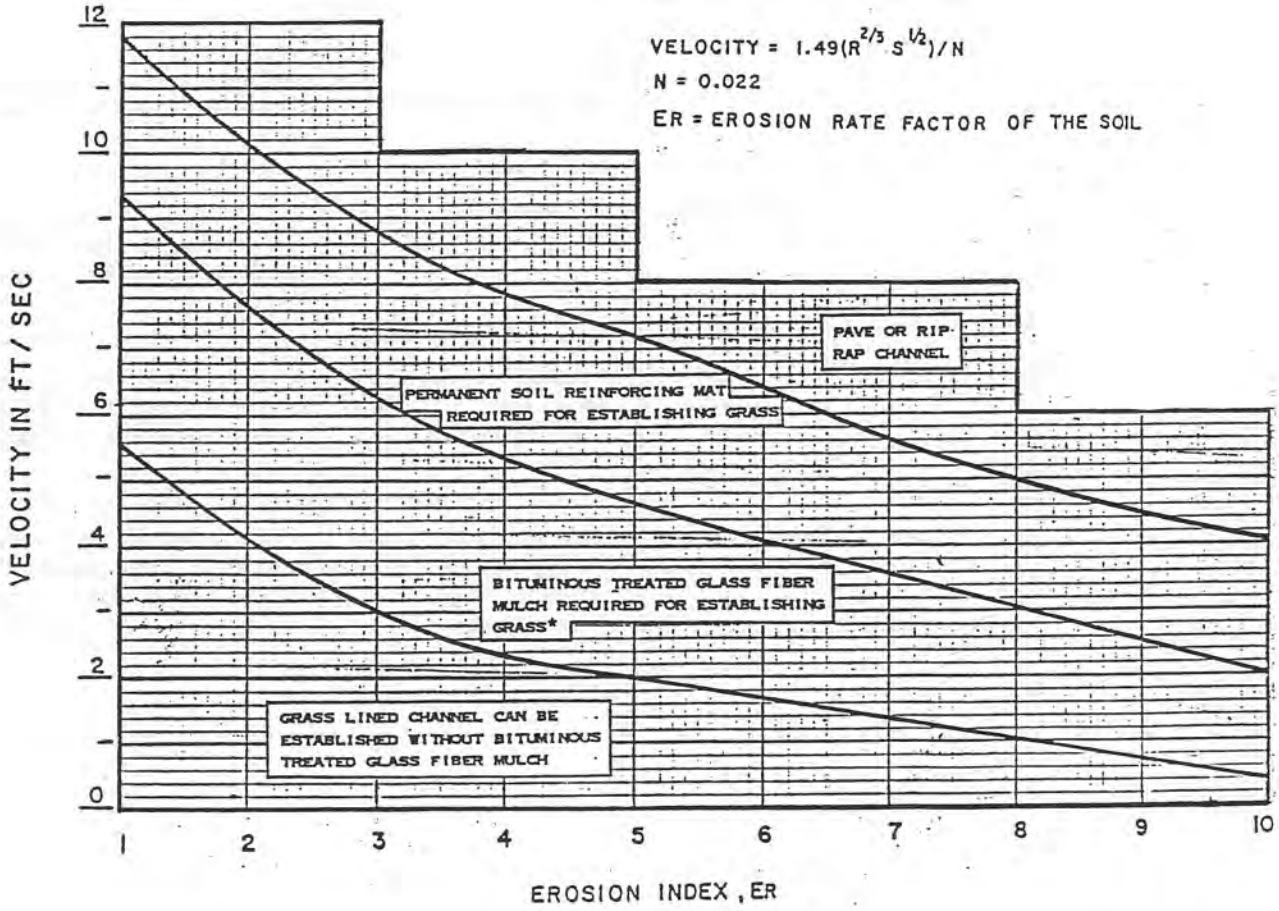


CHART 9-35

Source: Dept. of Transportation, State of Georgia, Channel Protection Criteria

Er Factor	-200 Range	Soil Description	Bare Soil Erosion
0 - 2	80% to 76%	A highly impervious and cohesive soil.	Steep cuts and embankments and narrow, deep ditches erode slowly without grassing.
2 - 4	76% to 50%	An impervious soil that is cohesive to moderately friable.	1 to 1 cuts, 2 to 1 embankments and deep, narrow ditches would erode moderately without grassing.
4 - 6	50% to 34%	Moderately cohesive to friable soil.	2 to 1 cuts, 4 to 1 embankments and wide, shallow ditches would erode moderately without grassing.
6 - 8	34% to 17%	Slightly cohesive to non-cohesive, highly friable soil.	3 to 1 cuts, 5 to 1 embankments and wide, shallow ditches would erode moderately without grass.
8 - 10	17% to 0%	Non-cohesive coarse to fine sand. Highly friable, highly porous soil.	4 to 1 cuts, 6 to 1 embankments and wide, shallow ditches would erode badly without grass.

EVALUATION OF PERFORMANCE OF WEDGE-SHAPED BLOCK PROTECTION SYSTEMS IN HIGH VELOCITY FLOW

Background

Considerable interest exists both in the US and the UK in low-cost and easily constructed forms of protection for embankments which may be subject to overtopping and for auxiliary spillways on dams. Protection systems presently used in high velocity applications include roller compacted concrete, rip-rap, gabion mattresses, cable-tied concrete blocks and soil/cement stabilisation. State-of-the-Art information on the use and performance of these systems was exchanged during the recent UK Overseas Scientific and Technical Expert Mission on Embankment Overtopping which visited US Army Corps of Engineers Waterways Experiment Station in July 1987, and subsequently liaised with other organisations involved with the ASCE Committee Force on Mechanics of Oveflow Erosion on Embankments.

The following proposal has been put forward as a result of the common interests identified during these discussions.

Technical literature(1, 2, 3) originating in the USSR indicates that Russian engineers have developed a special wedge-shaped protection block which is apparently very stable in high velocity flow (see Figure 1). The block has three key attributes:-

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.

Indicative size data for wedge-shaped concrete block protection in the terminal velocity zone on a typical 1 vertical: 3 horizontal slope is:-

Overtopping depth, H	Discharge intensity	Average block thickness	Terminal velocity, V_t
2.1m; 6.7ft	$5m^2/s$; 54 ft^2/s	0.22m; 9in	9m/s; 30ft/s
