First Technical Progress Report

Risk Assessment for Sea and Tidal Defence Schemes

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GLOSSARY

Coast protection
Works to protect the land against erosion or encroachment by the sea (NRA, 1993)

Coastal defences
Collective term covering protection provided to the coastline. This includes protection against erosion of the shoreline by waves (known as coast protection) as well as against flooding of low-lying land by the sea (known as sea defence) (NRA, 1993)

Coastal embankment
An embankment designed for wave action as the primary loading. A coastal embankment is usually a type of sea wall

Damage
A detrimental change to a structure which increases the probability of failure

Embankment
An artificial bank such as a mound or dike raised above the surrounding ground levels, generally constructed of soil, to hold back water and prevent flooding

Failure
Loss of ability of a structure to perform one of its principal functions. For the purpose of this study, that function will generally be defence against flooding from the sea, along the coast or in estuary or tidal waters. The term 'functional failure' may also be used.

Foreshore
The part of the shore lying between high water and low water

Functional failure
Synonymous with failure

Hazard
A situation that could occur during the lifetime of a product, system or plant that has the potential for human injury, damage to property, damage to the environment, or economic loss

Overtopping
Flow of water over the top of a structure as a result of wave run-up or high water levels (NRA,1993, slightly modified)

Revetment
A cladding of stone, concrete or other material to stabilise and protect the sloping surface of an embankment, natural coastline or shoreline structure against erosion by wave action or currents (NRA, 1993)

Risk
A combination of the probability, or frequency, of occurrence of a defined hazard and the magnitude of the consequences of the occurrence (British Standard 4778, 1991, Royal Society,

1992).

Risk assessment
The integrated analysis of risks inherent in a product, system or plant and their significance in an appropriate context (Royal Society, 1992)

Sea defence
Works to prevent or alleviate flooding of low-lying land by the sea (based on NRA, 1993)

Sea wall
A shoreline structure primarily designed to protect against erosion, flooding or a combination of both (NRA, 1993). Sea wall types range from vertical solid walls to revetted earth embankments. The primary loading is from wave action. This definition is not consistent with terminology used in the Sea Defence Survey, where the term 'wall' is used to describe a specific element type in the classificaiton which also includes 'embankment'

Stochastic
Governed by the laws of probability (NRA, 1993)

Structural failure
The state of a structure which has lost its ability to perform its principal functions due to excessive deformations. Structural failure can be considered to be an extreme form of damage, and will genrally lead to functional failure.

Tidal embankment
An embankment located in a sheltered area where wave action is not the primary loading

1. INTRODUCTION

This is the first Technical Progress Report for the NRA R&D project 'Risk assessment for sea and tidal defences'. The work is being carried out under Topic C06 - Coastal and Estuarine Works/Structures, and the Project Number is 0459. The project began in February 1993 and is due for completion in December 1995. The study has been divided into three phases. This report presents findings of Phase 1.

1.1 Terms of reference for study

The overall project objective is:

> 'To develop probabilistic design / analysis methods to assess the risks of failure for new or existing sea or tidal defence schemes, including areas at risk from flooding.'

Specific objectives are addressed under three interrelated themes: these are given below.

1) To derive data and develop understanding of the modes and risks of failure of sea and tidal defence structures using analysis of past failure, both functional and structural.

2) To develop appropriate techniques for the assessment of the probabilities of failure of such structures

3) To develop methods to assess the areas, extent and severity of flooding over the relevant range of risk levels

The project is divided into three phases. Activities to be carried out in each phase are summarised below, and are shown in the schedule of activities for the project (Figure 1.1):

Phase 1	Identify structures of interest, define principal elements, describe failure modes for each structure / element. Construct initial fault trees and make simple calculations of failure probabilities. Prepare inventory of data for detailed flood area mapping.
Phase 2	Develop failure modes and fault trees, carry out simplified probability modelling, analyse failure from existing databases. Develop appropriate risk assessment methods, and carry out initial applications. Evaluate flood area mapping techniques for three representative sites.
Phase 3	Calculate risks of failure for a range of real and / or idealised schemes.

1.2 Purpose of this report

This Technical Progress Report describes findings from Phase 1. It includes discussion of structure classification, structure types and failure modes. The report also sets out definitions of terms which will be adhered to throughout the project. The project is at an early stage, so the report also discusses several issues which have not yet been resolved. It should therefore not be seen as a final statement on any aspect: its purpose is to inform selected staff within the NRA of the progress of the project, to discuss and define its scope, and to raise issues which will be addressed in later phases of the project. It is hoped that it will generate comment, advice and input from within the NRA. This report also serves as a record of methods and/or data which have been considered and rejected.

Within the report, some paragraphs are written in square brackets [ ]. These discuss issues which are unresolved, on which comment would be particularly welcome.

2 STRUCTURE TYPES, PRINCIPAL ELEMENTS AND FAILURE MODES

This chapter identifies the main structures with which this study is concerned, presents their principal components, and identifies the main terminologies and classification which have been used. Damage mechanisms are identified for each structure type (Section 2.2). These are the main changes to a structure that can lead to failure. The terms 'failure' and 'damage' are defined in the glossary, and failure modes are discussed at greater length in Section 2.5.

The present project is concerned with defence against flooding of low-lying land by the sea in coastal, estuarine or tidal environments. The structures discussed here have a sea defence function. In some circumstances, a sea defence structure may also provide protection to the coastline against erosion by waves, in which case the structure has a coast protection role. Coastal defence is a collective term covering protection against flooding and against erosion.

2.1 Classification

A classification system for sea walls in the UK was presented in the CIRIA Technical Note 125, and is reproduced in Figure 2.1. (CIRIA, 1986b). Each sea wall is broadly classified according to the slope of its seaward face. The vertical category includes sea walls with battered faces between vertical and a slope of 1:1, and with re-curved faces. Sloping walls are defined as having seaward slopes less steep than 1:1. Sea walls may have a combination of vertical and sloping walls, in the form of a compound profile. Sloping walls may have a single slope, or the profile may include multiple slopes, possibly incorporating a berm.

The next level of categorisation distinguishes between porous and non-porous walls. Porous walls are designed to dissipate wave energy by the action of flow through voids in the structure. Examples of porous sea wall revetments include those armoured with rip-rap and specially shaped concrete units.

The final levels of classification identified in TN 125 are concerned mainly with structural aspects such as the materials used and the form of construction.

An alternative classification system was used for the Department of the Environment Coast Protection Survey (1980) in reviewing the Coast Protection Act (1949). A set of 80 1:25000 scale map sheets were produced of the coastline of England, and these were marked to indicate the existence and types of sea defences, beach types and areas of known accretion or erosion. The following classification system was used for defences:

- Embankment:
  - Clay
  - Concrete faced
- Gabions (or similar)
- Revetment:
  - Timber
  - Concrete blocks
  - Rock
- Sea Walls
- Groynes

- Dunes

Beach types were broadly classified by material (eg sand, shingle, mud, chalk etc. Cliff types are also classified by material (eg sandstone, chalk, clay etc.)

A preliminary classification system has been developed with the aim of describing the key features for a preliminary assessment of geotechnical and hydraulic stability. Twelve structure types are identified, and these are reproduced in Figure 2.2. Types 1 to 7 relate to sea defence embankments. Types 8 to 11 are labelled 'revetment', although this conflicts with the normal terminology where revetment refers to just the armoured surface of an embankment. The distinction between a 'revetment' and an embankment is the ratio of width, W, to height, h. In general it is assumed that a structure with W/h > 7 acts as a revetment used for coast protection, whereas other structures act as flood defence embankments. The need for this distinction is that some sea defence works which have wide crests will have failure modes more similar to coast protection revetments than embankments. The advantage of this type of classification is that the structure type indicates its vulnerability to damage. For example, an embankment with an unprotected rear face is potentially vulnerable to damage from overtopping flows.

Although each of types 1 to 7 and 8 to 11 is shown with the same overall geometry (with sloping front and back faces and a horizontal crest) it is noted that in fact the front and back slopes can be simple, bermed or vertical, of varying constructional forms. This secondary classification may be important because, for example, the slope and nature of the front wall influences the risk of toe scour.

The NRA's National Sea Defence Survey (SDS) provides a framework for recording types of structures, their properties and condition. A defence is considered to comprise one or more structures. The structures identified include the following:

apron	cliffs	shingle ridge
armour	dune fence	splash wall
bastion	embankment	stop-log
bank	gabions	tetrapod
breakwater	groynes	valve
breastwork	piling	wall
	revetment	wave return wall

Other categories include material (clay, rock, etc), position (eg hinterland, backshore, etc), and slopes, toe levels and crest levels.

The above list is rather inconsistent, incorporating a number of levels of information, and with some obvious omissions. For example, 'tetrapods' are only one specific example of concrete armour units (SHEDS, accropodes etc). However, the attraction of the SDS form of classification is that it enables information on different elements of a defence to be collected and recorded: the system is not limited to pre-defined generic structure types. This is particularly useful for the present project because the response of a structure depends not only on the type of structure, but also on the characteristics of individual elements, and their interraction.

In the CIRIA report on seawall design, Thomas & Hall (1992) propose that a sea wall can be divided into three main elements, as an aid to the design process:

- Body (including front face and core);
- Toe;
- Crest (including back face)

Each element has specific functions, and the ability to fulfil those functions depends on the components which make up each element..

The classification system adopted for a risk assessment methodology should incorporate features of several of the above systems. It may need to allow a broad classification of structure type, particularly if initial screening procedures are required. More detailed analysis will require information on the individual elements of a structure, and their functions. This will include, for example, what type of crest? is there a wave return wall? what is the level and strength of the wave return wall? The level of detail required will depend on the vulnerability of the structure and on the consequences of failure.

The following sections describe a number of structures which can have a sea defence role. Structures are broadly classified as sea walls, embankments, natural banks and other structures such as gates. Within each broad classification, structures are described with reference to their principal components.

[We have not yet drawn a definitive classification system, although the discussion in the above paragraphs and the structure of the sections below reflect our thoughts on the matter. NRA views would be welcomed]

2.2 Structure types and principal elements

2.2.1 Sea walls

These are structures built along the shore to protect the land from the sea. A sea wall may have both coast protection and sea defence functions. Sea wall types range from massive vertical retaining walls to sloping revetments. In general, many urban sea walls are vertical or near vertical monlithic type, whereas rural sea walls more often consist of a sloping layer of revetment material protecting the underlayer. For most sea walls, wave action is an important environmental loading.

Functions

- Protection to erodible coastline
- Flood protection, sea defence
- Protection to reclamation bunds
- Rehabilitation mound to existing vertical walls

Rock armoured revetment

Components

An example of a sea wall is shown in Figure 2.3. A layer of rock armour protects a filter layer, which is placed directly on the natural beach material. The rock armour may extend well below the sloping beach to protect the sea wall embankment fill from erosion by toe scour.

Damage mechanisms

- Slip failure of subsoil or core
- Erosion at toe (scour)
- Unstable armour: at crest, front slope, toe. Depends on wave and current loadings, and stone diameter, density, shape, and interlocking characteristics.
- Erosion of filter layer due to internal flows
- Erosion of subsoil due to internal flows
- Cracking of armour due to wave action or thermal effects. Depends on the intensity of the loading and the strength and properties of the rocks
- Liquefaction of subsoil: results in serious deformation and possibly collapse. Unsteady forces (from waves or earthquakes) lead to excess pore pressures and loss of strength of subsoil
- Outflanking ie erosion of natural beach at ends of defence

Sea wall with rock armoured revetment and wave return wall

Components

The sea wall shown in Figure 2.4 is similar to the rock armoured revetment described above, but also provides a wave return wall and roadway along the crest. In addition, the figure shows fill material forming a core for the revetment.

Damage mechanisms

As for the rock armoured revetment above, plus:

- Structural failure of the wave wall due to excessive impact pressures
- Sliding or tilting of the wave wall block due to excessive impact pressures
- Tilting or lowering of the wave wall block due to internal erosion of the core material
- Damage or deterioration to joints between adjacent sections of wall

Sea wall with blockwork revetment

Components

This type of sea wall generally consists of an embankment protected by a single armour layer of square or rectangular blocks laid on a suitable underlayer.

Damage mechanisms

As for the rock armoured revetment above, although displacement and damage to rocks does not apply: instead, these mechanisms become desplacement and damage to armour blocks. In addition, blockwork revetment slopes are genrally built with crets beams and toe beams, and these bocome damaged due to structural failure or settlement.

2.2.2 Embankments

Function

- Sea defence

An embankment is defined here as an artificial bank such as a mound or dike raised above the surrounding ground level to hold back water and prevent flooding. The primary function considered by this project is therefore sea defence. Embankments may be attacked directly by waves, when a revetment will be required to protect the seaward face of the embankment, or may be in sheltered estuary locations, or set back from the foreshore, and in these circumstances, waves may not be the most significant source of hydraulic loading. Flood embankments may be protected by vegetation or by artificial protection such as rip-rap.

Flood embankment without artificial protection

Components

The simplest form of flood embankment is normally constructed to a trapezoidal cross section with side slopes of, typically, 1:2. The top width is normally at least 2m, and a wider section may be required in order to gain access for maintenance or repair. Failure modes depend on the construction and materials of the embankment, and on the degree of exposure to waves, high water levels and other environmental factors.

Damage mechanisms

- Slip failure of subsoil or core. This is influenced by the water level and its effect on pore water pressures
- Seepage through the embankment
- Fissuring of clay embankments during dry weather. This is affected by properties of the embankment material and the type and cover of vegetation
- Erosion of the front face of the embankment due to waves and currents. The degree of erosion will depend on the type and cover of vegetation, the properties of the embankment fill material and on the hydraulic loading. Erosion may lead to slope failure of the front face, lowering of the crest, and possible overtopping and breaching.
- Overtopping due to a combination of waves and water levels
- Erosion of the crest of the embankment due to a combination of waves and water levels. May in turn lead to more overtopping and more severe erosion and breaching
- Erosion of rear (landward) face due to a combination of waves and water levels. May lead to slope failure of rear slope, lowering of the crest, increase in overtopping discharge and, consequently, more severe erosion and breaching

Protected flood embankment

The embankment may be protected on its front or rear faces or crest to reduce the likelihood of damage. Protective revetments include stone rip-rap or concrete blockwork. Damage mechanisms are the same as those for an embankment, except that the revetment protection may also be damaged. These additional types of damage are given below:

- Damage to revetment layer due to excessive wave action
- Thermal cracking of revetment materials
- Vandalism: removal of revetment blocks
- Erosion of embankment at edges of revetment layer
- Loss of grout between revetment units
- Loss of embankment material beneath revetment (eg revetment arches across voids)

2.2.3 Natural banks

Natural banks, often sand dunes or shingle banks, may perform a flood defence role, particularly in coastal areas.

2.2.4 Control structures and gates

Sea defences may be intercepted by drainage structures and flood gates. Possible failure modes depend on the type of structure, but the mode of failure may be due to:

- Mechanical failure (eg a flap valve fails to close due to trapped debris or a jammed mechanism)
- Operational error (eg failure to close a flood gate in a flood wall, possibly the result of operator error or a system failure)
- Failure due to design eg crest level of a flood defence gate too low for applied waves and water level

It is proposed to consider these structures within the present project, where they form an integral part of a sea defence structure. [In cases where a control structure forms a major element of a defence in its own right, we consider that this should fall outside the scope of this project. For example, a barrage or movable gate structure across an estuary will generally have complex control systems and will have been designed to withstand a certain return period event. This is likely to be well defined following detailed design studies, and it is not considered appropriate to include this type of defence structure within this project.]

2.3 Reported damage

In the survey of the design and performance of sea walls (CIRIA, 1986b), information on failure of sea walls was gathered in response to a questionnaire. The responses relate to sea walls in which wave action is the dominant design consideration, and are summarised below:

- Damage to sea walls may be minor and easy to repair, and may cause no immediate increase in risk. Some types of damage may result in serious failures if the damage is allowed to develop.
- The questionnaire requested information on damage history, 'especially serious

failures', the length of wall affected, the part of the wall affected (eg toe, crest, etc) and the mode of failure. Responses to the questionnaire included 188 incidences of damage, representing approximately 37% of the sea walls for which returns were received.

- The most common type of damage was erosion of the toe, which occurred at about 12% of all walls for which returns were received. Other types of damage included partial crest failure (5.1%) collapse/breach (3.1%), removal of revetment armour (3.7%), abrasion (3.0%) wash-out of fill material (1.9%) and concrete disintegration (1.7%). Damage classified as erosion of the toe therefore accounted for about a third of all reported damages incidences.
- Types of damage occurring at 1% or less of walls reported included structural member failure, landslip, corrosion, outflanking, uplift of armouring, settlement, spalling of concrete, damage to promenade and concrete cracking.
- The types of damage reported may not be independent. For example, washout of fill may have been the result of erosion to the toe. The format of the questionnaire did not encourage reporting of multiple failure modes.

It is important to bear in mind that the reported incidences of damage do not necessarily represent failures of the sea wall. For example, although removal of revetment armour is undesirable and will reduce the safety of the structure, it may not automatically lead to failure. Failure depends on the degree of damage to the structure and to the land and property it protects, and the environmental loading on the structure.

The influence of wall type on the three most serious types of damage is illustrated in Figure 2.5. This shows the percentages of each wall type reported to have suffered different types of damage. For example, over 15% of non-porous slope walls were reported to have suffered from toe erosion, compared with 12.9% of vertical walls and only 6.5% of porous slopes. The incidence of collapse/breach and wash-out is higher for slopes than it is for vertical walls.

As a result of investigations into the East Coast flooding of 1953, Cooling & Marsland (1953) describe four main initial causes of failure of earthen sea banks:

a) Erosion of the seaward face by wave action. This can be a very important factor in exposed locations, but is less likely to lead to damage at river or estuary embankments, or those fronted by extensive saltings.

b) Erosion of the landward face by overtopping. The evidence from the 1953 event indicated that surface erosion due to overtopping flows is unlikely to be a primary cause of failure except where severe overtopping occurs. Cooling & Marsland suggest that erosion due to overtopping is more likely to be a second stage following weakening of the surface layers due to shallow slips on the rear face (discussed below).

c) Slipping or slumping of the landward face caused by seepage of water through the bank. Seepage of water through a porous embankment causes drag forces on the particles of the embankment. The type of failure that may be initiated depends on the material of which the embankment is made. Pervious embankments of sandy material tend to fail by slumping or 'flowing' initiated at the toe of the bank. Banks constructed of soft highly shrinkable clay fail by a different mechanism. Slope failure is related to the tendency of these banks to crack and fissure on drying. The softening of the wetted clay and the drag caused by flow through the fissures may cause shallow slips.

d) Failure of underlying layers due to uplift pressures. This type of failure is possible when pervious layers beneath an embankment outcrop in the estuary bed. Rising water levels result in increased pore water pressures in these layers. This may cause large quantities of seepage and erosion to the landward face of the bank. In other cases, there may be little or no visible seepage, but the increase in pore water pressures can cause a serious reduction in the strength of the pervious layer. This can lead to rapid breaching of the bank.

Storm surge of February 1993

Significant flooding occurred along the East Coast of England on 21/2/93. The main cause was a surge of approximately 2m combined with a spring tide. High winds probably also contributed. The following notes summarize press reports of events:

- Gorleston, Gt Yarmouth: 400 people evacuated as the sea breached defences
- Walcott, Hemsby and Morston: Coastal villages evacuated. Coastal erosion lead to loss of 5 holiday bungalows at Hemsby
- A47 road closed: River Bure burst its banks at Acle
- Cley, North Norfolk: 1.5 miles of sea defences swept away
- BR trains between Norwich and Lowestoft cancelled
- Suffolk: many roads closed after overtopping of sea walls two hours before high tide
- Southwold: Beach huts and cafe 'swept away by the tide'. Police evacuated low lying properties
- Aldburgh: Police evacuated low lying properties
- Southend: Water level 9ft higher than normal
- Scarborourgh: Seafront evacuated as waves 25ft high swept over coastal roads
- Norwich (20 miles inland): Minor flooding

The NRA supplied HR Wallingford with their flood report for this event. The report is mainly concerned with procedural and communications matters, but the maps enclosed with the report give brief descriptions of observed damage and flooding:

Location	Type of damage, extent of flooding
North Norfolk Coast (Drg 7661/01)	COASTAL - Minor damage to gabion groynes - Scour and sand loss to dune frontage - Scour behind new uncompleted gabion works - Damage to dune faggot defences - Minor damage to dune frontage - Shingle bank: 15 breaches over 4km. Crest lowered. Shingle pushed landward by waves. Flooding. Bank reduced wave energy INLAND - Bank overtopping: minor flooding to fields
Happisburgh to Winterton (Drg 7661/02)	COASTAL - Overtopping at gate - Minor breach through pedestrian access in wall - leak onto road through gate - Water lapped over access - Gate damaged: emergency repairs undertaken - Beach badly eroded - Groynes damaged (missing boards etc)
Broadland (Drg 7661/03)	INLAND - Overtopping of inland defences, numerous locations - Breaches in inland defences, numerous locations - Damage to rear of wall - Concrete wall undermined - Leaking flood wall - RNLI gate leaked - Foundation to flood wall suspect
Suffolk District (Northern area drawing)	COASTAL - Numerour breaches in shingle banks - Severe erosion to landward face of shingle support bank - Transition wall: severe erosion to top and back of wall INLAND - Wall breached - Wall breached and slips on back of wall - Wall breached in 7 places and extensive slippage of wall back - 2 small slips - Slippage and blockwork damage

Suffolk District (Southern area drawing)	COASTAL - Confused seas cause overtopping: localised flooding INLAND - Front face of wall eroded: minor damage where overtopped - Leakage through and under concrete wall - Piled defences leaked causing localised flooding - Blockwork revetment damaged. Extensive erosion to face of wall

The damage to sea defences during this event was estimated to be in the of the order of £0.8 to £1.0 million.

