H.J. Verhagen, MSc Lecture Notes LN0062/07/1

Revetments, Sea-dikes and River-levees

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lecture notes on revetments, sea-dikes and river-levees

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1 Introduction

These lecture-notes were compiled using many other sources. Parts of the text are based on the books "Coastal Protection" and "dikes and revetments", edited by K.W. Pilarczyk. Also CUR-publications 141 (probabilistic design of flood defences), 142 (guide for the design of river dikes) and 154 (the use of rock in coastal and shoreline engineering) was used. Also some parts are based on the lecture notes on Bed, Bank and Shore Protection by G.J. Schiereck of the Delft University of Technology. In some cases information is used from suppliers of materials for the construction of dikes and revetments. The fact that this information is used does not imply that IHE fully supports these products.

1.1 dikes and revetments

Dikes and revetments are both types of shoreline protection. However, their aim is different. The aim of a dike is to prevent flooding of the area behind the dike. This can be both the prevention of temporary flooding or a permanent flooding. Permanent flooding occurs when the land is below normal high water level. Temporary flooding occurs when the land is above normal high water, but below the flood level. In case of a land level between normal low water and normal high water the area will be flooded twice a day, with the rhythm of the tide. These land are usually called intertidal land.

Revetments have a different aim. Their aim is to prevent loss of land (loss of shore-face) due to erosion. This erosion can be caused by currents, by waves, or by both. Sometimes this erosion occurs during storms only (like the erosion of dunes), otherwise it might occur mainly during the everyday conditions. This happens for example along the shores of a shipping canal due to the ship induced waves.

On a dike one usually also finds a revetment. The aim of such a revetment is the prevention of erosion of the dike-front, due to wave action. This can be both extreme or normal condition wave action. Very often the revetment on a dike is called slope-protection. In this way a distinction is made with a bottom-protection which has the task to prevent scouring of the bottom under water in front of a structure.

Sometimes on top of a revetment a parapet-type of construction is placed to prevent overtopping of the revetment. The aim of this parapet is to reduce the negative consequences of the overtopping quantities. The design of such a parapet sometimes equals dike design, but has a different function. Therefore the risk-level is usually different.

A dam is a dike-like structure in the mouth of a river or estuary to stop the inflow of seawater and the outflow of river water. Usually it is not possible to close of an estuary or river completely, and therefore a sluice is necessary to control the outflow of river water. The design of a dam is usually quite identical to the design of a dike. However, the construction is different, because the closing of the dam causes much problems. Due to the high current velocities in the decreasing closing gap, the material of the bottom and of the dam under construction is not stable any more. It is washed away by the strong tidal currents.

introduction

In these lecture notes the design of dikes, revetments, bottom protection and dams will be discussed. Also some information is given on the execution and maintenance of these works.

1.2 types of dikes

In general three types of dikes can be distinguished:

* sea dikes
* river dikes or levees
* dikes around lakes
* dikes along canals

Retaining structures along reservoirs etc. are not discussed, although may of the rules given in dike design are also valid for this type of retaining structures.

Note: *The word "levee" is mainly used in the USA, especially in the lower Mississippi area. In English literature sometimes the word "embankment" is used.*

In general a sea dike has to resist a combination of high water and wave action (storm surge) during a relatively short period (the top of one tidal cycle). The rise of the water is very fast and difficult to predict. A storm surge can be predicted approx. 48 hours before it occurs, but the exact height of the storm can only be determined a few hours before the maximum occurs. Also the exact wave height is a problem. Determination of the design wave-height at sea during a design storm is in most cases possible, but determination of the design wave-height near the dike is a problem.

For river-dikes the design waterlevel depends on the discharge of the river. In general the level can be predicted several days before the maximum level occurs. Also the top of the flood-wave in the river has a much longer duration. It may last for several days.

In (shallow) lakes there is a constant waterlevel, coupled with a surge effect due to wind. In a lake with a diameter of 50 km. there may occur water-level differences between one side of the lake and the other side of more than one meter.

The various types of load on the dikes result in various geometries. A typical river-dike has an other shape than a typical sea-dike. These differences will be discussed in more detail during these lectures.

In some situations there may be a combination. Along tidal rivers the waterlevel is determined by the discharge from the upper part of the river and also by the waterlevel in the sea. There the worst case is a combination of high river run-off and a storm surge at sea. However the probability of this combination is low. Special mathematical techniques have been developed to solve this combination. In the lectures on probabilistic design this will be worked out in detail in a numerical way.

1.3 history of dikes in the Netherlands

The Netherlands are mainly a floodplain of the big rivers Rhine, Meuse and Scheldt. The area is very fertile, but difficult to access. Water is everywhere, so transport over water is quite easy. These aspects are important enough for people to settle in these areas. The first known settlements in the intertidal area are from Roman times (approx. 2000 years ago). The fact that the area was difficult to access was a big advantage in those years. In the past it was very difficult to occupy this area with a military army.