6.0m; 19.7ft	$25m^2/s$ 269 ft^2/s	0.35m; 14in	14m/s; 46 ft/s

By any standards, this protection appears extremely economical. Pravidets and Slissky claim a 3-5 times reduction in block thickness compared with concrete blocks at similar discharge intensity and having the same superficial area but constant thickness.

The Russian literature indicates that the effectiveness of the blocks has been tested in the field at discharge intensities up to $60m^2/s$ (645 ft^2/s). Corresponding mean velocities were about 20m/s (66ft/s). It also explains that a special method of interlinking the blocks was developed which integrated the protection, yet allowed flexibility and the facility for replacing individual blocks (if damaged) to be retained.

As a first stage to developing an understanding of wedge-shaped blocks, laboratory research has been carried by Nouri[4] at University of Southampton in the UK to investigate the physical processes determining their stability. Model studies carried out at discharge intensities up to $0.05m^2/s$ confirmed the high stability.

The potential for application of this technology both in the UK and in UK projects overseas has been recognised by CIRIA's practitioners members, however the lack of a clear design methodology and of first-hand data on performance at large scale has inhibited its utilisation. In the US, the potential application of wedge-shaped blocks to dam protection, particularly in upgrading flood capacity, was acknowledged by the Federal Emergency Management Agency (FEMA) in its report on Safety of Existing Dams: Evaluation and Improvement[5].

Proposed programme of research

Given the substantial programme of research and development which has been carried out in the USSR, CIRIA considers that the most appropriate way to transfer this technology to the UK and US is to carry out a limited evaluation and demonstration programme with the following objectives:

1. To demonstrate to practitioners the effectiveness of wedge-shaped blocks as erosion protection.

2. To confirm the principal parameters (slope, discharge intensity, underlayer permeability, drainage holes, block geometry) affecting the stability of wedge-shaped blocks in high-velocity, unidirectional flow.

3. To provide a design method pertinent to the slopes and subsoil parameters relevant to UK and US practice.

4. To develop appropriate side and toe details (particular emphasis being given to the method of protection termination downslope and to appropriate detailing of slope protection in the region of a hydraulic jump).

