Performance of River Flood Embankments

R Bettess
C E Reeve

Report SR 384
April 1995

HR Wallingford

Address and Registered Office: **HR Wallingford Ltd.** Howbery Park, Wallingford, Oxon OX10 8BA
Tel: + 44 (0)1491 835381 Fax: + 44 (0)1491 832233

Contract

The report describes work commissioned by the Ministry of Agriculture, Fisheries and Food to investigate the Performance of Flood Embankments. The nominated officer was Mr J Goudie and the HR Wallingford Project Director was Dr W R White. The HR job number for the project was QPS00 13/E The work was carried out by members of the Rivers Group.

Prepared by R Bettess (name) Rivers Group (Job title)

Approved by .... Rivers Group Manager

Date 3/8/95

Summary

Performance of River Flood Embankments

R Bettess
C E Reeve

Report SR 384
April 1995

Embankments are often used to protect land on flood plains from river flooding. When a flood event passes down a river, a high head of water is contained by the embankment, preventing the flood water from inundating the flood plain.

A survey of the hydraulic performance of flood embankments in the United Kingdom is described.

Flood plains are commonly built up of highly permeable river sand and gravel deposits, over lain by a low permeability overburden of silt and clay alluvium. There is a potential for groundwater flow to take place within a flood plain aquifer, beneath the embankment. This can lead to surface ponding due to exfiltration of groundwater within the protected area. A description of the groundwater flow system is given. A method for making an approximate assessment of the degree of groundwater response to a given river hydrograph in a particular flood plain is described. Remedial measures are discussed briefly.

The pore water pressures within flood embankments fluctuate as a result of variations in both river and groundwater levels. These fluctuations in pore water pressure can result in stability problems and potentially embankment failure. The methods available for the study of embankment stability are described.

Flood embankments are designed for a particular flood return period. For floods greater than the design flood, there is the danger that the water will flow over the embankment. This can result in flooding of the land behind the embankment and in some circumstances it can be sufficiently severe as to cause the embankment to fail. The extent of this problem in the UK is assessed and methods available for its study are described.

Contents

Contents continued

Tables

Figures

Contents continued

1 Introduction

Embankments are often used to protect land on flood plains from river flooding. When a flood event passes down a river, a high head of water is contained by the embankment, preventing the flood water from inundating the flood plain.

These flood plains are commonly built up of highly permeable river sand and gravel deposits, over lain by a low permeability overburden of silt and clay alluvium. There is a potential for groundwater flow to take place within a flood plain aquifer, beneath the embankment. This can lead to surface ponding due to exfiltration of groundwater within the protected area.

The pore water pressures within flood embankments fluctuate as a result of variations in both river and groundwater levels. These fluctuations in pore water pressure can result in stability problems and potentially embankment failure.

The hydraulics of the groundwater system associated with flood embankments should be considered during the design of flood embankment schemes in order to identify areas at risk and to assess the true degree of flood protection provided.

Flood embankments are designed for a particular flood return period. For floods greater than the design flood, there is the danger that the water will overtop the embankment. This can result in flooding of the land behind the embankment and in some circumstances it can be sufficiently severe as to cause the embankment to fail.

2 Survey of hydraulic performance of flood embankments

In order to examine the hydraulic performance of embankments in the UK a questionnaire was circulated to the then Regional Manager responsible for flood defence in each of the National Rivers Authority regions. They were asked to identify problems and examples associated with embankments.

A copy of the questionnaire and a list of the correspondents is included in Appendix A. A total of 35 questionnaires were returned from 5 of the 10 National River Authorities contacted. General information was received from a further 3 regions.

The problems associated with embankments were :

- Groundwater seepage below or through embankment
- Flow over embankments
- Surface erosion resulting from:
    - flows over embankments
    - river flood flows
    - wave action
    - operation of sluices, gates
- Mass failure

- Land drainage problems due to embankments and flap gates
- Maintenance, control of vegetation

Seepage during periods of high flow was identified in many cases as a major problem. Many embankments are founded on silt and peat soils which permit seepage under embankments during floods as a result of the large applied hydraulic head. Methods mentioned for preventing such seepage included sealing the banks with clay or insertion of a butyl or bentonite membrane in the embankment.

Many of the replies received identified erosion endangering the stability of the embankments as a major problem. Erosion may be due to the effects of wind and waves; tides; wash from ship navigation; weathering and frost action; burrowing vermin and grazing stock; and high velocities during flood flows. These effects were exacerbated by loss of vegetation on the banks due to differential summer and winter levels in the river. Protection methods mentioned included revetments, gabions, stone batters, faggots and piles. Erosion of the inland face of the bank caused by overtopping was also identified as a problem. This leads to reduction in the surface width of the embankment and further instability.

During a separate project (NRA R & D project 459/C06) vermin activity was cited as probably the largest single cause of bank failure in the North West region. Traditional control measures include removing the vermin and stepping up the holes, but this can leave a labyrinth of tides within the body of the bank. Complete dismantling of the defence and reconstruction may be required to prevent heavy seepage and subsequent breaching.

A third major problem identified by the survey was the failure of an embankment immediately following the recession of a flood. This is due to the creation of a potential shear surface when pore water pressures remain high in the bank after water against the slope has been removed. This was identified as 'a common fault in flashy rivers'.

Responses to the questionnaire made a number of references to the problems caused by embankment overtopping. The regions that explicitly mentioned overtopping problems were: Anglian, Severn-Trent, Southern, South-West and Wessex. Discussions with National Rivers Authority engineers suggested, however, that overtopping of embankments is a general problem throughout the NRA, rather than limited to particular regions. It was mentioned more frequently in the context of tidal areas but whether this is a true reflection of the distribution of overtopping problems is not clear.

The survey revealed that settlement of embankment material may take place as a result of frequent saturation and then drying and this is more commonly a problem in tidal areas but may also occur with river embankments.

Many of the problems which were identified by the survey were subjects of normal maintenance procedures and very few cases of total collapse were reported. Very old banks and sand constructions were concluded to be the most at risk of collapse.

It should be noted that the initial source of a problem may not appear to be the cause of the final failure. Burrowing animals may weaken an embankment and promote piping though the embankment ultimately fails during a flood.

Thus failures frequently occur as a result of a sequence of changes due to a range of causes rather than as a result of a single mechanism. There may be interaction between different mechanisms which together lead to failure.

One of the main causes of the failure of coastal embankments in 1953 was initiated by flow through the embankment and then geo-technical failure associated with fissures in the crest of the clay banks.

A separate study has identified the following failure mechanisms:

Erosion due to direct wave attack
Erosion (landward face) due to overtopping
Erosion of crest and reduction of height
Scour of seaward toe leading to oversteepening
Under-seepage in permeable layer leading to piping
Seepage leading to slipping or sliding of landward face
Slip combined with lateral compression
Uplift pressure leading to floating
Uplift pressure leading to bursting
rapid drawdown leading to front face failure
Cracking of surface material leading to effective reduction in height
Densification of core material leading to effective reduction in height
Animal burrows causing general weakening and piping
Human damage, vandalism
Damage by ship collision
Bank instability in absence of flood
Foundation instability in absence of flood

The direction of this research project was altered in the light of the responses to the questionnaire but it was not possible, within the constraints of the project, to address all the problems that were identified. Consideration should be given to whether further research is required on these topics, particularly:

(1) Surface erosion resulting from:
- river flood flows
- wave action
- operation of sluices and gates.

(2) Mass failure

(3) Land drainage problems due to embankments and flap gates

(4) Maintenance and the control of vegetation

Item (4) falls outside the experience of HR but the other topics would certainly be amenable to further research. The responses to the questionnaire suggest that such research would be economically justifiable.

3 The groundwater system

3.1 General Description

To understand the source of the groundwater related problems associated with embankments and their solution, one must first develop a description of the groundwater system.

Figure 3.1 shows a cross-section through the type of system envisaged. The groundwater flow is initially in a steady condition, usually with a net flow of water to the river which acts to drain the flood plain (Figure 3.1a). With the onset of a flood event, the water level in the river rises, reversing the direction of groundwater flow as the river recharges the aquifer (Figure 3.1b). When the river level recedes, the high groundwater heads dissipate and the groundwater drains back toward the river (Figure 3.1c).

The speed and degree of response of the groundwater system to the river flood event (Figure 3.1d) depends on the geo-hydraulic properties of the flood plain; the hydraulic conductivities and storage coefficients of the aquifer and overburden, and also on the geometry of the groundwater system; the thickness of aquifer and overburden and the width of the river valley.

3.2 Types of Aquifer

There are three basic types of aquifer that may be considered: unconfined, confined or semi-confined aquifers, depending on the degree of influence of the alluvial overburden. The unconfined, or phreatic, aquifer is one above which the overburden is absent or has negligible effect. In the confined case, the overburden is considered to be totally impermeable. These are the two extremes when considering the influence of the overburden. The most likely situation to occur naturally, however, is that of a low, but not negligible, permeability overburden which semi-confines the aquifer but may also transmit high pore water pressures and possibly allow seepage to the surface.

Each case is considered in a little more detail below:

(i) Unconfined aquifer, Figure 3.2a.

The unconfined aquifer contains a water table which reflects the hydraulic head elevation in the aquifer (phreatic surface). If a volume of water is added to the aquifer the water table will rise, say from level A to B in Figure 3.2a, in accordance with the available pore space or specific yield of the aquifer. This storage is the difference between the unsaturated and saturated moisture contents of the aquifer material. For an unconsolidated granular soil, the available storage may be in the region 15 - 35 % of aquifer volume, depending on the particle size grading.

As there is no restricting layer in an unconfined aquifer, if the hydraulic head in the aquifer rises above ground level, level C, exfiltration and surface ponding of groundwater will occur.

(ii) Confined aquifer, Figure 3.2b

The confined aquifer is fully saturated. The hydraulic head elevation is given by the piezometric surface, which is above the top of the aquifer, level A on Figure 3.2b. If this were not the case, an unconfined condition would exist, level B. There can, therefore, be no storage due to the specific yield of the

soil. Instead, a small degree of storage is available due to the elastic storage of the aquifer. This storage coefficient incorporates the slight compressibility of water and the stress-strain relationship of the soil matrix.

In comparison to specific yield, the elastic storage is very small, typically in the region 0.01 - 0.5 % for unconsolidated granular aquifers. This means that groundwater responses will be more pronounced for confined than unconfined aquifers.

As no exfiltration of water can occur through the overburden, there is no danger of seepage when the piezometric surface exceeds the ground surface, level C. There is, instead, a danger of uplift pressure due to the excess head above the top of the aquifer, becoming greater than the downward soil pressure, resulting in flotation and rupturing of the overburden. This mode of soil failure caused by high piezometric responses to embanked river floods was noted as being highly destructive in the Bay of Plenty, New Zealand (Raudkivi and Callander, 1976) and also in Ise Bay, Japan (Naguchi, 1989). This potential for generating uplift pressures is an important aspect when considering the flood protection of urbanized flood plains.

(iii) Semi-confined aquifer, Figure 3.2c
In this case we need to consider the hydraulic properties of the overburden material as well as the aquifer, in order to take the hydraulic head in the overburden into account.

High uplift pressures may evolve similarly to confined aquifers but the excess head this time is due to the difference in head between levels A and B, Figure 3.2c. Another mode of soil failure that may occur in this situation is that of high groundwater heads evolving within the overburden. These high pore water pressures result in a lowering of the effective stress and subsequent loss of soil strength which can lead to subsidence around foundations.

An important point to note here is that in the aquifer, vertical gradients are very small and so the groundwater velocity is always predominantly horizontal; v_1, due to head difference h_1 in Figure 3.2c. The groundwater head changes much faster in the aquifer than in the overburden, due to the large difference in permeability between them. This generates predominantly vertical groundwater velocities in the overburden; v_2 due to head difference h_2 on Figure 3.2c.

Semi-confined aquifers present more complex modelling challenges than fully unconfined and confined aquifers. Models of these latter two types of aquifers may suffice in many situations. If soil stability calculations are to be performed, however, the response of groundwater pressure in the overburden material needs to be assessed in as much detail as possible.

4 Seepage beneath flood embankments

4.1 Approximate Assessment of groundwater response to a flood event

In their studies of baseflow recessions of stream-aquifer systems, Singh and Stall (1971) introduced the dimensionless constant, τ, to characterize aquifers whereby

$$\tau = Tt / SL^2 \qquad (1)$$

where
- T = aquifer transmissivity
- S = aquifer storage coefficient
- L = distance from river to catchment edge
- t = time of recession

The transmissivity (product of hydraulic conductivity and depth of flow) and the storage coefficient may be found from conducting on-site pumping tests.

Watkins (1988) used this equation to characterize flood plain aquifers by substituting

t = period of flood cycle
L = width of flood plain

Using a simple, full hydraulic connection, river boundary condition and neglecting the overburden, the response to sinusoidal flood cycles was examined for various values of τ. When $\tau = 1$, the response is considerable and groundwater problems are likely to occur. At $\tau = 10$, 99 % of the river flood peak is transmitted to the aquifer location furthest from the river. Thus τ was used to assess the degree of response to the aquifer.

The parameter SL^2/T can be interpreted as the aquifer response time (Nutbrown and Downing, 1976). Designating the aquifer response time as τ_*,

$$\tau_* = SL^2/T \qquad (2)$$

and applying the values of τ considered above,

$\tau = 0.1$ $\quad$ $\tau_* = 10\,t$
$\tau = 1$ $\quad$ $\tau_* = t$
$\tau = 10$ $\quad$ $\tau_* = 0.1\,t$

The aquifer response time, τ_*, may, therefore, be used to characterize the response of a flood plain aquifer to a flood event. If τ_* is of the order of ten times the flood cycle period or more, then groundwater responses will be small. If τ_* is of the same order as the flood cycle period then groundwater responses may be considerable and are liable to cause problems. If τ_* is around one tenth of the flood cycle period or less, the aquifer will respond fully to the flood hydrograph right across the valley.

Note that an order of magnitude is not a large variation when considering field measurements of T and S.

This approach for evaluating seepage beneath flood embankments is very simple and it has the advantage that it allows a rapid assessment of the magnitude of the problem which may be expected. Groundwater models can be used for site specific studies. These allow a number of additional factors such as the degree of saturation, the properties of the overburden and vertical flow components to be taken into account.

4.2 Remedial Measures

Vertical impermeable barriers constructed in the aquifer beneath the embankment may restrict seepage but will also affect groundwater discharge and impede the river draining the flood plain naturally. It may also be possible to construct partial barriers that impede groundwater responses enough on the time-scale of a flood event whilst still allowing relatively unimpeded steady-state drainage. The balance between the design of an effective and ineffective scheme, however, is delicate.

A more reliable method of controlling the groundwater responses is by pumped drainage. A drain situated at the inside toe of the embankment may be used to control groundwater within the protected area. Because the drain would be used when ambient groundwater and river levels are high, the discharge would need to be pumped. The feasibility of such a scheme, the rate at which pumping would be required and design details may be studied by the application of numerical models.

5 Stability of flood embankments

Slope instability can result in material falling, sliding or flowing down a embankment, possibly damaging structures or endangering life. Assessing the stability of an embankment is therefore a common concern for the engineer or engineering geologist. It is for this reason that attempts have been made over many years to understand the processes which govern embankment stability. This has resulted in the development of methods of slope stability analysis, in order to make a quantitative assessment of the stability of a particular slope.

A major problem is the failure of an embankment immediately following the recession of a flood. This is due to the creation of a potential shear surface when pore water pressures remain high in the bank after water against the embankment has been removed.

5.1 Fundamental factors affecting the stability of embankments

5.1.1 Soil characteristics

The nature of the soil forming the embankment is an important factor affecting its stability. Soil consists of particles of rock, or rock forming minerals, with air and water filling the pore spaces between. Rock forming minerals can be either massive (eg. quartz) or clay minerals (eg. illite), and whereas massive minerals generally form angular blocks of silt or sand, clay minerals form much smaller flake shaped particles.

The type of minerals present affects the ability of the soil to absorb water, and therefore determines the physical interaction between soil particles. In general, massive minerals do not absorb much water. However, clay minerals are often

electrically charged and absorb water readily, leading to the soil swelling and shrinking as the soil environment changes.

The type of minerals present also affects the permeability of the soil. Soils consisting of large particles, such as sands, contain more voids and thus have a larger permeability than soils which consist of fine particles such as clays. Clay materials commonly exhibit very low permeabilities. These properties are important when considering flow through or under embankments.

5.1.2 Effective stress

The principle of effective stress is fundamental to soil mechanics. Stress may be transmitted through a soil by the soil skeleton and by the pore fluid. Pore fluid can exert only an overall stress, whereas the soil skeleton is able to transmit normal and shear stresses through the soil. It is for this reason that the soil strength is controlled by the stresses transmitted by the soil skeleton. A high pore fluid pressure will reduce the *effective stress* on the soil, as there is a reduction in the stress transmitted through contacts between soil particles.

5.1.3 Soil strength

As mentioned previously, soil strength is essentially derived from the soil skeleton, ie. the contacts between soil particles. Friction between soil particles results in a *shear strength* that is controlled by the effective stresses on the soil. If a soil is only just supporting the stresses imposed on it, its shear strength is fully mobilized and plastic deformations occur.

Cohesionless soils such as sands and gravels, have no shear strength when they are unconfined, however when confined some shear strength is derived from the interlocking of soil particles and the friction between them. When sheared, initially the confined soil dilates giving a peak strength. If shearing is continued, the shear strength drops until shearing at a constant volume occurs. Unconfined cohesionless soils tend to contract when sheared, until the constant volume condition or the *critical state* of the soil is reached.

In contrast, cohesive soils such as clays have some shear strength when unconfined. This is mainly due to sub-atmospheric pore pressures, but there can be bonding between clay particles. For this reason, soil permeability and drainage are important to the shear behaviour of clays. The plate-like shape of clay particles means that they will often align themselves with a shear plane, therefore reducing the shear strength of the plane in relation to the surrounding soil. The reduced shear strength along the plane is termed the residual strength.

Fully drained cohesive soils behave in a similar way to cohesionless soils when sheared, eg. over-consolidated clays will initially dilate, whereas normally-consolidated clays will contract on shearing. If shearing is continued, then the critical state is reached, and eventually a shear plane is formed. The shear strength of the clay is then reduced to the residual value.

Soil drainage is very important to the soil strength in both cohesive and cohesionless soils. For instance, a saturated soil with a constant volume (ie. undrained) will have a constant strength. If drainage is allowed however, the soil strength will change with different loads.

5.1.4 *Pore water pressure and groundwater*

Pore water pressure has a direct effect on the effective stresses which control the shear strength of soils, and therefore on the stability of an embankment. When using an effective stress method of slope stability analysis, the determination of the pore water pressure distribution within the embankment is essential. The evaluation of the pore water pressures may involve field measurements or modelling soil seepage, which is detailed in such texts as Cedergren (1967).

Embankment instability can occur when pore water pressures are out of equilibrium with the boundary conditions of the embankment. This may be due to changes in the pore water pressures as a result of undrained loading and unloading, or a recent change in hydraulic boundary conditions. As equilibration of pore water pressure occurs, the stability of the soil changes. For instance where an embankment is constructed, the soil beneath it will gradually become more stable as pore water pressures are reduced. Where a cutting is made, the side slopes will gradually become less stable as pore water pressures are increased. The behaviour of embankments and cutting is complicated by the nature of any drainage, and by the type and zoning of soil involved.

Where there is rapid drawdown behind an embankment or dam, and the permeability of the soil is such that it impedes outflow, the response of the pore water pressures in the soil may lag behind the drawdown of the reservoir. When the loading of water on the embankment is removed, there may be some residual high pore water pressures remaining in the embankment which could render it unstable.

It is clear then that pore water pressures are very important when considering the stability of an embankment, and vital if the stability is to be evaluated in terms of effective soil stress. Information on pore water pressure may be represented as a piezometric surface, or as a *pore-pressure ratio* r_u. The former, although useful in simple regimes, cannot be used to describe complex pore water distributions. The pore-pressure ratio is given by the pore water pressure at a point in the soil divided by the vertical total stress at that point, (which may be calculated roughly using the soils unit weight and the depth below ground level). This ratio may be averaged along a slip surface, or for zones of different soil types, however, it should be used with caution when modelling slope stability, as averaging will often result in large reductions in accuracy.