[The failures resulting from this storm surge are particularly relevant to this project because of the range of problems encountered:

- coastal erosion
- coastal flooding due to overtopping caused by surge, high tide and waves
- Breaching of coastal flood defences, probably due to a number of different mechanisms in different areas eg wave action, erosion from overtopping etc
- Inland flooding due to high water levels: waves probably not significant. Levels may have been exacerbated by freshwater flows
- structural failure of inland embankments]

2.5 Failure modes

A failure mode is an event, or a sequence or chain of events, which lead to failure.

A failure is defined as unacceptable discharge across a sea or tidal defence. Failure of a defence therefore occurs when it fails to perform its sea defence role.

A failure is not necessarily the result of damage to a structure, although the degree of damage will in many cases affect the likelihood of failure. Indeed, a structure may be so damaged by a particular storm event, without actually failing, that it is very likely to fail during the next event, even though this may not be an extreme event. [It is tempting to describe this as a partial failure, but this terminology is inconsistent with the definition of failure. If we wish to talk about degrees of failure, then a complete failure could be formation of a large breach, while a partial failure would be a small amount of overtopping. It is important to distinguish between damage and failure.].

The consequences of failure depend on many factors, including the magnitude and duration of the discharge across the defence, the topography and land use of flooded areas, and the effectiveness of advanced warning systems and other emergency measures.

Failure modes may be shown on flow charts, as in Figure 2.7. [This drawing is not complete, and does not include some failure modes which are described below, although it illustrates a potentially useful way of presenting potential failure modes]. This diagram is not strictly a fault tree or event tree, (see Section 3.2.1) and cannot be used for probabilistic analysis. It does however provide a convenient method for summarizing the failure modes. Additional useful examples failure mode diagrams are given by Thomas and Hall (1992). Again, they are labelled fault trees, but in fact resemble more closely the type of presentation given in Figure 2.7, presenting chains of events leading to failure. Diagrams are presented for several types of failure including flow under or over a sea wall, damage to the front face of a sea wall, and events leading to geotechnical and slope instability.

The following sections describe failure modes for sea and tidal defences. These are considered to belong to three categories:

- failure during a storm event, when the failure is not related to or associated with any structural change to the defence as a result of the storm. Failure therefore depends only on the loading, in terms of the severity of the storm, and on the state of the structure before the storm. Example - overtopping

- Failure as a result of damage during the storm. In this case, failure is affected by changes to the structure during the storm. Example - formation of a breach following excessive overtopping

The two categories above can be thought of as passive failure mechanisms: they are controlled by the loading on the structure and the properties of the structure, in terms of its ability to withstand the loadings placed on it. The third category is identified here as 'active' failure mechanisms:

- these are normally the result of human error, operational errors, or mechanical fault: failure is not dependent on the inherent ability of the structure to withstand the forces imposed on it. Example - failure to close a flood defence gate in a sea wall

Each category of failure mode is discussed in more detail in the following sections.

2.5.1 Failures not associated with structural damage

This section considers failure modes which do not involve any damage to the structure. These failure modes are often relatively simple to analyse because there is no need to formulate damage functions for the structure. It is assumed that the structure is unchanged during the storm. The failures may, however, be influenced by the condition of the structure prior to the storm. For example, overtopping discharge may be increased if settlement has cause lowering of the crest level.

The main modes of failure are given below:

- Flow over the defence, due to a combination of waves and water level (Figure 2.8a). Normally divided into two cases: overflowing due to water levels exceeding the crest level, and overtopping due to waves, where the water level may be below the crest level. This failure mode is influenced by previous settlement of the embankment which causes a lowering of the crest level.

[Steady overflowing would normally be calculated by a type of weir equation, and wave overtopping by the Owen formula. The project will later need to consider the range of applicability of these methods. Specifically, we may need to address how to calculate overtopping discharge when neither formula seems totally applicable, such as the case where tthe water level is above the crest with significant waves?]

- Groundwater flow through the defence, driven by the static head difference between seaward and landward faces. This may also be influenced by waves which modify the pore water pressure in the embankment. This type of failure will depend on the water level difference across the defence, and on material and dimensions of the embankment or sea wall. Groundwater flow may be increased if permeable layers or fissures are present.

- Groundwater flow beneath the defence. This depends on the underlying geology, the permeability of sub soil and the presence of permeable strata.

  [Groundwater flows are unlikely to be major causes of failure in tidal areas, due to the low permeability of flood embankments and sea walls and the relatively short duration high water levels in tidal waterways. However, groundwater flows may cause damage as discussed later, and this damage can lead to more severe failure such as breaching.]

- Leaking gates, stop-logs etc. This depends on the design and condition of the gate.

- Leaking joints in concrete crown walls and flood walls.

- Flows through animal burrows.

2.5.2 Failures due to storm damage

The most severe failures are usually the result of damage to a structure during a storm. For example, damage to the rear (landward) face of an embankment due to overtopping flows can lead to erosion of the face and breaching. Damage caused by the storm loading leads to failure during the storm.

The probability of failure also depends on the state of the structure prior to the storm. For example, unrepaired damage to a blockwork revetment, perhaps the result of a previous storm or vandalism, will increase the risk of failure of the armour layer during subsequent storms. Changes in the strength of, and loadings on, a structure with time are illustrated in Figure 2.6: damage during storms or from other events reduces the strength, and a general reduction with time may also occur, as materials deteriorate. Maintenance increases the strength. The other line in the figure shows the loadings on the structure in time, which will usuallty be from waves and water levels. At some point, when the loading exceeds the strength, structural failure will occur, invariably resulting in functional failure. The probability of failure during a particular storm depends not only on the intensity of the storm and the 'strength' of the structure as designed, but also on the degree of damage, or condition, of the structure prior to the storm. [An aspect which this project will consider is significance of structure condition on the probability of failure, and how the condition of a structure can be taken into account as part of a risk assessment procedure.]

Breaches

The most dramatic and severe mode of failure is a breach in the flood defence. A breach may be initiated over a short length of embankment, with the development of a small gap. Causes of this initial small gap are discussed below. The discharge through a gap may be small, but these flows may cause erosion, particularly in the case of earth embankments. The breach will then tend to increase in size, allowing higher discharges to cross the embankment. This chain of events can lead to a large breach, with high discharges across the defence line. At some stage, falling seaward water levels or rising landward (flood) water levels will tend to reduce the discharge through the breach, and the growth of the breach will cease.

The mechanisms which cause localised failure and the formation of a gap are not the same as those which cause growth of a breach. A risk assessment procedure should consider the probabilities of initial failure, and the conditional probabilities that given an initial failure, the gap will grow in size to form a breach.

[A potential source of complexity here is the connection with flood area mapping. The area that is flooded depends on the discharge through a breach, itself a function of water level and the dimensions of the breach. Breach enlargement depends on erosion and geotechnical failure of the base and sides of the breach, and is a function of the landward and seaward water levels. This is a complex process about which little is known at present. It may be necessary to simplify and assume that either a gap develops into a 'large' breach, or it remains as a 'small' local failure. We will need somehow to define 'large' and 'small' in order to estimate breach discharge.]

Failure modes causing initial breach

The following sequences of events lead to an initial breach in a flood defence structure. The initiating event for each failure mode will normally be a storm, defined as a combination of waves and water levels which result in extreme loading on the structure. Other initiating events may cause damage which leads to failure. These include earthquakes, terrorism and ship impact.

Each failure mode is a sequence of clearly defined events leading to the initial breach. The failure modes are sequences of *ordered* events: the failure mode gives the order in which the events occur. For example, erosion of the crest can only happen after overtopping, not before. But the event will not necessarily occur, even if all previous events have taken place. The crest may be resistant to erosion and overtopping will not necessarily result in erosion of the crest. The progress of events which comprise each failure mode depends on the structure and on the initiating event, or storm intensity.

In order to consider specifically defined failure modes and event sequences, a series of illustrations has been prepared (Figures 2.8b - 2.8 k). The sequences which they illustrate are now described. In the descriptions, overtopping is taken to mean flow over a defence by any combination of waves and water levels.

* Overtopping → erosion of crest → lowering of crest level → breach (Figure 2.8b)

* Overtopping → erosion of landward face by overtopping flows → erosion of core (reduction in dimensions) → loss of stability leading to breakthrough → breach (Figure 2.8c)

* Applied overturning moment exceeds resistance → breach (Figure 2.8d)

* Applied horizontal force (hydrostatic + wave impact) exceeds resistance → sliding along approximately horizontal surface → breach (Figure 2.8e)

* Slope failure of landward face (seepage pressure within embankment) → reduction in dimensions → loss of stability leading to breakthrough → breach (Figure 2.8f). (Slope failure of landward face may greatly increase susceptibility to erosion damage on earth embankments.)

* Damage to seaward face (revetment) → erosion of core (reduction in dimensions) → breakthrough → breach (Figure 2.8g)

* Breakage of revetment blocks → damage to seaward face (revetment) → erosion of core (reduction in dimensions) → breakthrough → breach

* Slope failure of seaward face → erosion of core (reduction in dimensions) → breakthrough → breach

* Erosion to seaward toe → slip failure of seaward face → damage to seaward slope → erosion of core (reduction in dimensions → breakthrough → breach (Figure 2.8h)

* Seepage through internal layer → piping → internal erosion → breach (Figure 2.8i)

* Seepage through permeable foundation layers → elevation of pore pressures in foundation → slip failure within foundation layers → breach (Figure 2.8j)

* Seepage through permeable foundation layers → piping at landward toe → erosion of landward toe → slipping of landward face → reduction in dimensions → breakthrough → breach (Figure 2.8k)

* Liquefaction due to impact forces → stability failure

* Breakage of crown wall → gap in wall

* Displacement (sliding, tilting) of crown wall → gap in wall

Growth of breach

The failure modes above describe events leading to an initial breach or gap in a flood defence. The growth of this gap to form a major breach is largely independent of the failure mode leading to the initial breach. The following mechanisms thought to be largely are responsible for breach growth:

* Erosion of the base of the breach due to direct hydraulic action → deepening of breach

* Erosion of the sides of the gap due to direct hydraulic action → widening of breach

* Geotechnical failure of sides of breach → widening of breach

2.5.3 Failure resulting from operational or mechanical faults or other hazards

This category of failure was defined earlier as an 'active' failure, as it is not directly related to the inherent strength of the structure to withstand storm loading. This category includes operational error such as failing to close a flood gate, ship impact, earthquake loading, and terrorism. These mechanisms may weaken a structure or reduce its water-retaining capacity to such an extent that failure may result during the next high tide.

[2.6 Example structures selected for further study

Our initial view is that the three failures discussed below, all occurring during the storm surge of 21 February discussed earlier, could be of particular relevance. They encompass a wide range of loading conditions, structure types and failure modes.

- Overtopping of the Manor Wall at Felixstowe. This is a sloping sea wall with concrete blocks and a vertical wave wall. The sea wall is not sheltered from wave attack. The sea wall protects a built-up area of Felixstowe, and the ground behind the wall is well below the crest level. This is an example of wave overtopping without any damage to the structure. HR has wave height and direction data for the storm surge period, from three wave buoys positioned near to the Shipwash Bank, approximately 20km offshore.

- Damage to blockwork revetment along the southern bank of the Orwell estuary, with extensive erosion to the face of the wall. This was probably due to a combination of waves and water levels.

- Major breach of the River Yare embankment near Hassingham. This is an inland tidal waterway, and wave action is limited. The breach caused flooding over approximately 1km^2, and threatened a railway line. The initial cause of the breach is not known to us at present: crest erosion due to overtopping, or geotechnical failure of the rear slope are possibilities. The breach appears to have developed to a considerable size, over 100m wide.

HR has data on Overtopping from Arun District Council, in the Littlehampton area, which could be analysed, but this does not include events leading to damage. The well-publicised failure of sea defences at Towyn in 1990 would be appropriate. A storm caused breaching of the sea wall at Towyn, resulting in serious flooding. HR has offshore wave for the North wales coast, but further enquiries are needed to establish whether more detailed information on nearshore conditions is available, and whether information relating to this failure could be published.

We are currently establishing the availability of data for these events. Suggestions from the NRA of other failures which could be used as case studies would be welcome, particularly where data on the extent of flooding and the type of failure is available.]

3 PROBABILISTIC RISK ASSESSMENT TECHNIQUES

This chapter begins by outlining previous studies which are particularly relevent to the present project (Section 3.1). This is not intended to be an exhausive literature review, but illustrates the types of methods which can be used, and their strengths and limitations. Risk assessment techniques are explored and discussed in Section 3.2, and probabilistic techniques are explained in Section 3.3, including description of sources of uncertainty. Some of the terminology introduced in Section 3.1, particularly in connection with fault trees, is explained in the later sections of this Chapter.

3.1 Previous studies of sea defences

This section summarizes several key studies which are relevant to the current project. It is not intended as a comprehensive review of the subject, but the studies illustrate the state of knowledge and practice in the areas of probabilistic design and risk assessment. In addition to the literature related to sea defences, the project team has access to a range of literature on risk assessment in other areas such as industrial and nuclear installations.

CIAD (1985): The aim of this work was to investigate the use of computer models to optimise the design of breakwaters. The project focused on risk analysis in the design procedure. The project considered many failure mechanisms, and attempted to apply models and stability functions to estimate the probability of failure. Failure was defined as 'Wave penetration in the harbour too large'. It was assumed that this would occur when the crest level has fallen to 0.5m below the design level. The aim of the study was therefore to assess the probability of the crest level falling to 0.5m below the design level. A general fault tree for this failure mode for rubble mound breakwater was devised (reproduced in Figure 3.1). Clearly this is too complex to analyse, and a simplified version was developed (Figure 3.2).

The failure mechanisms for which failure probabilities were calculated are given below. For some mechanisms, the failure probability was disregarded because either it was judged that the failure probability was very low, or, presumably, because no adequate description of the mechanism could be found. This analysis applies to the particular breakwater in the study. Other breakwaters and structures in different environments will have different failure probabilities and different dominant failure mechanisms. The table below indicates the range of mechanisms that were considered.

Failure mechanism	Probability of failure (yr^{-1})
Excessive deformation of sub-soil	1×10^{-7}
Collapse of sub-soil cavities	1×10^{-7}
Excessive deformation of core	Disregarded
Base-plate fails due to unfavourable support or slamming	Disregarded
Wave wall of crest element collapses	5×10^{-8}
Crest element slides	12×10^{-3}
Crest element tilts without direct wave loading	13×10^{-3}
Overloading of support stone beneath base	Disregarded
Washout of base-plate support material	Disregarded
Support from inner slope disappears	Disregarded
Crest wall tilts after armour blocks have disappeared	26×10^{-3}
Unfavourable support of armour blocks	Disregarded
Fracture of armour during placing	20×10^{-3}
Fracture of armour due to direct wave action	1×10^{-3}
Fracture of armour due to production method	1×10^{-3}
Hydraulic instability of outer slope	6×10^{-3}
Geotechnical instability of outer slope	7×10^{-3}
Hydraulic instability of armour at berm on outer slope	0.9×10^{-3}
Erosion at outer toe	Disregarded
Local geotechnical instability of toe	8×10^{-3}
Washout of inner material below armour	Disregarded
Failure due to earthquake action	23×10^{-3}

In order to combine the failure probabilities to obtain an overall estimate of the structure failure probability, all mechanisms with probabilities below 10^{-4} were disregarded. The remaining mechanisms were assumed to act in series, connected by an OR gate. Taking the two extremes of complete dependence and complete independence gave upper and lower bounds for the annual structure failure probability of 0.185 and 0.075 respectively. The concept of the fault tree as a logical description of the interactions between different failure mechanisms was therefore lost. It was assumed that failure of any one mechanism would result in the TOP-event.

The CIAD report contains no discussion of the consequences of failure, except the assumption that a 0.5m lowering of the crest results in excessive wave action behind the breakwater.

PIANC (1992) summarises recent work carried out by number of organisations into the safety of breakwaters. This included identification of formulae for failure modes of rubble mound breakwaters, discussion of the treatment of environmental data and extreme events in probabilistic design and assessment, and the development of partial safety coefficients for conventional multi-layer rubble mound breakwaters. HR Wallingford (1993) presents probabilistic analysis, using Monte-Carlo sampling, for rubble mound armour layer stability and for overtopping of sea walls. The advantage of the sampling approach is that it allows greater flexibility in the response functions and distribution functions that can be considered. Development of formulae for partial safety coefficients entails considerable work, even for a single failure mechanism. These studies both focused in some detail on specific failure mechanisms.

The study by CUR/TAW (1990) is concerned with design of flood defences, and includes discussion of inundation characteristics, damage due to flooding and associated costs, as well as failure probabilities. Reflecting the approach of CIAD (1985), CUR/TAW begins with a discussion of fault trees and event trees, and presents a complex fault tree for inundation due to failure of a flood defence structure (reproduced in Figure 3.3). The report considers the following failure mechanisms: overflowing; wave overtopping; macro-instability (ie slope failure along a large failure surface); micro-instability of slopes, typically resulting from seepage in non-cohesive soils, and piping resulting from entrainment of soil particles by seepage flows. For each mechanism, reliability functions are proposed, based on empirical formulae or modelling results. The report also discusses how the length of a dike can be taken into account. The reliability functions are two dimensional, and failures tend to occur at one or more localised points along the length of the dyke, so a long length of defence will generally have a higher failure probability than a short length. This is not taken into account by the standard reliability functions, and depends on the correlation between strength and loading parameters along the length of the dyke.

The report includes discussion of the consequences of inundation, including factors affecting the degree of damage, and includes estimates of costs incurred from inundation of different land use types to different depths.

Two criteria for acceptable risk are considered: the degree of risk acceptable to the individual, and the degree of risk acceptable to society. The latter is approached in two ways: an economic evaluation is possible based on the cost of raising a defence and the expected cost saving due to a reduction in risk. Alternatively, a socially acceptable level of risk can be formulated, in terms of a number of deaths per year.

An example of the design of a river flood defence dike is given. The failure mechanisms identified above were evaluated for this example, and it was found that only the mechanisms for overtopping and piping were significant: macro- and micro-instability were found to have low failure probabilities. The probabilities from these two mechanisms were combined assuming that the mechanisms are independent. The lowest failure probability was associated with the highest crest level and the flattest slopes. An optimisation procedure can then be carried out to determine the most economic solution, given the constraints and limitations of the analysis.

The NRA Level Of Service (LOS) approach (NRA, 1990) was designed to enable the NRA to define the standard to which the NRA provide flood defence. The method can be applied to non-tidal river, tidal river, estuary and coast. Under this system, reaches of between 4 and 7km in length are defined. The maximum known extent of flooding is established for each reach, and the land use is taken into account. The land use is categorised on the basis of the House Equivalent (HE) per kilometre of riverbank. Estuary and coastal reaches have only one bank per reach. For each type of land use, a target range of HE's affected by flooding per year per kilometre is proposed. The score for each reach, in HE/km/year is the unit which specifies the Level Of Service for the reach. A corresponding standard of protection in terms of return period is also recommended. Target standards for flooding by sea water are higher than those for river flooding to reflect the extra damage caused by saline flood water.

Two methods are proposed for assessing the frequency of flooding. These are designed to take account of a) observed flooding over a short (5 year) timescale, and b) predicted flooding for a range of return periods.

The latter method may be adapted to account for perceived reduction in performance of an asset due to, for example, poor condition. Poor asset condition may increase the score (HE/km/year). The interim recommendation for carrying out the asset assessment is that the advice of operations personnel is used to define the maximum return period against which the defence provides protection. This is used to modify the score based on the predictive method, so that asset condition is taken into account in addition to the design standard.

A probabilistic method for asset assessment was developed , but was not recommended for implementation pending further studies on objective assessments of asset condition. The approach is described here as it includes simplified probabilistic assessment.

Three factors are proposed. Each is divided into 5 categories representing the severity of the factor:

1. Structural condition (eg good, average, bad, etc)
2. Beach or river bank/bed condition (eg accreting, volatile, rapid erosion,etc)
3. Overtopping condition. (eg overtops rarely with >50 year return period, or overtops often, greater than once a year)

The asset is classified using each of the above factors. A simple table is proposed which enables the probability of damage resulting from the design storm to be related to the assessment of the three factors mentioned above. For example, the conditional probability of an asset in 'moderate' structural condition suffering damage due to the condition of the structure is given as 0.1. Damage probabilities are combined using a simple scheme to calculate the conditional probability of flooding given the design storm. The performance of the asset is compared with the design performance, expressed as a percentage. Thus the simple classification system is used to convert the reported characteristics of the asset to an estimate of performance relative to the design performance.

The system was not recommended for implementation, awaiting methods to reduce the subjectivity of the condition assessments.

Pro-formas for data collection are included with the procedure: these include data collection sheets for flood banks and/or diversion channels, control structures or drainage pumping stations, and estuary and sea defences. Data sheets typically include information such as position, purpose, description, material, structure type, and dimensions. Other information recorded is the condition of structure and beach, overtopping frequency and potential failure modes.

Discussion of findings from other studies

a) 'Event trees' and 'fault trees' are constructed and presented, but are not applied as logic diagrams except for individual structures which are the subject of very detailed study.
b) 'Event trees' and 'fault trees' have almost always served as diagrammatic representations of failure modes, rather than as strict logical representations of failures.
c) Some studies have aimed at being very broad, covering failure modes, consequences and costs, while others have focused on particular damage mechanisms in detail. This has enabled more thorough probabilistic methods to be applied to a limited range of mechanisms. This demonstrates the difficulty of acheiving a satisfactory compromise between engineering and mathematical detail and rigour, and developing a procedure that can be widely applied across a range of structure types.
d) The Level of Service (now Standards of Service) method establishes a technique for relating land use to a recommended return period of flooding, but does not include flood area modelling or consideration of structural failure.

3.2 Risk assessment techniques

The risk assessment process can be divided into separate processes: hazard identification, listing of failure modes and failure mechanisms, representation of failure modes in the form of logic diagrams (eg fault trees), calculation of failure frequency, and prediction of consequences.

Using the definition of risk as the combination of the probability of an event and the consequences of that event, we can conveniently show risk levels on an exceedance curve as shown in Figure 3.4. For any consequence (for example, a given area of land flooded) the chart shows the probability, or expected annual frequency of exceedance of that consequence. Integration of this curve gives the total expected risk, expressed in terms of the expected flooded area per year.