Method

1. Establish best practicable contacts with Russian engineers responsible for development and utilisation of wedge-shaped blocks in the USSR at the Kuibyshev Institute of Civil Engineering, Moscow to obtain further data on application. CIRIA has already put this into progress.

2. Set up liaison group under CIRIA comprising US engineering experts, representatives of Hydraulics Research Wallingford (HR; premier UK organisation dealing in applied civil engineering hydraulics research), University of Salford (UoS) and UK practitioners to review and appraise programme of research.

3. Utilise existing $0.25m^3/s$ ($8.9ft^3/s$) capacity, 0.6m (2ft) wide x 4m (13ft) high laboratory chute spillway installation at UoS (see Figure 2). Design of test layout to be undertaken jointly by UoS and HR. Test programme to investigate performance of largest block size (initially estimated as 15mm thick average, 75mm long, 65mm wide) which will fail under maximum flow conditions on the 1V:2.5H slope of laboratory chute. Monitor velocity of flow and time-variable pressure distribution on top and bottom block. Also consider obtaining series of turbulence and air entrainment measurements.

4. Carry out matrix of tests confirming sensitivity of block stability to changes in slope, underlayer permeability, contact friction, drainage area and block geometry. (Compare with base-line test data obtained under 1987 series of tests carried out at UoS on constant thickness blocks under Science and Engineering Research Council funding.)

5. Build side slope to one half of chute spillway and review stability of preferred block type and bedding condition on side slope.

6. Investigate stability of blocks under high tailwater condition with plunging hydraulic jump on slope. Consider methods of improving stability as appropriate.

7. Review data and prepare report in consultation with liaison group.

Programme duration

24 months to report submission.

Indicative cost estimate

£80,000.

References:

1. Grinchuk, A.S. and Pravdivets, Y.P.
   Precast reinforced-concrete revetment of earth slopes used for discharging water. (In Russian).
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   Tests of earth slope revetments permitting flow of water at large specific discharges. (In Russian).
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3. Pravdivets, Y.P. and Slissky, S.M.
   Passing floodwater over embankment dams, Water Power & Dam Construction, July 1981.

4. Nouri, B.M.A.
   Performance of wedge-shaped blocks for channel protection in water flow. PhD thesis, University of Southampton, 1984.