5.2 Stability analysis of embankments

5.2.1 *Factor of safety*

It is usual to express the results of slope stability analyses in terms of an index of relative stability called the *factor of safety (F)*. This is the ratio of the actual strength available to that mobilized ie.

$$F = \frac{\text{Available shear strength}}{\text{Shear strength required for stability}} \quad . \qquad (3)$$

In the analysis of stability the factor of safety traditionally has several functions,

1 To take into account uncertainty of shear strength parameters due to soil variability, and the relationship between the strength measured in the laboratory and the operational field strength.

2 To take into account uncertainties in the loading on the slopes (eg. surface loading, unit weight, pore pressures)

3 To take into account the uncertainties in the way the model represents the actual conditions in the slope, which include,

   (a) the possibility that the critical failure mechanism is slightly different from the one which has been identified, and,
   (b) that the model is not conservative.

4 To ensure deformations within the slope are acceptable.

The factor of safety does not allow for the possibility of gross errors, for example a bad choice of failure mechanism, such as ignoring the presence of existing shear surfaces in a slope.

A disadvantage of the Factor of safety is that it does not necessarily represent the probability of failure. The failure probability is an important element of risk analysis used to compare the costs and benefits of alternative schemes. A probabilistic approach takes account of the uncertainty in soil parameters, in pore pressures and, if necessary, in the soil behaviour model, to evaluate the probability that the slope will fail.

5.2.2 Methods of stability analysis

An exact stability analysis of embankments would involve solving simultaneously the conditions of equilibrium and compatibility throughout the embankment. Clearly a complete knowledge of the stress-strain behaviour of the soil would be required and the calculations would be very complicated. For this reason, the simpler limit equilibrium approach is often used to analyse the safety of a slope, even though these methods will not give details of deformation under stress.

Limit equilibrium methods

There are a number of different methods of stability analysis, the procedures of which are generally similar in concept. The embankment and embankment material being considered are modelled theoretically, taking account of the loadings on the embankment. Although embankment instability may result in rock falls or mud flows, most failures start or progress by sliding along surfaces within the soil mass. For most engineering purposes then, sliding models are considered to be sufficient, and these 'limit equilibrium methods of analysis' are widely used for the analysis of slopes, embankments and excavations.

Simple sliding methods of analysis are adequate for most engineering applications. These are used to determine the factor of safety for the critical slip surface, and therefore, the embankment. The presence of bedding planes or other soil features in some embankments will result in a slip surface approximately parallel to the embankment surface. Other embankments,

where soil formations are predominantly cohesive, will have more deep seated slip surfaces, which can often be described by the arc of a circle.

When using a limit equilibrium method of stability analysis, the first step is to assume a surface along which the slope is likely to fail. The mobilized stress along this surface is then calculated and compared with the stress required to cause failure along the surface, the result being a factor of safety for that particular slip surface.

Limit equilibrium methods of analysis may be broadly divided into two categories: linear methods and non-linear methods. Linear methods are simpler to use, as the factor of safety can be calculated directly from a linear equation. However, the assumptions made in these methods will result in conservative factors of safety. Non-linear methods have non-linear factor of safety equations, which must be solved using an iterative procedure. Assumptions must also be made in these analyses, however most methods are held to produce acceptable factors of safety.

Linear methods

Where shallow forms of embankment failure are likely to occur over a large cross-sectional slope area, the *infinite slope analysis* (Taylor, 1948) may be the most appropriate method of analysis. In this case, failure is assumed to occur along a plane slip surface which is parallel to the ground surface. Forces acting on the top and toe of the slide are considered negligible and are ignored (ie. the slope is infinite in length). The soil properties and groundwater conditions are assumed to be constant along the slip. This method is also described by Skempton and Delory (1957).

Wedge analysis, or the graphical wedge method may be used where the slip surface may be simply approximated by a few straight lines. For instance where the geology of the embankment dictates the position of a slip surface. The soil is divided into a number of blocks or wedges. The factor of safety is then obtained by solving horizontal and vertical equilibrium equations for each block, and constructing force polygons. Assumptions about interwedge forces must be made.

The circular arc method of analysis (Fellenius, 1936) is commonly used to assess embankment stability. Failure of the embankment is assumed to occur by the soil mass sliding along a cylindrical slip surface, where the undrained soil strength can be mobilized. The simplest method which assumes a circular arc slip surface, is the $\phi_u = 0$ method. Here the shear strength of the soil is assumed to be purely cohesive, as the angle of friction for the undrained soil is ignored. This assumption simplifies the calculation of the maximum available resisting moment, ie. the sum of all the cohesive strengths multiplied by the areas or lengths over which they act, and the radius of the slip circle. The factor of safety is then calculated by dividing the maximum available resisting moment by the moment of disturbing forces. The undrained shear strength is used in this method of analysis, which implies that effective stresses and pore water pressures in the soil have not yet reached equilibrium. This method is appropriate for the analysis of problems in the short term, immediately after construction.

The ordinary method of slices is an effective stress analysis where failure is again assumed to occur by rotation of the soil mass about a cylindrical slip

surface. The method requires that the variation of the normal stress around the slip surface is determined, and in order to do this the mass of soil is divided into slices. It is assumed that the resultant of the interslice forces on each slice is parallel to its base. The moment equilibrium about the centre of the slip circle is examined to obtain the factor of safety of the embankment. This is the simplest method of slices to use, however, the assumptions about the inter-slice forces can lead to large under-estimations of the factor of safety, and so it is not often used (Nash, 1987).

Non-linear methods

In order to determine the effective stresses around a failure surface, the failure mass of soil is divided into slices. However, many of the forces acting on a slice are unknown at the beginning of the analysis. The number of unknown variables is greater than the number of equations available. This means that assumptions for some variables must be made in order to solve the equations for the remainder. The assumptions may be about the normal stress distribution over the slip surface; the position of the line of thrust of interslice forces or the inclination of interslice forces. Most non-linear methods of slices assume that the normal force acts on the centre of the base of each slice. This is acceptable if the soil mass is divided into a relatively large number of slices. The assumptions made, and whether an overall force or moment equilibriums are considered, distinguish the different methods of slices.

The general or conventional method of analysis examines the overall moment equilibrium about an assumed centre of rotation, or overall force equilibrium in order to obtain two expressions for the factor of safety. The slip surface may be circular or non-circular, and an assumption must be made about interslice forces.

Bishop (1955) improved on the conventional method of slices; which takes no account of interslice forces, by assuming that the interslice forces are horizontal, and resolving all the forces for a slice vertically. In this method, a centre point for the trial slip circle is specified, along with a point through which the circle passes (eg. the toe of the slope). Moments are taken about the centre of the circle and by examining the overall stability a factor of safety for the slip circle is obtained. A detailed description of Bishops simplified or 'routine' method of slices may be found in his paper of 1955, and in a number of geotechnical texts (eg. Bromhead (1986); Anderson & Richards (1987)).

In order to determine the most likely slip surface for the slope, a number of different circles are examined, the centres of which are usually specified as a rectangular grid. Contours of factor of safety for each circle centre point can then be drawn and the surface with the lowest factor of safety against sliding may be determined.

In certain circumstances when using Bishop's method, problems can arise when evaluating the factor of safety at the toe of a embankment. When the soil at the toe segment has a high angle of friction compared with the soil of the rest of the slope, this may be incompatible with the overall low factor of safety for the embankment.

Janbu's method (Janbu et al. 1956) developed Bishop's routine method so that it could be applied to any shape of slip surface. Although the general rules of Bishop's method are followed, this method uses the force rather than moment

equilibrium equation. The interslice shear forces are assumed to be zero, but the introduction of an empirical correction factor which is applied to the converged factor of safety, allows for them. The sliding mass should be divided into narrow slices when using this method.

A method developed by Spencer (1967) examines both overall force and overall equilibrium, so that two expressions for the factor of safety are produced. The interslice forces are assumed to be at a constant inclination and the inclination is found at which both expressions give the same factor of safety. This method may be applied to non-circular slip surfaces.

Morgenstern and Price developed a method of analysis which could be applied to both circular and non-circular slip surfaces. This method was specifically devised to overcome problems with programming other methods of stability analysis for use with computers. The method assumes that interslice shear forces (X) are related to the interslice normal forces (E) by the following:

$$X = \lambda\, f(x)\, E$$

where $f(x)$ is a function which varies continuously across the slip surface and λ is a scale factor. The value of λ and the factor of safety (F) are computed for the assumed function $f(x)$. The factor of safety of a slip surface has been shown to be relatively insensitive to the $f(x)$ used, not varying by more than about 5%, for this reason it is commonly assumed that $f(x) = 1$.

In this method the sliding mass is divided into a relatively small number of vertically sided sections which are generally much wider than the slices used in other similar methods of analysis. Within each of these sections, a slice of infinitesimal width is considered, and with the above assumption regarding the relationship between X and E, the equations for force and moment equilibrium are solved simultaneously.

The analysis involves a complex process of iteration and is therefore, intended to be used with the aid of a computer.

Methods such as the Morgenstern and Price procedure have been modified and improved continually. Detailed accounts of commonly used methods of slope stability analysis are given in several texts eg. Terzaghi and Peck,(1967); Bromhead,(1986); Anderson and Richards (1987); Cedergren,(1967).

The results of the various slope stability analyses above are usually expressed as a factor of safety for the particular embankment. However, the analysis may be adapted to give details such as the slope angle at which a embankment would fail. For circular failure surfaces, the factor of safety is often contoured by writing it against the centre of each slip circle marked on a cross-section of the embankment.

Stress - strain analysis : Finite element methods

Numerical techniques such as finite element analysis can be used to obtain an approximate distribution of the stresses and strains throughout a slope. With modern computers these techniques are extremely powerful and they are particularly useful for analysing the conditions in a stable slope or embankment when it is subjected to changes of loading or geometry. However their use for analysing slopes which are on the point of failure is less satisfactory and in

general their use is limited by the difficulty of modelling the stress-strain behaviour of soil.

This is a sophisticated method which requires very accurate input data if the analysis is to be useful.

5.2.3 Procedure for stability analysis

It is important to understand the theoretical basis of a particular method before applying it in practice.

A thorough site investigation is essential to establish the soil and groundwater conditions, followed by careful soil testing.

Once the soil and groundwater conditions have been established, a method of analysis may be chosen which is appropriate to the anticipated mechanism of failure. In practice the simplest methods are often used initially to give an indication of the magnitude of the problem. When the problem demands more sophisticated methods these can then be used. It is often useful to ascertain the sensitivity of the result of the analysis to small changes in the assumed parameters so that engineering decisions may be based on a full understanding of the problem. One approach is to use probabilistic methods to give a full description of the situation.

5.3 Computer software for stability analysis of embankments

5.3.1 STABLE

There is a range of computer software currently available which can be used to carry out stability analyses of embankments. Though their details vary the approach of many of them is similar. The program STABLE was selected for use in this study as being typical of a class of software. STABLE is a program for assessing the stability of circular or non-circular slip surfaces. Slip surfaces may be in a natural slope, a cutting or excavation, an embankment, or an earth dam. The program can analyse 'pre-existing' slip surfaces i.e. where slippage has already occurred. The program can accommodate soil reinforced slopes.

The slope stability analysis program 'STABLE' allows the embankment geometry and other characteristics such as piezometric surface, to be defined graphically using a CAD (computer aided design) software package. Results of the analysis can also be shown on the graphical display. The program calculates the factor of safety of the slope against sliding, and tabulates information derived from the analysis.

The program is able to apply any of four methods of analysis based on different assumptions as follows,

- Bishop's Method : circular path
- Morgenstern & Price's method : non-circular path
- Sarma's method : non-circular path
- Greenwood's method : circular path.

Bishop's method
If the Bishops slip circle method of analysis is chosen, then the rectangular boundary to the grid of circle centres is drawn directly on to the cross-section of the embankment. The user has the choice of defining a point which the trial circles intercept as either a single point (or tangent), or two points (or tangents) in which case a specified number of interpolated points are taken in between the two. The range of slip circles with different centre points and radii are then analysed, and the surface with the lowest factor of safety is given. If the centre of this circle lies on the edge of the rectangular grid, then analysis should be repeated with another grid of circle centres.

Morgenstern & Price's method
When using the Morgenstern and Price option in 'STABLE', the geometry of the slope under consideration is defined in the same way as for the Bishops option. The slip surface is described however, by a series of straight lines rather than by points defining a circle.

Sarma's method
Sarma's method assumes a non-circular wedge-shaped slip surface, as does Morgenstern and Price, but Sarma is more rigorous, not constraining subdivisions of wedges to vertical slices. By iteration the program locates the optimum slip surface and optimum inter-wedge inclinations.

Greenwood's method
Greenwood's method assumes a circular slip surface - as for Bishop's method - but employs a different governing equation. Greenwood's method permits the modelling of soil reinforcement as a set of straight lines seen in section. Each strip has an associated 'force' value.

5.3.2 CRISP (CRItical State Program)
CRISP (Britto and Gunn, 1990) is a finite element program incorporating critical state models of soil behaviour. The model has been developed by members of the Cambridge University Engineering Department soil mechanics group. CRISP-90, a PC version, can be run in INTEL 80386 processor with a minimum of 2 MB RAM, a maths co-processor, VGA colour monitor, mouse and MS DOS 3.30 or higher.

The model may be used to perform an undrained analysis (to predict behaviour in the short term), a drained analysis (for long term behaviour), or a coupled consolidation analysis. Two dimensional plane strain, three dimensional and axisymmetric solid bodies can be analysed.

Various soil models are available within the package including linear elastic, anisotropic elastic and inhomogeneous elastic properties; critical state soil models including cam clay, modified cam clay and Horslev surface; elastic perfectly plastic models Tresca, von Mises, Mohr-Coulomb and Drucker-Prager.

The program has the ability to specify non-zero in-situ stresses. Boundary conditions include prescribed displacements, pressure loading and pore water pressures.

5.3.3 Software testing

As part of the research, copies of STABLE and CRISP were obtained; a course on CRISP was attended by a member of HR Wallingford staff. The software was used to study the performance of a typical embankment section.

Embankment geometry

The cross-section of the embankment geometry, which was analysed in this study is given in Figure 5.1. It was chosen in order to compare the minimum factor of safety (F) for the different slopes with varied soil types.

Soil characteristics

Three soil types were used in this study; clay, gravel and silty clay. The respective soil characteristics which were required for the stability analyses, are shown in Table 5.1.

Table 5.1 Soil characteristics used in stability analysis

SOIL	ANGLE OF FRICTION ϕ'	COHESION C' (kN/m^2)	UNIT WEIGHT γ
Clay (A)	18°	3	19
Gravel (B)	40°	0	19
Silty clay (C)	21°	2	19

Pore water pressures in the embankment soil were calculated from the position of the specified piezometric surface. This line is shown on Figure 5.2.

Programme of analyses

For each embankment geometry, STABLE using Bishops' method of analysis (ie. circular and non-circular slip surfaces), and CRISP were used to assess the stability of the embankment. These analyses were carried out for the three soil types described above, and for the homogeneous soil cross-section

5.4 Comparison of limit equilibrium and stress-strain methods of embankment stability

Brinkgreve and Bakker carried out analysis of embankments using a finite element formulation.

The finite element method was used to estimate the factor of safety for an embankment under the most dangerous conditions. For the same situation the critical slip circle according to Bishop's method was calculated. A drained analysis was carried out for both the finite element approach and the Bishop approach. Whilst the positions of the slip circles were somewhat different for the two approaches the safety factor in both Bishop's method and the finite element calculation was exactly the same. From this analysis it might be concluded that the finite element method has no advantage in comparison with, simpler, more conventional methods. In the simple case investigated this happens to be true but the finite element method will also detect non-circular slip surfaces and associated factors of safety.

5.5 Impact of changes in pore pressure on embankment stability

As described above, pore water pressures are very important when considering the stability of a embankment. Embankment instability can occur when pore water pressures are out of equilibrium with the boundary conditions of the embankment. This may be due to a recent change in hydraulic boundary conditions. This situation was investigated using the STABLE software.

5.5.1 Embankment geometry

The cross-section of the embankment which was analysed in this study is given in Figure 5.2.

5.5.2 Soil characteristics

Two soil types were used in this study; a clay and a silty clay. The respective soil characteristics which are required for the stability analyses, are given in Table 5.1.

Two soil type arrangements were tested for the embankment, ie. homogeneous soil over the whole cross-section.

5.5.3 Pore water pressures

Pore water pressures in the embankment soil were calculated from the position of the specified piezometric surface. Locations of the piezometric surface at different times during the passage of a flood are presented in Figure 5.2; these were calculated from the seepage model described in Chapter 3.

5.5.4 Programme of analyses

For each condition STABLE, using Bishops' methods of analysis (ie. circular and non-circular slip surfaces), was used to assess the stability of the embankment. These analyses were carried out for the two soil types described above, for the homogeneous soil cross-section and for the different locations of the piezometric surface.

5.5.5 Result

The analysis showed that during a flood event the factor of safety can be significantly reduced. The factors of safety obtained using transient porewater pressure distribution that can occur during a flood event were substantially lower than those under static equilibrium conditions. In some cases the reductions were up to a factor of 10 for the least stable configurations. The lowest factors of safety were obtained at the height of the flood when the water level was at its highest on the outside face of the embankment. The results conclusively indicate that in determining the stability of an embankment it is not sufficient to consider static, equilibrium pore-pressures and that it is necessary to consider the transient pore-water pressure that can develop during the progress of a flood.

6 Flow over Embankments

This section addresses some of the problems related to flow over flood embankments.

6.1 Description of problem

Flood embankments are designed for a particular flood return period. For floods greater than the design flood there is the danger that the water will flow over the embankment. On this report we will distinguish between the case, frequently found in coastal situations, where the mean water level is below the crest of embankment but waves overtop the embankment. We will reserve for this case the term 'overtopping'. Where the mean water level exceeds the height of the embankment and flow takes place over the embankment continuously then this will be referred to as flow over the embankment. In the case of flow over embankments flow takes place down the landward side of the embankment. Depending on the degree of over flowing, the slope of the embankment, the material of the embankment and the protection on the downstream slope, erosion of the embankment may take place. In some circumstances this can be sufficiently severe as to cause the embankment to fail. It is important, therefore, to ensure that embankments should be capable of surviving the expected degree of overflowing.

When flow begins to take place over an embankment the flow velocities on the crest of the embankment are usually low. As the water flows down the landward face, however, it accelerates which can result in very much higher velocities. The flow velocities achieved depend upon the depth of flow and the slope of the embankment. These velocities can be such that the shear stress applied to the embankment may erode material from the embankment face.

In extreme cases the flow down the embankment can become super-critical. Where the flow meets the base of the embankment or, if there is a tailwater level, part way up the embankment, a hydraulic jump can form. This can result locally in high shear stresses being applied to the embankment fill and can result in severe erosion.

Where an embankment has a consistent height then the flow over it is distributed uniformally along the length of the embankment. If there are local features or anomalies in the height of the embankment or its profile, these may act to concentrate the flow in particular areas. Instead of large areas being subject to quite modest shear stresses, which the embankment can withstand, this may result in large shear stresses concentrated at particular locations. This may result in erosion thus leading to the failure of the embankment. Thus the detailing of the design and the maintenance of the embankment can be important.

Small degrees of overflowing may initiate more serious forms of failure. The water overflowing the embankment may percolate into the soil of the embankment and by raising pore water pressures induce slip failures in the inner slope. Alternatively the water percolating into the embankment may trap air inclusions which again may lead to reductions in strength and subsequent failure.

It should be noted that there is frequently a difference in the nature of flow over an embankment between rivers and coastal areas. In a river spillage over an embankment frequently takes place over long lengths and so the spill discharge can be high. It can frequently be significant relative to the total river discharge. In this case the spillage influences the water levels in the river and so limits the depth of flow over the embankment. Except in localised areas, therefore, in a fluvial context the depth of flow over an embankment is normally small.