The concept of risk illustrated in Figure 3.4 is fundamental to this project. There will inevitably be compromises, complications and approximations; we intend to adhere to the concept of *risk* as the *expected (ie average) annual consequence, comprising the sum of consequences from a variety of mechanisms.*

[This definition is consistent with the Middlesex University Flood Hazard Research Centre (FHRC) definition, used for financial appraisal and Cost Benefit Analysis. The present project will not include consideration of land values, and will be restricted to predicting areas of flood risk, but the consistency of approach will make later implementation of financial consequences more straightforward]

3.2.1 **Fault trees, event trees and event chains** (ie failure mechanism diagrams)

This section considers methods for combining the failure probabilities from individual mechanisms to give the failure probability for the structure as a whole.

There are several ways of presenting information about failure modes.

Fault tree analysis

A fault tree is, in its strictest definition, a graphical description of the logical interconnection between various component failures and events within a system. Fault trees are used widely for analysing the probabilistic behaviour of systems of linked components such as safety systems for industrial plants, and electronic circuitry. This section will firstly discuss use of fault trees to assess the reliability of system designs in industry, and will then discuss their possible application to risk analysis of flood defence schemes.

The fault tree is constructed from events and gates. Gates are logical operators used to combine events to give an event at a higher level. Gates are built from Boolean operators AND, OR and NOT. The highest level of event is known as the TOP event. At the lowest level are primary events, normally component failures.

The main purpose of constructing a fault tree is to enable the identification of minimal cut sets, and thereafter to enable the probability of the TOP event occurring. A minimal cut set is a group of minimum component failures and events which are necessary to lead to the TOP event. Each minimal cut set has a probability of occurrence, depending on the probabilities of failure of individual components, and on the way these components are linked to form the system.

Software is available for assembling and analysing fault trees. This automates the process of deriving the minimal cut sets and carrying out the algebra to arrive at a probability for the top event. It is possible to put uncertainties on the basic component failure possibilities. (eg one piece of software on which we have technical information enables the user to specify a Normal variation, and then carries out Monte Carlo sampling to obtain a measure of uncertainty in the TOP event failure probability) It is important to note that fault tree analysis is essentially based on events which occur or do not occur, such as a valve which fails or operates successfully.

Many attempts have been made to construct fault trees for flood defence and coastal structures. For example, Figure 3.3 is taken from CUR/TAW (1990) and shows a fault tree for a flood defence structure. (The structure of this fault tree was not used for analysis). The strength of this type of illustration is that is an efficient way to communicate to the engineer the possible modes of failure for a structure, together with causes of that failure. However, there are problems in applying this type of fault tree to calculate failure probabilities. Three particular difficulties have become clear:

- There is a difficulty in the definitions of the basic events, at the ends of the branches. Take the case of breaching due to erosion of inner slope, caused by excessive wave run-up. The basic events causing this are 'water level too high' and ' waves too high'. It is not possible to define 'too high' without reference to events higher up the tree. In general, each branch of the fault tree would require considerable effort to define the basic events that would lead to the TOP event.
- In order to be able to combine probabilities of different events to obtain the probability of the TOP event requires an assumption to be made about the exclusivity of events. For example, it would be easy to assume that wave overtopping, overflowing and breaching are exclusive events, which would enable the combined probability to be calculated by simply adding the three individual probabilities. However, it is possible that overtopping and breaching can occur together, but it is uncertain how the individual probabilities should be combined. This difficulty is compounded by the complex nature of structural failures of sea defences. One failure mode is often crucially influenced by another, for example, scour may affect the likelihood of failure of the front armour layer. This behaviour does not fit into the fault tree approach.
- Fault trees are essentially binary in character: a componenet either fails or does not fail. On the other hand, components of sea defences undergo various degrees of damage in response to storms of various magnitudes.

It is difficult to construct a rigorous system based on fault trees to enable risk assessment of flood defence schemes.

Event trees and event chains

Event trees can be used to describe the possible changes to a flood defence resulting from a given initiating event, or storm. The event tree should represent all possible relevant changes to the flood defence in response to a particular type of initiating event. Event trees can also be used to represent the response of organisational systems, such as whether flood warnings have or have not been broadcast. In the context of flood defence works, the initiating event is likely to be a storm, meaning an extreme set of wave heights and water levels. Each branch of the event tree results in two or more possible outcomes. The probabilities of each of these outcomes depends on the way in which the flood defence system responds to the initiating event. This may be determined by fault tree analysis. Using this approach, therefore, the event tree can be seen as the progression of events leading from the particular storm to a certain outcome. The probability of a particular outcome depends on the probabilities of all events leading to that outcome. The probability of each individual event depends on the probability of failure of the system which determines whether the event occurs, together with the frequency of the initiating storm.

A drawback of conventional event trees is that they do not formally allow joining up of different paths. Furthermore, the rigidity imposed by a tree structure limits the ability of the tree to represent the behaviour of a defence. In reality, several events may occur simultaneously and independently, whereas the tree structure imposes a fixed order to many events. A further difficulty with event trees and fault trees is their binary nature; the decision on which branch to follow is based on a yes/no answer. For example, there may be a question such as 'Does overtopping occur?'. But the *amount* of overtopping may be important. Furthermore, the overtopping discharge which leads to failure depends on complex factors such as whether flow slides on the rear face have occurred. These issues are not easily included in an event tree approach.

An event tree is an attempt to represent all sequences of events from an initiating event. In view of the difficulties in constructing rigorous event trees, and the complexities in using these for numerical analysis, a simplified approach is proposed. It is possible to identify chains of events which can be analysed independently. These correspond to the failure modes listed in Section 2.5.2. Each event chain can be thought of as a route through a complete event tree. Chapter 4 presents a simplified example of the use of event chains, or accident sequences, to calculate the failure probability from the probabilities of individual failure mechanisms.

Consequence - cause diagrams

These are an alternative type of logic diagram which attempt to account for the recursive nature of failures of sea defences. An example of a simplified consequence - cause diagram is shown in Figure 3.5.

3.2.2 Interaction between failure modes

It is important to define clearly the meaning of the events which comprise failure modes. For example, in the failure mode of overtopping followed by erosion of the crest, what is meant by 'erosion of crest'? If only a limited degree of erosion takes place, this will not necessarily lead to further damage to the embankment. In order for the failure modes to be useful in probabilistic assessment, the definition of each event needs to be precisely defined as follows. We will use the first failure mode, crest erosion due to overtopping, as an example.

Assume that a particular initiating event acts on the structure. This is assumed to be a particular storm, defined, typically, by a combination of wave condition and water level. Analysis allows the frequency of the storm (in terms of annual probability) to be estimated. All event probabilities can now be expressed as *conditional* on the occurrence of this storm. Given these storm conditions, we can calculate an overtopping discharge, or, more usefully, we can take account of probabilistic variations to calculate a frequency distribution of overtopping discharge. Each overtopping discharge (more precisely, each increment of overtopping discharge) has an associated probability of occurrence, given the initiating storm. The event 'overtopping' in the failure mechanism can therefore be expanded to mean '*The conditional probability distribution of overtopping discharge given the occurrence of a particular initiating storm*'.

Crest erosion depends on the overtopping discharge, and each discharge causes a degree of damage, with associated variability. For each overtopping discharge, there is therefore a conditional probability distribution of damage. If each of these conditional probability distributions are multiplied by the conditional probability of discharge given the initiating storm, this gives a set of probability distributions of damage which can be combined to give a conditional probability of damage given the initiating event. The term 'crest erosion' can therefore be expanded to mean '*the conditional probability distribution of damage to the crest given the occurrence of the initiating event*'.

This takes into account the variability in the overtopping discharge and in the damage function, and enables the assessment to take account of the fact that the event tree will not necessarily terminate at the final failure of a breach. It is quite possible to terminate at a particular event. For example, we could set a damage threshold below which lowering of the crest will not take place. Only events leading to damage above this threshold will contribute to the probability of initial breaching.

Two or more failure modes may occur simultaneously, and there may be interaction between different modes. While armour on the front face of a sea wall is being damaged by waves, erosion to the toe may be taking place which is increasing the likelihood of slip failure to the seaward face, leading to increased susceptibility to further armour damage. This is an example of recursive behaviour. It may be difficult to take this into account in a risk analysis procedure. The approach of previous studies, giving upper and lower bounds by assuming complete dependence and complete independence, is reasonable for the initial stages of this study.

The next stages of the study will address the issue of recursiveness and interaction between different failure modes. We may find it necessary to recommend a 'weakest link' approach, in which the most likely failure mode is identified for each class of structure and environment. Alternatively, we may develop simplified failure mechanisms which acknowledge the recursiveness.

3.2.3 Consequence modelling

The consequences of failure must be predicted in order to carry out a complete risk assessment. For the purposes of this project, the consequence will be expressed primarily in terms of the area and location of flooding. The depth and duration of flooding may also be significant, and these aspects will be considered. Assessment of the consequences will be achieved by the flood area mapping and modelling. the link between the environmental loading and structure failure probability and the flood area modelling will be in terms of the discharge crossing the defence line. It is expected that it will be necessary to transfer a set of discharge values from the failure analysis component to the flood area modelling component. Each discharge will have an associated probability of occurrence. This will be, effectively, a discretisation of a discharge exceedance curve.

The findings to date of the flood area modelling part of the project are given in Section 5.

3.3 <u>Probabilistic techniques</u>

Probabilistic design means that lack of knowledge about the true, realised, values of significant parameters is taken into account by assigning suitable probability distributions to the parameters. The possibility of large or small values of parameters is accepted, and the probability of extreme values is given by the form of the probability distributions. These distributions may apply to properties of the structure, to the environmental loading, or to the parameters ('constants') of the design equations used to characterise a particular response of the structure. These distributions are fed into the relevant design equation, which has been re-arranged to be in the form of a failure function. The result is a probability distribution of the failure function. Design values for the structure, such as crest height or armour size, can then be chosen and adjusted to produce a target failure probability. This contrasts with the deterministic approach where a single set of expected values of all independent parameters is chosen, the design parameter is calculated and a safety factor is applied. Using this latter method, the probability of failure cannot be estimated, and the sensitivity to changes in design cannot be quantified in terms of risk.

Probabilistic risk assessment implies that the risk inherent in the system is not known with certainty. Uncertainty in the loadings and strength of the structure, and uncertainty in the response functions used to design or assess the structure imply that the behaviour of the structure cannot be predicted with certainty. Furthermore, uncertainty in the behaviour of a failed structure, in terms of the growth of breaches, and uncertainty in ground level data mean that it is not possible to predict the exact flooded area. These factors lead to a range of possible outcomes, each with an associated probability of exceedance. Integration of all possible outcomes, weighted by their probability gives the expected risk.

3.3.1 Sources of uncertainty

This section summarises types of uncertainty. Examples of each type of uncertainty are discussed, but are not intended to be exhaustive. It should be noted that 'uncertainty' does not imply that the issue cannot be quantified: probabilistic methods are designed to do this. At this early stage in the project, it is important to acknowledge uncertainty wherever it may exist.

- Uncertainty in identification of all significant hazards and failure modes. If significant hazards are omitted, then the risk assessment procedure will be flawed. Similarly, there may be considerable uncertainty in the sequence of events leading to failure: this is particularly true of flood defences where detailed observation of failures is difficult or impractical.

- Uncertainty about the development of damage leading to failure. Design equations and models of structure response are usually intended to represent a single mechanism only, such as damage to a rubble armour layer. This is the initial damage event in a sequence of events which must be realised to cause failure. These subsequent events may be difficult to identify qualitatively, and impossible to represent quantitatively. In some cases, the initial damage event in a sequence may be the strongest link: It may be valid (and necessary) to assume that once the armour layer fails, breaching is inevitable. There remains the difficulty of predicting the dimensions of the breach.

- Uncertainty in loading parameters due to inadequate data. Accurate determination of extreme value wave heights and water levels require long-term data which may not be available. Measurement of water levels and, in particular, wave heights, is subject to error which affects assessment of extreme values.
- Uncertainty due to future change in climate. Future loading may differ from past or present loading, due to long-term climate change, but the changes are not known with any certainty.
- Uncertainty due to stochastic (ie probabilistic) nature of loading. Even if the statistical properties of the wave and water level climate could be determined accurately, there remains the possibility that events in the future will not conform, due to the unpredictability of individual events.
- Uncertainty in realised values of structure parameters. Many structural properties may not be known precisely, due to variability in time or in space. For example, the geotechnical properties of an earth embankment will vary along its length, and a functional representation of this variation cannot be determined without extensive measurements. Furthermore, measurements of soil parameters may themselves give values which are not representative of the realised values, due to flaws in the sampling, measurement or analysis technique.
- Uncertainty in the form and constants of the response equations. The behaviour of most structures is represented by design equations or modelling of some kind. Design equations are generally based on experiments carried out at laboratory scale. Model results are not usually exactly repeatable, giving scatter in results and uncertainty in fitting parameters. Assumptions are made about the form of the equations, and these may be incorrect. Model experiments are normally designed to study the response to variations in a small number of variables, others being kept constant. In the absence of site-specific model testing, prototype conditions will usually be different to model conditions on which the design is based.
- Uncertainty in flood characteristics. Errors in ground level data, approximations in flood modelling or mapping techniques and localised features such as ditches and walls within the flooded area mean that the precise flood area which will be realised cannot be determined with certainty. The location of flood flows through the defence will affect the flooded area, and this also will not be known.

3.3.2 Probabilistic calculation methods

Techniques are available for carrying out probabilistic design of specific damage mechanisms: Note that a failure mode may comprise several damage mechanisms, so a complete probability assessment would entail several sets of analysis.

Probabilistic methods are often identified as Level 1, 2 and 3 approaches. These are not described here, except to say that Level 1 methods rely on partial safety factors which have been developed for specific design equations, and are functions of the required failure probability. Level 2 methods place limitations on the form of the distribution functions but can be applied more readily. They are ideally suited to determining the importance of input parameters to the probability of failure: they enable parameters to be ranked according to the sensitivity of the failure probability to uncertainty in each input parameter. Level 3 methods rely on repeated sampling of parameter values from their distribution functions to build up an approximation to the failure function probability distribution.

3.4 Appropriate techniques for probabilistic risk assessment of sea and tidal flood defences

The above sections illustrate the potential complexity of probabilistic risk assessment. This implies that we should aim for a flexible approach, where we establish a framework which is easy to understand and where assessments can be done at a simple level, using subjective indicators and engineering judgement where appropriate. The amount of analytical effort can be increased if resources are available, and these more sophisticated techniques (probabilistic methods) can be slotted into the basic framework. An alternative approach is to develop a separate initial screening procedure based on simple indicators, followed by more sophisticated risk analysis of priority defences.

4 SIMPLIFIED PROBABILITY MODELLING

This section gives an example of the type of simple modular approach that may be appropriate for carrying out risk assessment for numerous sea and tidal defences.

The method is based on the theory of total probability: this states that, given a set of mutually exclusive exhaustive events B_1, B_2, ...B_n, then the probability of another event A can be expressed as:

$$P[A] = P[A \cap B_1] + P[A \cap B_2] + ... + P[A \cap B_n]$$

This is equal to the following sum:

$$P[A] = \sum_{i=1}^{n} P[A|B_i]P[B_i]$$

The usefulness of this approach is that the total probability of a particular consequence, represented by event A, can be calculated from the individual conditional probabilities of that consequence, given the occurrence of each of a number of causes, B_i.

For example, consider the event A to be defined as an overtopping discharge between 0.1 and 1.0l/s/m. Each element B_i is the probability of a specific wave height condition, specified in terms of a wave height within a given range. Each wave height condition has a probability of occurrence which can be calculated from existing data, modelling etc. For each wave height condition i, there is a certain probability that the overtopping discharge will fall within the range defined for event A. This is the conditional probability $P[A|B_i]$. This is multiplied by the probability of the wave height condition to give the probability that event A *and* B_i occur. Summation of these probabilities over all wave height conditions gives the total probability of event A.

We can extend this to consider a number of different overtopping discharges. Solution can now best be implemented using matrix arithmetic. Use the following definitions:

[IE] column matrix, n rows by 1 column, with each element containing the frequency of occurrence (yr^{-1}) of a particular storm. The storm is the Initiating Event

[Q|IE] matrix containing conditional probabilities of a number of overtopping discharges for given storm events

For example, assume that overtopping depends only on wave height. Previous analysis has yielded the following probabilities of wave height falling within defined intervals:

Hs (m)	P (yr^{-1})
1-2	0.1
2-3	0.01
>3	0.001

[It has been suggested that the timescale should be per tide rather than per year, ie P ($tide^{-1}$) as there will be many events (storms) per year. The merits of this will be considered during the next phase]

$$\text{thus } [IE] = \begin{matrix} 0.1 \\ 0.01 \\ 0.001 \end{matrix}$$

Note that the total probability is less than 1.0: we have not included the lowest wave heights in our analysis domain, as these are assumed to cause no significant overtopping.

For each wave height, we can carry out probabilistic analysis to obtain a probability distribution for overtopping discharge.

Select a number of ranges of overtopping discharge eg

Q (l/s)
<1
1-10
>10

For each combination of wave height and discharge, use the pdf of discharge for that wave height to obtain a conditional probability of the particular discharge, given the wave height. Thus assemble matrix [Q|IE].

$$\text{eg } [Q|IE] = \begin{matrix} 0.8 & 0.4 & 0.2 \\ 0.2 & 0.3 & 0.4 \\ 0.0 & 0.3 & 0.4 \end{matrix}$$

Note that the columns add to 1.0: For each wave height condition, it is assumed that the selected discharge events are exclusive and exhaustive.

Multiplying [Q|IE][IE] gives the total probability of occurrence of each discharge:

$$[Q|IE][IE] = \begin{matrix} 0.0842 \\ 0.0234 \\ 0.0034 \end{matrix}$$

(At this stage, before damage is considered, these probabilities could be converted to an overtopping exceedance relationship, which could be integrated to give an estimate of the expected annual overtopping volume)

The overtopping may be the start of a more complex event chain: storm - overtopping - crest damage - breach. We can extend the above calculations to give the probability of a breach. We need to assemble the following probability matrices:

[S|Q] matrix containing probability of a number of degrees of damage, given each overtopping discharge

[B|S] matrix containing probabilities of no breach and breach formation given each degree of damage

These matrices are assembled from knowledge of the structure response mechanism (eg damage in response to overtopping) or using engineering judgement, and taking account of probabilistic uncertainty. The following calculation gives the probability of breach and no breach:

$$[B|S][S|Q][Q|E][E] = [B]$$

where [B] is a column matrix containing the probability of no breach and breach.

Having selected a number of damage values, the conditional probability of each damage value given the occurrence of each discharge gives the probability matrix [S|Q]. For example for damage levels S = 2, 4, 8 ,16:

$$[S|Q] = \begin{matrix} 0.6 & 0.4 & 0.2 \\ 0.3 & 0.4 & 0.5 \\ 0.1 & 0.2 & 0.3 \\ 0.0 & 0.0 & 0.1 \end{matrix}$$

For each damage level, assign probabilities that the structure will not breach, and a probability that the structure will breach due to erosion of the crest, to give the matrix [B|S]:

$$[B|S] = \begin{matrix} 1.0 & 0.9 & 0.7 & 0.5 \\ 0.0 & 0.1 & 0.3 & 0.5 \end{matrix}$$

Carry out matrix multiplication to obtain the probability of no breach / breach [B]. In this example,

$$[B] = \begin{matrix} 0.103 \\ 0.008 \end{matrix}$$

Thus, in this simplified example, the probability of a breach due to the specific event chain analysed is $0.008 yr^{-1}$.

This procedure can be repeated for the other event chains. This will yield several probabilities of breaching which can be combined to estimate the overall probability of breaching for the defence.

Breaches due to different failure modes may arise from the same initiating events: the events will not, in general, be exclusive, and a conservative (upper bound) estimate of the probability of breach will be obtained by summing the probabilities from the individual failure modes. A lower bound estimate will be obtained by taking the maximum probability given by all of the failure modes. At a later stage, this issue may be addressed in more detail by considering representation of all failure modes on a single event tree, rather than as separate event chains. Such an event tree is likely to be complicated and difficult to construct rigorously.

5. FLOOD AREA MODELLING

5:1 Introduction

The main objectives of the flood area modelling component are to develop methodologies to assess the areas, extent and severity of flooding over a relevant range of probability levels ie. not the modelling of flooding, but the mapping of flood risk. The work will link to Tasks 2 and 3 through contoured flood risk surfaces relating a number of stages on discharge frequency diagrams to flood ares (via the discharge volumes). Spatial analysis of the flood maps will provide the information to produce flood area/frequency plots. The procedure is defined in more detail later.

5.2 Identification of the main variables

The first stage of the study identified the key variables affecting modelling of flood areas. The driving component is the presence of a water level, adjacent to a defence, above the lowest level of the land. The area at risk can initially be defined as any point into which the water can flow. flood water can cross a defence by overtopping or overflow with or without damage to the structure. The volume will depend on the nature of the failure and the prevailing environmental conditions.

The space occupied by the water is determined by three major factors:

- the relief of the land;
- loss of water from the flood area (eg seepage);
- hydraulic resistance: surface drag, and form drag from obstacles.

The initial risk mapping will be based on steady equilibrium conditions. In these circumstances, drag factors are not relevant and would only significantly modify the flood outline in cases of rapid and severe failures. The study will assess its influence, possibly by dynamic modelling of overland flow from breaches. However, modelling of the dynamics of flooding is unlikely to be a general recommendation of the final methodology as it is likely to be too time-consuming and data-intensive for widespread use.

Of the remaining factors relief is dominant, with the the relative importance of the second group, seepage, increasing as the mean gradient within the flood area falls. The second stage of Task 4 will examine aspects of modelling terrain-dominated floods. The third stage will investigate the nature of factors modifying the flood outline.