5. Federal Emergency Management Agency.
   Safety of Existing Dams: Evaluation and Improvement, Washington, 1983.

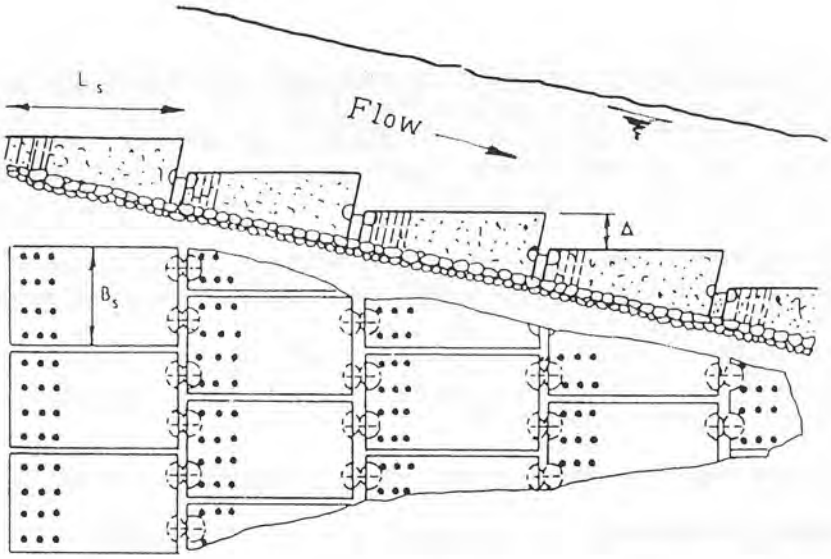

Figure 1 Wedge-shaped block slope protection

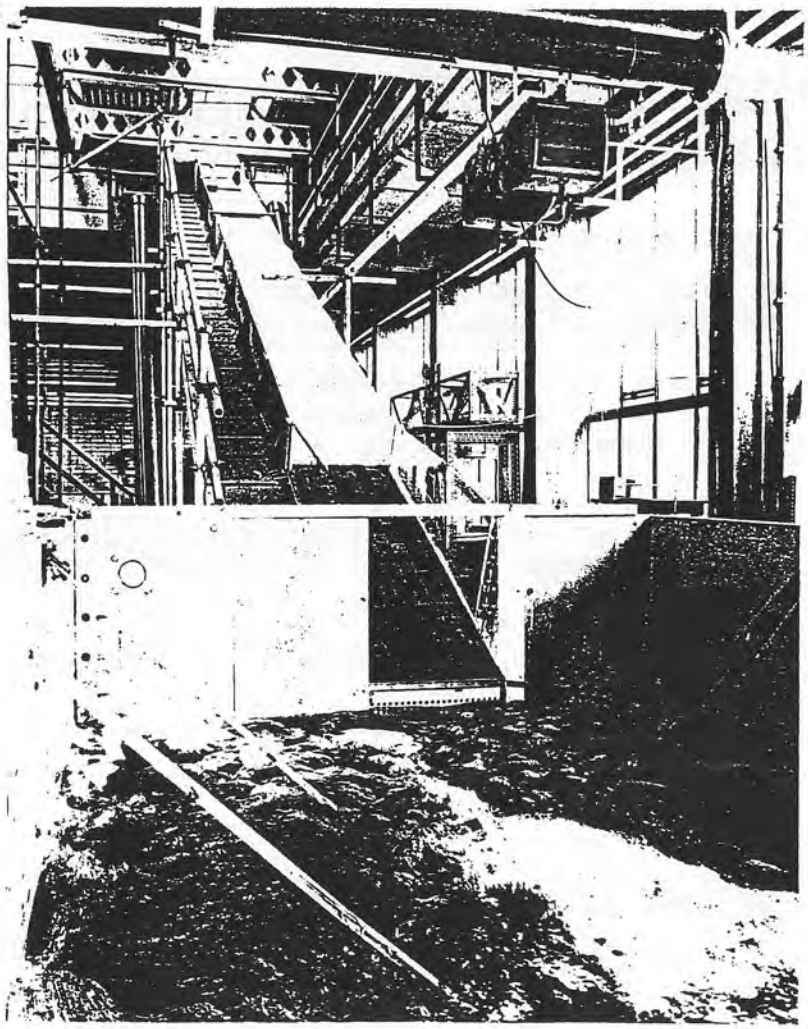

Figure 2 Chute spillway installation at University of Salford (with parallel-sided blocks in place)

US Army Engineer Waterways Experiment Station
PO Box 631, Vicksburg, Miss. 39180-0631

VISIT OF REPRESENTATIVES OF UK DEPARTMENT OF TRADE AND INDUSTRY OVERSEAS SCIENCE AND TECHNOLOGY EXPERT MISSION (OSTEM) SCHEME

16 JULY 1987

WES AUDITORIUM

0830	REGISTRATION AND INTRODUCTIONS	C.L. MCANEAR
0835	WELCOME	COL D.G. LEE
0840	WES STORY: THREE SCREEN BRIEFING	L.A. SANDERFORD
0915	OSTEM STORY	M.E. BRAMLEY
0955	WES PUBLIC TOUR	PAO
1130	LUNCH	
	GL CONFERRENCE ROOM: 1ST FLOOR	
1230	OVERTOPPING OF EMBANKMENT-DAMS	S.P. MILLER &
	SYNOPSIS OF CASE STUDIES	P.A. GILBERT
	VIDEO: CLARENCE CANNON DAM	
	VIDEO: MICHIGAN FLOOD-1986	
1430	STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION; SECTION 32 PROGRAM	N.R. OSWALT
	VIDEO	
	OVERVIEW OF R&D & DEMONSTRATIONS	
1630	MISSISSIPPI RIVERBANK ARTICULATED CONCRETE REVETMENT MATTRESS	C. NETTERVILLE
	MOVIE DEPICTING PLACEMENT	
	TOUR: DELTA CASTING PLANT	
1800	RETURN TO MOTEL	