In a coastal context, the important factor is the seaward water level. In most cases the impact of spills on these water levels is small. Under extreme tidal conditions, therefore, large depths may occur over the top of the embankment.

6.2 CIRIA work on use of grass

The majority of UK embankments are covered with grass or other vegetation and this has a significant effect on the ability of a surface to withstand shear stresses. CIRIA have carried out a number of studies into the use of grass in an engineering context which are valuable in assessing problems associated with embankment overtopping or overflowing (Coppin and Richards, 1990).

Vegetation reduces or prevents soil erosion in a number of ways. The vegetation increases the hydraulic roughness of the surface from that of an unvegetated surface and hence, for a given discharge, reduces the flow velocities. In addition the vegetation can form a protective layer over the soil, preventing the direct application of shear stress to the soil. The root structure of vegetation may also act to stabilise the soil and inhibit erosion.

6.2.1 Hydraulic roughness of grassed surfaces

The hydraulic roughness of a surface can be described by a parameter such as Mannings n, which relates the average flow velocity to the hydraulic radius R and the slope S of a channel using the equation

$$v = R^{2/3} S^{0.5} / n$$

The hydraulic roughness depends upon the morphology of the vegetation, the density of growth and the height of the vegetation. When the flow depth is shallow relative to the height of the vegetation most of the energy is dissipated as form losses caused by the flow around the individual plant stems. As the flow depth increases relative to the height of the vegetation, the plant stems begin to move in the flow, further enhancing energy loss. As the flow depth and velocity increases further the vegetation begins to be flattened by the flow. The vegetation thus occupies less of the flow area and most of the energy loss arises from skin friction. This is normally significantly less than the form losses which occur at smaller depths. The general pattern is thus that as flow depth increases the resistance initially increases and then reduces substantially.

The hydraulic roughness of short grass vegetation is comparable with that for bare soil. As the grass length increases the roughness increases (Figure 6.1). This can lead to a conflict between the need to carry out cutting to maintain the quality of the grass cover and the need to maintain the length to increase the roughness.

For very shallow flow, in which the vegetation is rarely fully submerged, the roughness may be high (Morgan, 1980).

6.2.2 Protection from hydraulic action provided by grass covered surfaces

The protection provided by vegetation can be assessed in terms of the velocity of flow, the quality and length of the vegetation cover and the duration of the flow. Figure 6.2 assesses erosion resistance in terms of velocity and duration for a range of types of cover. If these conditions are exceeded then erosion is likely to occur. Once erosion has been initiated then further loss of vegetation cover and erosion of the soil will occur rapidly. Hewlett et al (1987) gave the following recommendations on permissible velocities that can be withstood by a well selected and maintained grass cover.

2m/s for prolonged periods of over 10 hours
3 to 4m/s for periods of a few hours
5m/s for brief periods of less than 2 hours.

In general, grass can be expected to provide an adequate protection for velocities up to 1m/s. This limiting velocity is likely to decrease, however, as the quality of the grass cover diminishes.

The length of the grass cover is a major factor in the amount of protection provided. Other important factors are:

(a) the density of the stems, foliage and the surface mat
(b) the state of growth and structure of the sward, this is related to the species present and their management
(c) the uniformity of the sward. The presence of tussocks or clumps can lead to concentrations of flow and local weaknesses which can significantly reduce the degree of protection
(d) the flexibility and strength of the stems affect both the hydraulic roughness and the capacity to withstand the hydraulic forces
(e) the season of the year.

A further important factor that must be considered is the capacity of the grass cover to recover from a flood event. A grass sward will require some time to recover its full strength following a flood event. If two floods occur in quick succession then the grass cover will not have a chance to recover from the first attack before it is inundated further. In these circumstances, if there is no opportunity to carry out remedial work then it is the total duration of flooding that is important.

6.2.3 Other forms of embankment protection

The above work has been based upon the assumption that the downstream surface of the embankment is grassed. The methods developed, however, can be applied to consider the risk of eroding any surface. The only requirement is that the hydraulic roughness of the surface and the critical velocity for erosion are known. The equation for the discharge overflowing the embankment and the Mannings equation still apply to the flow characteristics. The appropriate calculations can then be performed to determine the degree of flow over the embankment that will result in surface erosion downstream.

6.3 Description of flow

The flow down the landward side of the embankment is determined by the discharge that flows over the embankment.

An extensive laboratory study was carried out by the US Geological Survey on the discharge characteristics of embankment shaped weirs, (Kindsvater, 1964). This study looked specifically at the hydraulics associated with flow over embankments. As part of the study a series of laboratory experiments were carried out at scales ranging from 1 in 6 to 1 in 12. These tests looked at many aspects of the problem including:

- water-surface profiles,
- velocity distributions,
- influence of boundary layer development,
- influence of boundary form on flow characteristics,
- influence of surface roughness,
- influence of geometry.

If more information is required on any of these aspects than is contained in this report then reference should be made to Kindsvater (1964) and the references contained therein.

The study identified different flow conditions dependent on the influence of the tailwater level. For low tailwater levels, the discharge is determined by the upstream head. This was referred to as free-flow by Kindsvater but using weir terminology is now more frequently referred to as modular flow. At high tailwater levels, the discharge is also influenced by the tailwater level. This was referred to as submerged by Kindsvater but the normal weir terminology would now be drowned. In the case of flow over of embankments the more severe conditions generally occur during modular flow. Depending on the nature of the landward conditions and the rate of rise of the river, it is likely that drowned conditions will only occur following a prolonged period of modular flow. For this study attention has, therefore, been restricted to the case of modular flow.

The flow over embankments bears some similarities to flow over a broad-crested weir. It follow that the type of parameters involved are very similar: these are:

- the profile of the weir,
- the upstream water level, and
- if the flow is non-modular, the downstream water level.

6.3.1 Modular-flow discharge equation

Both theoretical considerations of the flow of an ideal fluid and comparisons with other weir equations leads one to the consideration of a discharge equation of the form:

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

where q is the discharge per unit length and H is the upstream head relative to the crest of the embankment. Kindsvater showed that, for a range of heads, the coefficient C had a value of approximately 3.0 when q and H were expressed in imperial units. It is this equation that has been used in the

subsequent analysis. Thus the discharge over the embankment is dependent on the upstream head. In the context of an embankment this is the water level on the river or seaward side. For an estuarial or sea embankment, the occurrence of flow over the embankment is unlikely to affect the upstream head. For a river embankment, however, the spillage of water over an embankment may influence the water level. This may lead to a change in the upstream head along the length of the embankment. In these situations the embankment will act as a side-weir. For the analysis of such cases reference should be made to standard hydraulics text books such as Chow (1959).

6.3.2 Flow on landward face of embankment

For modular flow the unit discharge is determined by the upstream head. The velocity and depth of flow on the landward side is then determined by the slope of the embankment and the roughness of the surface. For embankments covered with vegetation the roughness is determined by the hydraulic roughness of the vegetation.

The CIRIA study of the use of vegetation in civil engineering provides information on the relationship between the hydraulic roughness of vegetation, as expressed by the Mannings n value, and the product of the velocity and depth VR. Different relationships are given for different vegetation lengths, see Figure 6.1. For a given vegetation length this provides an equation linking the n value with the velocity and hydraulic radius.

The further one progresses down the embankment the closer will the flow approximate to normal flow. This can then be used as a method of estimating the flow characteristics. The assumption of normal flow provides an equation linking the velocity, hydraulic radius, slope and roughness.

6.4 Acceptable degree of overtopping

The question for normal flow together with the discharge equation for the embankment can be solved to give the corresponding value for the flow velocity. The flow velocities that are derived depend upon the upstream head of water relative to the embankment and the slope of the embankment. The velocities can then be compared with the information on the velocities that can be withstood by vegetation that were discussed above.

6.4.1 Avoiding supercritical flow on the downstream face

If supercritical flow occurs on the downstream face then at or near the base of the embankment an hydraulic jump is likely to form. This will subject the embankment to high shear stresses and could cause severe erosion. We will now derive the conditions necessary to prevent supercritical flow occurring on the downstream face. We will assume that:

(a) The overtopping discharge is given by.

$$q = C\,h^{3/2} \tag{6.1}$$

(b) Sufficiently far down the embankment the flow approximates to normal flow.

The condition that the flow is sub-critical is that the Froude number is less than 1, that is,

$V^2 < gd.$ (6.2)

Equation (6.1) implies that

$vd = C\ h^{3/2}.$ (6.3)

Assumption (b) implies that

$$v = \frac{d^{2/3}S^{0.5}}{n},$$

So that Equation (6.3) becomes

$$\frac{d^{2/3}S^{0.5}}{n}\ d = C\ h^{3/2},$$

or,

$$d = \left(\frac{C\ h^{3/2}n}{S^{0.5}}\right)^{3/5}.$$

From Equation (6.3) we have that

$$V = \frac{C\ h^{3/2}}{d} = C\ h^{3/2}\left(\frac{S^{0.5}}{C\ h^{3/2}n}\right)^{3/5} = \frac{C^{2/5}h^{3/5}S^{3/10}}{n^{3/5}}.$$

The inequality (6.2) becomes

$$V^2 = \left(\frac{C\ h^{3/2}}{d}\right)^2 < gd = g\left(\frac{C\ h^{3/2}n}{S^{0.5}}\right)^{3/5},$$

that is,

$$\left(\frac{C^{2/5}h^{3/5}S^{3/10}}{n^{3/5}}\right)^2 < g\left(\frac{C\ h^{3/2}n}{S^{0.5}}\right)^{3/5}$$

thus to ensure sub-critical flow

$$h^{3/10} < \frac{g\ n^{9/5}}{C^{1/5}S^{3/10}}$$

This condition gives, for a specified downstream slope S, the limit on the overtopping depth h before super critical flow occurs.

6.5 Development of a breach in an embankment

There is little available information on the way in which a breach in an embankment develops. As the discharge over an embankment increases the flow velocity increases and it may reach a point at which the flow begins to erode the embankment. Normally this first happens locally. As erosion increases the size of the breach and lowers the crest levels, the discharge increases. The rate of erosion can be large and the increase in the size of the breach can be rapid. If the tailwater level rises or the upstream level falls then the erosion rate will gradually reduce until the flow velocity is unable to erode any further material.

Once the breach has developed so that it is a substantial proportion of the height of the embankment then the highest flow velocities occur immediately downstream of the breach. The scour in this area may be substantial.

There are limited observations on the width development of breaches (Marsland, 1984 and CUR, 1990). These demonstrate that breach development is most rapid initially and gradually reduces in time, see Figure 6.3. The ultimate width of the gap must depend upon upstream water levels, the inundated area and the composition of the embankment, but as yet there is no reliable method for estimating the breach width.

In Section 6.1 the differences between flow over embankment in the river and coastal situations was discussed. It is likely that these differences will also affect the development of a breach. As a breach develops in a river flood embankment then the flow through the breach will reduce the river level and so act to inhibit the breach development. In a coastal situation, the upstream water level is frequently imposed by the tidal level and will not be affected by the flow through the breach. In this situation there are fewer constraints on the breach development. In the short term the development of a breach is likely to be the same for both situations but the long term development of the breach is likely to be very different between the two cases.

6.6 Probabilistic approach to flow over embankments and damages

For any embankment there is always the risk that it will fail. One of the modes of embankment failure is overtopping and so the risk of overtopping should be considered. An important decision that a designer has to take is the appropriate height of the crest of the embankment. In the past this has frequently been based on the flood levels with a specified return period, with an appropriate allowance for free-board. Though the selection of the appropriate return period has normally been based on the nature of the area to be protected it is rare that detailed calculations are performed of the damage costs associated with overtopping. If these are carried out then a sounder basis for the economics of the construction of a flood embankment can be obtained. Such an approach has been developed and described in 'Probabilistic design of flood defences' which was written by the Dutch Technical advisory committee on water defences CUR(1990). This section is loosely based on this work. The approach is based on:

(a) estimating the probability of a given level of overtopping
(b) for the specified level estimating the volume of water that overtops
(c) converting the given volume of water into a depth of flooding
(d) for the given depth of flooding determining damage costs
(e) integrate damage costs with original probabilities to give overall risk of damage due to overtopping

The marginal cost of incrementing the height of the embankment can be compared with the resulting reduction in damages. As a result it is possible to determine the optimum economic height of the embankment. It must be realised that such an economic analysis may not take into account all the factors that may affect the degree of protection offered.

In this section attention is restricted to the risk of overtopping. The method can, however, be extended to include other possible failure mechanisms. Thus the probability of breaching can be included in this type of analysis.

For each water level, there is a conditional probability that the embankment will breach, which may be determined by, for example, probabilistic slope stability analysis. The breach will lead to a certain damage cost. This can be included in the risk analysis by multiplying the water level frequency with the conditional breach probability, and integrating the probabilities with cost as above. This is the approach being recommended by HR Wallingford for use by the NRA. (Risk Assessment for Sea and Tidal Defence Schemes, NRA R & D Contract CO6/459).

6.6.1 Probability of overtopping

The embankment will be overtopped when the water level exceeds the embankment level. The probability of overtopping can be considered by assessing the probability distributions for the water level and embankment heights. We can define the function Z by

$$Z = h_a - S_v - S_p - Z_k,$$

where h_a is the construction height of the embankment, S_v is the water level, S_p is the uncertainty in the water level and Z_k represents the effect of settlement of the embankment. The probability of overtopping is then equivalent to the probability that Z is less then zero. To assess this probability one must estimate the probability of a given water level. Within the context of the UK, the methods described in the Flood Studies Report can be used to determine flood hydrographs with a range of return periods. The output from such an analysis is normally in terms of discharge but hydraulic calculations can be carried out to determine corresponding water levels. If consideration is only being given to one point then these calculations will be relatively straight forward. If a long reach of a river is being studied then recourse will have to be made to a numerical model. Whichever method is adopted, the calculations will provide the water levels on the channel side of the embankment during the flood event.

6.6.2 Volume of overtopping

If the water levels exceed the specified embankment height then overtopping will take place. The discharge over the embankment for a given upstream water level can be determined by regarding the embankment as acting as a weir. An extensive laboratory study was carried out by the US Geological

Survey on the discharge characteristics of embankment shaped weirs (Kindsvater, 1964). If the discharge over the embankment is substantial then the impact on the river discharge should be considered. These calculations can usually be carried out by hand. If a numerical model is being used to determine the river levels then it is likely that the same model could be used to determine the overtopping discharge.

By summing over the hydrograph the total volume of water that overtops the embankment can then be determined. By carrying out this calculation for a range of hydrographs the overtopping volume associated with given return periods can be determined.

If substantial overtopping occurs then consideration should be given to the impact on the embankment. The downstream face may be eroded leading to erosion of the crest and possibly even to partial or total failure of the embankment. In these circumstances the total volume passing through the embankment may increase substantially. Once a breach occurs in an embankment then scour can lead to a rapid increase in the size of the breach. In time the headloss across the embankment will reduce and erosion will cease. The rate of increase of the breach and the ultimate size of the breach will depend upon the hydraulic conditions both upstream and downstream of the breach and the composition of the embankment and sub-soil.

6.6.3 Converting the volume of overtopping into a depth of flooding

Once water has overtopped an embankment then its ultimate fate depends upon the local topography behind the embankment. The water may just pond immediately behind the embankment or it may flow a considerable distance. In some cases it may even return to the river further downstream. The assessment of the fate of such water can only be based on a local inspection of the area. In the following analysis it will be assumed that the water will pond in a known area and that for this area it is possible to determine a relationship between depth and volume. Thus for a given volume of overtopping the corresponding depth of flooding could be determined.

6.6.4 Determination of the damage costs

Damages due to flooding are normally related to the depth of flooding and in some cases to the velocity of flow. The amount of damage is related to:

(1) the depth and duration of inundation and the quality of the water,
(2) the characteristics of the flooded area in terms of size and nature. Other factors may be important in certain circumstances such as the availability of reliable flood warning.

Much work has been done on the assessment of damages related to flooding, see for example Penning-Rowsell and Chatterton (1977) and Parker et al (1986). These works provide methods to assign damages to different depths of flooding.

6.7 Assessment procedure

There is always a risk that any flood embankment will be overtopped. There is a further, smaller risk that the degree of overtopping will be sufficient to cause erosion of the downstream face of the embankment. To assess the conditions under which erosion could take place one must know the following:

(1) the crest height of the embankment,
(2) the downstream slope of the embankment,
(3) the nature of the vegetation cover on the embankment,
(4) the flood levels and durations for different return periods. The information given above can be used to assess the risk of overtopping resulting in damage to the embankment:

(a) the nature of the vegetation can be used to select an appropriate figure from Figures 6.4 to 6.12,

(b) for the selected return period the duration is used to determine the appropriate curve on the figure,

(c) from the crest height and the flood level, the amount of overtopping can be determined. This together with the downstream slope can be used to determine whether damage will occur.

Following this procedure curves have been produced which indicate the maximum amount of overtopping relative to the crest that can be tolerated before damage is likely to occur, see Figures 6.4 to 6.12. There are different sets of figures for poor, medium and good vegetation cover. In each set of figures there are figures for short, medium and long grass cover. In each figure the maximum overtopping head is plotted against downstream slope, with different curves for different durations.

6.7.1 Use of figures

For an existing embankment, the quality and length of the grass cover can be used to select the appropriate figure. Information on the duration of floods at the site can be used to select the appropriate curve and then the downstream slope can be used to determine the amount of overtopping that can be tolerated before damage is likely to occur. The figures can also be used to assess the designs of embankments.

The engineer must be aware of the problems that might arise due to multiple flood events. Account should be taken, therefore, of the frequency of such events and the nature of any maintenance work that might be carried out.

The above calculations cannot take into account the effect of local features on the flow. Any local features that could act to concentrate the flow will reduce the return period at which damage may occur. Also any local damage to the vegetation cover may also lead to premature damage.

The dependence of the limiting velocity on the state of the vegetation cover highlights the need to consider the maintenance of the vegetation to ensure that it is always in a fit state to afford the required degree of protection. Both surface cover and root density decline during dormancy which in the UK is unfortunately when the period of greatest erosion risk occurs. Regular

inspection is normally required and if there are areas of vegetation failure then remedial action should be taken as soon as possible.

6.8 Maintenance

During overtopping the main threat to embankments comes from local anomalies and unevenness in the height of the embankment. Maintenance should be directed at minimising these.

A maintenance programme should be prepared to maintain the required properties of the vegetation. This programme should address the problems of the establishment and continued management of the vegetation. Such management is usually aimed at controlling the height and density of the sward and as a contribution to this manipulating the species that are present. Consideration has to be given to the impact of cutting, grazing and the use of chemicals and fertilisers. For more information on this subject the reader is directed to Coppin and Richards (1990).

There is normally a conflict between the need to cut vegetation to maintain the sward and the desire to maximise the length of the vegetation to provide the greatest degree protection. How this conflict is resolved will depend upon local circumstances and the risk of varying levels of overtopping.

7 Conclusions and recommendations

Embankments are frequently used in the UK for flood defence of coastal or flood plan areas. As a result the UK has an extensive system of flood embankments. A survey was carried out by questionnaire of the hydraulic performance of flood embankments. This questionnaire was circulated to all the regions of the National Rivers Authority. The responses to the questionnaire indicated that the problems associated with embankments are typically:

Groundwater seepage below or through the embankment,
Flow over embankments,
Surface erosion resulting from:

- flows overtopping the embankment,
- river flood flows,
- wave action,
- operation of sluices and gates.

Mass failure.
Land drainage problems due to embankments and flap gates.
Maintenance and the control of vegetation.

The direction of this research project was altered in the light of the responses to the questionnaire but it was not possible, within the constraints of the project, to address all the problems that were identified. Consideration should be given to whether further research is required on these topics, particularly:

(1) Surface erosion resulting from:
- river flood flows
- wave action
- operation of sluices and gates.

(2) Mass failure

(3) Land drainage problems due to embankments and flap gates

(4) Maintenance and the control of vegetation

Item (4) falls outside the experience of HR but the other topics would certainly be amenable to further research. The responses to the questionnaire suggest that such research would be economically justifiable.