5.3 Establishing appropriate terrain models

Terrain models are fundamental to the task of flood area modelling. The study will develop methodologies for producing digital terrain models (DTMs) with an appropriate level of vertical accuracy and density of data points. The appropriateness is governed by five main considerations:

(a) relief (the minimum level of acceptable accuracy is proportional to the mean gradient within the flood area)

(b) balance between vertical accuracy and density of data points (eg 4 points to an rms error of 1cm will produce a less reliable DTM than 20 points to an rms error of 10cm)

(c) magnitude of losses (there is little point in collecting DTM data to an accuracy much greater than the influence of these factors)

(d) cost of data capture

(e) application of the risk mapping (the level of expense may be influenced by the investment within the study area)

Points (a) - (d) are identifiable factors which will be addressed by the study; (e) is dependent on the application of the methodology. As the relative importance of (a) - (c) reflects the nature of the study site, three cases will be considered:

- steep relief;
- moderate relief with mixed land cover;
- low relief with open land cover.

5.3.1 Methodology for producing appropriate DTM data

This requires determination of a suitable level of vertical accuracy and density for data points collected at an appropriate cost. Possible data sources for the UK include:

- OD enhanced DTM data (spot heights and contours)
- Air photo survey with interpretation by:

  (a) analytical plotter
  (b) automated analytical plotter

  with ground control provided by:

  (a) Global Positioning System (GPS)
  (b) conventional survey
  (c) OS ground control

- Ground survey:

  (a) GPS
  (b) conventional survey

- Satellite imagery

OS DTM data

OS provide 1:50,000 and 1:10,000 DTM data nationally; the 1:10,000 data comprise contours at 5m intervals (the first at 5m ODN), MLW and MHW. They have a rms error of between 1 and 1.5m. The 1:10,000 data can be enhanced with spot heights from the 1:1,250 and

1:2,500 maps with a rms error of less than 0.01m. All sets are available in 5km tiles. However the spot heights are a special service product and the density of coverage varies considerably. No data is provided below MHW.

While the data is readily available and the vertical accuracy of spot heights is suitable, the great range in data density may preclude its use for many locations. Halcrow currently hold a sample data set which has been used to investigate flood area mapping and also in evaluating software for the purposes of this study.

Coverage of OS data is shown in Figure 5.1. This shows good coverage within Grimsby, but extremely low densities in the industrial and rural areas to the north-west and south-east. Other problems may arise, reflecting the tendency of the spot heights to be located along roads. Because of the contours' rms error it is unlikely that they would be suitable for mapping flood risk, although they may provide a good basis for screening sites for further study.

Air photo survey

This is a widely used techniques for collecting DTM data for detailed modelling as it can provide a good balance between an appropriate density of data and positional accuracy. It has been used by Halcrow for engineering work and is currently used by some regions of the NRA for beach level monitoring.

Data density does not vary spatially and collection is not limited to areas where access can be arranged. Only limited survey work is required to position and correct the data points obtained. The source information is available for re-examination and, where available, provides one of the only reliable sources of historic mapping for the coast.

Its main disadvantage is that suitable data are not generally available off the shelf necessitating commissioning a survey and/or analysis of the photographs (this may include establishing ground control). The issue of GPS or conventional survey ground control are discussed under their respective headings. OS ground control points are of limited spatial availability.

GPS survey

For the purposes of the study GPS offers the most cost effective method of capturing surveyed data. Recent work has shown the resolution provided by GPS to match that of conventional survey at considerably lower cost in terms of time and resources.

A range of techniques exist. Full descriptions are not given here as thay have already been documented in detail.

While the potential accuracy is greater than that readily derived from air photo interpretation, both GPS and conventional survey techniques suffer from the need to gain access to the survey site. This is a major consideration in in urban areas. Both also have greater resource requirements. It should also be noted that some GPS techniques require lines of sight to be maintained with a number of satellites to optimise collection times - this may be a problem in areas with tall structures or other obstructions.

Conventional survey

The advantages are similar to those of GPS; however, there are numerous disadvantages which make conventional survey less attractive. The major consideration is the greater investment in time and resources required. Work by the Thames Water Land Survey Unit comparing identical surveys using conventional and pseudo static GPS indicated a time saving of around 75% and reduced the survey team by one (although an extra vehicle was needed).

Examination of the effects of different data sources

With the exception of the OS DTM data, the above sources are capable of yielding positional information over a range of accuracies and densities depending on the amount of money invested. Determination of an appropriate balance will be based on comparison of flood areas derived from DTMs which have been constructed from data sets with a range of z tolerances. A series of DTMs can be produced for each data set by 'wobbling' the height value. This involves adjusting each data point within the DTM by a random value within the range of half its rms error. The flood areas derived for each data source can then be compared using overlay techniques.

The effect of data density will be investigated by comparing flood areas derived from a data set with varying degrees of thinning. For the OS data this will involve comparison of urban and rural areas to examine the effect of the lower density outside of built-up areas.

For these studies to yield the greatest benefit the modelled flood areas should be evaluated against a well-recorded flood event, ideally with stereo pair coverage.

5.4 Flood risk mapping methodology

The basic principle underlying the risk mapping is the conversion of discharge exceedance probabilities into risk areas. This involves generating flood vulnerability maps for discharges read from the probability/discharge plot for a range of probabilities. The various flood outlines can then be superimposed to produce a contoured mapping of flood risk.

This basic method will be modified to represent the two major sources of uncertainty: discharge and conversion of discharge into a flood area. Each discharge on the probability/discharge plot has an associated error fringe. Flood areas can be generated for the discharges at each end of the error band giving an horizontal representation of the uncertainty on the resulting risk area maps. The full risk map can be produced by overlaying the risk areas for a range of probabilities and discharges.

Flood areas can be generated for the same water level on two DTMs derived from the same data adjusted to either extreme of the confidence range of their positional tolerance. Areas for a range of water levels can then be overlain to give a mapping illustrating to what degree uncertainties in the source data affect the risk maps. The method can be used to incorporate fringes relating to other sources of uncertainty, such as water loss. The risk maps can then be viewed in the context of uncertainty relating to the flood area derivation technique.

5.4.1 Evaluation of MGGA as a tool for modelling flood risk

MGGA is a raster GIS (ie. the data is stored in grid cells) with a scientific applications language, GOAL. The software has been evaluated using an OS DTM data set (Figure 5.2), to establish its suitability for flood risk mapping. Contouring of the OS data shows quite clearly the problems caused by the low density of information and reliance on roads.

An initial model has been developed to translate flood volumes into areas based on the study site terrain. Early results are encouraging (Figure 5.3) and have highlighted some of the basic problems that need to be addressed in order to reproduce realistic flood risk outlines.

The model considers a level flood surface and does not allow for the influence of local reservoirs/cells on the area at risk.

6. DISCUSSION AND CONCLUSIONS

This report has reviewed the progress made during Phase 1 of the study. Three interrelated themes have been addressed, although not at equal levels of intensity. The report is therefore not intended to give a balanced view. It is intended more to record the findings, discuss the technical arguments and the conclusions drawn from them and to identify aspects of the work which require discussion with the NRA.

Structure classification and failure modes

The scope of the study, in terms of the range of structures which it is planned to include, has been presented. Various method of classification have been reviewed. These generally enable identification of the main elements, but their interrelations and sub-classes need to be reviewed. It is recommended that the classification system adopted should be based on that used for the National Sea Defence Survey, but additional information will need to be included. This will include more detail, where available, such as the type and size of rock armour, and will also include additional information to enable initial estimate of the vulnerability of the structural elements to damage.

Mechanisms for damage have been given for the main structure types. A failure mode is defined as an event or sequence of events leading to failure. There are many possible failure modes, sometimes subtely different. The individual mechanisms which make up a sequence of events leading to failure may be difficult or impossible to calculate. Reports of failure may include description of the main mechanism thought to be responsible, but there is generally very little evidence to suggest the details of how different damage types interract to give failure. Existing methods for failure analysis are based on a single cause and single effect. The report identifies broad categories of failure mode, and for each category gives the main failure modes as sequences of events.

Several events have been selected for possible use in future case studies.

Probabilistic risk assessment techniques

This part of the project has not yet attempted to produce a methodololgy for risk assessment. Rather, we have concentrated on establishing the fundamentals on which a methodology should be based.

A proposed definition of risk is presented, as 'the combination of the probabilty of an event and its consequences'. For this project, it is proposed that the consequences are expressed in terms of the area flooded. The report discusses the concept of integrating over a range of possible flood events to arrive at a risk value which gives the expected (ie average) area flooded per year. In fact this is likely to entail some approximations, such as considering a limited number of events, rather than a continuum. The project team has also considered probabilistic effects, and how uncertainty can be taken into account as part of an overall procedure.

One of the early findings in the work on methodologies was that approaches using fault trees or event trees may be suitable for simple binary operations that fail or do not fail. These are entirely insufficient on their own for the present problem. The project has developed initial probabilistic techniques based on event chains and a total probability approach. There remain important questions about correlation and interraction between failure modes and these will be addressed in the next Phase.

The project team has discussed and reviewed the scope and overall form of the methodolgy. Two approaches are under consideration:

a) A broad screening procedure to enable rapid identification of defences for which a risk assessment is required, followed by a risk assessment methodology ro be carried out on the selected defences. The screening tests would be based on responses to simple questions about the defence and its functions.

b) A single methodology devised to be very flexible. The method could be applied using minimal resources and data, but would be devised in such a way to enable additional data and analysis to be applied where appropriate, ie where risk levels dictate that more refined assessment is required.

At this preliminary stage, it seems likely that a 2-stage process will be required, particularly in order to reduce the number of elements and failure modes to be addressed, and to overcome the limitations in the information available on the structural responses. On the other hand, there may be benefits in a single unified approach capable of refinement where required. [NRA comments on this would be welcomed]

We have established the form of linkage between the failure analysis and the consequence modelling. It is proposed that the failure analysis component of the methodology should produce results expressing the probability distribution of discharge across a defence. This may be approximated by a number of discrete discharges, each with an associated exceedance probability. In some cases, the flood water level will be assumed to equal the sea wate level, in which case the discharge value is not important. Future work will include analysis to identify the importance of estimating the discharge across a defence, selection of appropriate methods to estimate this discharge, and definition of a defence.

Flood area modelling

This component of the project has considered possible sources of data, and has discussed benefits and drawbacks of different types of data for establishing suitable representations of the ground surface. This has drawn attention to the variable density of OS spot height data, and has led us to examine alternative sources such as GPS and air photo data. The next Phase will seek to establish sources of data which are available from the NRA, such as coverage of air photo surveys and recorded flood outlines, and to acquire appropriate data. These will be important for calibrating and validating flood outlines produced by the flood area modelling.

A trial model of a flood area has been established, assuming steady equilibrium conditions (ie a horizontal water surface). A given quantity of flood water is assumed to fill the flood area up to a horizontal level, which is calculated iteratively. This simple representation does not take account of connectivity between flood cells, or features which affect the flood area such as embankments and other water retaining features on the flood plain. The next Phase will examine ways to overcome this limitation. This will also require a method for defining the location of the source of flood water along the defence. The next Phase will also examine the importance of other factors such as losses and hydraulic resistance.

Future work will be based on example data sets developed in conjunction with the failure analysis methodology, and the next phase will also therefore explore the linkage between the failure analysis and the flood area modelling

BIBLIOGRAPHY

The items below have been collected for the use of the project team, in connection with this project and with the ongoing DoE project on probabilistic methods for the design of coastal structures. This is not intended to be a definitive collection of references: the list will grow as the project proceeds.

Apostolakis G E 1992. Reliability engineering & system safety. Elsevier Applied Science, Vol. 39, NO. 2, 1993.

Ashkar F & Rousselle J 1981. Design discharge as a random variable: A risk study. Water Resources Research, Vol. 17, NO. 3, June, pages 577-591.

Bakker W T & Vrijling J K 1980. Probabilistic design of sea defences. Rijkwaterstaat report WWk-80.V012. June

Benjamin J R & Cornell C A 1970. Probability, statistics and decision for civil engineers. McGraw-Hill Book Company.

Brennan W A & Stickland I W.1981. The probabilistic approach to the design of maritime structures. Transactions of the Institutions of Engineers of Ireland, Vol 105,

Bruun P & Gunbak A R 1978. Stability of sloping structures in relation to surf similarity parameter risk criteria in design. Coastal Engineering, Vol. 1, NO. 4, March, pages 287-322.

Bruun P & Manohar M 1963. Coast protection for Florida: development and design. Bulletin series No. 113, Florida Engineering and Industrial Experiment Station, College of Engineering, University of Florida, August

Burcharth H F 1992. Introduction of partial coefficients in the design of rubble mound breakwaters. Proceedings of 1991 Conferencee of the Intstitution of Civil Engineers, Coastal Structures and Breakwaters. Thomas Telford, London,

Burcharth H F 1990. Introduction of a partial coefficient system for the design of rubble mound breakwaters. PIANC Working Group 12, Sub-Group F, draft report. November

CIAD 1985. Computer aided evaluation of the reliability of a breakwater design. Final report, CIAD project group, July

CIRIA 1986. Maintenance of coastal revetments. CIRIA Techniocal Note 124

CIRIA 1986. Sea Walls: Survey of performance and design practice. CIRIA Technical Note 125

CIRIA/CUR 1991. Manual on the use of rock in coastal and shoreline engineering. London/Gouda, CIRIA Special Publication 83, CUR Report 154

Clark P B 1987. A probablistic risk assessment of an earth embankment dam.

Coles S G and Tawn J A 1991. Statistical methods for multivariate extremes: an application to structural design. Read before The Royal Statistical Society, 10 March

Cooling L F & Marsland A 1953. Soil mechanics of failures in the sea defence banks of Essex ank Kent. Conference on the North Sea Floods of 31 January - 1st February, 1953. Institution of Civil Engineers, December , pp 58-73

Cullen N 1990. Risk assessment of earth dam reservoirs. Final report to the department of the environment, January. Commercial - in - confidence

CUR/TAW 1990, Centre for civil engineering research and codes / technical advisory committee on water defences. Probabilistic design of flood defences. Report 141. June

Dawdy R & Lettenmaier P 1987. Initiative for risk-based flood design. Journal of Hydraulic Engineering, Vol. 113, NO. 8, August, pages 1041-1051.

Dover A R & Bea R G 1979. Application of reliability methods to the design of coastal structures. Coastal Structures '79 Vol 2. ASCE,

Eberhardt A 1979. The game of engineering. Water Power & Dam Construction, December, pages 31-34.

Franco L et al 1986. Evaluation of risk applied to the designed breakwater at Punta Riso at Brindisi, Italy. Coastal Engineering Vol. 10, NO. 2, July, pages 169-191.

Freudenthal A M 1968-70. Probablistic evaluation of design criteria for maritime structures. PIANC Bulletin No. 2, pages 17-31

Fujita Y & Tamura T 1987. Enlargement of breaches in flood levees on alluvial flood plains. natural Disaster Science, Volume 9, No 1, November 1, pp. 37-60

Fukuda N, Uno & Irie I 1974. Field observations of wave overtopping of a wave absorbing revetment. Coastal Engineering in Japan. Vol. 17,

Goda Y 1971. Expected rate of irregular wave overtopping of seawalls. Coastal Engineering in japan. Vol. 14;

Hague R C 1992. UK South Coast shingle study: joint probability assessment. HR Wallingford report SR 315. November

Halcrow, 1992. Identification of UK coastal flood areas for ABI - pilot study. (Confidential to project team)

Hallam M G 1977, Heaf N I & Wootos I R. Rationalisation of safety and sericeability factors in structural codes. CIRIA Report 63, London, . **To be obtained**

Helton J C & Breeding R J 1992. Calculation of reactor accident safety goals. Reliability Engineering and System Safety. Vol 39 No 2 pp129-158.

Howell G L 1985. Proc Workshop on Measurement and analysis of structural response in concrete armor units. Thomas Telford Ltd., May.

HR Wallingford 1992. National Rivers Authority, Thames Region: Satndards Of Service

Reach Specifiction Methodology. Summary Report EX 2652 September

HR Wallingford, 1993. Probabilistic assessment of breakwater safety (Interim report) IT 392. In preparation.

HSE 1989. Risk criteria for land-use planning in the vicinity of major industrial hazards. Health and Safety Executive, HMSO,

Karlsson P O & Haimes Y Y 1988. Risk-based analysis of extreme events. Water Resources Research, Vol. 24, NO. 1, January, pages 9-20.

Konecny F & Nachtnebel H P 1985. Extreme value processes and the evaluation of risk in flood analysis. Applied Mathematics Modelling, Vol. 9, February, pages 11-15.

Lee H-L & Mays L W 1983. Improved risk and reliability model for hydraulic structures. Water Resources Research, Vol 19, No 6, pp. 1415-1422, December.

Lumbers J P & Cook, S C 1993. Risk management in the water industry. The Institution of Water and Environmental Management Yearbook.

Marsland A 1966. The design and construction of earthen floodbanks. River Engineering and Water Conservation Works, Ed Thorn R B. Butterworth,

McCartney B L 1976. Survey of coastal revetment types. US Armay Corps of Engineers Coastal Engineering Research Centre Miscellaneous Report No. 76-7. May

Midgley D C 1967. A flood risk analysis for choosing the height of a coffer dam. Reprint Commission Internationale des Grands Barrages Istanbul, pages 169-179.

Mol A, Ligteringen H, & Paape A 1983. Risk analysis in breakwater design. Delft Hudraulics Laboratory Publ. NO. 301, May.

National Rivers Authority 1990. Flood defence levels of service, Stage 2. Project 262. December

National Rivers Authority 1993. Revetment systems and materials. R&D Note 116.

Ostergaard C & Schellin T E 1980. Use of risk analysis to evaluate the seaworthiness of offshore structures. Applied Ocean Research, Vol. 2, NO. 3, pages 113-117.

Owen M W 1983. The hydraulic design of sea-wall profiles. HR Wallingford.

Owen M W 1980. Design of sea walls allowing for wave overtopping. Report No. EX 924, Hydraulics Research Station Wallingford, June

Owen M W 1982. Overtopping of sea defences. HR Wallingford.

PIANC 1987. Risk consideration when determining bank protection requirements. Supplement to PIANC Bulletin NO. 58.

PIANC. Breakwater design. Report of Subgroup C.

PIANC, 1992 Analysis of rubble mound breakwaters. Report of Working Group 12 of the Permanent Technical Committee.

Royal Society 1992. Risk: Analysis, perception and management. Report of a Royal Society Study Group. The Royal Society, London,

Royal Society 1983. Risk assessment. Report of a Royal Society Study Group. The Royal Society, London.

Rydz B 1979. The practical application of reliability criteria. Proc IAHR 18th Congress Cagliari Italy Vol.2, September, pages 307-313.

Shuttler R M 1982. Some aspects of modelling slope protection. Reprint from British National Committee International Commission on large dams autumn conference: Performance of dams and reservoirs, Keele University, September, pages 13-26.

Smith K V H 1986. Probablistic approach to the stability analysis of rock protection for earth weirs. Proc ICE Part 2:Research and Theory Vol. 81, June, pages 243-253.

Sobey R J 1992. The distribution of zero-crossing wave heights and periods in a stationary sea state. Ocean Engineering, Vol. 19, No. 2, pp101-118.

Shaw E M 1984. Hydrology in practice. Van Nostrand Rhinhold.

Stans J C 1979. Model investigations and probabilistic design. Coastal Structures '79 Vol 2. ASCE

Thoft-Christensen P & Baker M J 1982. Structural reliability theory and its applications. Springer-Verlag.

Thomas R S & Hall B 1992. Seawall design. CIRIA.

Tung Y K & Mays L W 1981. Optimal risk-based design of flood levee systems. Water Resources Research, Vol. 17, NO. 4, august, pages 843-852.

van den Graaff J 1985. Probablistic design of dunes A Dutch approach. Undated paper, 34 pages.

van der Graaff J 1986. Probablistic design of dunes: An example from the Netherlands. Coastal Engineering Vol. 9 NO. 5, February, pages 479-500.

van der Meer J W 1986. Deterministic and probablistic design of breakwater armour layers. Dock & Harbour Authority Vol.67 NO. 785 December, pages 177-180.

van der Meer J W 1988. Deterministic and probabilistic design of breakwater armour layers. ASCE Journal of Waterways, Port, Coastal and Ocean Engineering, Vol 114, No 1, January

Vrijling J K 1986. Probabilistic design of waterretaining structures. Engineering Reliaility and Risk in Water Resources. L duckstein and E J Plate, Eds. Martinus Nijhoff, Dordrecht, The Nertherlands

Vrijling, J K. Probabilistic methods in hyraulic engineering. PIANC Bulletin 80, pp 56-61

Wunderlich W O. Probablistic approaches to maintenance. ASCE.

Yen B C 1975. Risk based design of storm sewers. HR Wallingford, Report NO. INT 141, July.