17 JULY 1987

0830	OVERFLOW EROSION OF EMBANKMENTS	M.E. BRAMLEY
0930	DAM SAFETY ACTIVITIES IN UK	M.F. KENNARD
1030	NEW WES RIPRAP FACILITY	N.R. OSWALT
	DESCRIPTION & TOUR	
1130	LUNCH	
1230	RIPRAP IN HIGH VELOCITY FLOWS	M.L. DOVE
	CURRENT PRACTICE	P.G. COMBS
	RESEARCH	N.R. OSWALT
1330	SCOUR EROSION IN SPILLWAY CHANNELS	
	REMR VIDEO: OPEN CHANNEL FLOW	N.R. OSWALT
	VIDEO REPORT: DISPL ROCK BLOCKS	J.H. MAY
	LAB EROSION FLUME STUDIES	J.H. MAY
1500	ALTERNATIVES TO RIPRAP	N.R. OSWALT
	LMVD PERSPECTIVES	C.M. ELLIOTT
	ENVIRONMENTAL/BIO-ENGR SYSTEMS	D. Shields
OPEN	CLOSING REMARKS-ADJOURNMENT	UK/WES

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P O BOX 25007
BUILDING 67, DENVER FEDERAL CENTER
DENVER, COLORADO 80225-0007

PROPOSED ITINERARY FOR BRITISH MISSION DURING VISIT AT
BUREAU OF RECLAMATION
July 20 through July 24, 1987

Monday, July 20, 8:30 a.m. - Visit Bureau of Reclamation Building 67, Room 1324 on the Denver Federal Center and meet George Powledge, Chief, Technical Review Staff.

9:00 a.m. - Go to Room 1410 for video presentation on Bureau of of Reclamation. (17 minutes)

10:00 a.m. - Welcome and introductions - Room 1410

Dennis Schroeder - Deputy Assistant Commissioner Bldg. 67, Room 1420
Neil Parrett - Chief, Division of Dam and Waterway Design, Bldg. 67, Room 1440
John Smart - Chief, Embankment Dams Branch, Bldg. 67, Room 1290
Ralph Atkinson - Chief, Concrete Dams Branch, Bldg. 67, Room 1290
Frank McLean - Chief, Division of Research and Laboratory Services, Bldg. 56, Room 1015
Phil Burgi - Chief, Hydraulics Branch, Bldg. 56, Room 1009
Paul Knodel - Chief, Geotechnical Branch, Bldg. 56, Room 1009
Lloyd O. Timblin, Jr. - Chief, Applied Sciences Branch, Bldg. 56, Room 1001
Danny King - Chief, Office of Project Management and Review, Bldg. 67, Room 1420
Jim Graham - Chief Division of Dam Safety, Bldg. 67, Room 1470

Each of the above E&R Center personnel will give a brief description of their functions for the Bureau of Reclamation.

Visitors will present the purpose of their mission and what they want to see and discuss while they are in the E&R Center. Meetings and activities for remainder of week will be identified.

11:30 a.m. - Break for Lunch.

Monday, July 20, 1:00 p.m. - Visit Neil Parrett's office, Bldg. 67, Room 1440.

2:00 p.m. - Visit Jim Graham's office, Bldg. 67, Room 1470.

3:00 p.m. - Remainder of afternoon will be spent in individual meetings.

Tuesday, July 21, 8:00 a.m.	-	Visit Vern Resler of Office of Foreign Technical Services Bldg. 67, Room 418.
8:30 a.m.	-	Visit Frank McLean's office of Division of Research and Laboratory Services, Bldg. 56, Room 1015.
9:00 a.m.	-	Tour of Division of Research and Laboratory Services Laboratory.
10:00 to 11:45 a.m.	-	Individual meetings with Messrs. Burgi, Knodel, and Timblin.
11:45 - 12:45	-	Break for lunch.
1:00 to 3:00 p.m.	-	Lecture by M. E. Bramley in the Auditorium of Building 56 on "Overflow Erosion of Embankments," with time included for questions.
3:30 p.m.	-	Meet with Darrell Webber, Assistant Commissioner - Engineering and Research.
Wednesday, July 22 9:00 to 11:00 a.m.	-	Lecture by M. F. Kennard in the Auditorium of Building 56 on "Dam Safety and Reservoir Activities in England," with time included for questions.
11:15 a.m.	-	Break for lunch.
1:00 to 4:00 p.m.	-	Half-day seminar
6:30 p.m.	-	Dinner at Mt. Vernon Country Club.
Thursday, July 23 8:00 a.m.	-	Bob Schranz, Property and Services Branch (Extension 66722) will pick up team at front entrance to Compri Hotel for tour of Colorado-Big Thompson Project, East Slope features. Travel to Loveland Colorado (60 miles) arriving approximately 9:30 a.m. Briefing by R. H. Willms, Project Manager, Eastern Colorado Projects Office, followed by orientation and tour of facilities. Return to Denver by early evening.
Friday, July 24	-	Visit to Ft. Collins, Colorado, to talk to representatives of Simon, Li and Associates.
Saturday, July 25	-	All of the team will depart for the United Kingdom, except for M. E. Bramley.