A study was carried out of the problem of seepage below a flood embankment. The study showed how the dimensionless constant used by Singh and Stall (1971) to characterise aquifers could be modified to characterise the response of the aquifer to a flood cycle.

If t is the period of the flood cycle
L is the width of the flood plain or flood berm
S is the aquifer storage coefficient and
T is the aquifer transmissivity

then τ is given by Tt / SL^2

and the aquifer response time τ_* is given by SL^2 / T

The nature of the response of the aquifer to the flood event can therefore be characterised by comparing the aquifer response time to the duration of the flood event.

$\tau \geq 10t$ If τ_* is of the order of ten times the flood cycle period or more then the groundwater response will be small.

$\tau \approx t$ If τ_* is of the same order as the flood cycle period then the groundwater responses may be considerable and are liable to cause problems.

$\tau \leq 0.1t$ If τ_* is of the order of one tenth of the flood cycle period or less then the aquifer will respond fully to the flood hydrograph right across the valley.

Where it is identified that groundwater response may reduce the effectiveness of an embankment, remedial measures should considered.

The factors affecting the stability of an embankment are discussed and methods to determine the stability are described. In applying such methods it is normal practice to assume steady pore-water pressures in the embankment corresponding to normal water levels in the region of the embankment. Analysis using transient pore-water pressures obtained during the course of a flood event indicates that in these circumstances the stability of the embankment may be reduced. This explains why failures are sometimes observed to occur during the recession of the flood. During the flood, pore-water pressures in the embankment are raised and during the recession these are not dissipated as quickly as the flood level falls. The transient pore-water pressures act to reduce the stability of the embankment. The present report indicates that methods of transient groundwater flow and

stability analysis can be used to predict this reduced stability. It is recommended that such a transient analysis is carried out when designing or assessing embankments so that failure due to transient pore-water pressure effects may be avoided in the future.

Embankments are sometimes used to protect against floods with relatively modest return periods. In these cases the overtopping of such embankment is not unusual. In these circumstances the embankment should be designed to withstand overtopping.

In this report previous work on the ability of grass to withstand flow and the hydraulic characteristics of flow over embankments has been put together to provide curves which describe the acceptable degrees of overtopping in terms of the embankment characteristics and the condition of the embankment. Figures 6.3 to 6.12 give the acceptable degrees of overtopping for different embankment geometries and degrees of vegetation cover. The degree to which flow may safely occur over an embankment depends upon the nature of the vegetation on the embankment. Thus maintenance of the embankment and the vegetation is important. A suitable maintenance programme should therefore be prepared.

It is also advisable to avoid supercritical flow developing on the downstream face of the embankment. A condition is given that must be satisfied to avoid supercritical flow.

Once a breach is created there is little information on the rate at which the breach will develop. The rate at which a breach develops affects the volume and hence extent of flooding that will occur in the event of a failure. With the increasing use of risk analysis in the design of embankments there is increasing emphasis on determining both the probability and the consequences of a failure. It is in determining the consequences of failure that the rate of breach development is of great importance. It is recommended that further research is carried out on this topic. Ideally one would carry out experiments at full scale but this has many practical difficulties. It is likely that one would have to resort to a combination of physical model, numerical modelling and material testing of the soil properties of existing embankments. Such research is necessary if the risk analysis of river embankments is to have any justification.

There is always a risk that flow will occur over an embankment and flooding will result. The design of the embankment should take into account the probability of such flow and the damage that results. A framework for carrying out such an analysis is described.

8 References

Anderson, M.G. and Richards, K.S. (ed.)(1987) 'Slope Stability -Geotechnical Engineering and Geomorphology' Wiley.

Bishop, A.W. (1955). The use of the slip circle in the stability analysis of earth slopes. Geotechnique 5, pp 7-17.

Brinkgreve, R.B.J. and Bakker, H.L. (????). Non-linear finite element analysis of safety factors.

Britto, A.M. and Gunn, M.J. (1987). Critical State soil mechanics via finite elements. Ellis Horwood Ltd.

Britto, A.M. and Gunn, M.J. (1990). CRISP 90. User's and Programmers's Guide.

Bromhead, E.N. (1986). 'The Stability of Slopes' Blackie & Son Ltd.

Cedergren, H. R. (1967). 'Seepage, Drainage and Flow Nets' Wiley, New York.

Chow, V.T. (1959). Open-Channel Hydraulics. McGraw-Hill.

Cole, J. 1994, Breaching of flood embankments, Program Report, Oxford Brookes University/HR Wallingford.

Coppin N.J and Richards I.G. (1990). Use of vegetation in Civil Engineering. Butterworths.

CUR (1990), Probabilistic design of flood defences, Centre for Civil Engineering Research and Codes, The Netherlands, Balkema

Fellenius, W. (1936) Calculation of the stability of earth dams. Trans. 2nd Congr. on Large Dams, Washington, 4. pp 445-459.

Hewlett H.W.M, Boorman L.A, and Bramley M.E. (1987). Design of reinforced grass waterways. CIRIA.

Janbu, N, Bjerrum, L, and Kjaernsli, B. (1956). Soil mechanics applied to some engineering problems. Norwegian Geotechnique Institute Publication 16.

Kindsvater C.A. (1964). Discharge Characteristics of embankment shaped weirs. United States Geological Surveys Water Supply Paper 1617-A.

Marsland, A. (1984), Controlled breaching of a full scale clay flood bank, Proc Int Conf of Case Studies in Geotech Enging. St Louis, Missouri, USA, 1, p465.

Morgan, R.P.C. (1980), Field studies of sediment transport by overland flow. Earth Surface Processes, 5, pp 307-316.

Morgenstern, N.R. and Price, V.E. (1965). A numerical method for solving the equations of stability of general slip surfaces. Computer Journal 9, pp 388-393.

Naguchi, M. (1989). Hydraulic and Hydrological Problems in Japan, Nagasaki University, Japan.

Nash, D. (1987) 'A Comparative Review of Limit Equilibrium Methods of Stability Analysis' IN Anderson, M.G. and Richards, K.S. (ed.)(1987) 'Slope Stability - Geotechnical Engineering and Geomorphology' Wiley.

Nutbrown, D.A. and Downing, R.A. (1976) Normal-Mode Analysis of the Structure of Baseflow Recession Curves, Journal of Hydrology, 30, pp 327-340.

Parker D.J, Green C.H and Thompson P.M. (1986). Urban Flood protection benefits : a project appraisal guide.

Penning-Rowsell E.C, Chatterton J.B, Farrell S.J, Salvin S.H and Witts R.C. (1977). The benefits of flood alleviation : A manual of assessment techniques.

Raudkivi, A.J. and Callander, R.A. (1976). Analysis of Groundwater Flow. Edward Arnold, London.

Skempton A.W. and Delory F.A. (1957). Stability of natural slopes in London Clay. Proc. 4th Int. Con. on Soil Mechanics Foundation Engineering, London 2, pp 378-381.

Singh, K.P. and Stall, J.B. (1971). Derivation of Baseflow Recession Curves and Parameters, Water Resources Research, 7(2), pp 292-303.

Spencer E.E. (1967). A method of the analysis of the stability of embankments assuming parallel inter-slice forces. Geotechnique 17, pp 11-26.

STABLE. (1992). Program for the stability analysis of slopes. Mott MacDonald software.

Taylor, D.W. (1948) 'Fundamentals of Soil Mechanics' Wiley, New York.

Terzaghi, K. and Peck, R.B. (1967) 'Soil Mechanics in Engineering Practice' Wiley, New York.

Watkins, D.C. (1988). Groundwater Flow Beneath Flood Embankments, Hydraulics Research Report No. SR 169.

Figures

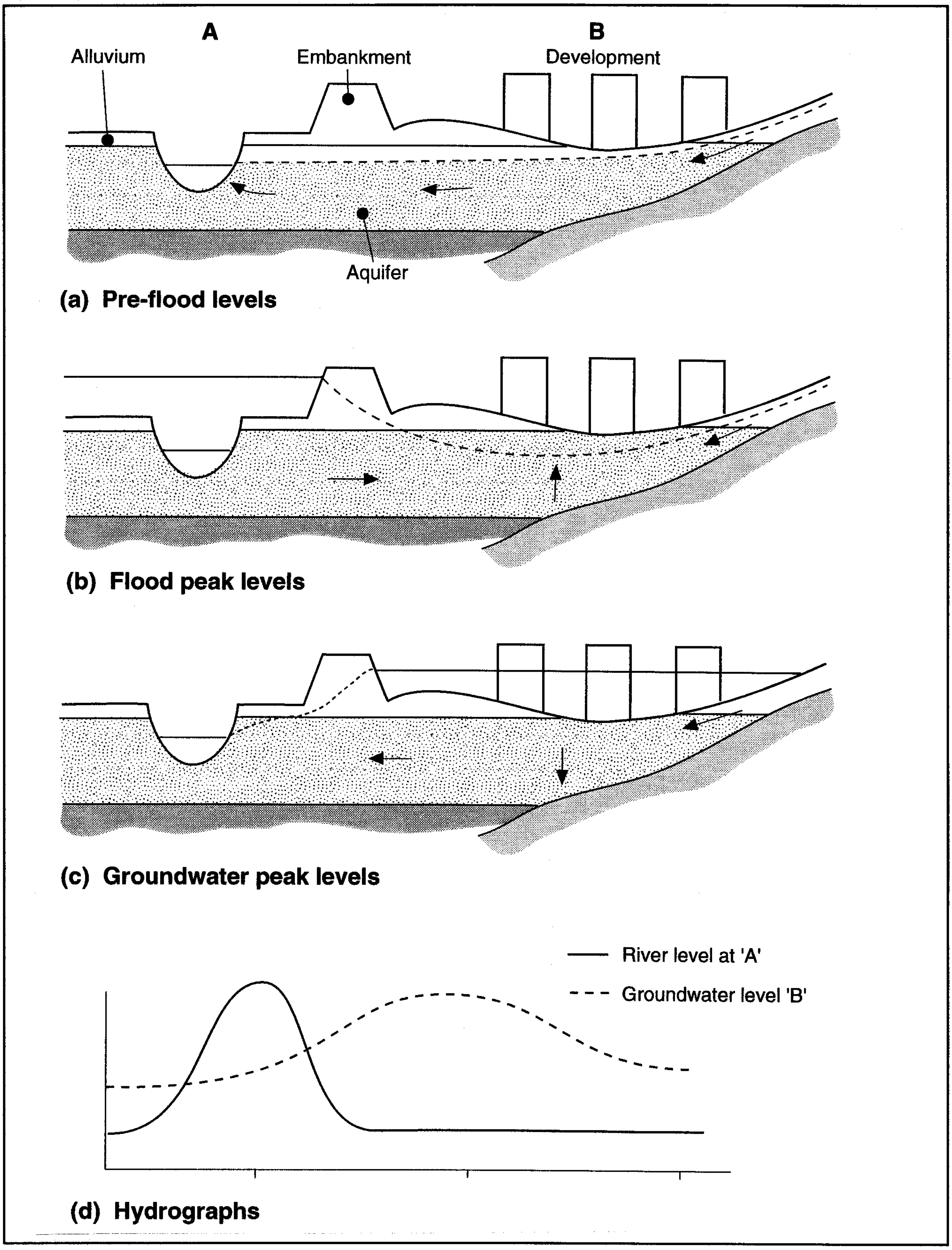
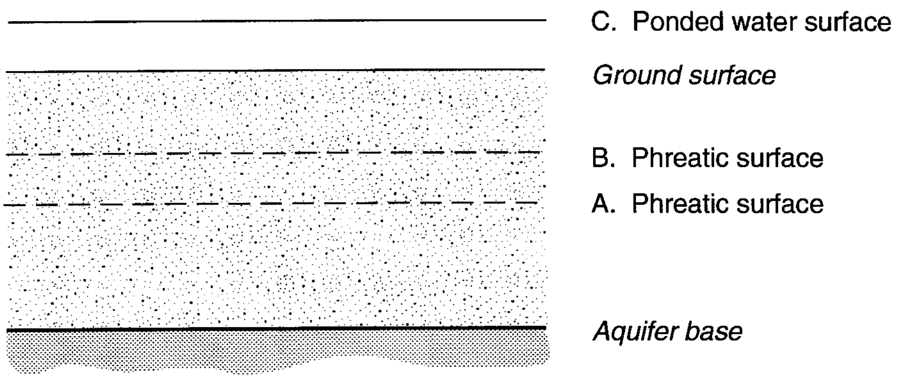
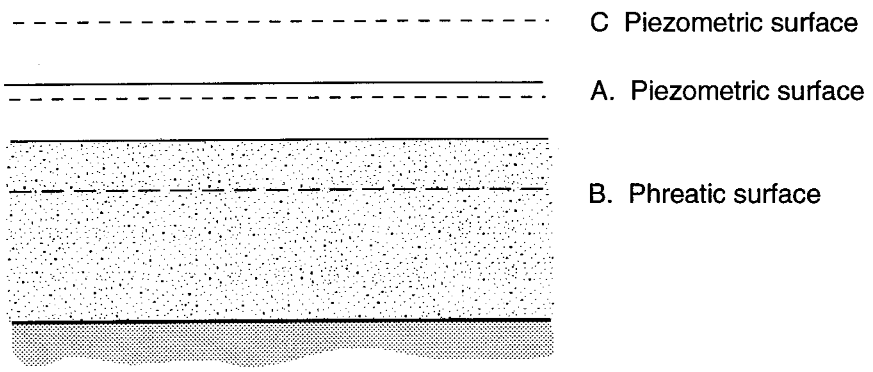


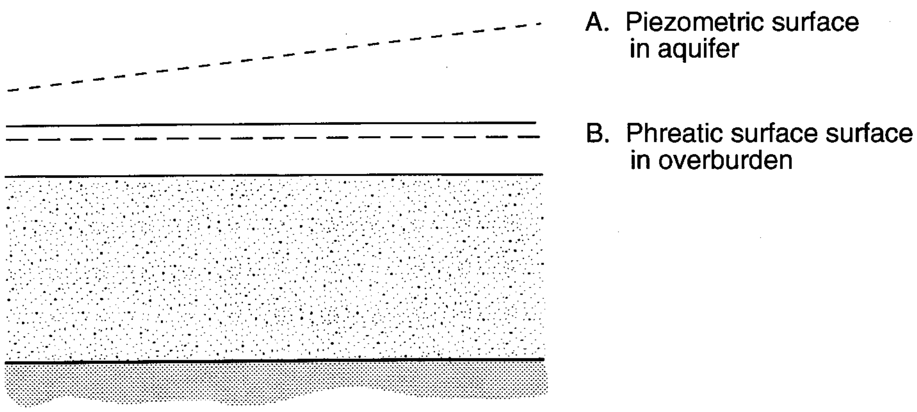
Figure 3.1 The groundwater flow system



(a) Unconfined aquifer



(b) Confined aquifer



(c) Semi-confined aquifer

Figure 3.2 Types of aquifer

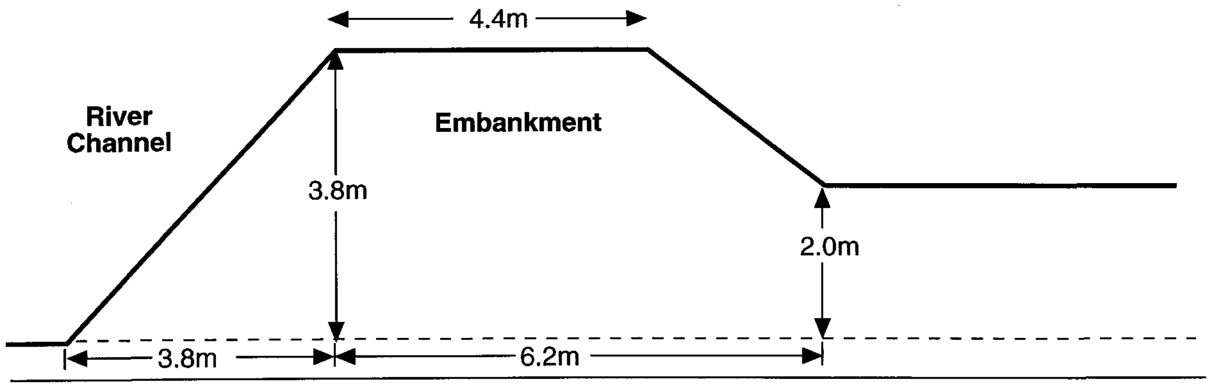
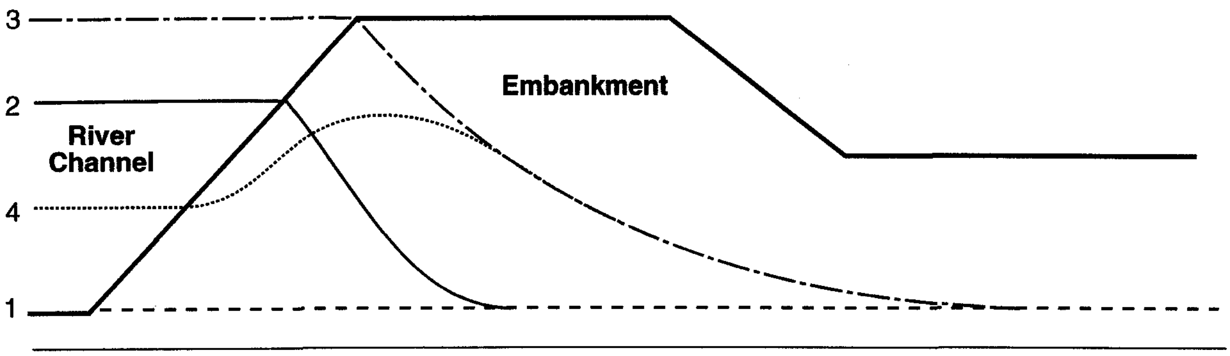


Figure 5.1 Cross-section of embankment geometry used in software testing



Key: Water level in channel and piezometric surface in embankment

1 - - - - - - *No water in channel*

2 ——— *Water in channel rising*

3 —·—·— *Water level in channel reaches top of embankment*

4 ········· *Water level in channel falls as flood passes*

Figure 5.2 Piezometric surfaces used in testing the impact of pore pressure on embankment stability