FIGURES

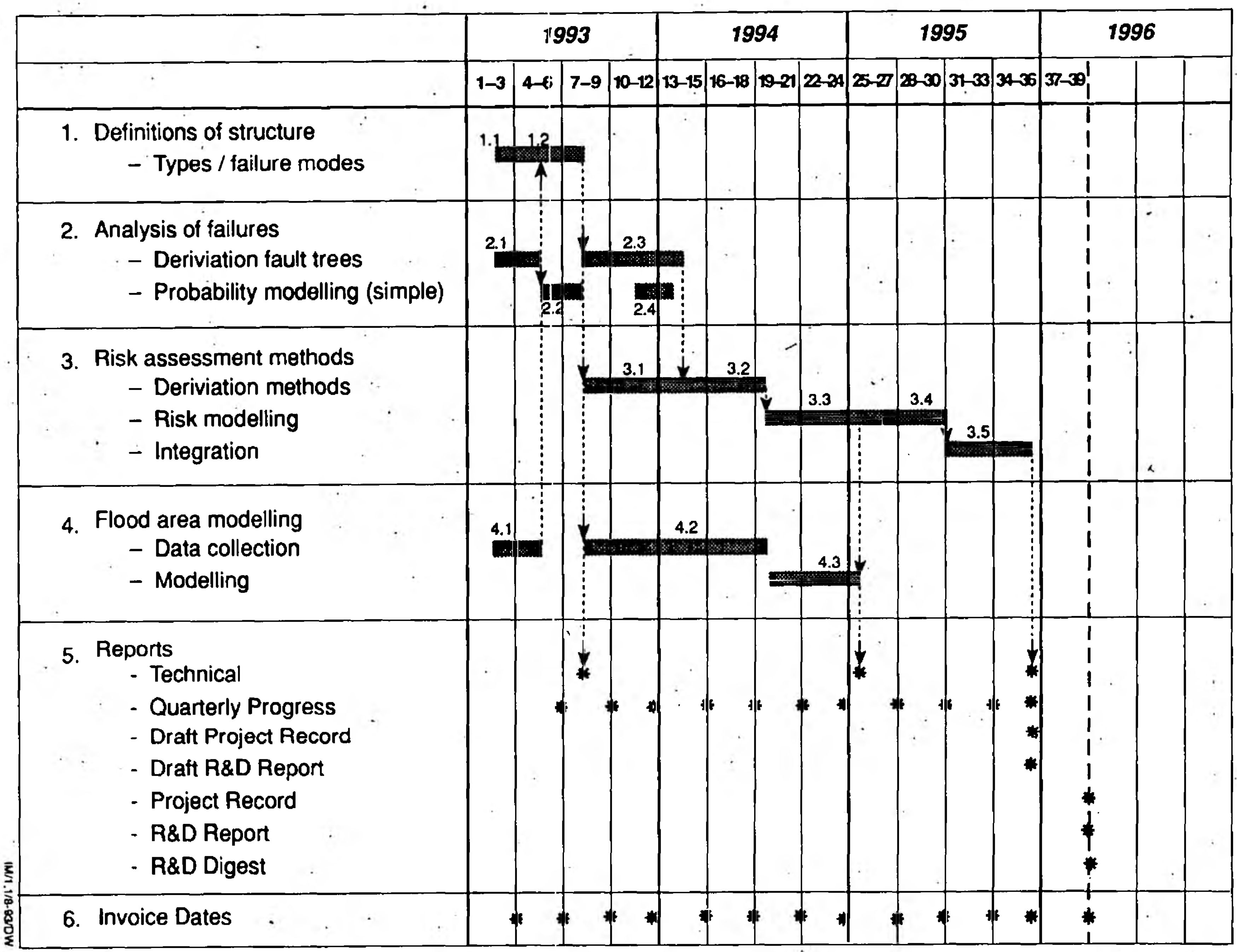


Figure 1.1 Project activity Chart

IM/1.1/8-93/DW

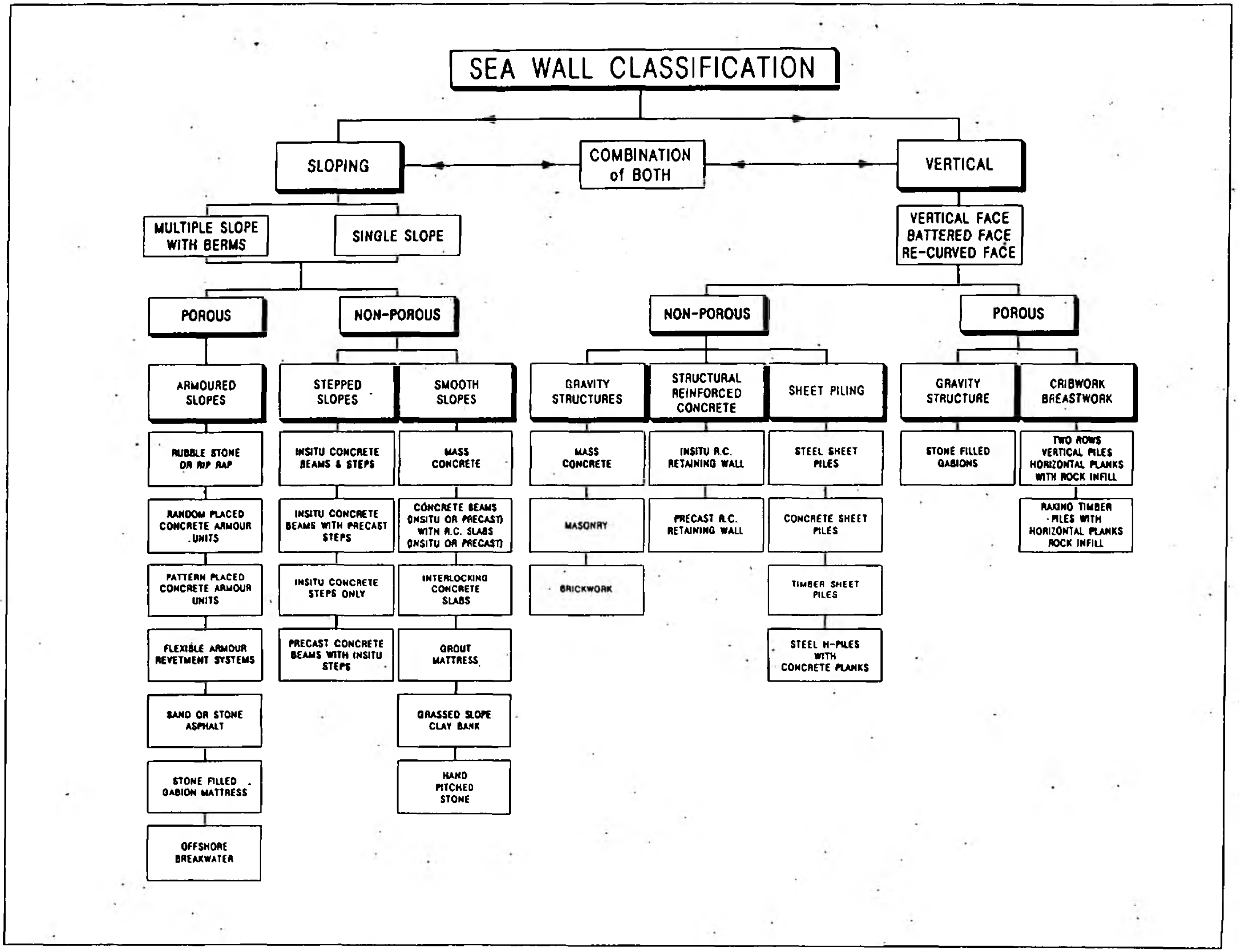


Figure 2.1 Sea wall classification *(reproduced from CIRIA, 1986b)*

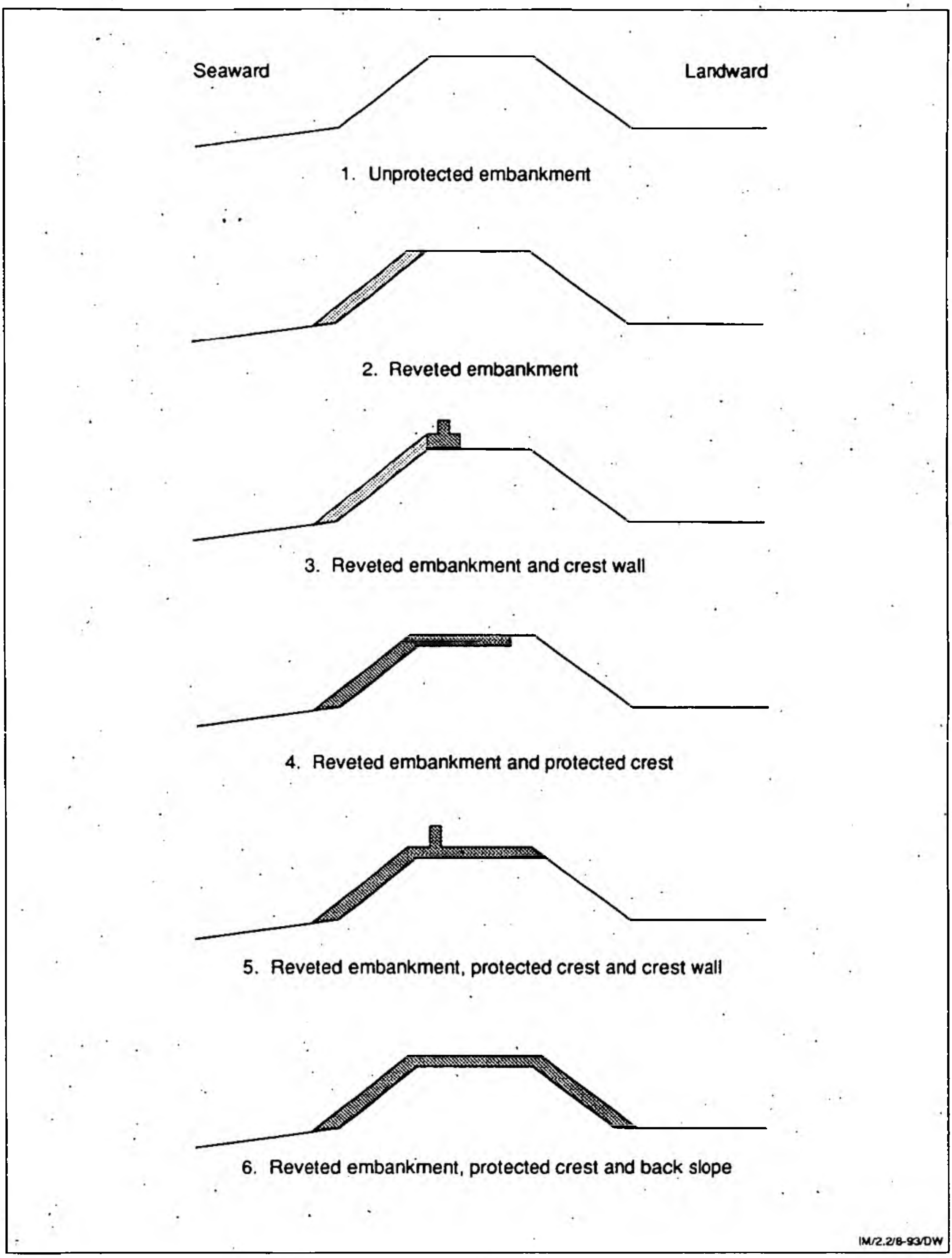


Figure 2.2 Classification of structure types: preliminary classification based on susceptibility to geotechnical and hydraulic instability

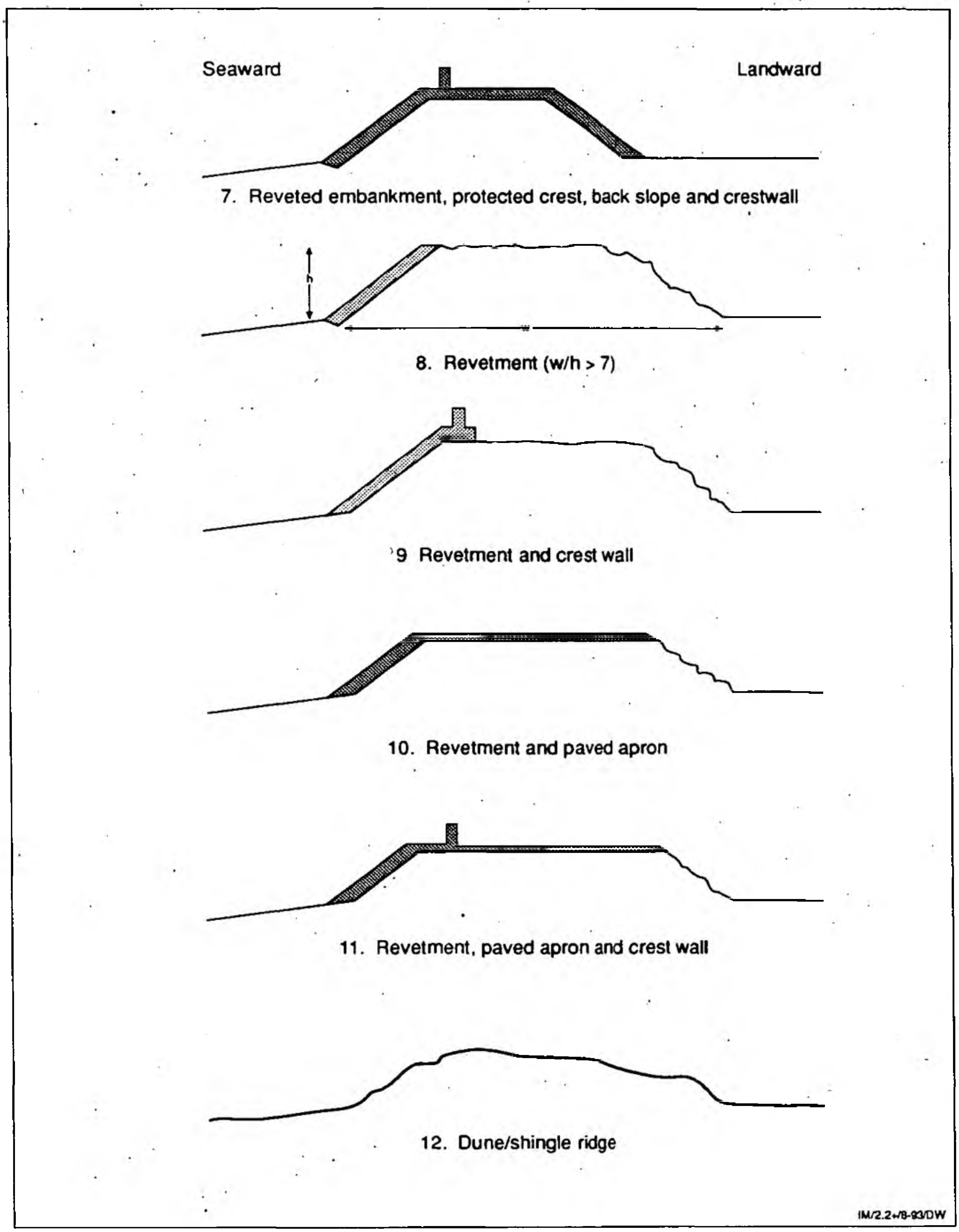


Figure 2.2 Classification of structure types: preliminary classification based on susceptibility to geotechnical and hydraulic instability (cont'd)

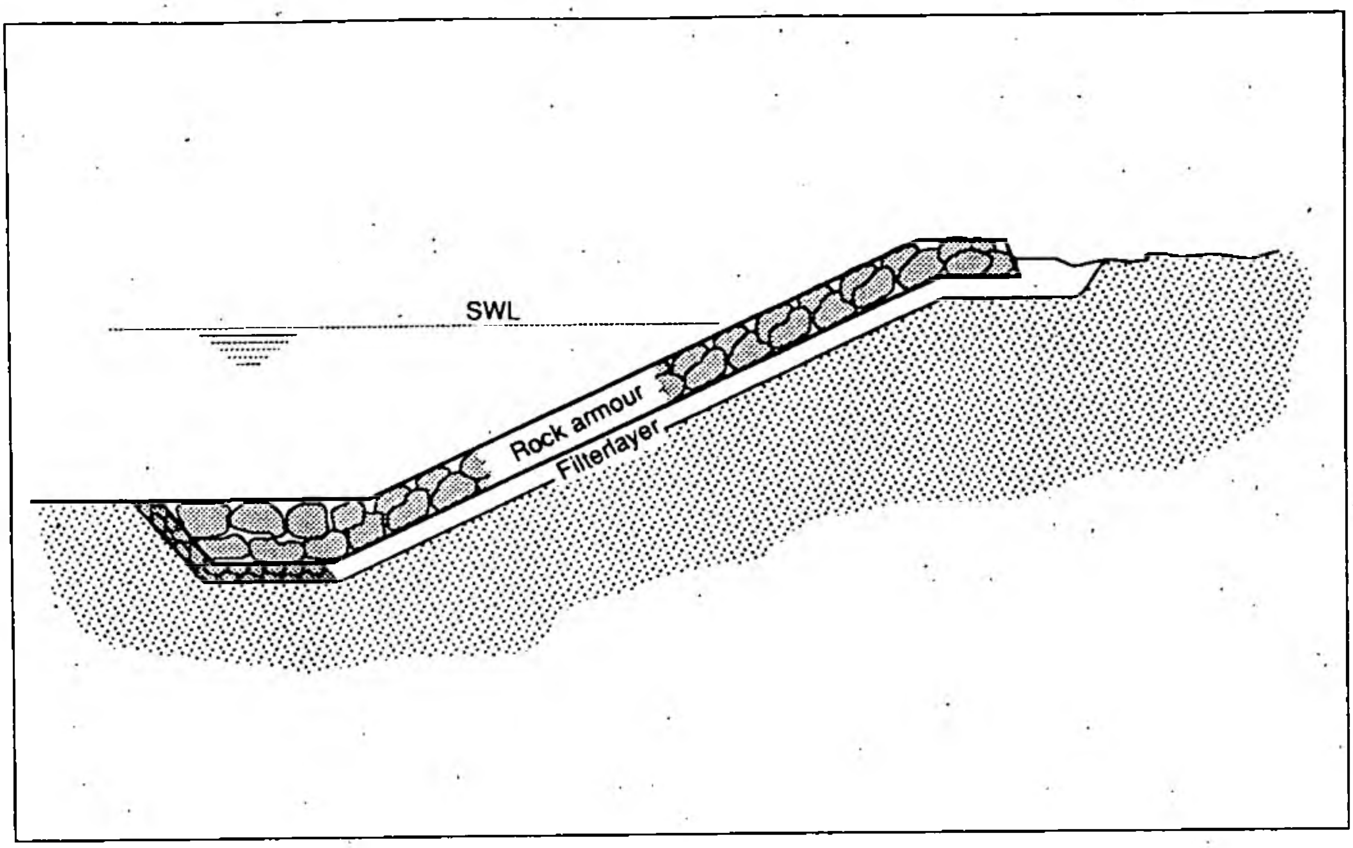
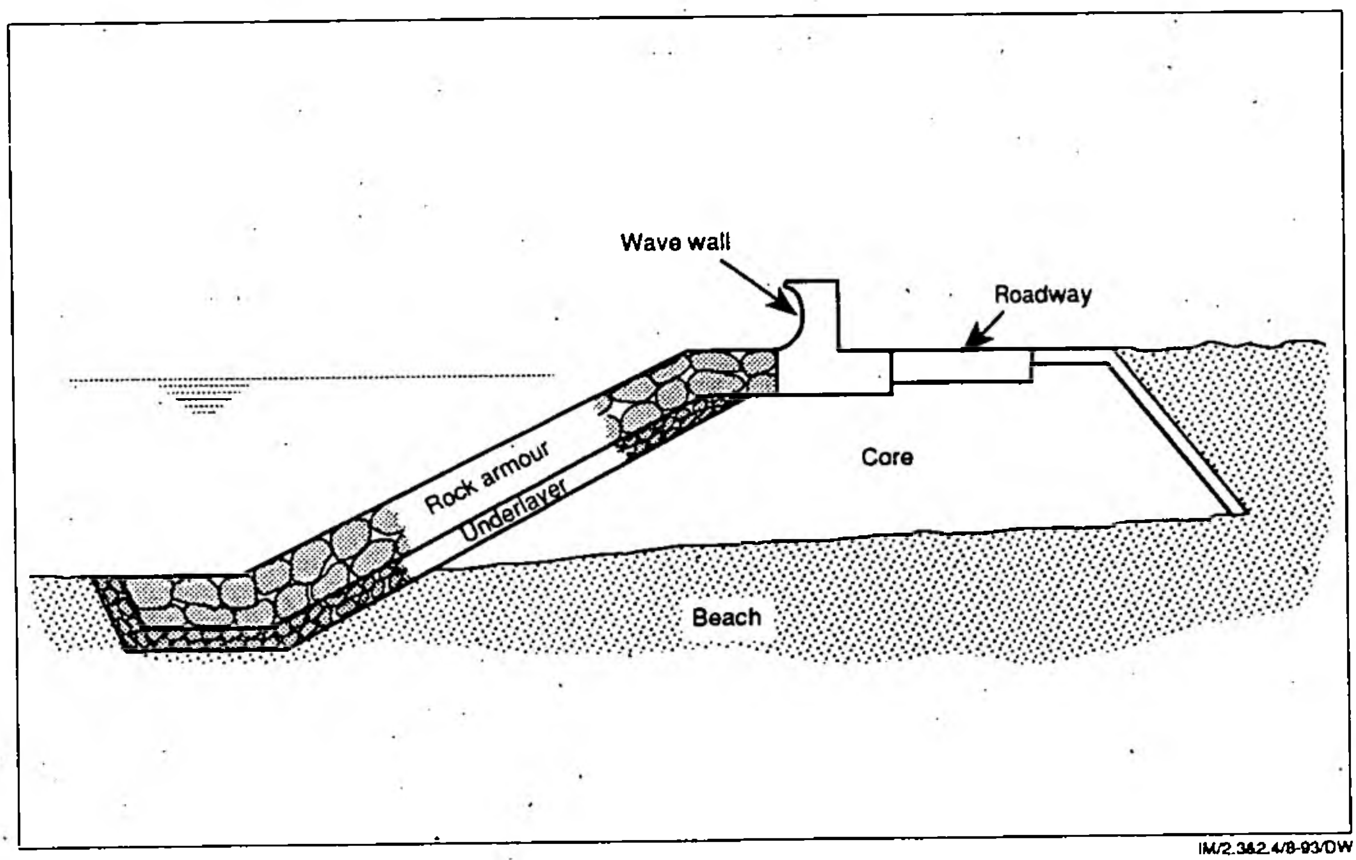