To protect themselves against flooding people started to raise artificial mounds. On these mounds the first villages were build. Later-on, when agriculture developed on the area around high water, people started to protect the area against the yearly storm surges by constructing small dikes.
Later-on these dikes were improved.
In the middle-ages the area between high and low water (the intertidal zone) was cultivated. The area was drained by automatic sluices, which opened during low water and closed during high water.
In the 17th century, after development of the windmill, also the area below low water was cultivated. All the drainage water had to be removed from the polder by artificial means. In the beginning this was done by windmills, later-on also by steam, diesel and electrical pumps.

Because of all this pumping the peat-layer compacted. The effect was a subsidence of the land. This subsidence made it necessary to increase the height of the dikes. Besides this subsidence of the land, we are also confronted with a rise of the sea level.
Both sea level rise and subsidence make it necessary to increase the height (and the strength) of the dike.

As can be imagined, a dike which is only made higher, but not wider, becomes too steep, and eventually becomes geotechnically unstable. The consequence is that during a storm surge this dike will collapse and cause inundation (flooding).

Constructing a dike is one thing, but to maintain it is another problem. Maintenance of a dike is not only conserving the present state, but also adapting the dike to the varying boundary conditions (=rise of the water levels). After a storm surge the people (the first generation) are aware of the problem. The second generation remembers the stories of the storm surge disaster from their parents, they maintain the dikes, keep it in good condition. However, the third generation forgets the problem. The dike was "always" there in their memory, it always worked well, why put a big effort in maintenance? And consequently after the third generation a new storm surge disaster will occur. We have had that problem several times in the Netherlands. We unfortunately have a long history of disasters.

Between 1200 AD and 1953 we had at least 140 flood-disasters with casualties. The first known was in 839, when 2400 houses were flooded, which was a considerable number for those years. In 1492 there was a very unstable political situation in the Netherlands. Because of that unstable situation, maintenance of the dikes was neglected. In that year the Said. Elizabeth-flood caused an enormous damage. In total 65 villages disappeared totally from the map.
Not all floodings were caused by nature. For example the flood of 1944 on the island of Walcheren was caused by Allied bombers, to move the German forces out of the island.

design practice for river dikes until 1953
The height of dikes was in history always a problem. Usually dikes were designed at a crest level of 0,5 m above the highest known water level, with a surcharge for wave run-up. After a serious flood, much dikes in the coastal zone were improved. The height was increased up to a level related to the highest storm. However, because of bottom subsidence the crest height of the dikes became also lower in respect to level of the sea. The river dikes in the Netherlands were also designed on a highest known water-level. Also special "green rivers" were in operation to divert water if the water in the river raised to a too high level. Because after 1926 there were no serious problems with river dikes, there was less political pressure to improve the dikes than it was in the coastal area. After the second world war engineers and mathematicians warned that this approach is not correct, and that an extreme value statistic should be applied. The consequence of this approach would have considerable financial consequences, and no political decision was made.

1.4 types of revetments

The simplest type of revetment is turf which is formed either by sodding, or soiling and seeding. Because of its great disadvantage of impeding the flow as it grows, it is usually only used for flood banks or side slope protection above normal water level. Puddled clay is still an effective method of lining and sealing channels flowing through permeable soils and brushwood or shingle, held in position by wire netting anchored to the bed and slide slopes, are equally effective in preventing the fissuring of cohesive soils in dry conditions.
Random tipped stone is commonly used on rivers and occasionally on canals. Although simpler to place, particularly under water by tipping, than any other system, it is not easy to control the thickness, and wastage occurs in attempting to obtain the minimum requirement. As it is not bonded in any way, additional allowance has to be made for losses caused by erosion.
Pitched stone, which is economical in material, was a very popular method in western europe, until the 1960's but has since become expensive in labour. Masonry and brickwork which are also expensive because of their skilled labour contents are now generally only used in short length of protection adjacent to bridges and other structures. They, unlike tipped and pitched stone, are rigid in character and likely to rupture if affected by settlement.
Concrete is used in a great many forms: in-situ, mass or reinforced; pre-cast in slabs or in interlocking blocks; in mattress form by injecting it into man-made fibre quiltings. Concrete provides a smooth surface when placed in-situ or as pre-cast slabs and gives a very economical channel cross section. However, this form of construction is impermeable and inflexible and should be used with caution in ground conditions which give rise to settlement or uplift. Pre-cast blocks which may be keyed so that they interlock, or joined with steel pins or ties, provide a more flexible and semi permeable lining but a rougher surface. The injected mattress type will conform to any profile when initially placed, but is rigid thereafter. Some of these latter are semi-permeable, all are usually rough faced.
Asphalt, asphaltic concrete, sand asphalt and asphalt-mastic grouted stone are all impervious but flexible, and linings of them may rough or smooth faced. Membrane linings of butyl-

rubber, polyvinyl or polythene are completely impermeable and flexible, but have only a limited life due to their lack of resistance to puncturing and weathering.
Gabions (steel wire mesh boxes, filled with stones) are rough, permeable and flexible; Their weak point is that after damage to the weir mesh (by corrosion or vandalism) the structure looses it structural integrity.
From the variety of materials mentioned, the designer must choose one which has toe qualities of (im-)permeability, robustness, flexibility, durability and economy to suit his requirements.