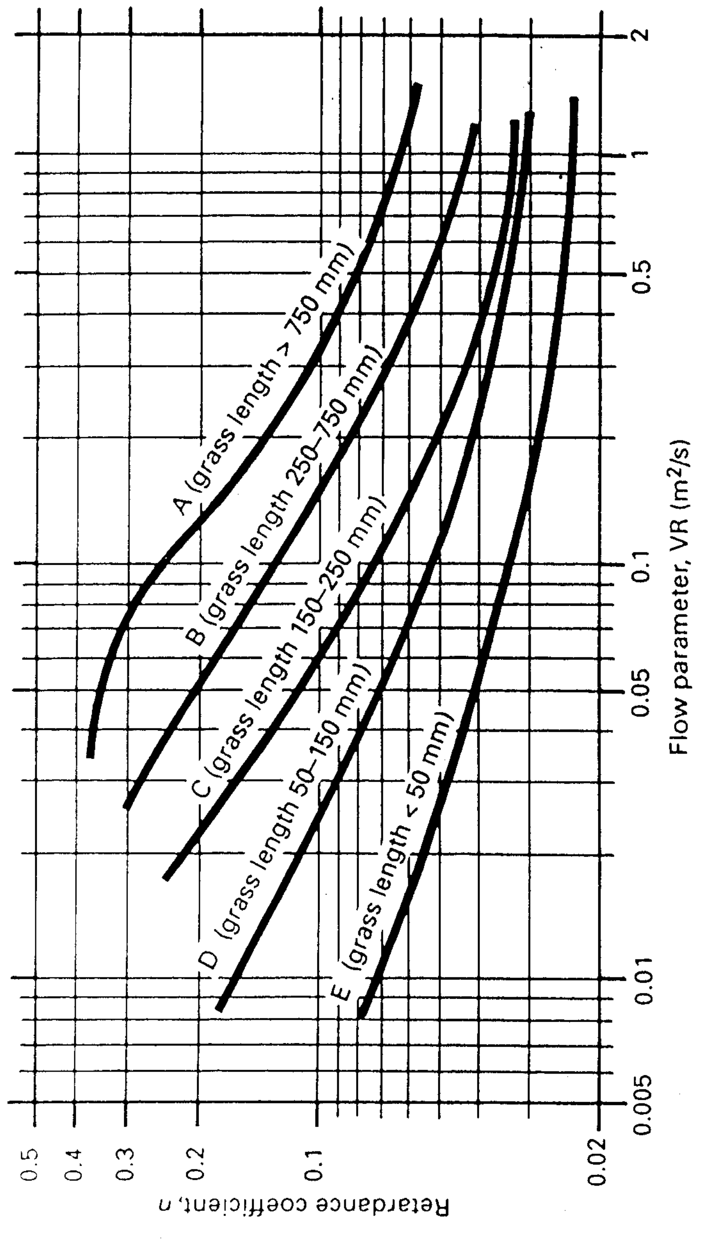
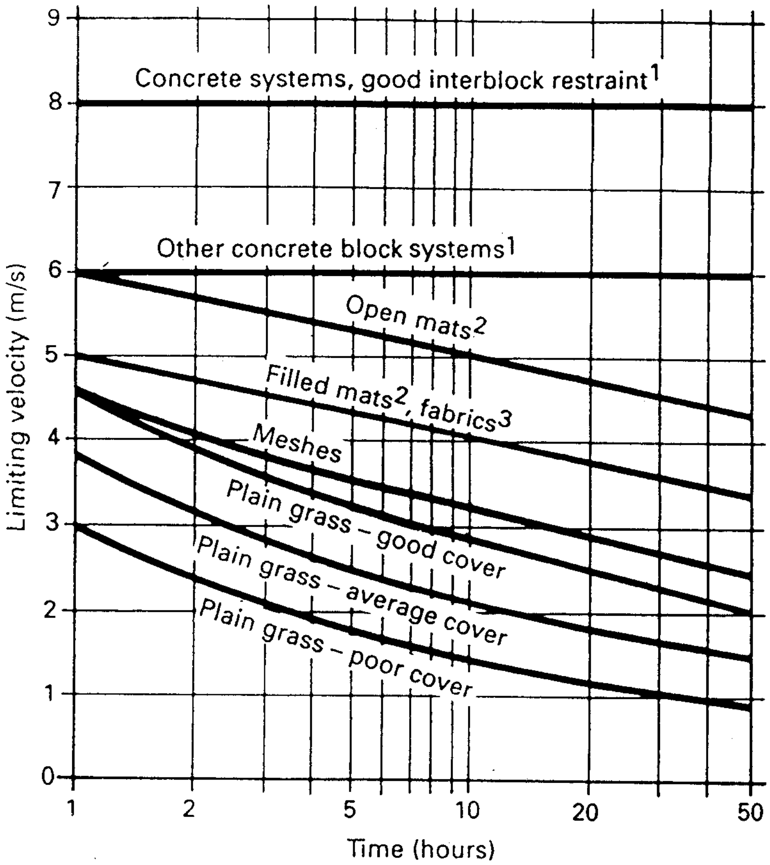


Figure 6.1 Frictional resistance of grass swards for various retardance categories (after USSCS, 1954)



Notes

1 Minimum superficial mass 135 kg/m^2
2 Minimum nominal thickness, 20 mm
3 Installed with 20 mm of soil surface or in conjunction with a surface mesh

All reinforced grass values assume well established good grass cover.

Figure 6.2 Recommended limiting velocities for erosion resistance of plain and reinforced grass against unidirectional flow (after Hewlett *et al*, 1987)

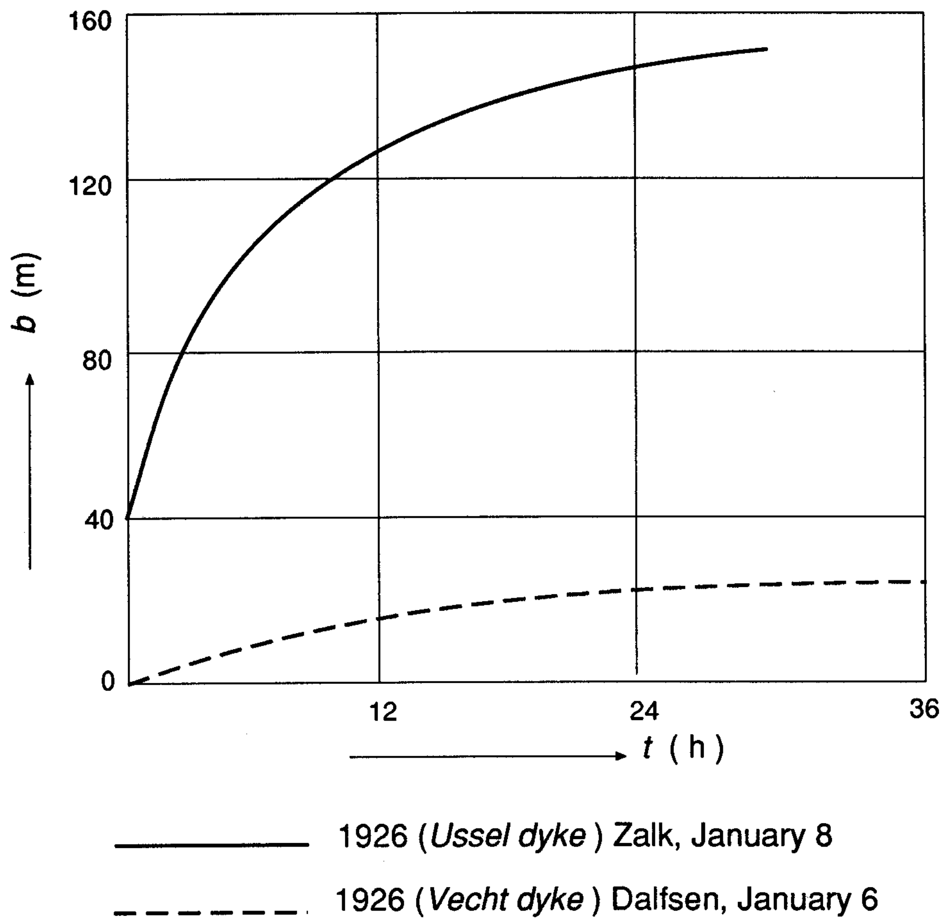


Figure 6.3 **Growth of a gap with width *b* in the case of two dyke breaches**

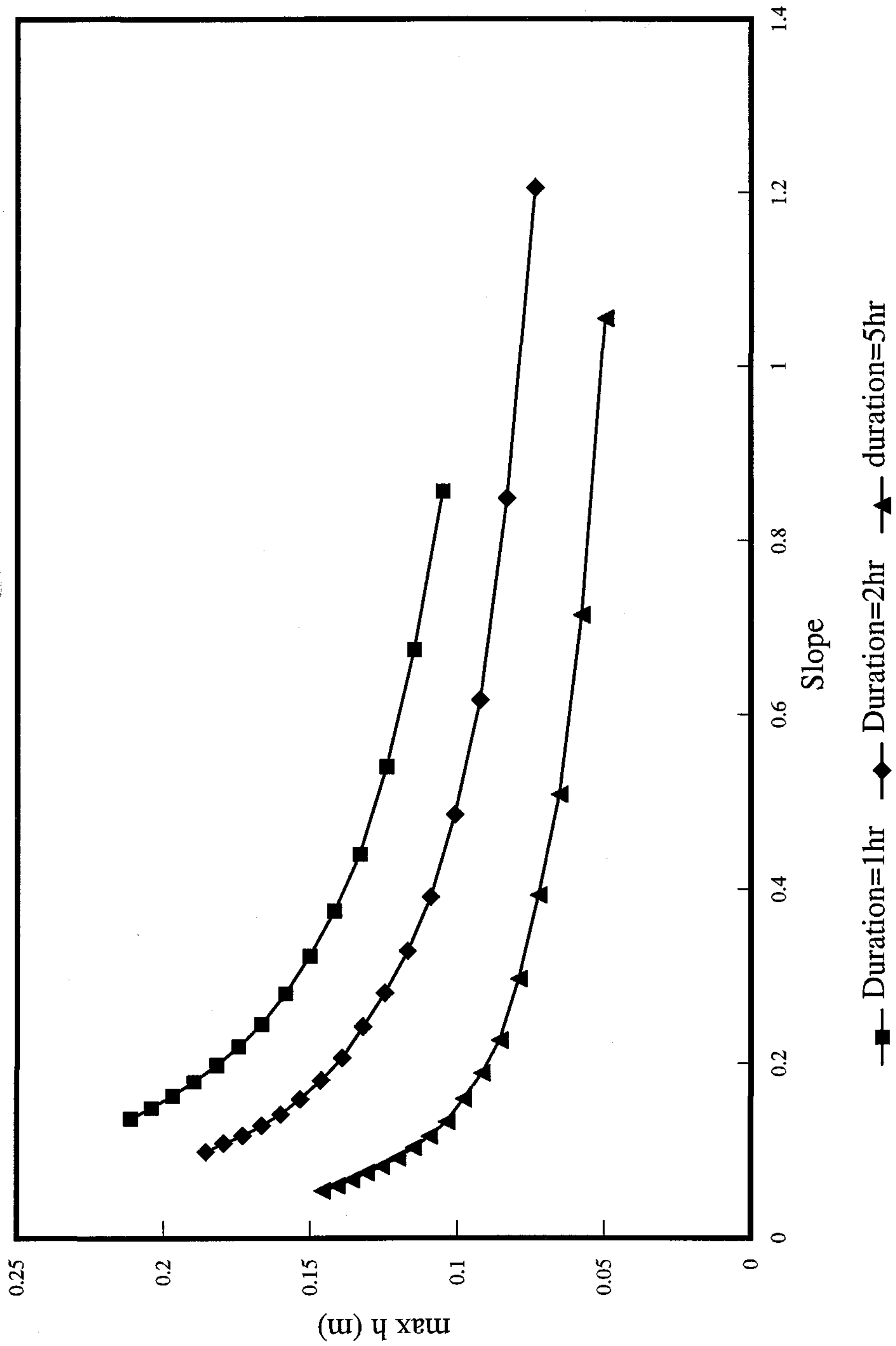


Figure 6.4 **Maximum allowable depth of overtopping: Poor cover, short grass (<50mm)**

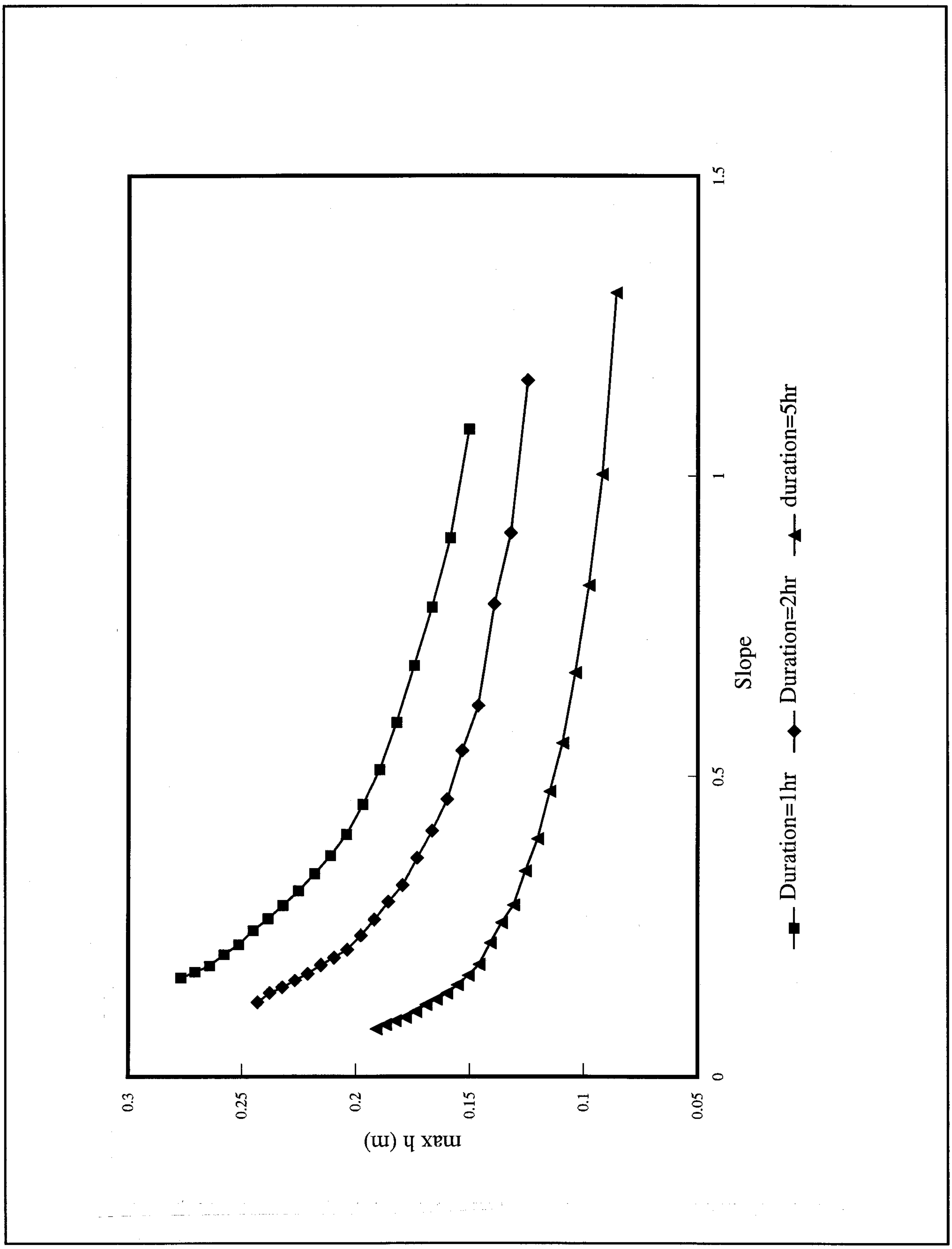


Figure 6.5 Maximum allowable depth of overtopping: Poor cover, medium grass (50-250mm)

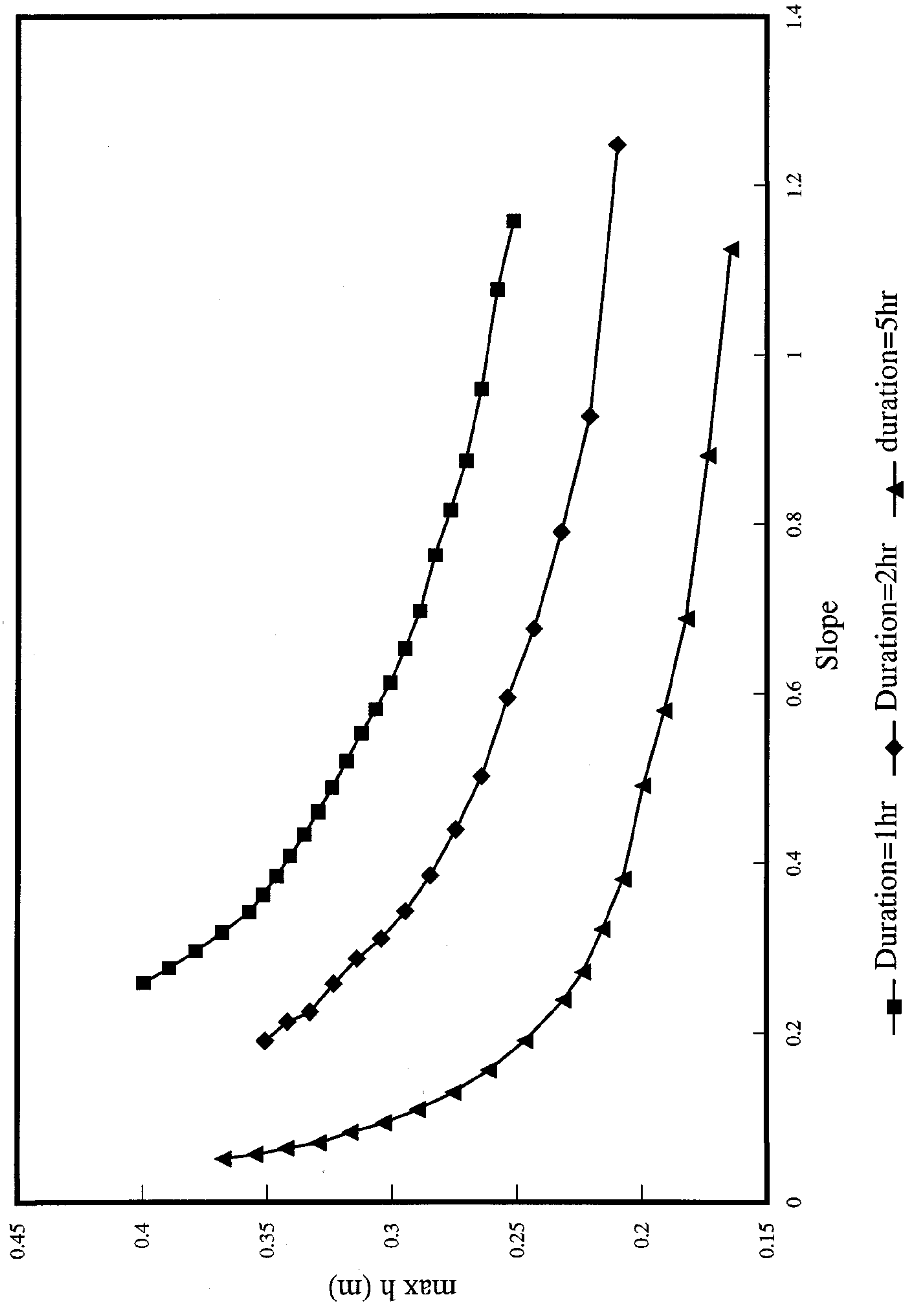


Figure 6.6 **Maximum allowable depth of overtopping: Poor cover, long grass (>750mm)**

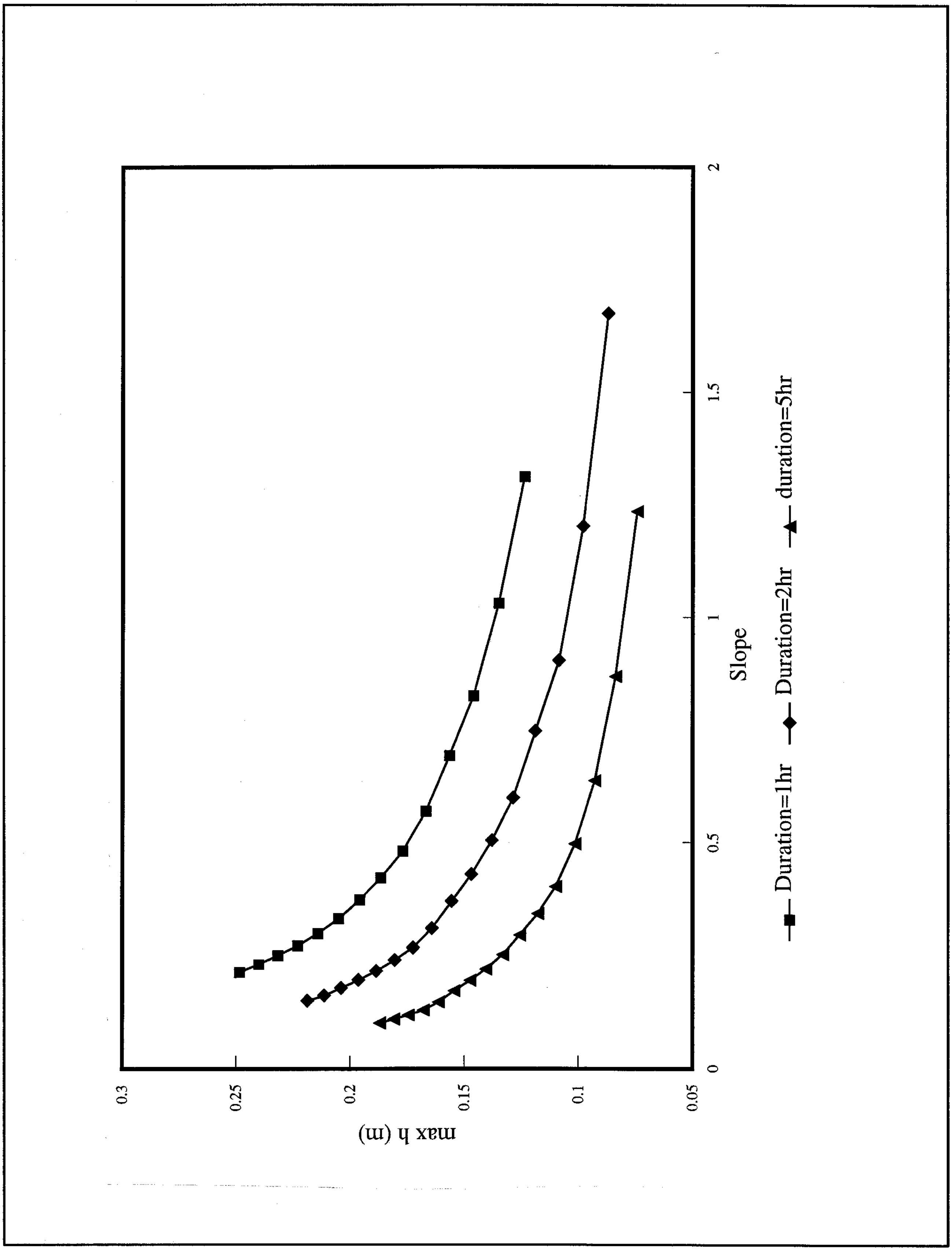


Figure 6.7 Maximum allowable depth of overtopping: Average cover, short grass (<50mm)