Figure 2.3 Rock armoured revetment



IM/2.3&2.4/8-93/DW

Figure 2.4 Sea-defence revetment with rock armour and wave wall

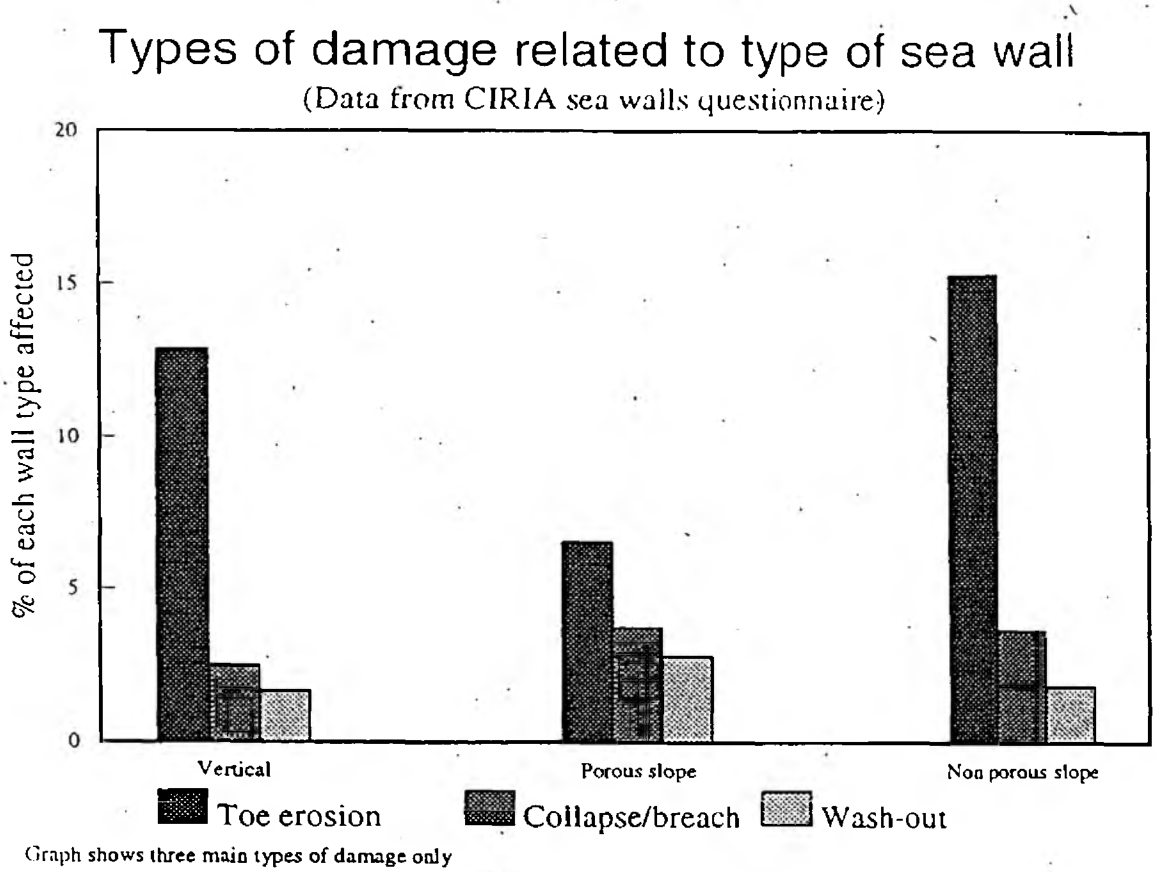


Figure 2.5 Incidences of reported damage related to type of sea wall

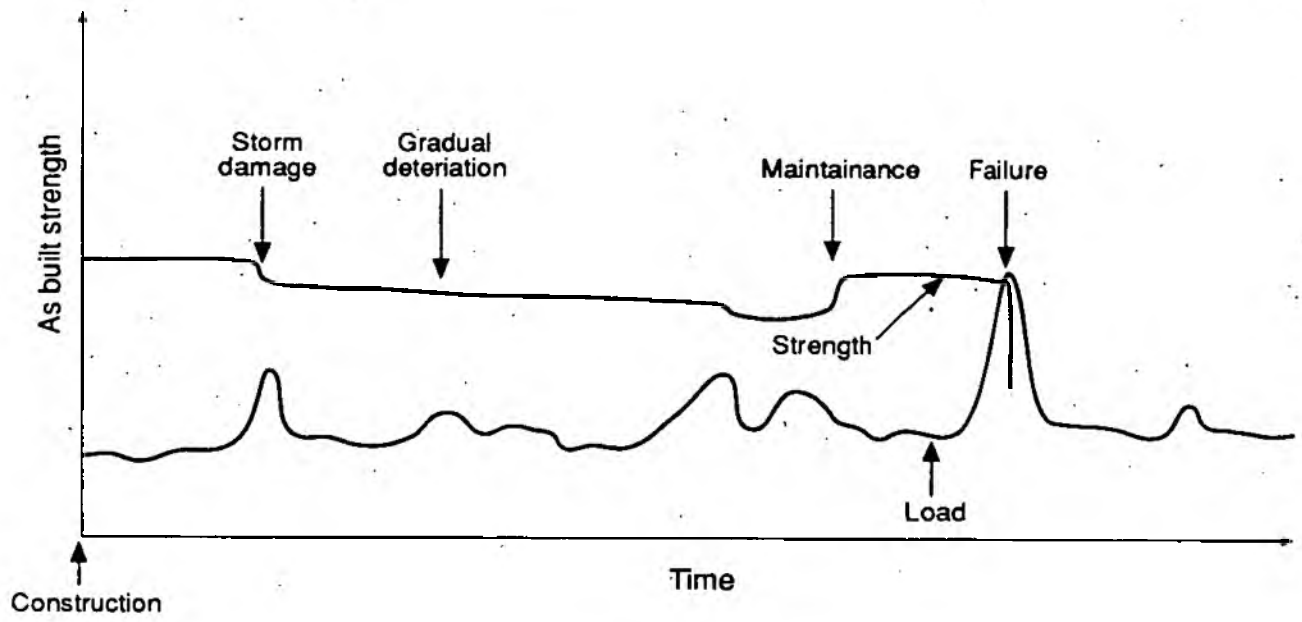


Figure 2.6 Change to strength and load during liftime of a structure (illustrative)

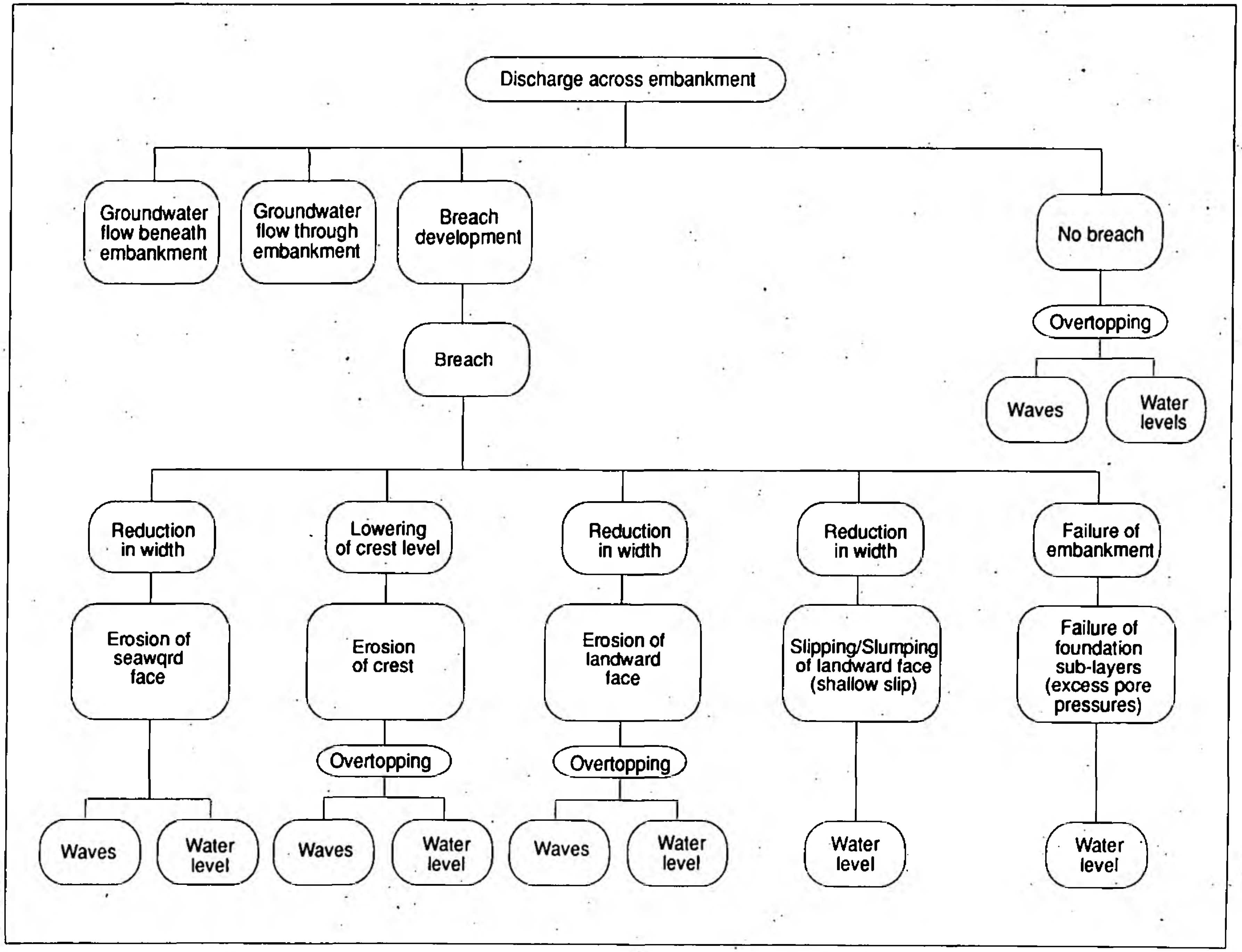


Figure 2.7 Flow chart of some failure modes for an earth flood embankment

IM2.7/8-93/DW

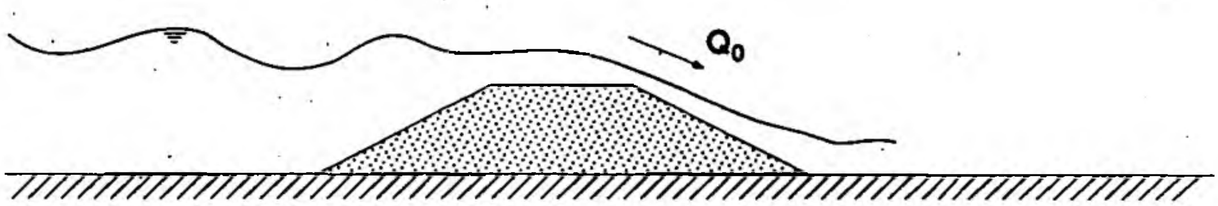
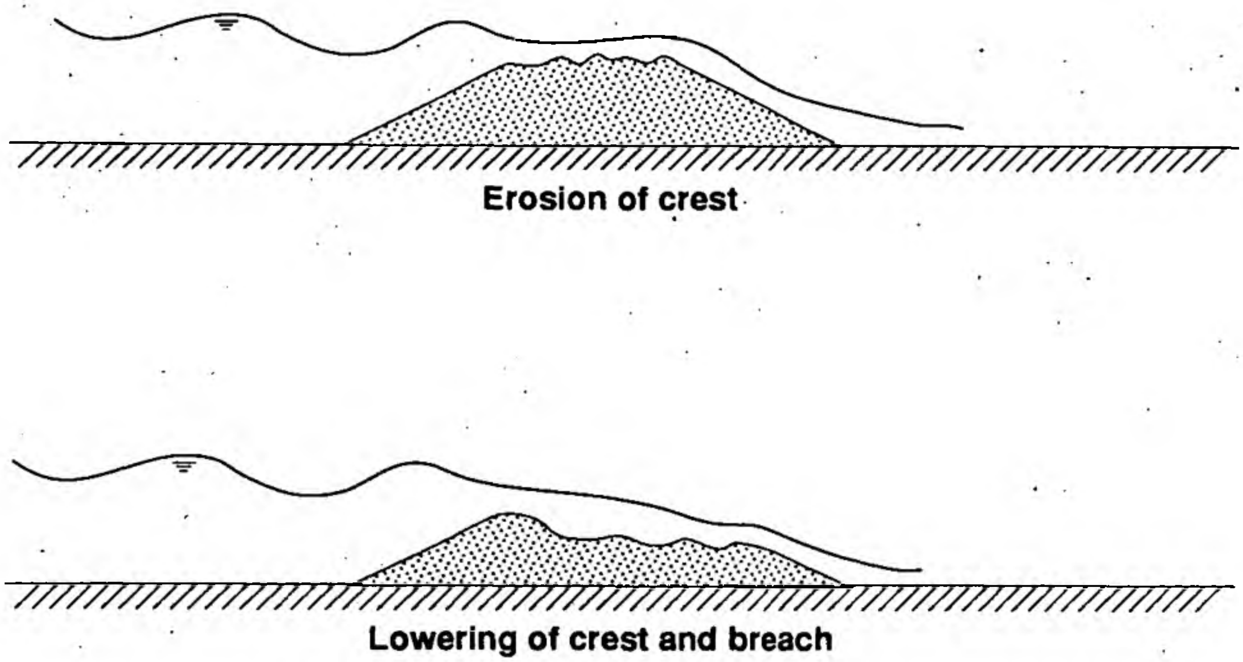
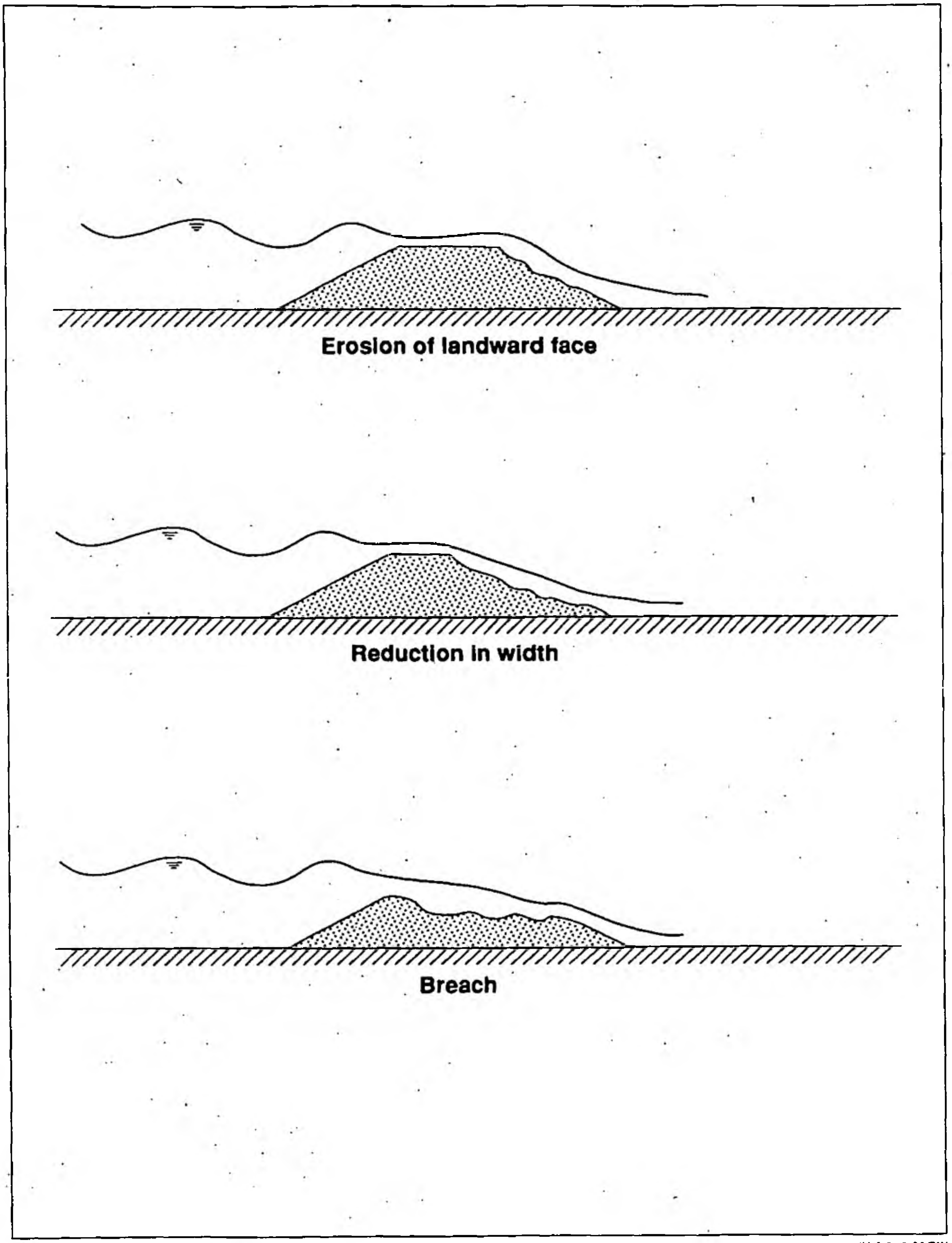


Figure 2.8a Illustrations of failure modes for an earth flood embankment



IM/2.8b/8-93DW

Figure 2.8b Illustrations of failure modes for an earth flood embankment (cont'd)



IM/2.8c/8-93/DW

Figure 2.8c Illustrations of failure modes for an earth flood embankment (cont'd)

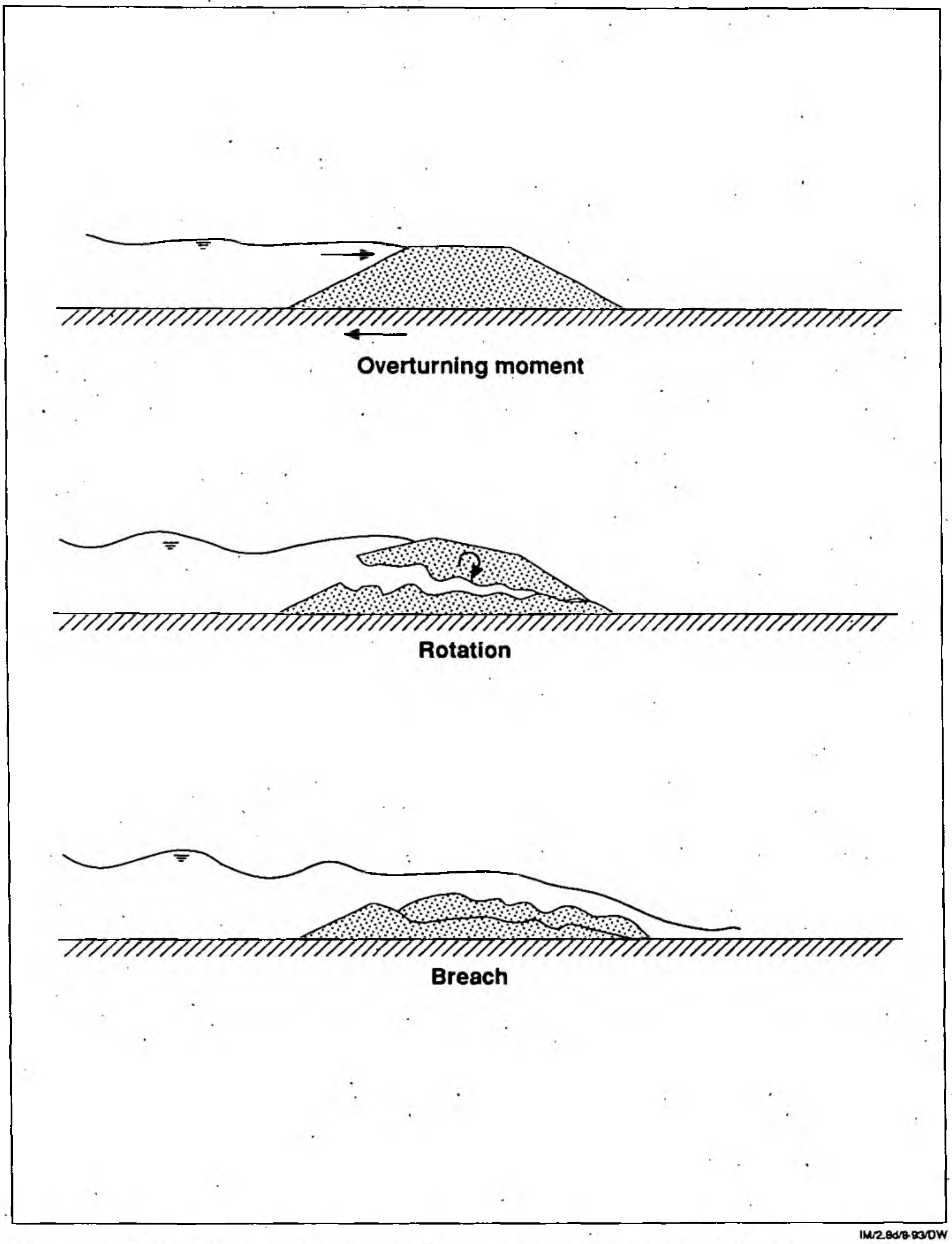
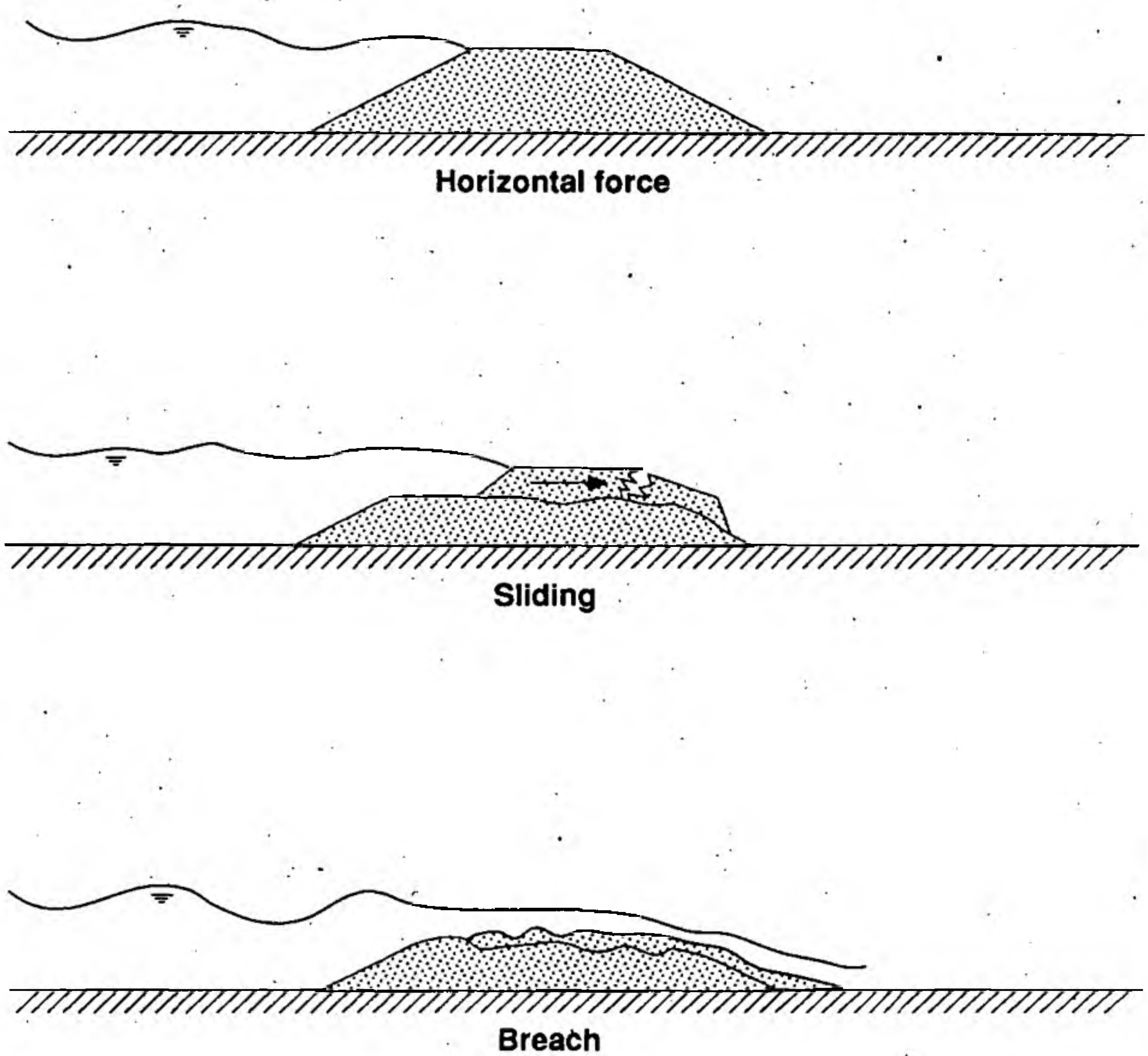
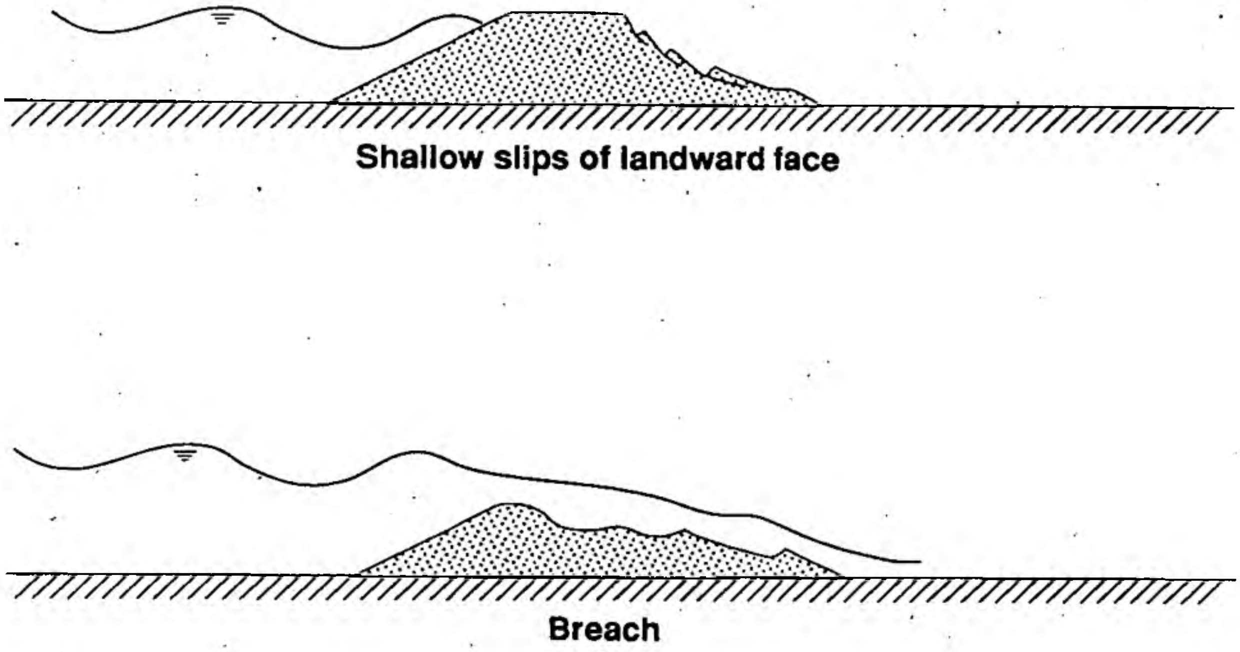