Meetings were held with the following persons/organisations additonal to CoE and USBR staff during the Mission visits to Vicksburg and Denver. Many of these were present at the half-day seminar with consultants, contractors and manufacturers held on 22 July:

S. R. Abt	Colorado State University
R. P. Anderson	Tensar Corporation
G. Bach	Geoweb Inc.
D. Bogan	Engineering Consultants International (ECI)
L. E. Buck	Tennessee Valley Authority
B. Call	Lonestar Corporation
P. E. Clopper	Simons, Li & Associates, Inc.
R. Daniels	Contech, Construction Products Inc.
G. I. Dodson	BASF Corporation, Fibers Division
J. W. France	Geotechnical Engineers, Inc.
T. Kelley	Bowman Construction
J. McDermott	Kiva Construction & Engineering Inc.
R. A. Mussetter	Simons, Li & Associates, Inc.
S. Nicolaon	Engineering Consultants International (ECI)
A. Pearson	Division of Water Resources, State of Colorado
W. Schlenzig	Contech, Construction Products Inc.
B. Urbonas	Denver Urban Drainage & Flood Control District
D. M. Vick	Consultant

ATTENDANCE BY M. E. BRAMLEY AT ASCE NATIONAL CONFERENCE ON HYDRAULIC ENGINEERING, WILLIAMSBURG, VIRGINIA, 3-6 August 1987

Principle interests were sessions on Overflow on Embankments organised by ASCE Task Committee on Mechanics of Overflow Erosion on Embankments, Embankment Dams, and Dam Safety: Spillway Design Flood Selection. This included presentation of invited lecture on Reinforcement of Steep Grassed Waterways outlining the CIRIA research programme in UK.

Discussions were held with the following additional persons/organisations concerning the subject area of the mission, and with whom contact had not been established earlier:

D. I. Bray	University of New Brunswick
F. A. Locher	Bechtel Civil & Minerals, Inc.
W. A. Moler	Morrison-Knudsen Engineers, Inc.
D. W. Newton	Tennessee Valley Authority
S. J. Poulos	Geotechnical Engineers, Inc.
D. C. Ralston	Soil Conservation Service
J. Sterling Jones	Federal Highways Administration
D. M. Temple	USDA Agricultural Research Service.

FULL SCALE TESTING OF CABLE-TIED BLOCK SYSTEMS

This note updates information presented in Sections 4.4 and 7.3.3 of the report with preliminary results of the full scale tests on cable-tied concrete block protection systems which were carried out in Autumn 1987 after the mission visit.

Tests were carried out on the effectiveness of three proprietary cable-tied concrete block systems in protecting the embankment. The systems - Armorflex, Dycel and Petraflex - had superficial weight ranging from 160 to 200 kg/m^2 and each provided different interblock restraint due to varying:-

- cabling systems (one way and two way)
- configuration of adjacent blocks (stretcher and stack bond)
- face to face contact
- use of gravel wedging between blocks.

The systems were laid on a high-permeability, geotextile fabric underlayer. A drainage mesh was also incorporated into the underlayer of two systems. Anchorage at the crest and toe, and on the slope was as shown in Figure 1.

A range of overtopping tests were carried out at overtopping heads of 1 ft, 2 ft and 4 ft, mainly in the free fall condition (see Section 4.4).