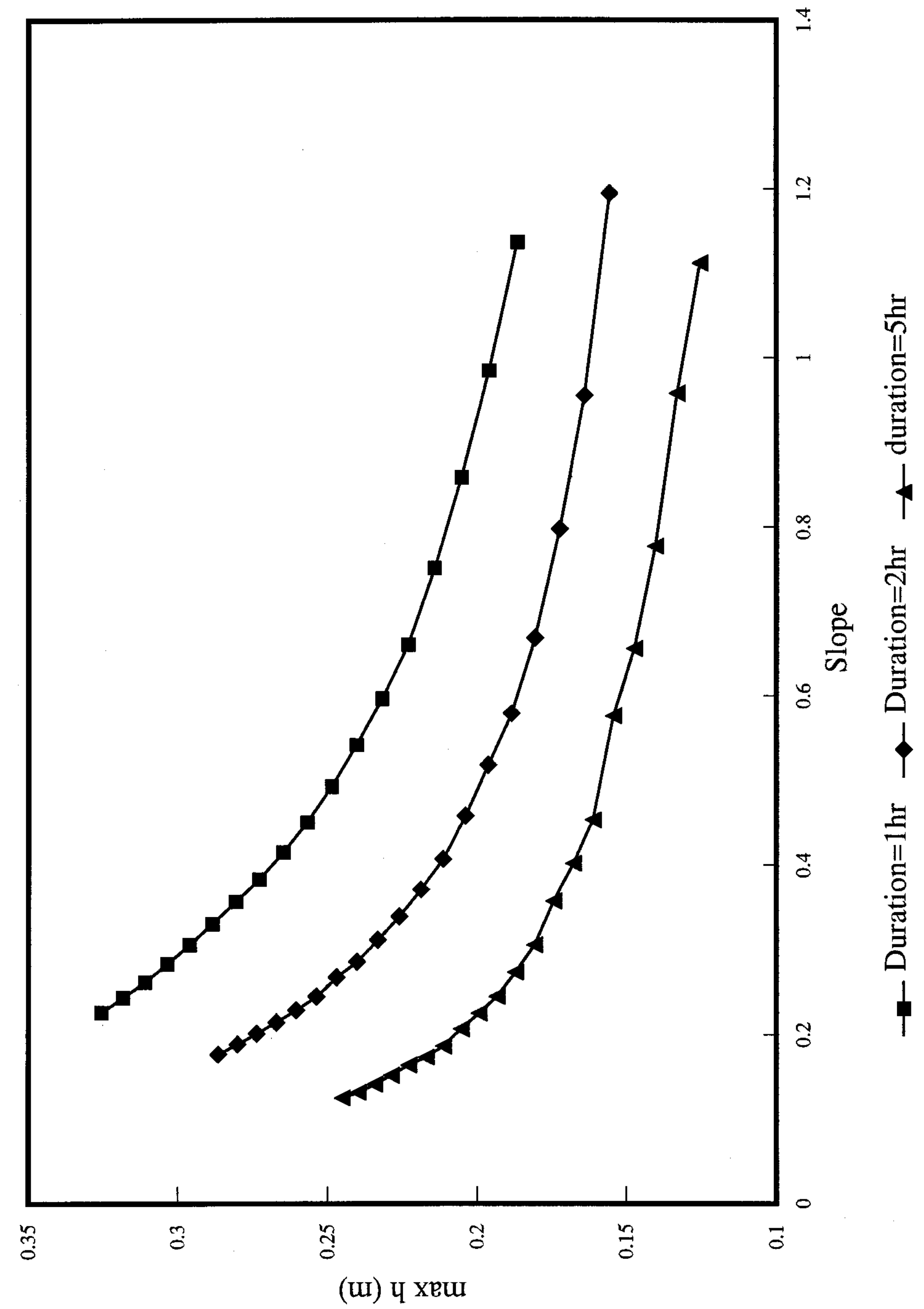


Figure 6.8 Maximum allowable depth of overtopping: Average cover, medium grass (50-250mm)

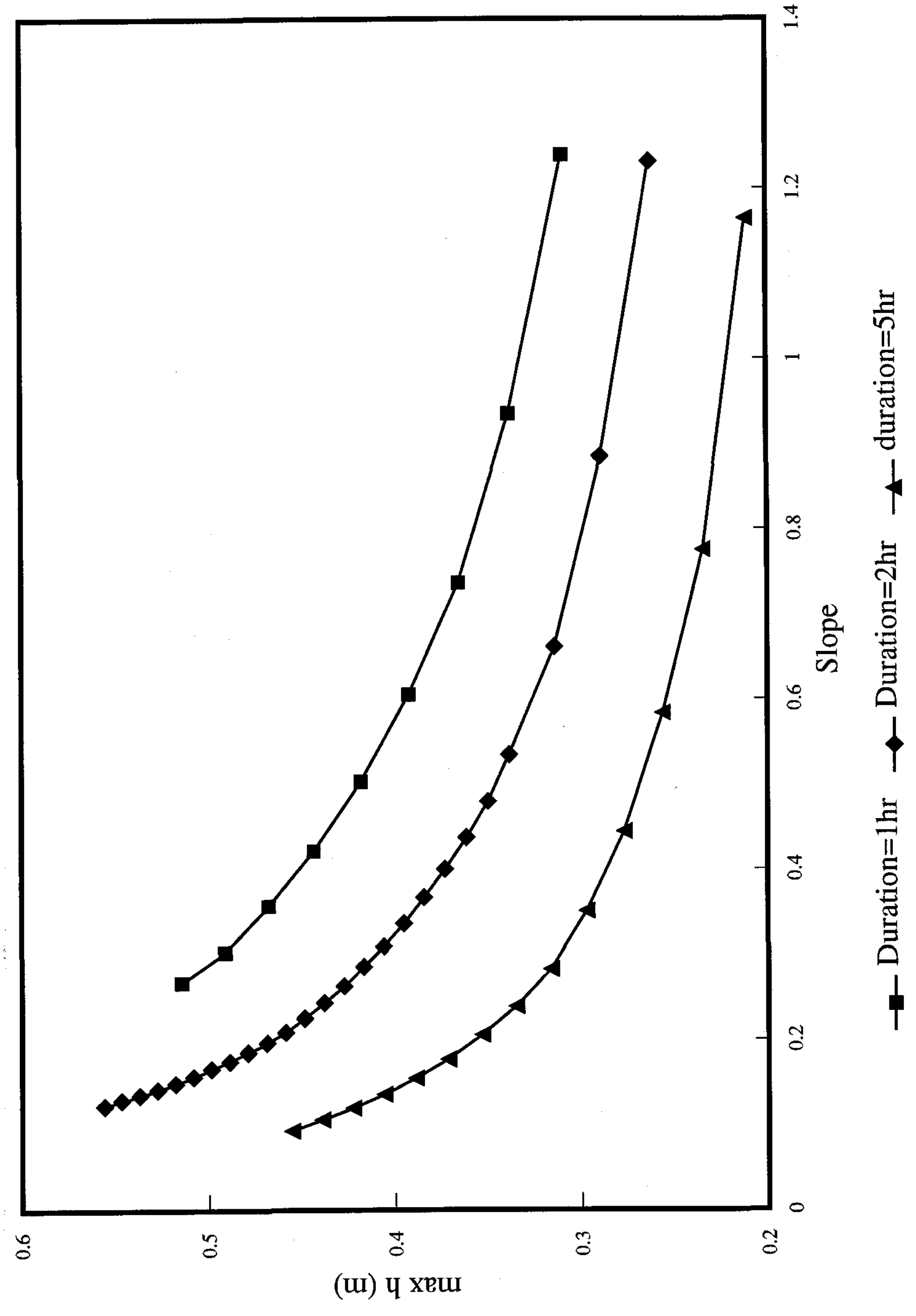


Figure 6.9 Maximum allowable depth of overtopping: Average cover, long grass (>750mm)

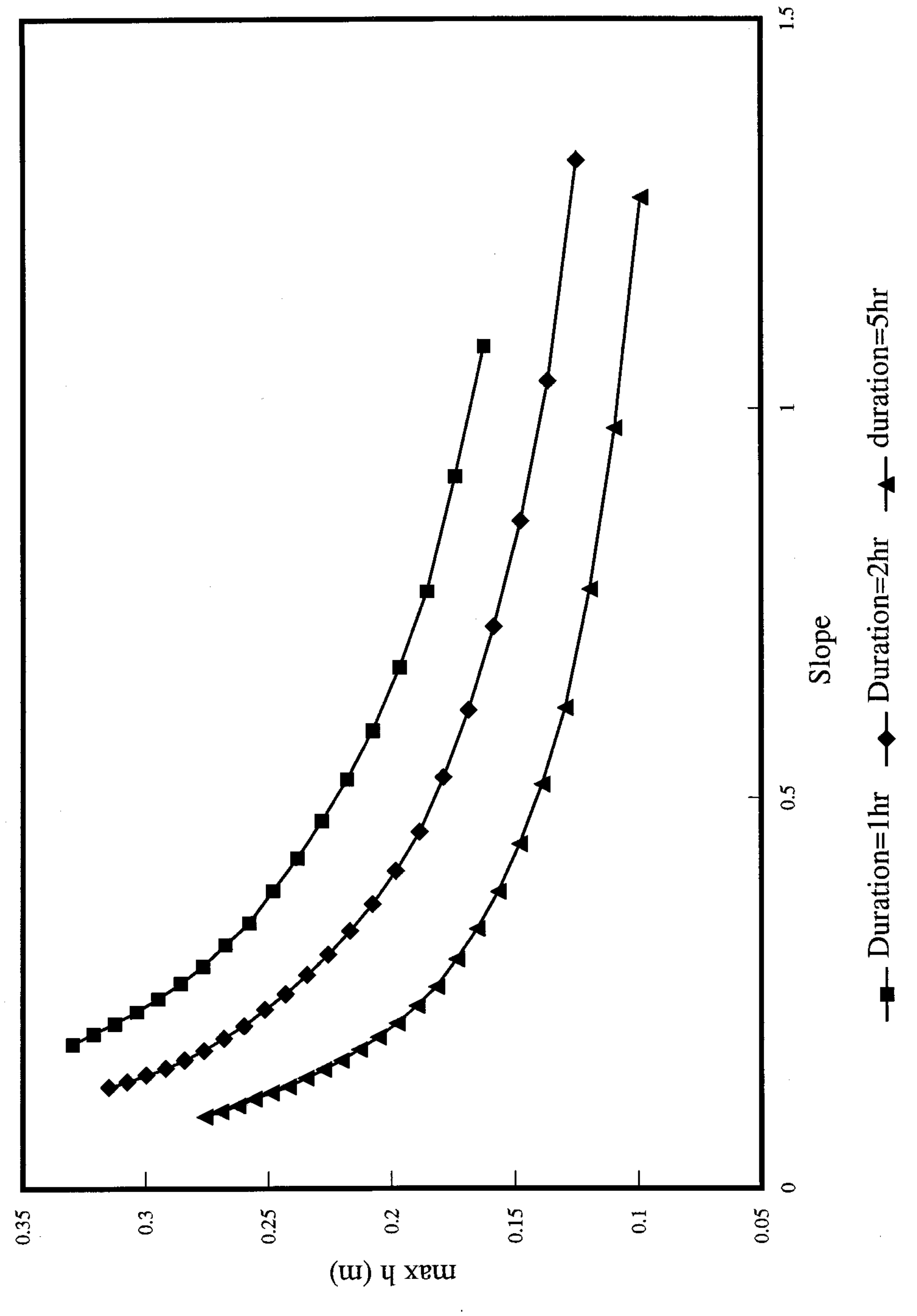


Figure 6.10 Maximum allowable depth of overtopping: Good cover, short grass (<50mm)

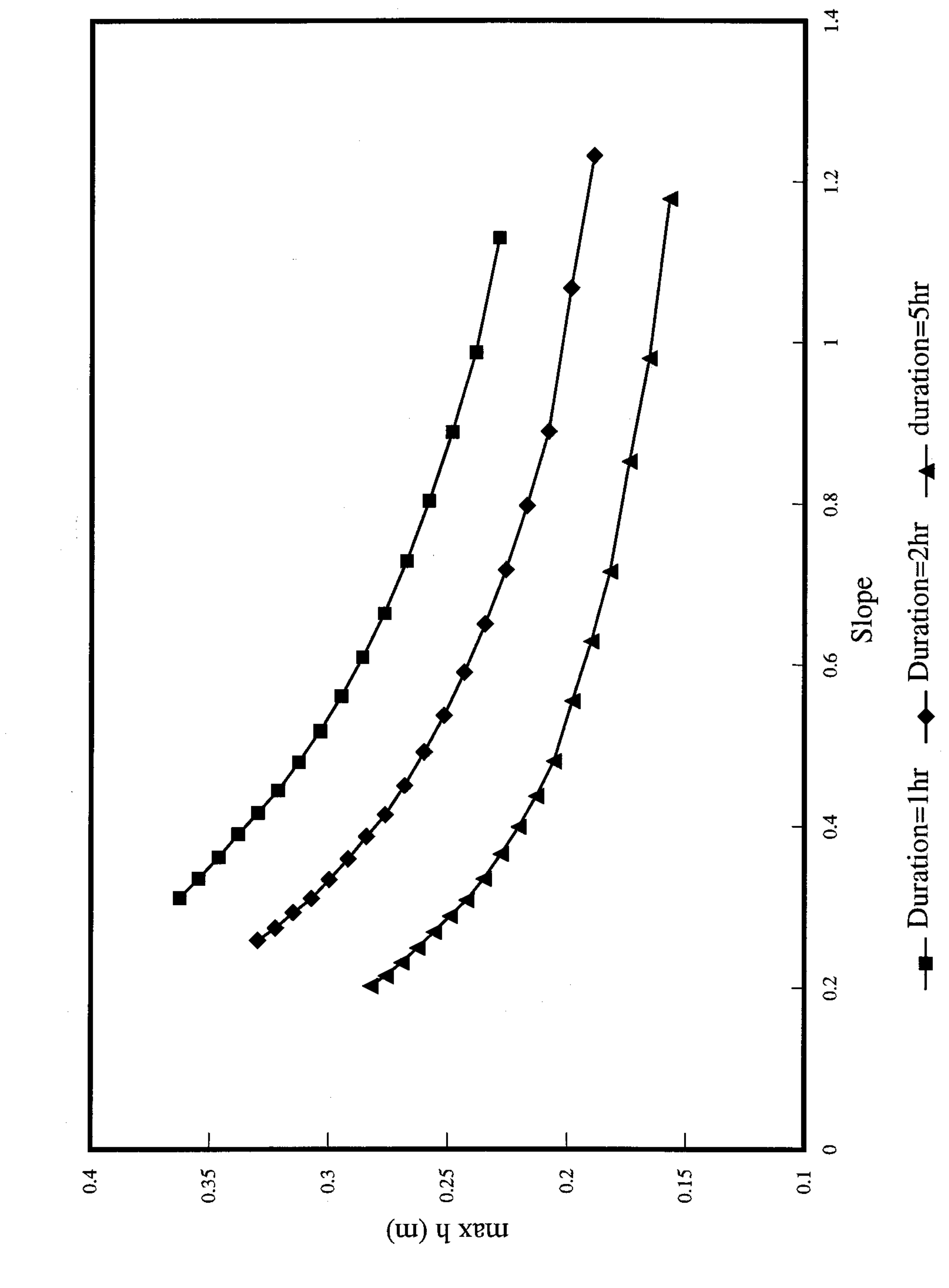


Figure 6.11 Maximum allowable depth of overtopping: Good cover, medium grass (50-250mm)

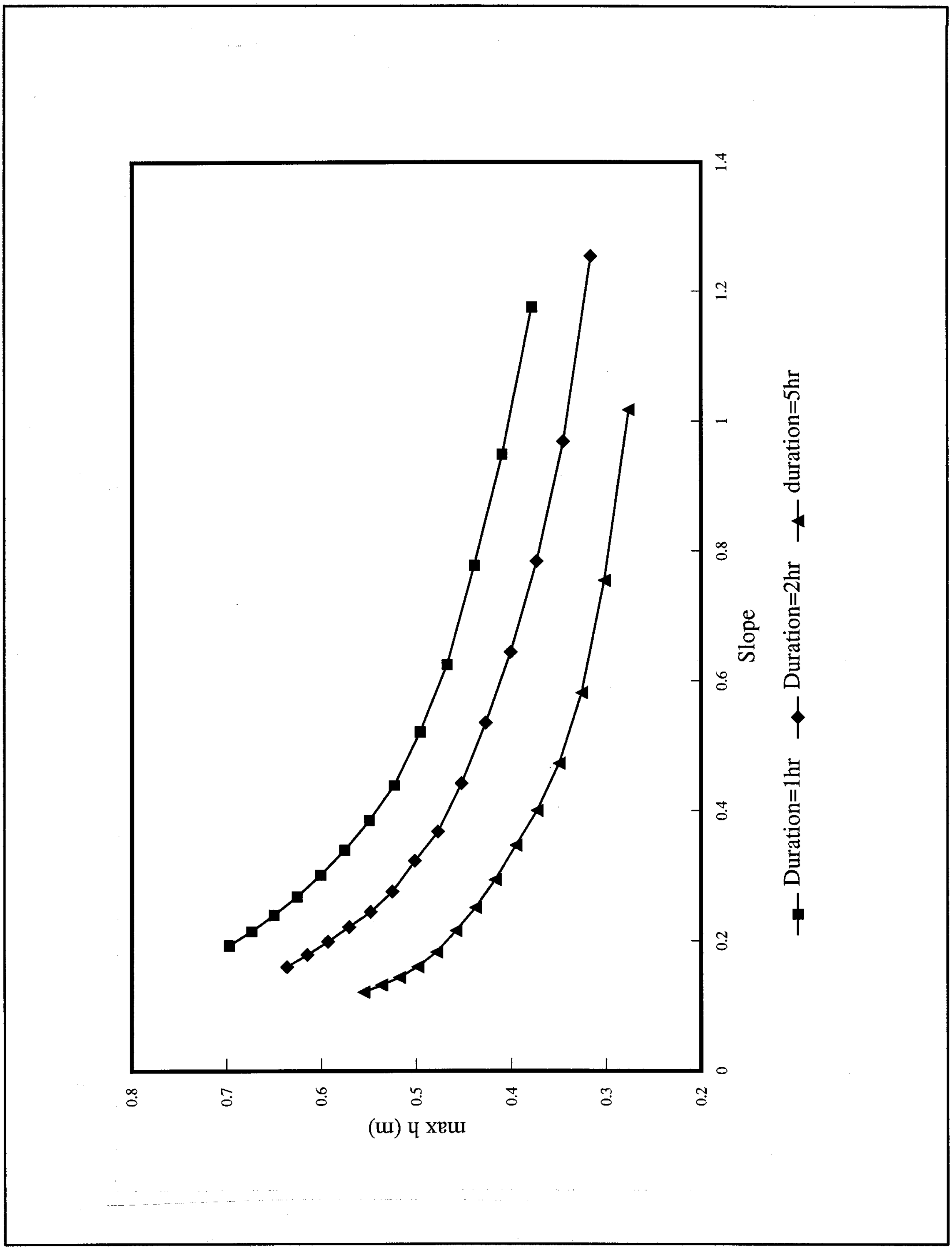


Figure 6.12 Maximum allowable depth of overtopping: Good cover, long grass (>750mm)

Appendices

Appendix A

Returned questionnaires

W&L RB/ICM

NRA

National Rivers Authority
Anglian Region

23 April 1990

I C Meadowcroft, Esq
Hydraulics Research Limited
Wallingford
Oxon
OX10 8BA

Our Ref: AHB/LS/RC/29

Your Ref:

Dear Mr Meadowcroft

Hydraulics Performance of Flood Embankments

I refer to Clive Mason's letter to you dated 23 March 1990.