Figure 2.8d **Illustrations of failure modes for an earth flood embankment (cont'd)**



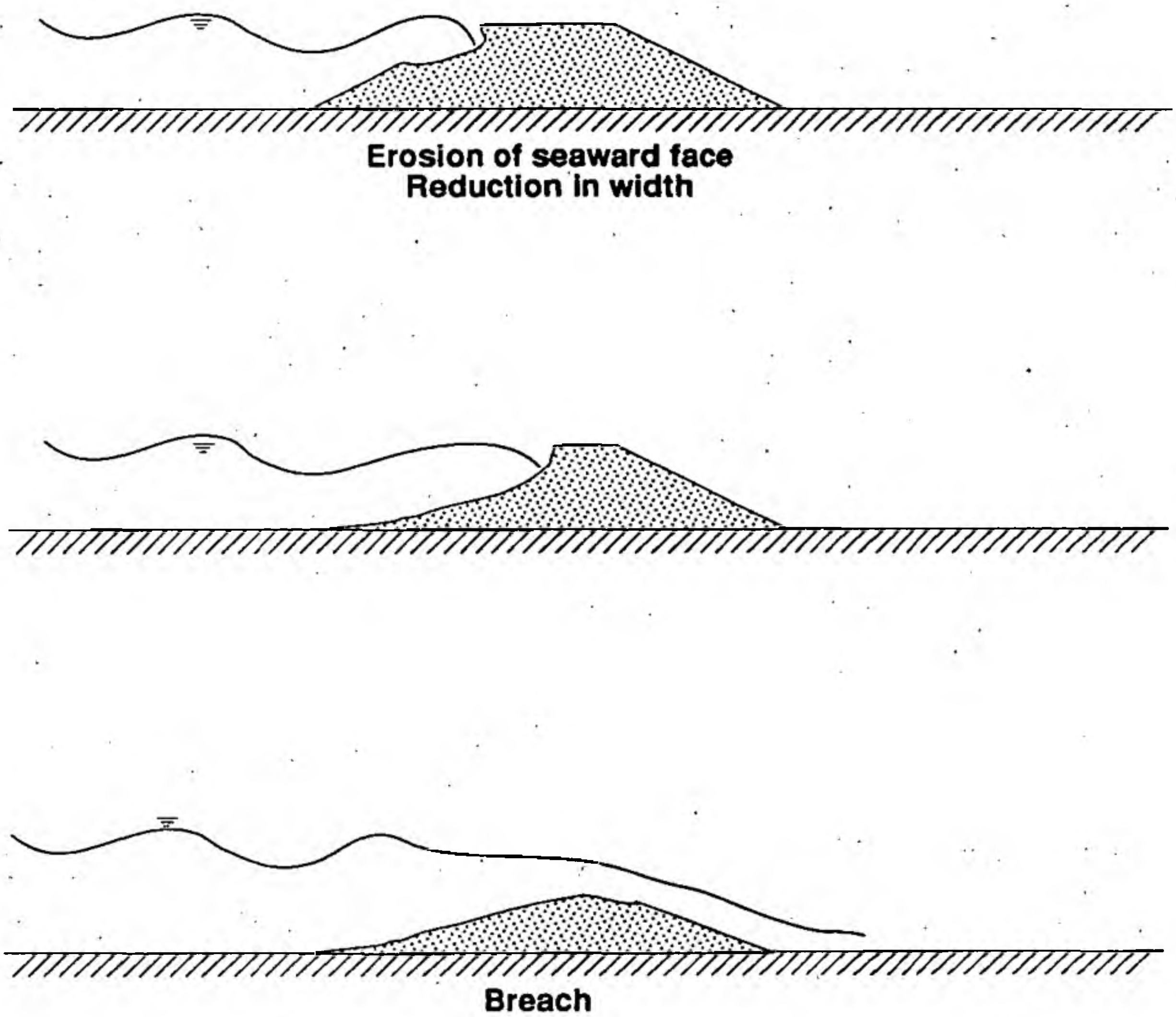
IM2.8e/8-93/DW

Figure 2.8e Illustrations of failure modes for an earth flood embankment (cont'd)



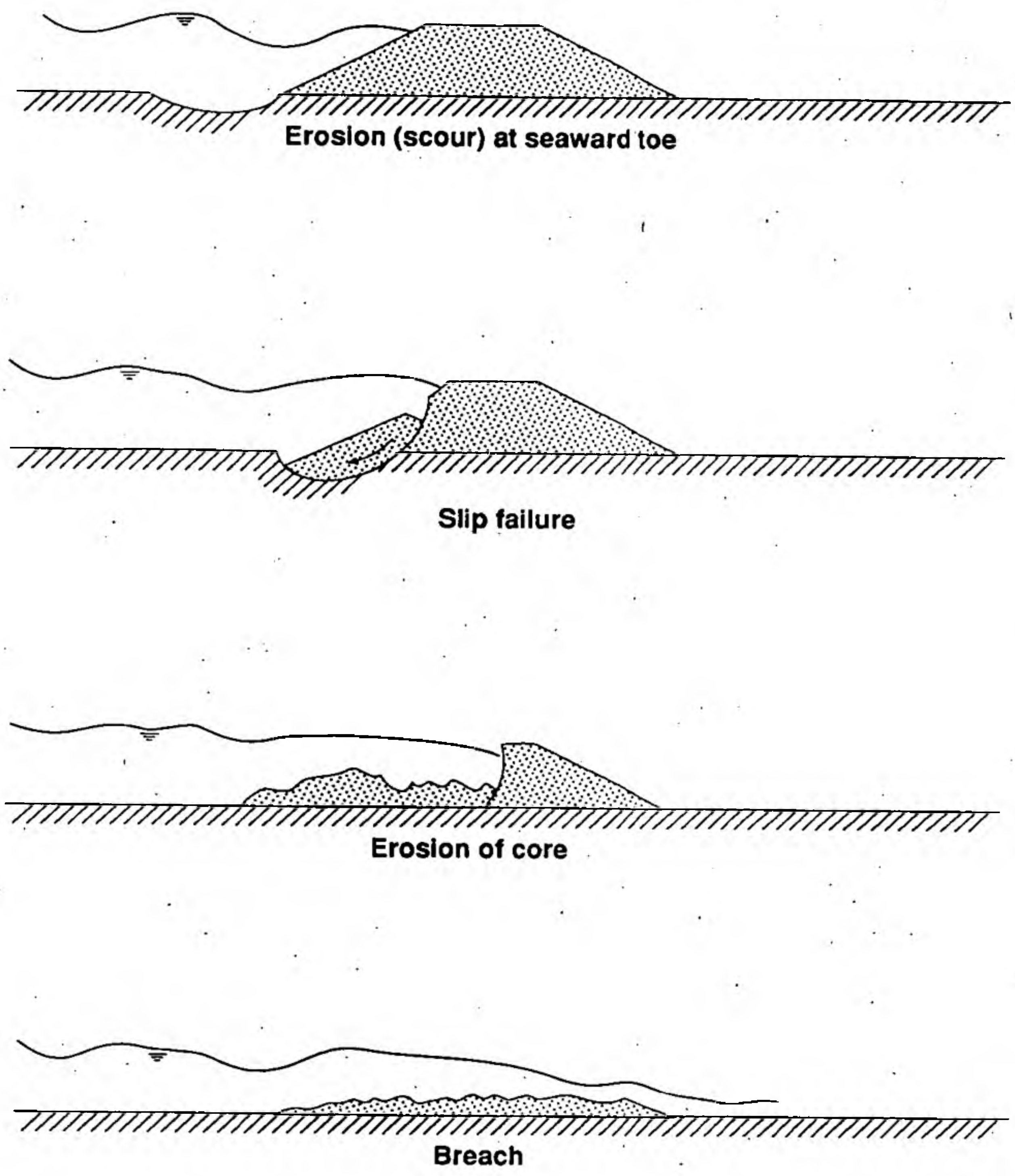
IM/2.8f/8-93/DW

Figure 2.8f Illustrations of failure modes for an earth flood embankment (cont'd)



IM/2.8g/8-93/DW

Figure 2.8g Illustrations of failure modes for an earth flood embankment (cont'd)



IM/2.8h/8-93/DW

Figure 2.8h Illustrations of failure modes for an earth flood embankment (cont'd)

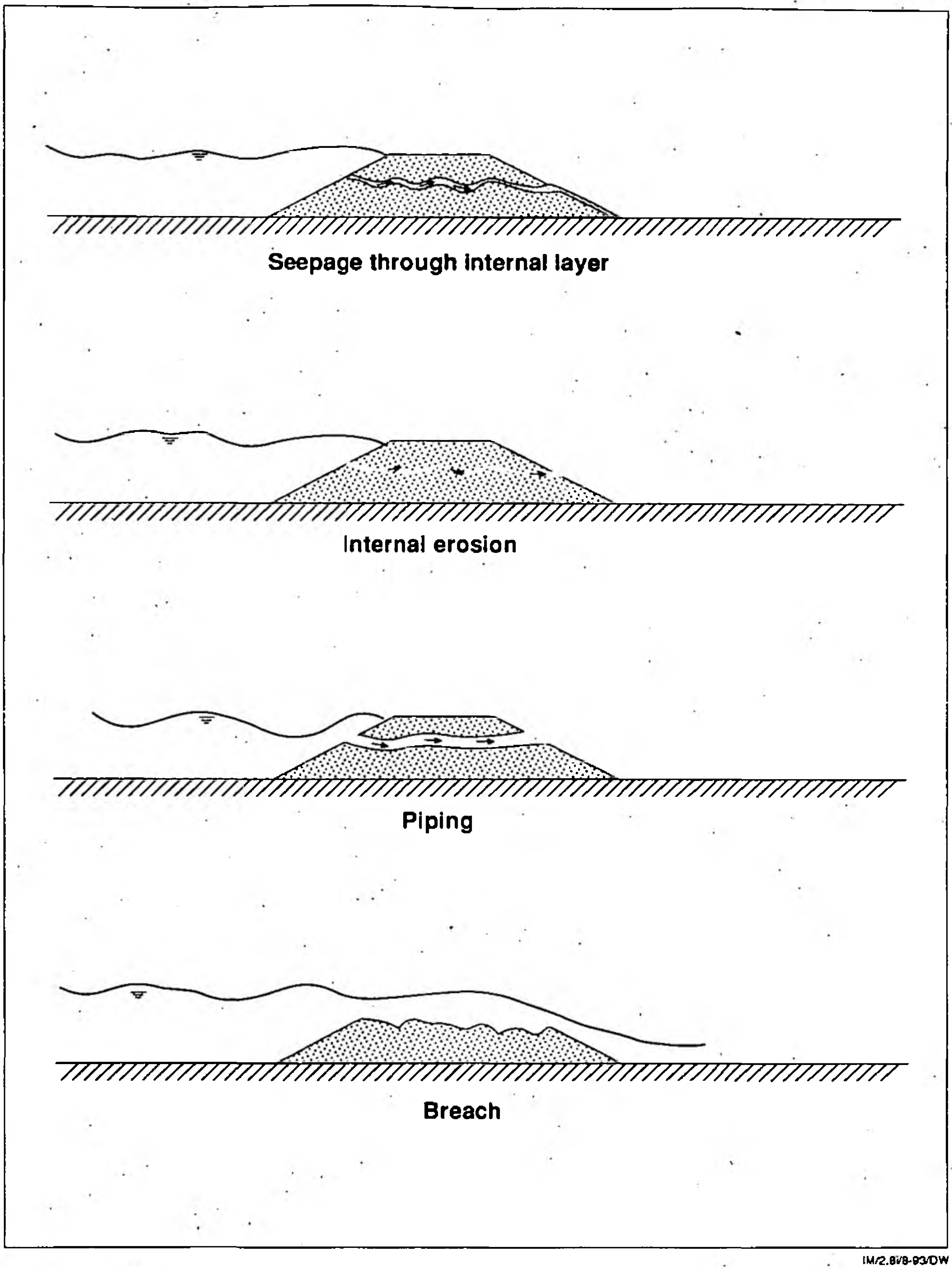
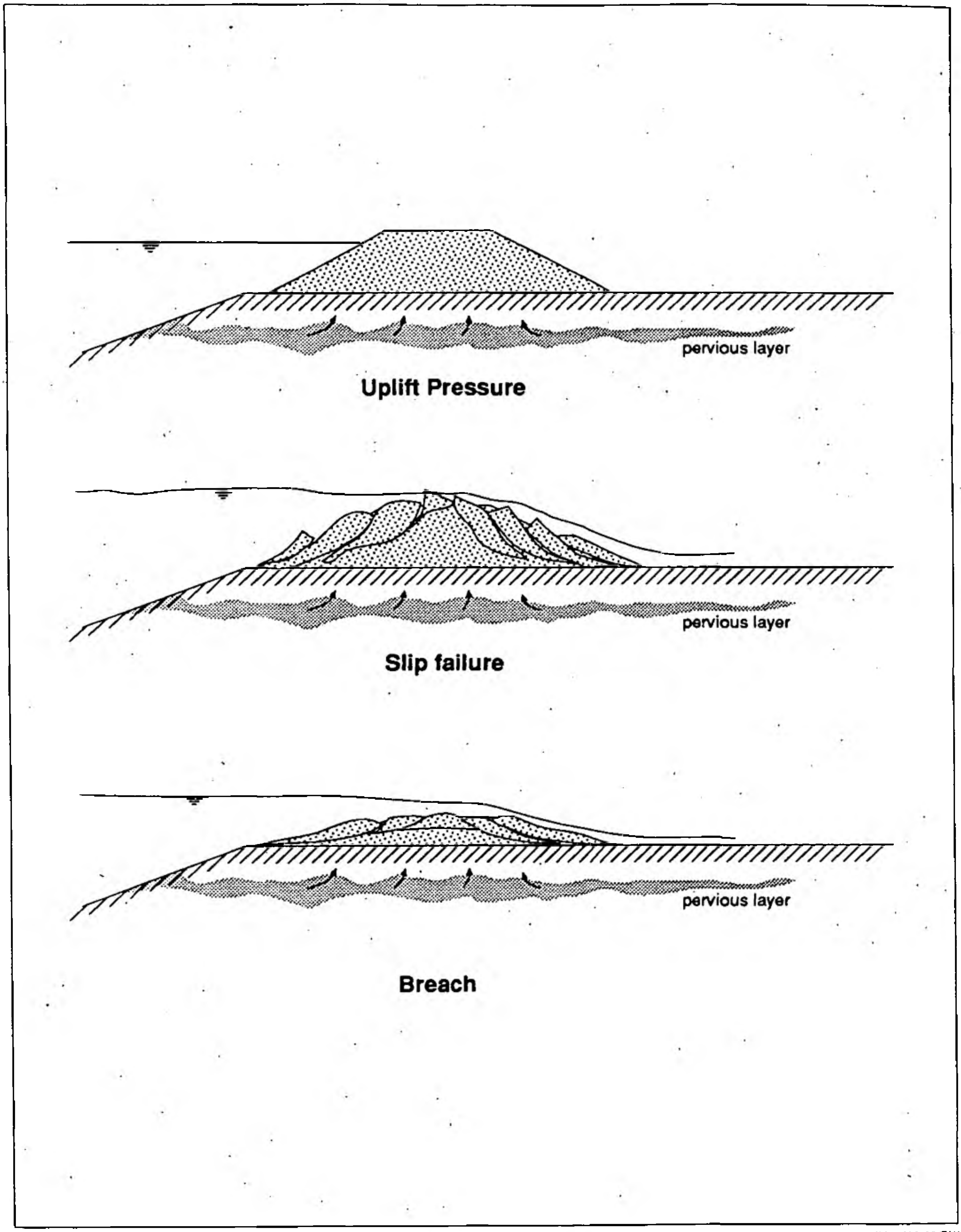
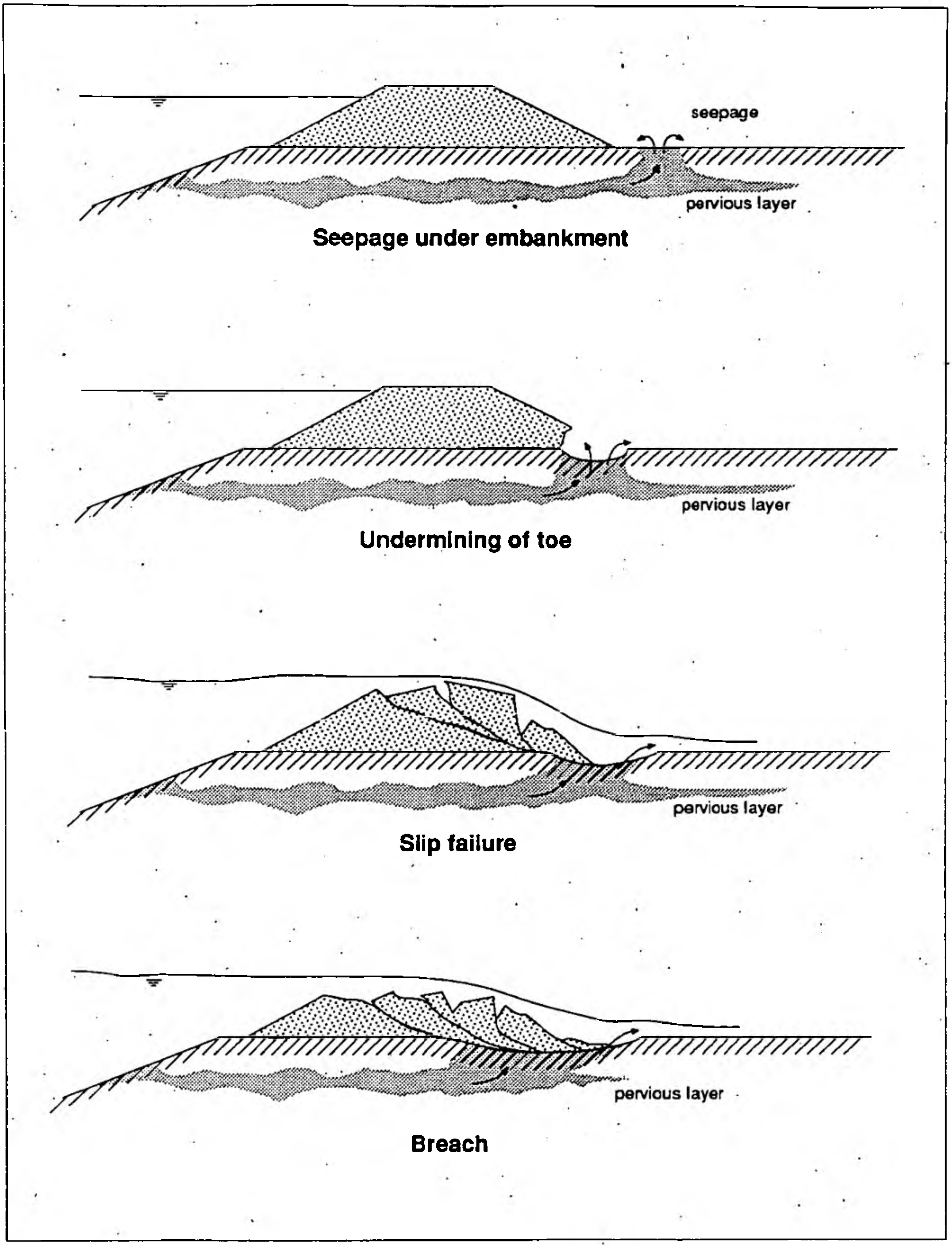