With one of the systems, the protected embankment survived all tests, including 30 hours of testing at 4 ft overtopping. For some of this time the slope and toe anchors were removed.

With two of the systems, failure of the underlying embankment was not prevented. Failure apparently occurred through saturation and liquifaction of the subsoil in the surface region at the toe of the embankment. Overtopping heads at failure ranged from 1 ft to 4 ft for different systems and combinations of underlayer type and anchorage. When failure occurred, this was generally after about 1 hour of overtopping.

The researchers recognise the difference between the integrated armour layer concept of reinforced grass as developed in the UK for a stable, low-permeability subsoil and the extreme test condition provided by the Phase II embankment material with a non-integral armour system. For the Phase II test situation, they consider that the differing performance of the various systems is influenced by superficial weight, the type of interblock restraint and the ability of the flow to penetrate below the blocks.

The USBR and other US agencies having potential interest in the use of cable-tied systems are currently considering an extension of the Phase II test programme into Spring 1988 to carry out further tests on cable-tied systems. The objective of these tests will be to obtain further information on the limitations of different types of cable-tied concrete block systems, and the effects of seepage flow and subsoil.

No analysis or conclusions of the Phase II test programme has yet been published by the USBR and FHA, and further comment is not possible at this point in time.

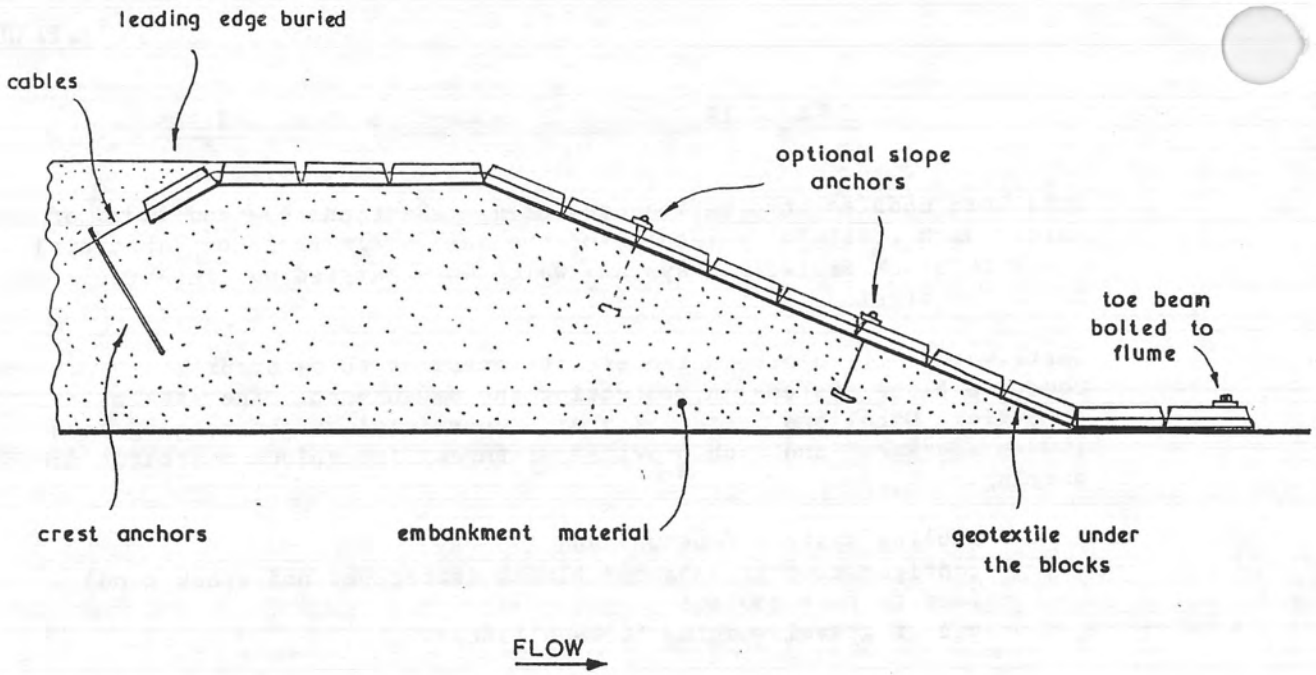

Figure 1 Cross-section of concrete block system in test flume