I am now returning a set of completed proforma, which relate primarily to our Eastern Area.

Yours sincerely

L. Spencer.

p.p Andrew Hunter-Blair
Regional Co-ordinator

DR. KEVIN BOND
Regional General Manager

Kingfisher House
Goldhay Way
Orton Goldhay
Peterborough
PE2 0ZR
Tel: 0733 371811
Fax: 0733 231840

QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS

19/02/90

Sheet No 1

Name of embankment scheme :

Location of embankment :

River Bure at Runham, Nr Great Yarmouth

Nature of problem :

Seepage and instability of toe piling

Details of problem : **Date :**

Remedial action taken or considered necessary :

Ad-hoc repairs to seepage boreholes and emergency repairs to collapsed steel piling.

Person to contact should more information be required :

Name... John Ash
District Engineer .... **Tel No** ... (0603) 662800 ....

Address

National Rivers Authority ....

79 Thorpe Road ....

NORWICH NR1 1EW ....

....

QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS 19/02/90

Sheet No 2

Name of embankment scheme :

North Breydon flood defences.

Location of embankment :

North Breydon, Great Yarmouth

Nature of problem :

Poor water pressure problems leading to liquifaction.

Details of problem :

Date :

Remedial action taken or considered necessary :

Restoration of semi-permeable revetment armouring.

Person to contact should more information be required :

Name John Ash
District Engineer

Tel No (0603) 662800

Address

National Rivers Authority

79 Thorpe Road

NORWICH

NR1 1EW

Sheet No 3

Name of embankment scheme :

Location of embankment :

Brightlingsea Railway Embankment, Colne Estuary

Nature of problem :

Permeability problems resulting from change of use of structure.
Vulnerability to frontal attack of revetment armouring.

Details of problem : **Date :**

Remedial action taken or considered necessary :

Emergency repairs completed after winter gales, 1990.

Person to contact should more information be required :

Name Steve Worrall
District Engineer
Tel No (0376) 72091

Address

National Rivers Authority
Rivers House, Threshelfords
Business Park, Inworth Road
Feering, Kelvedon, Essex
CO5 9SE

Sheet No 4

Name of embankment scheme :

Location of embankment :

Orplands Marsh Wall, Bradwell, Blackwater Estuary.

Nature of problem :

Dessication and fissuring.

Details of problem : **Date :**

The problem remains to be tackled and may be insuperable because of difficulties with economic justification.

Remedial action taken or considered necessary :

Problem has yet to be addressed

Person to contact should more information be required :

Name Chris Ramsden
District Engineer **Tel No** (0245) 478065

Address

National Rivers Authority
Brook End Road
Chelmsford
Essex. CM2 6NZ

RECEIVED
30 MAR 1990
RIVERS DEPT
HYDRAULICS RESEARCH

NRA

National Rivers Authority
Anglian Region

23rd March, 1990

I.C. Meadowcroft Esq.,
Hydraulics Research Limited,
Wallingford,
Oxon,
OX10 8BA.

Our Ref: AHB/JC/RC/29

Your Ref:

Dear Mr. Meadowcroft,

Hydraulics Performance of Flood Embankments

Thank you for your letter of 19th February. I am pleased to return a set of completed proforma, which relate primarily to our Northern Area. In respect of our Central Area I would comment that we have hundreds of kilometers of embankments which have been investigated over many years by many consultants and organisations. These include for example the Middle and South Level Banks of the Bedford River System and the Tidal Embankments of the River Ouse. We feel that such embankments do not require any further research at this stage.

I will be grateful to be kept informed about the project.

Yours sincerely,

Clive Mason

Clive Mason,
Regional Manager
(Flood Defence & Operations).

PETER BULLOCK
Regional General Manager

Kingfisher House
Goldhay Way
Orton Goldhay
Peterborough
PE2 0ZR
Tel: 0733 371811
Fax: 0733 231840

Sheet No

Name of embankment scheme :

CAMBRIDGESHIRE LODES.

Location of embankment :

BURWELL LODE, REACH LODE, SWAFFHAM LODE

Nature of problem :

Seepage, instability of embankments, crest level / freeboard above design flood.

Details of problem : Date :

Over a number of years, deterioration of the existing embankments has continued due to a number of factors.

Remedial action taken or considered necessary :

Major scheme now in progress to refurbish the embankments to a satisfactory level of service.

Person to contact should more information be required :

Name G. C. Beel Tel No 0480-414581

Address

....

....

....

....

Sheet No ☐

Name of embankment scheme :

Steeping River Revetment Scheme

Location of embankment :

Downstream of Wainfleet, Lincolnshire

Nature of problem :

The river is retained for water resource reasons during the summer and lowered for the winter. Erosion occurred on the unvegetated faces.

Details of problem : Date : 1978

Wind/wave action and weathering/frost action caused erosion endangering the stability of the embankments.

Remedial action taken or considered necessary :

Where the batters of the banks were not too steep, revetment was provided in the form of frost resistant stone toe and bank protection laid on a filter sheet. On steeper sections, stone filled box gabions were provided.

Person to contact should more information be required :

Name M Pettifor Tel No Sth Cockerington (050782) 8102

Address

Guy Gibson House

Manby Park

Louth

Lincs

Sheet No ☐

Name of embankment scheme :

River Freshney

Location of embankment :

Right Bank Upstream of Railway Line, Grimsby

Nature of problem :

Seepage occurred during periods of high flow.

Details of problem : **Date :** 1989

Material comprising embankment largely silt and vulnerable to attack by burrowing vermin.

Remedial action taken or considered necessary :

Bank sealed with clay.

Person to contact should more information be required :

Name M Pettifor **Tel No** Sth Cockerington (050782) 8102

Address

Guy Gibson House
Manby Park
Louth
Lincs

Sheet No ☐

Name of embankment scheme :

River Ancholme Improvement Scheme

Location of embankment :

South Ferriby to Brandy Wharf - South Humberside/North Lincolnshire

Nature of problem :

Seepage occurs during flood conditions and in 1981 breaches occurred due to combination of seepage and overtopping. Erosion is also a problem.

Details of problem : **Date :** Current

The bank material is variable but generally a light silty soil susceptible to seepage which is made worse by burrowing animals and entails frequent control of vermin. The channel is a navigation and subject to variation in retention level summer/winter. Unvegetated faces between these levels is subject to weathering/frost damage/wave attack and erosion by wash from boats.

Remedial action taken or considered necessary :

Works planned include the reprofiling of the embankments to improve standard and stability, the insertion of a butyl membrane (vertical) to combat seepage and toe protection consisting of timber toe revetment, filter sheet and backfill to combat erosion.

Person to contact should more information be required :

Name M Pettifor **Tel No** Sth Cockerington (050782) 8102

Address

Guy Gibson House

Manby Park

Louth

Lincs

Sheet No ☐

Name of embankment scheme :

Witham Haven Stoning Schemes

Location of embankment :

Tidal Embankments of the Haven Downstream of Boston

Nature of problem :

Erosion and slipping of the tidal river banks.

Details of problem : **Date :** Continuous

The tidal outfall of the River Witham is also a busy navigable channel serving Boston Docks. Erosion and slipping is caused by weathering, the wash from cargo vessels and high velocity during flood flows.

Remedial action taken or considered necessary :

A variety of methods have been employed including gabion mattresses, placing of stone on filter sheets and the laying of interlocking panel revetment.

Person to contact should more information be required :

Name M Whiley **Tel No** Lincoln (0522) 513100

Address

Aqua House

Harvey Street

Lincoln

LN1 1TF

Sheet No ☐

Name of embankment scheme :

River Witham - Greetwell to Fiskerton

Location of embankment :

Downstream of Lincoln

Nature of problem :

Erosion of the berm on the riverside of the embankment endangered the stability of the embankments in periods of high flows.

Details of problem : **Date :** 1989

The River Witham is an embanked channel, a navigable river, protecting very large areas of grade 1 and 2 arable land. The banks are generally of light silty soil and are afforded protection by the berm. The berm was eroded by a combination of waterbourne traffic, grazing stock and weathering/frost damage/wave action.

Remedial action taken or considered necessary :

Berm revetment provided by installation of Enkomatt supported by timber piles driven along the edge of berm and backfilled with dredged material. Top edge of berm also given protection after settlement.

Person to contact should more information be required :

Name M Whiley **Tel No** Lincoln (0522) 513100

Address

Aqua House

Harvey Street

Lincoln

LN1 1TF

Sheet No ☐

Name of embankment scheme :

River Witham - Kirkstead to Tattershall

Location of embankment :

Both Banks of the Witham near Woodhall Spa, Lincs

Nature of problem :

Erosion of the berm on the riverside of the embankments endangered the stability of the embankment in periods of high flows.

Details of problem : **Date :** 1978

The River Witham is an embanked channel, a navigable river, protecting large areas of Grade 1 and 2 land. The banks are generally of light silty soil and are afforded protection by the berm. The berm was eroded by a combination of weathering/wave action and boat wash particularly in the unvegetated zone between summer and winter retention levels.

Remedial action taken or considered necessary :

Berm protection was provided by placing frost resistant limestone on a filter sheet to rebuild the toe.

Person to contact should more information be required :

Name M Whiley **Tel No** Lincoln (0522) 513100

Address

Aqua House
Harvey Street
Lincoln
LN1 1TF

Sheet No ☐

Name of embankment scheme :

South Forty Foot Drain

Location of embankment :

Upstream of Boston, Lincs

Nature of problem :

Erosion of the berm on the riverside of the embankments is endangering their stability and causing access problems.

Details of problem : **Date :** 1991

Soils are light Grade 1 fenland silts and the berms are susceptible to erosion by weathering and wave/wind attack.

Remedial action taken or considered necessary :

Revetment (form yet to be decided) to be provided after reinstatement of the berm.

Person to contact should more information be required :

Name M Whiley **Tel No** Lincoln (0522) 513100

Address

Aqua House

Harvey Street

Lincoln

LN1 1TF

Sheet No ☐

Name of embankment scheme :

Kyme Eau

Location of embankment :

Upstream and Downstream of South Kyme, Lincs

Nature of problem :

Seepage occurs during periods of flood flow.

Details of problem : **Date :** 1993

The embankments are founded on silt and peat which permits seepage during high flows.

Remedial action taken or considered necessary :

Seepage remedial works yet to be decided possibly in the form of a membrane inserted in the bank.

Person to contact should more information be required :

Name M Whiley **Tel No** Lincoln (0522) 513100

Address

Aqua House

Harvey Street

Lincoln

LN1 1TF

Sheet No

Name of embankment scheme :

Branston Delph Bank Improvement

Location of embankment :

East of Lincoln (Tributary of the River Witham)

Nature of problem :

The banks are on a deep peat area and susceptible to seepage at high water levels.

Details of problem : **Date :** 1994

An embanked channel serving as a highland carrier across the fen between the River Witham and higher land to the south west, it is therefore dependent on levels in the River Witham for discharge and high water levels are retained for long periods in storm events.

Remedial action taken or considered necessary :

The banks have been clayed in the past but a more reliable system is required to prevent or significantly reduce seepage.

Person to contact should more information be required :

Name M Whiley **Tel No** Lincoln (0522) 513100

Address

Aqua House
Harvey Street
Lincoln
LN1 1TF

Sheet No

Name of embankment scheme :

TIDAL WELLAND

Location of embankment :

FROM WASH OUTFALL TO SPALDING

Nature of problem :

EROSION OF BERM AND BANKS DUE TO TIDAL AND SHIPPING ACTION

Details of problem : **Date :** 1988

THE TIDAL CHANNEL IS ARTIFICIAL AND RUNS THROUGH SILT SOILS SUSCEPTIBLE TO EROSION. PAST PROTECTION HAS BEEN AFFORDED BY TRADITIONAL THORN FAGGOTS. THESE ARE NOW COMING TO THE END OF THEIR USEFUL LIVES AND RESULTING IN AN INCREASE IN LOSS OF BERM AND BANK.

Remedial action taken or considered necessary :

REINSTATEMENT OF BERM AND BANK WITH PROTECTION NOW GIVEN BY RANDOM PITCHED STONE BATTER AND BANK.

Person to contact should more information be required :

Name J. ULYATT **Tel No** SPALDING (0775) 762123

Address

STEPPING STONE WALK
WINFREY AVENUE
SPALDING
LINCOLNSHIRE

Sheet No ☐

Name of embankment scheme :

TIDAL NENE

Location of embankment :

FROM WASH TO FOUL ANCHOR

Nature of problem :

EROSION OF BERM AND BANKS DUE TO TIDAL AND SHIPPING ACTION

Details of problem : **Date :** 1988

THE TIDAL CHANNEL IS ARTIFICIAL AND RUNS THROUGH SILT SOILS SUSCEPTIBLE TO EROSION. PAST PROTECTION HAS BEEN AFFORDED BY TRADITIONAL THORN FAGGOTS. THESE ARE NOW COMING TO THE END OF THEIR USEFUL LIVES AND RESULTING IN AN INCREASE IN LOSS OF BERM AND BANK.

Remedial action taken or considered necessary :

REINSTATEMENT OF BERM AND BANK WITH PROTECTION NOW GIVEN BY RANDOM PITCHED STONE BATTER AND BANK.

Person to contact should more information be required :

Name JOHN ULYATT **Tel No** SPALDING (0775) 762123

Address

STEPPING STONE WALK

WINFREY AVENUE

SPALDING

LINCOLNSHIRE

Sheet No ☐

Name of embankment scheme :

MAXEY CUT

Location of embankment :

BETWEEN PEAKIRK AND TALLINGTON

Nature of problem :

EROSION OF FLOOD BANKS

Details of problem : **Date :** 1985

THE BED OF THE MAXEY CUT FALLS RAPIDLY FROM ITS HEAD AT TALLINGTON TO ITS OUTFALL INTO THE WELLAND AT PEAKIRK. ITS FUNCTION IS TO ACT AS A FLOOD RELIEF CHANNEL TAKING HIGH FLOWS OFF THE MAIN WELLAND CHANNEL. THE BED AND BANKS ARE MADE UP OF GRAVELS AND SANDS AND IN TIMES OF FLOOD LARGE SCOUR HOLES WOULD RAPIDLY APPEAR WITHOUT WARNING THREATENING THE SAFETY OF THE EMBANKMENT.

Remedial action taken or considered necessary :

PROTECT THE BANKS FROM SCOUR BY APPLYING AN ENKAMAT REVETMENT SYSTEM AND STABILISING THE BED BY INSTALLATION OF WEIRS.

Person to contact should more information be required :

Name..JOHN ULYATT.................... **Tel No** .SPALDING (0775) 762123.....

Address

.......STEPPING STONE WALK...........

.......WINFREY AVENUE.................

.......SPALDING.......................

.......LINCOLNSHIRE...................

Sheet No

Name of embankment scheme :

DEEPING HIGH BANK CRADGE BANK

Location of embankment :

RIVER WELLAND U/S OF SPALDING

Nature of problem :

BANK AND BERM CONSISTS OF SILT MATERIAL SUSCEPTIBLE TO LONG TERM EROSION

Details of problem : **Date :** 1982

AN ARTIFICIAL CHANNEL WAS CUT IN 1950. THE BANKS WERE PROTECTED AT THAT TIME AND SINCE BY THORN FAGGOTS TO RESIST WAVE ACTION AT NORMAL RETENTION LEVEL. THE FAGGOTS ARE NOW FAILING LEADING TO A LOSS OF BERM WIDTH AND EMBANKMENT.

WORK IS ONGOING.

Remedial action taken or considered necessary :

REINSTATEMENT OF BERM WIDTH AND PROTECTION OF RIVER FACE USING A VARIETY OF METHODS INCLUDING:-

(a) Asbestos Piles
(b) Random Pitched Stone
(c) Duracem Piles
~~(d) Lightweight Steel Piles~~

Person to contact should more information be required :

Name JOHN ULYATT **Tel No** SPALDING (0775) 762123

Address

STEPPING STONE WALK
WINFREY AVENUE
SPALDING,
LINCOLNSHIRE

Sheet No 1

Name of embankment scheme :

River Glen Akeld Flood Protection Scheme

Location of embankment :

NU 940 310 to NU 971 314

Nature of problem :

Failure of flood banks due to percolation of water under banks during high flows.

Details of problem : Date : 1983.

The Glen has a heavy bed load of gravel. This had raised the bed level and water levels over a period. The banks were formed of local material most of which was of high permeability. The river drains a highland catchment and most flood flows are of short duration.

Remedial action taken or considered necessary :

Channel dredged to lower bed and material used to raise and widen flood bank

Person to contact should more information be required :

Name A.J. CLARKE Tel No 091 2130266

Address

N.R.A. NORTHUMBRIA REGION
ELDON HOUSE
REGENT CENTRE
GOSFORTH
NEWCASTLE UPON TYNE

Sheet No 2

Name of embankment scheme :

—

Location of embankment :

Various

Nature of problem :

Water collects behind flood bank during flood and bank breaches when river level falls rapidly. This is a common fault with flashy rivers.

Details of problem : Date :

Source of water may be run off from fields or from over topping at some point further up stream.

Remedial action taken or considered necessary :

Flapped pipes put through flood banks with invert level set at or near to field level.

Person to contact should more information be required :

Name..A.J.CLARKE.............. Tel No 091.2130266......

Address

N.R.A. NORTHUMBRIA REGION
ELDON HOUSE REGENT CENTRE
GOSFORTH
NEWCASTLE UPON TYNE

Sheet No 3

Name of embankment scheme :

GREATHAM CREEK

Location of embankment :

GREATHAM CREEK - TIDAL EMBANKMENT

Nature of problem :

PENETRATION IN STORM TIDE CONDITIONS

Details of problem : Date :

WATER PENETRATING THROUGH EMBANKMENTS (CONSTRUCTED WITH SLAG & SAND) RESULTING IN TOTAL COLLAPSE - Approx 4 YEARLY INTERVALS

Remedial action taken or considered necessary :

REPLACE BREACH WITH CLAY

Person to contact should more information be required :

Name D. CLARKE Tel No 0325- 480849

Address NATIONAL RIVERS AUTHORITY

STRESSHOLME S.T.W.

SNIPE LANE

BLACKWELL

DARLINGTON CO. DURHAM.

Sheet No 4

Name of embankment scheme :

Newsham Grange, Yarm. - River Tees

Location of embankment :

Newsham Grange, Yarm, Cleveland. Grid Ref Square NZ 3709.

Nature of problem :

See enclosed report.

Details of problem : Date :

See enclosed report.

Remedial action taken or considered necessary :

See enclosed report.

Person to contact should more information be required :

Name D. Clarke. Tel No 0325 480849.

Address National Rivers Authority.
Streessholme S.T.W.
Snipe Lane
Blackwell
Darlington.
Co. Durham.