Figure 2.8i Illustrations of failure modes for an earth flood embankment (cont'd)



IM/2.8j/8-93/DW

Figure 2.8j Illustrations of failure modes for an earth flood embankment (cont'd)



IM/2.8k/8-93/DW

Figure 2.8k Illustrations of failure modes for an earth flood embankment (cont'd)

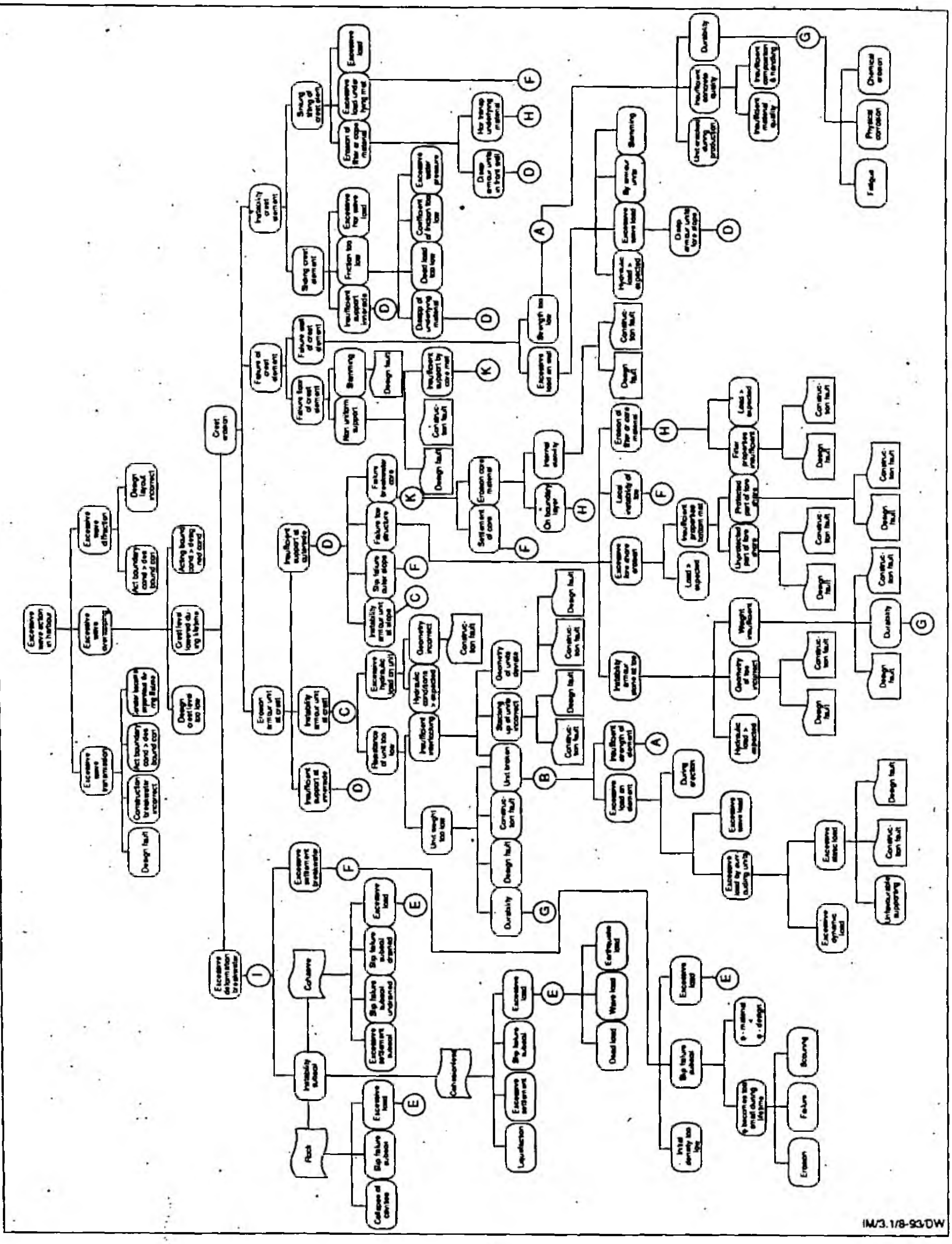


Figure 3.1 Fault tree for excessive waves behind a rubble mound breakwater *(reproduced fromCIAD, 1985)*

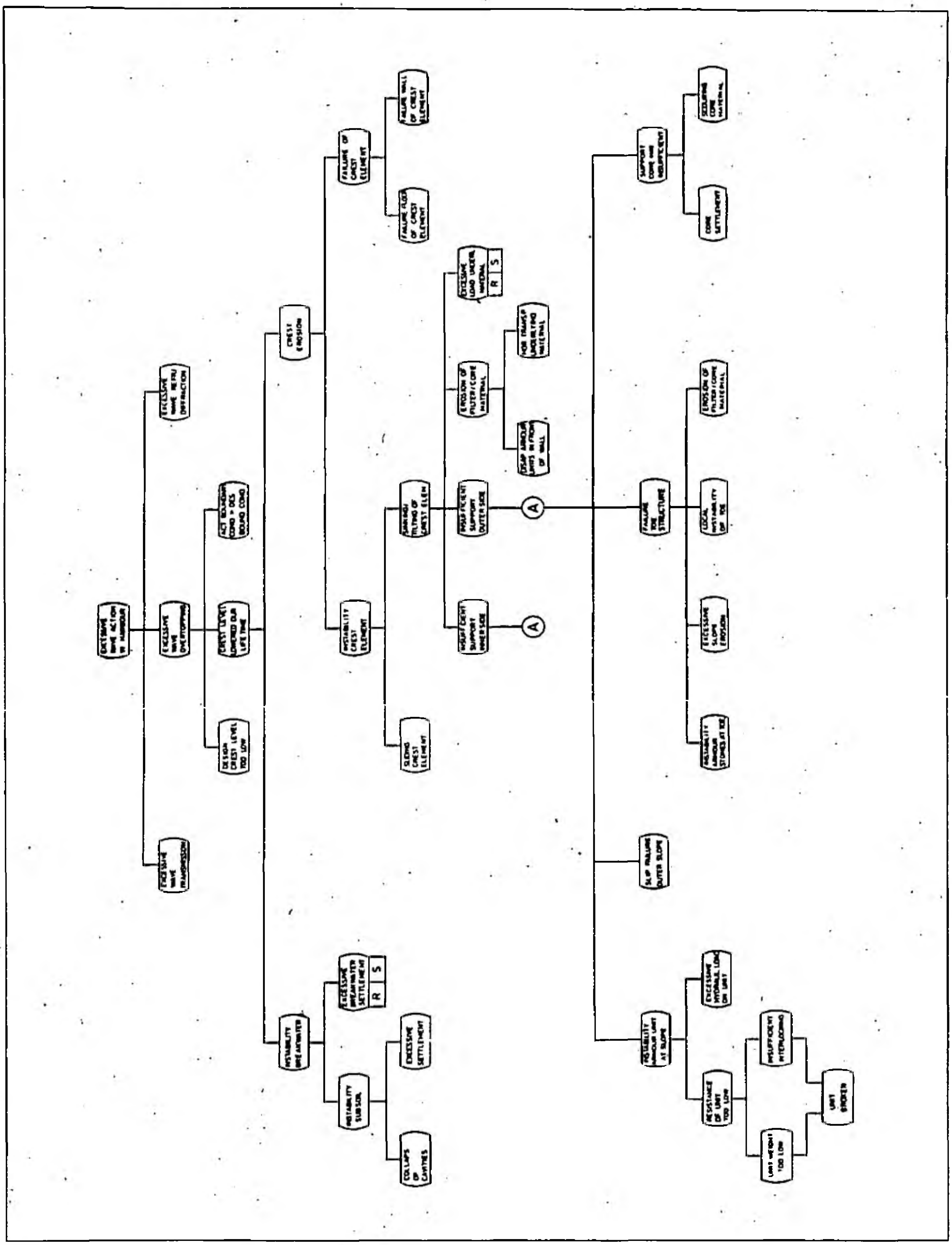


Figure 3.2 Simplified fault tree for excessive waves behind a rubble mound breakwater *(reproduced from CIAD, 1985)*

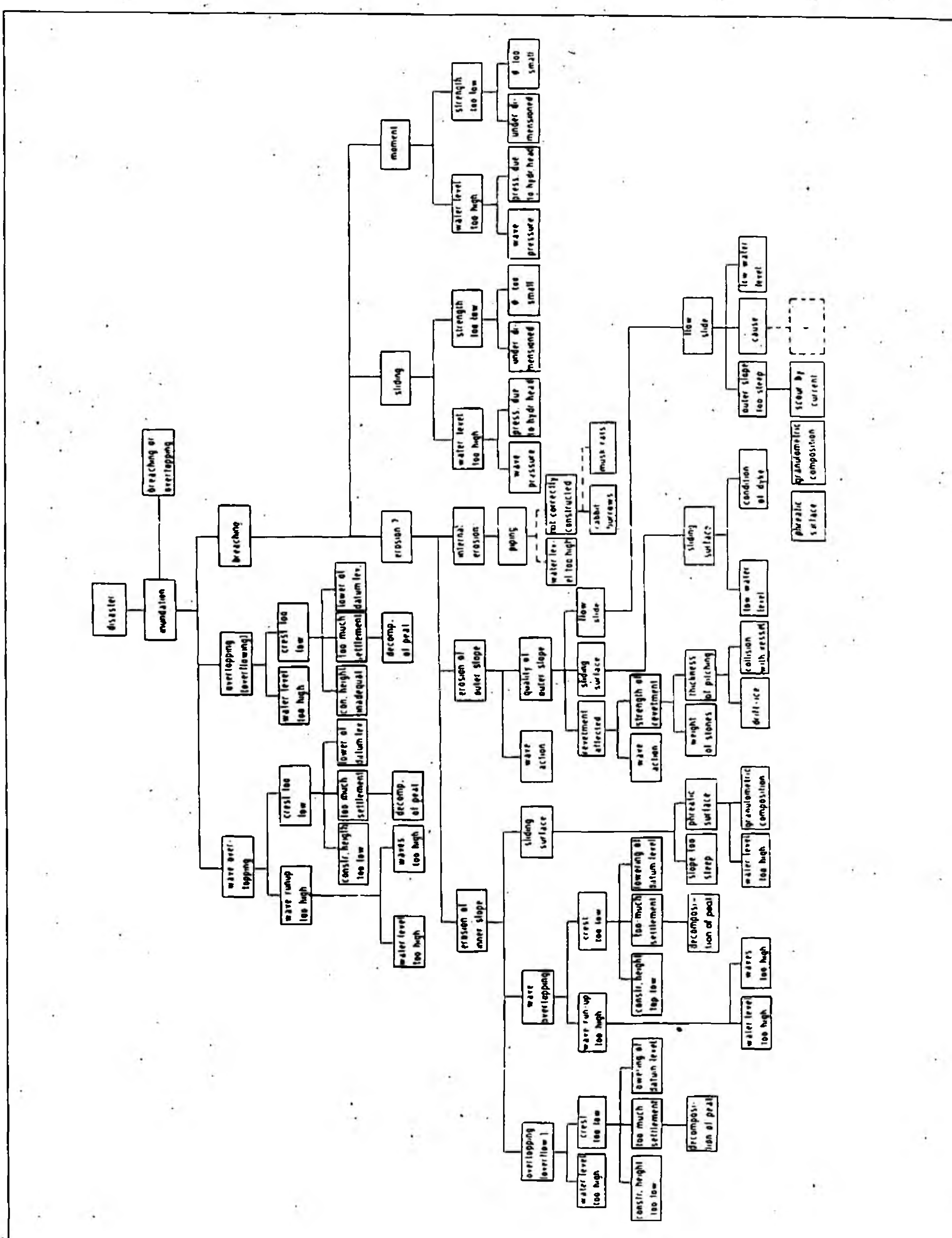
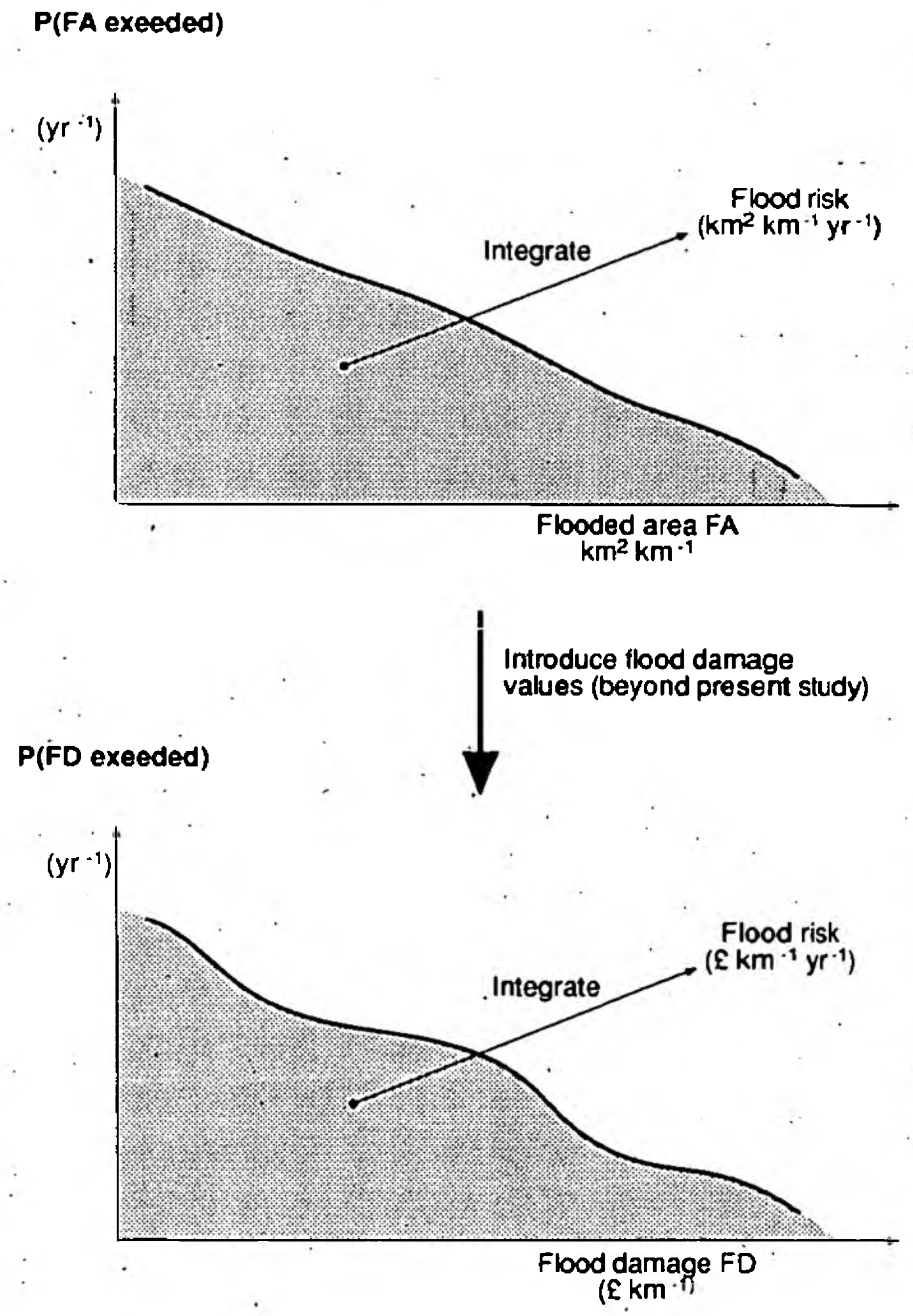


Figure 3.3 **Fault tree for inundation due to failure of a flood defence** *(reproduced from CUR/TWAW, 1990)*

Project purpose statement

'To develop methods to assess the risk of tidal and coastal flooding. The risk of flooding, or flood risk, is defined as the expected area of land flooded per km of defence per year'

Risk of flooding, or flood risk, is obtained from a flooded area - Frequency diagram:



IM/3.4-8-93/DW

Figure 3.4 Illustration of the concept of risk

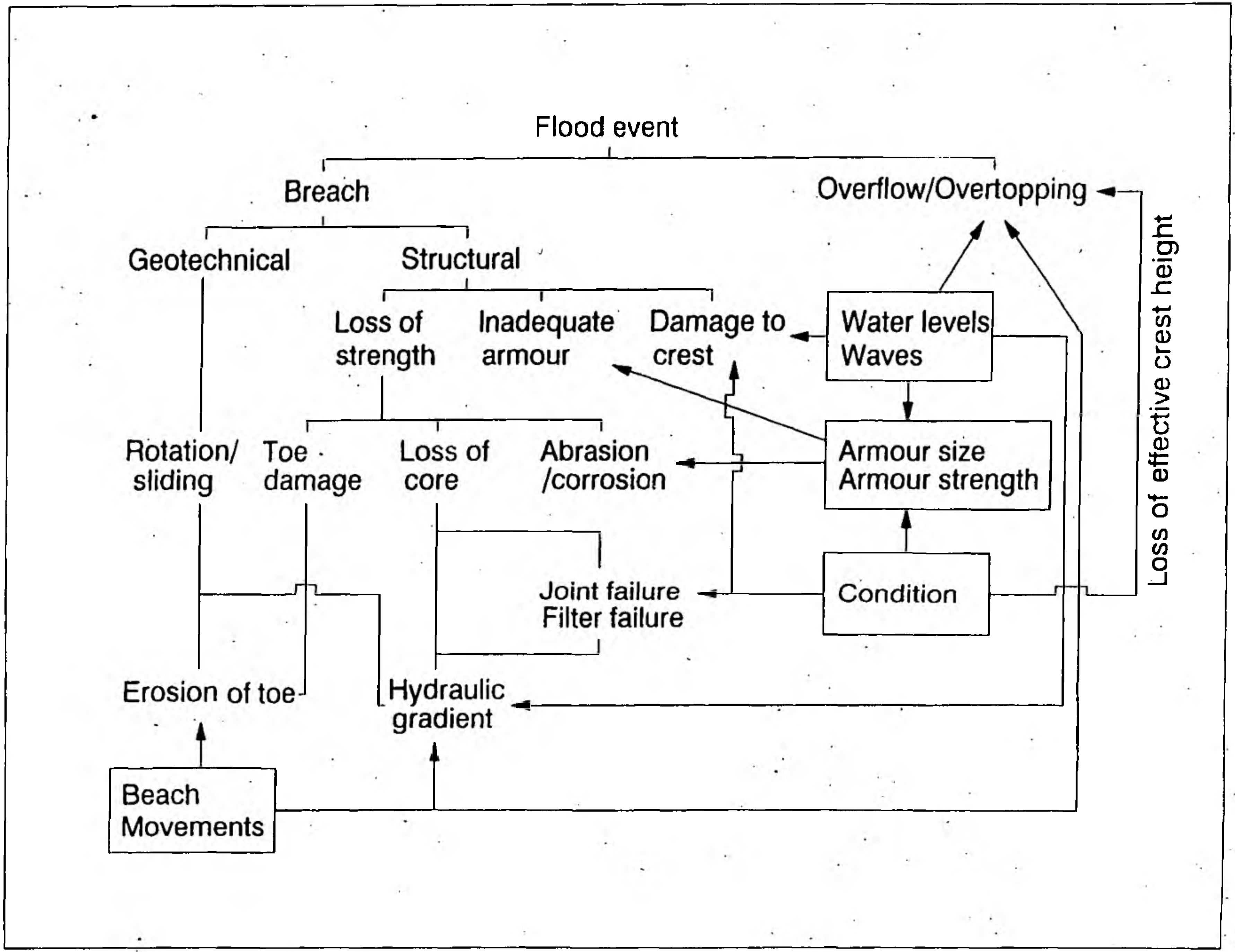


Figure 3.5 Simplified consequence - cause tree for coastal flooding

Figure 5.1 OS spot height and contour data around Grimsby

Figure 5.2 Terrain model data around Grimsby

Figure 5.3 Terrain model data (pink) and flood area and depth contours (blue) around Grimsby