W&GW RB/ICM

RECEIVED
25 APR 1990
RIVERS DEPT
HYDRAULICS RESEARCH

NRA

National Rivers Authority
North West Region

Our ref FD11/12/PW/SS1
Your ref

Date 23rd April 1990

Ian Meadowcroft Esq
River Engineering Department
Hydraulics Research
Wallingford
Oxfordshire
OX10 8BA

Dear Mr Meadowcroft

HYDRAULIC PERFORMANCE OF FLOOD EMBANKMENTS

Please find enclosed a completed questionnaire on the hydraulic performance of embankments in respect of our Northern Area. I understand that my Central and Southern Area Managers have already replied to you directly and this completes the response from the North West Region.

Yours sincerely

Peter Woods

f

DR PETER D WALSH
Regional Flood Defence Manager

P.O. Box 12 Richard Fairclough House Knutsford Road Warrington WA4 1HG Tel: (0925) 53999 Telex: 628425 Fax: (0925) 415961

Sheet No ☐

Name of embankment scheme :

River Freshney

Location of embankment :

Right Bank Upstream of Railway Line, Grimsby

Nature of problem :

Seepage occurred during periods of high flow.

Details of problem : **Date :** 1989

Material comprising embankment largely silt and vulnerable to attack by burrowing vermin.

Remedial action taken or considered necessary :

Bank sealed with clay.

Person to contact should more information be required :

Name M Pettifor **Tel No** Sth Cockerington (050782) 8102

Address

Guy Gibson House
Manby Park
Louth
Lincs

19/02/90

HR QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS

Sheet No 1

Name of embankment scheme :

Millom Embankment (SEA DEFENCE)

Location of embankment :

From Millom, North East for 3Km.

Nature of problem :

Loss of vegetation.

Details of problem :

Date : March 1990

The embankment is constructed of sand/silt and protected with grass. The front face was originally turfed and the back face seeded. The grass has been slowly replaced by moss.

Remedial action taken or considered necessary :

Last year, the embankment was limed and this will be followed by fertiliser. This is a problem with other embankments in South Cumbria.

Person to contact should more information be required :

Name: R.V. Williams Tel No 05395 60567

Address

Beathwaite,

Levens,

Kendal,

Cumbria. LA8 8NL.

HR QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS 19/02/90

Sheet No 2

Name of embankment scheme :

River Pool

Location of embankment :

Helsington Pool Bridge - Low Pool Bridge

Nature of problem :

Structural failure.

Details of problem : Date : March 1990

This embankment was raised about 10 years ago. Extensive cracks have occurred parallel to the embankment on both faces.

Remedial action taken or considered necessary :

Three months ago the cracks were filled with the silts similar to the embankment constructions and the site is being monitored for any further movement.

Person to contact should more information be required :

Name R.V. Will Tel No 05395 60567

Address

Beathwaite,

Levens,

Kendal,

Cumbria. LA8 8NL.

HR QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS 19/02/90

Sheet No 3

Name of embankment scheme :

Sampool

Location of embankment :

River Kent (tidal section) adjacent to Sampool Caravan Park

Nature of problem :

Erosion of river bed and lwoer banks adjacent to a flood embankment.

Details of problem : Date : March 1990

This section is subject to erosion from freshwater flows which are undermining the river bank, whilst the adjacent flood bank (tidal) is endangered. It is not possible because of the proximity of static caravan to move the flood bank.

Remedial action taken or considered necessary :

Placing of large rocks in the bed.

Person to contact should more information be required :

Name ....R.V. Willi.......... Tel No ....05395 60567....

Address

...Beathwaite,...

...Levens,...

...Kendal,...

...Cumbria... LA8 8NL...

Sheet No ☐

Name of embankment scheme :

MOSSBAND GRETNA

Location of embankment :

BETWEEN RIVER SARK AND RIVER ESK, APPROX. NY 330 662 & NY 335 650.

Nature of problem :

Erosion of inland face of bank.

Details of problem : Date :

Over-topping causing erosion of inland face of bank and consequent reduction in top width on this grassed bank.

Remedial action taken or considered necessary :

Backfil and repair. Raising of bank.

Person to contact should more information be required :

Name G. A. NOONAN .... Tel No ....

Address

....

....

....

....

Sheet No ☐

Name of embankment scheme :

DURRANHILL BECK FLOOD STORAGE RESERVOIR

Location of embankment :

NORTH OF WARWICK ROAD, CARLISLE - N.G.R. NY 423 562.

Nature of problem :

Seepage into storage basin reducing its capacity.

Details of problem : Date :

Seepage is through underground gravel layers fed by ground water from the River Eden.

Remedial action taken or considered necessary :

Cut-off of gravel layers by bentonite or butyl seal.

Person to contact should more information be required :

Name G. A. NOONAN Tel No ....

Address

....

....

....

....

Our ref 556.R/JR/JV
Your ref R/S/0013E

Date 2nd April, 1990

RECE[illegible]
09 APR 1990
RIVER[illegible]
HYDRAUL[illegible] [illegible]EARCH

NRA

National Rivers Authority
North West Region

Hydraulics Research Limited,
Wallingford,
Oxfordshire,
OX10 8BA

FOR THE ATTENTION OF MR. IAN MEADOWCROFT.

Dear Sir,

Please find enclosed a completed copy of your embankment performance questionnaire which I hope will be of use to you in your current research project.

Should you require further information do not hesitate to contact this office.

Yours faithfully,

HT Davidson

H. T. DAVIDSON,
SENIOR ENGINEER,
SOUTH FLOOD DEFENCE AREA.

This matter is being dealt with by Mr. J. Ruckledge.

'Mirwell' Carrington Lane Sale M33 5NL Tel: (061) 973 2237 Fax: (061) 973 4601

Sheet No 1

Name of embankment scheme :

RIVER MERSEY REHABILITATION SCHEME

Location of embankment :

BETWEEN ASHTON WEIR (SJ 773 936) AND STOCKPORT E.T.W. (SJ 870 890)

Nature of problem :

EROSION AND SILT DEPOSITION DURING FLOOD FLOWS

Details of problem : Date :

BANK EROSION AT FLOOD FLOWS.
SILT DEPOSITION, CAUSING REDUCTION IN FLOW CAPACITY OF CHANNEL.

Remedial action taken or considered necessary :

REMOVAL OF SILT OVERBURDEN AND RECONSTRUCTION OF BANKS.

Person to contact should more information be required :

Name JOHN RUCKLEDGE Tel No 061 973 2237 (Ext. 168)

Address

NATIONAL RIVERS AUTHORITY,
SOUTH FLOOD DEFENCE AREA,
MIRWELL,
CARRINGTON LANE,
SALE, M33 5NL,
CHESHIRE.

IM

Our ref BFW\LR301\01
Your ref

Date 26 March 1990

River Engineering Department
Hydraulics Research Ltd
Wallingford
OX10 8BA

NRA

National Rivers Authority
North West Region

RECEIVED
27 MAR 1990
RIVERS DEPT
HYDRAULICS RESEARCH

For the attention of Mr Ian Meadowcroft

Dear Sir

HYDRAULIC PERFORMANCE OF FLOOD EMBANKMENT

Further to your letter of the 19 February 1990 to our Regional Flood Defence Manager I attach details of two problems of possible interest.

Yours faithfully

B F Whelan

B F WHELAN
AREA MANAGER
CENTRAL FLOOD DEFENCE

Lostock House Holme Road Bamber Bridge Preston PR5 6AE Tel: (0772) 39882 Fax: (0772) 627730

Sheet No 1

Name of embankment scheme :

SLUICE EMBANKMENT

Location of embankment :

BETWEEN SLUICE & BACK DRAIN SOUTH OF WATER LANE, CROSSENS

Nature of problem :

SEEPAGE FROM HIGH LEVEL CARRIER TO LOW LEVEL DRAIN, BOTH IN CROSSENS PUMPED SYSTEM

Details of problem : Date :23/3/90

THE WATER LEVEL DIFFERENTIAL IS ABOUT 2.1M TO 2.4M, SEEPAGE IS MONITORED BY INSPECTION AND WORKS ARE UNDERTAKEN AS NEEDED TO CONTROL THE FLOW AND STABILISE THE BATTER

Remedial action taken or considered necessary :

OLD METHOD WAS TO SUPPORT SLIPPING EMBANKMENT USING TIMBER REVETMENT AND STONE BACKING - SLOW AND COSTLY. CURRENT MUCH CHEAPER AND EFFECTIVE METHOD UTILISES ICI FILTRAM IN A TRENCH PARALLEL TO THE LINE OF THE EMBANKMENT TO INTERCEPT THE SEEPAGE.

Person to contact should more information be required :

Name BERNARD WHELAN Tel No 0772 - 39882

Address

NATIONAL RIVERS AUTHORITY
LOSTOCK HOUSE
HOLME ROAD
BAMBER BRIDGE
PRESTON
PR5 6AE

Sheet No 2

Name of embankment scheme :

RIVER DOUGLAS TIDAL EMBANKMENT

Location of embankment :

NORTH OF A59, BANK HALL BRIDGE, TARLETON

Nature of problem :

LOSS OF MATERIAL FROM LOWER BATTER AND BERM DUE TO SLIPPAGE CREATES A POTENTIAL PROBLEM OF SECURITY OF TIDAL EMBANKMENT

Details of problem : Date :23/3/90

V. FINE SILTY MATERIAL HAS LITTLE STRENGTH AND STABILITY WHEN LUBRICATED.

Remedial action taken or considered necessary :

HEAVY STONE TOE PITCHING - EXPERIMENTAL LENGTH TO BE UNDERTAKEN IN 1990.

Person to contact should more information be required :

Name D RANSON Tel No 0772 - 39882

Address

NATIONAL RIVERS AUTHORITY

LOSTOCK HOUSE

HOLME ROAD

BAMBER BRIDGE

PRESTON

PR5 6AE

HR QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS 19/02/90

Sheet No ☐

Name of embankment scheme :

RIVER FLOOD EMBANKMENTS TO RIVERS OUSE, ADUR & ARUN

Location of embankment :

OUSE – NEWHAVEN TO BARCOMBE
ADUR – SHOREHAM TO HENFIELD
ARUN – LITTLEHAMPTON TO PULBOROUGH
} ALL IN SUSSEX

Nature of problem :

All the problems described in your letter of 19/2/90 occur frequently except for mass failure which is fairly rare. General settlement of course is an on-going problem

Details of problems in general Date :

OUSE Wave action erosion, bank leakage, tide flap operation
Bank failure Jan 1990 at Southease

ADUR Bank leakage, overtopping, flood flow erosion, land drainage problems, conservation problems with bank mowing.

ARUN Severe wave + flood flow erosion (Arundel to Littlehampton).
overtopping, seepage, land drainage problems

Remedial action taken or considered necessary :

OUSE Several miles of chalk revetment placed annually. Southease bank failure being repaired now.

ADUR Some chalk revetment and bank raising done annually

ARUN 'Endless' (!) revetment schemes

Person to contact should more information be required :

Name ADRIAN BIGGS Tel No WORTHING (0903) 820692.

Address

NRA SOUTHERN REGION

GUILDBOURNE HOUSE

CHATSWORTH RD

WORTHING, SUSSEX

Sheet No

Name of embankment scheme :

Witham Haven Stoning Schemes

Location of embankment :

Tidal Embankments of the Haven Downstream of Boston

Nature of problem :

Erosion and slipping of the tidal river banks.

Details of problem : **Date :** Continuous

The tidal outfall of the River Witham is also a busy navigable channel serving Boston Docks. Erosion and slipping is caused by weathering, the wash from cargo vessels and high velocity during flood flows.

Remedial action taken or considered necessary :

A variety of methods have been employed including gabion mattresses, placing of stone on filter sheets and the laying of interlocking panel revetment.

Person to contact should more information be required :

Name M Whiley **Tel No** Lincoln (0522) 513100

Address

Aqua House

Harvey Street

Lincoln

LN1 1TF

Sheet No ☐

Name of embankment scheme :

South Forty Foot Drain

Location of embankment :

Upstream of Boston, Lincs

Nature of problem :

Erosion of the berm on the riverside of the embankments is endangering their stability and causing access problems.

Details of problem : **Date :** 1991

Soils are light Grade 1 fenland silts and the berms are susceptible to erosion by weathering and wave/wind attack.

Remedial action taken or considered necessary :

Revetment (form yet to be decided) to be provided after reinstatement of the berm.

Person to contact should more information be required :

Name M Whiley **Tel No** Lincoln (0522) 513100

Address

Aqua House
Harvey Street
Lincoln
LN1 1TF

Sheet No ☐

Name of embankment scheme :

TIDAL WELLAND

Location of embankment :

FROM WASH OUTFALL TO SPALDING

Nature of problem :

EROSION OF BERM AND BANKS DUE TO TIDAL AND SHIPPING ACTION

Details of problem : **Date :** 1988

THE TIDAL CHANNEL IS ARTIFICIAL AND RUNS THROUGH SILT SOILS SUSCEPTIBLE TO EROSION. PAST PROTECTION HAS BEEN AFFORDED BY TRADITIONAL THORN FAGGOTS. THESE ARE NOW COMING TO THE END OF THEIR USEFUL LIVES AND RESULTING IN AN INCREASE IN LOSS OF BERM AND BANK.

Remedial action taken or considered necessary :

REINSTATEMENT OF BERM AND BANK WITH PROTECTION NOW GIVEN BY RANDOM PITCHED STONE BATTER AND BANK.

Person to contact should more information be required :

Name J. ULYATT **Tel No** SPALDING (0775) 762123

Address

STEPPING STONE WALK

WINFREY AVENUE

SPALDING

LINCOLNSHIRE

Sheet No

Name of embankment scheme :

DEEPING HIGH BANK CRADGE BANK

Location of embankment :

RIVER WELLAND U/S OF SPALDING

Nature of problem :

BANK AND BERM CONSISTS OF SILT MATERIAL SUSCEPTIBLE TO LONG TERM EROSION

Details of problem : **Date :** 1982

AN ARTIFICIAL CHANNEL WAS CUT IN 1950. THE BANKS WERE PROTECTED AT THAT TIME AND SINCE BY THORN FAGGOTS TO RESIST WAVE ACTION AT NORMAL RETENTION LEVEL. THE FAGGOTS ARE NOW FAILING LEADING TO A LOSS OF BERM WIDTH AND EMBANKMENT.

WORK IS ONGOING.

Remedial action taken or considered necessary :

REINSTATEMENT OF BERM WIDTH AND PROTECTION OF RIVER FACE USING A VARIETY OF METHODS INCLUDING:-

(a) Asbestos Piles
(b) Random Pitched Stone
(c) Duracem Piles
~~(d) Lightweight Steel Piles~~

Person to contact should more information be required :

Name JOHN ULYATT **Tel No** SPALDING (0775) 762123

Address

STEPPING STONE WALK

WINFREY AVENUE

SPALDING,

LINCOLNSHIRE

Sheet No 3

Name of embankment scheme :

GREATHAM CREEK

Location of embankment :

GREATHAM CREEK – TIDAL EMBANKMENT

Nature of problem :

PENETRATION IN STORM TIDE CONDITIONS

Details of problem : Date :

WATER PENETRATING THROUGH EMBANKMENTS (CONSTRUCTED WITH SLAG & SAND) RESULTING IN TOTAL COLLAPSE – Approx 4 YEARLY INTERVALS

Remedial action taken or considered necessary :

REPLACE BREACH WITH CLAY

Person to contact should more information be required :

Name D. CLARKE Tel No 0325- 480849

Address NATIONAL RIVERS AUTHORITY

STRESSHOLME S.T.W.

SNIPE LANE

BLACKWELL

DARLINGTON CO. DURHAM.

HR QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS 19/02/90

Sheet No 3

Name of embankment scheme :

Sampool

Location of embankment :

River Kent (tidal section) adjacent to Sampool Caravan Park

Nature of problem :

Erosion of river bed and lwoer banks adjacent to a flood embankment.

Details of problem : Date : March 1990

This section is subject to erosion from freshwater flows which are undermining the river bank, whilst the adjacent flood bank (tidal) is endangered. It is not possible because of the proximity of static caravan to move the flood bank.

Remedial action taken or considered necessary :

Placing of large rocks in the bed.

Person to contact should more information be required :

Name R.V. Willi Tel No 05395 60567

Address

Beathwaite,

Levens,

Kendal,

Cumbria. LA8 8NL.

Sheet No 1

Name of embankment scheme :

RIVER MERSEY REHABILITATION SCHEME

Location of embankment :

BETWEEN ASHTON WEIR (SJ 773 936) AND STOCKPORT E.T.W. (SJ 870 890)

Nature of problem :

EROSION AND SILT DEPOSITION DURING FLOOD FLOWS

Details of problem : Date :

BANK EROSION AT FLOOD FLOWS.

SILT DEPOSITION, CAUSING REDUCTION IN FLOW CAPACITY OF CHANNEL.

Remedial action taken or considered necessary :

REMOVAL OF SILT OVERBURDEN AND RECONSTRUCTION OF BANKS.

Person to contact should more information be required :

Name JOHN RUCKLEDGE Tel No 061 973 2237 (Ext. 168)

Address

NATIONAL RIVERS AUTHORITY,
SOUTH FLOOD DEFENCE AREA,
MIRWELL,
CARRINGTON LANE,
SALE, M33 5NL,
CHESHIRE.

HR QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS 19/02/90

Sheet No

Name of embankment scheme :

RIVER FLOOD EMBANKMENTS TO RIVERS OUSE, ADUR & ARUN

Location of embankment :

OUSE – NEWHAVEN TO BARCOMBE
ADUR – SHOREHAM TO HENFIELD
ARUN – LITTLEHAMPTON TO PULBOROUGH
} ALL IN SUSSEX

Nature of problem :

All the problems described in your letter of 19/2/90 occur frequently except for mass failure which is fairly rare. General settlement of course is an on-going problem

Details of problems in general Date :

OUSE Wave action erosion, bank leakage, tide flap operation
Bank failure Jan 1990 at Southease

ADUR Bank leakage, overtopping, flood flow erosion, land drainage problems, conservation problems with bank mowing.

ARUN Severe wave + flood flow erosion (Arundel to Littlehampton). overtopping, seepage, land drainage problems

Remedial action taken or considered necessary :

OUSE Several miles of chalk revetment placed annually. Southease bank failure being repaired now.
ADUR Some chalk revetment and bank raising done annually
ARUN 'Endless' (!) revetment schemes

Person to contact should more information be required :

Name ADRIAN BIGGS Tel No WORTHING (0903) 820692

Address

NRA SOUTHERN REGION
GUILDBOURNE HOUSE
CHATSWORTH RD
WORTHING, SUSSEX

Sheet No 1

Name of embankment scheme :

Woodholmes, British Coal Rechargeable

Location of embankment :

Between Ferrybridge and Beal on R. Aire.

Nature of problem :

Groundwater seepage through embankment.

Details of problem : Date : Jan 1990

Field behind bank was fairly wet prior to bank raising/improvements. Key trench was excavated on line of new bank but it appears that a lens of sand was missed (?) or at deeper level than impervious key.

Remedial action taken or considered necessary :

Boreholes taken.
Sheet Piling – to cut through pervious layer.

Person to contact should more information be required :

Name B.M. Bramman Tel No Leeds 440191

Address
21, Park Square South
Leeds
LS1 2QG

DLC. 12
484

HR QUESTIONNAIRE ON PERFORMANCE OF EMBANKMENTS 19/02/90

Sheet No 2

Name of embankment scheme :

Ea Beck - British Coal, Rechargeable

Location of embankment :

Norwood near Thorpe Marsh, South Yorkshire.

Nature of problem :

Numerous slips in old bank.

Details of problem : Date : 1988/89.

Numerous slips and slumping to an old bank which had been poorly constructed and compacted. High moisture content. Weak material.

Remedial action taken or considered necessary :

Banks rebuilt to new cross section and using imported colliery shale (weathered)

Person to contact should more information be required :

Name....B.M. ~~Bamms~~..... Tel No ....

Address

as sheet 1

....

....

....

....

DEC.12