2 Risk-Analysis

2.1 differences in risk for dikes and revetments

The function of a dike is to protect the land behind the dike against flooding. When the dikes does not function well, the result is a flood. This is a rare event, causing much damage.
A revetment has to protect a shoreline. In case of failure, there will be damage. Usually it is not too difficult to repair the damage, however it cost some money.
Because of these differences, the acceptable risk level for dikes and for revetments is completely different. Failure of a dike is only accepted with very small probabilities (less than once in every .... years). Damage to a revetment can be accepted with a much higher frequency. Every year some damage to a revetment does not cause serious problems, when this damage is repaired regularly by a good maintenance scheme. The acceptable damage level is in this case completely determined by the economic optimum between initial costs and maintenance costs.

2.2 socially acceptable risk-levels for inundation

The first problem in dike-design is to establish a frequency failure of the structure. It has to be decided how often flooding of an area is acceptable. This is not a problem of hydraulic engineering, but an economic and political problem. One can imagine that an optimum relation can be found between the cost of dike building and the value of the protected goods. (higher dikes costs more, but are economic when you have to protect more valuable infrastructure in the polder area). However this problem cannot be solved only in a monetary way. What is the economic value of a natural reserve, a pension of old-aged people, a museum of classical art ???. So in fact it is a mixture of politics and economics.
Thus the level of safety depends on the willingness of investing money in safety and (of course) on the availability of money.
Because in The Netherlands diking started by (small) private land-owners working together, they build up a tradition of investing in the property they used to work on. In the Netherlands we have hardly any absentee-landownership.
In principle there are several systems for handling the flood-problem:

- defend the area against flooding by a dike or levee system
- allow flooding, but ensure that the damages during flood are minimal
- reduce the peak discharge by constructing some retention basins upstream.

A method to increase your safety on the cost of your neighbour is making your dike 10 cm higher than the dike of your neighbour. In case of a flood, the dike of your neighbour will flood first, and consequently the level will drop somewhat. And so you are safe. This is called "overdiking". Already in the 17th century in the Netherlands we had law which made overdiking illegal.

The choice between these options is mainly a political one. In the United States of America in most cases the second option is selected, while in the Netherlands the first option is selected.

2.2.1 flood insurance and flood management in the U.S.A.

In the United States the *Federal Emergency Management Agency (FEMA)* administers the *National Flood Insurance Program (NFIP)*. Under this program everyone can insure their property by buying and insurance policy from the federal government. This flood insurance covers part of all costs of damage to properties until a certain maximum (for a single-family dwelling this maximum is for example US$ 185,000). For more details is referred to the general information brochure of Fema [FEMA, 1992].

Besides being responsible for the administration of the NFIP, FEMA sponsors other activities intended to reduce losses attributable to flooding. In pursuit of the latter goal FEMA has sought to:

- encourage wise land-use and watershed management practices;
- encourage better integration of natural and social systems;
- encourage appropriate design and construction practices in flood-prone areas.

The emphasis nowadays is both on structural control measures and on nonstructural strategies:

Structural control measures:

- dams and reservoirs
- levees and dikes
- channel improvements

Non structural strategies:

- Land-use planning
- Urban redevelopment and preservation
- Land acquisition in flood-prone areas
- Floodproofing
- Forecasting, warning and emergency preparedness
- post-disaster relief and rehabilitation

land use planning and management

The principal nonstructural strategy for reducing flood damage is to effect better use of water and land resources. This goal is achieved through comprehensive planning for and management of these resources throughout riverine watersheds and coastal environs.

Planning and management, as a strategy to reduce flood damage, addresses the critical need to better integrate the natural and built environments. This approach to the problems of floodings is based on the knowledge that, while floods cannot and should not be totally eliminated, the built environment can nevertheless be successfully developed if it respects the natural system. Planning and management, in practice are base on compiling technical data on topography, drainage, soil composition, climate and other natural characteristics and analyzing it in light of the physical, social and economic aspects of the built environment. This analysis is then used to determine appropriate locations for both the encouragement and prohibition of building. Implementation then relies on regulations, such as zoning ordinances, subdivision regulations, and health and building codes, or on incentives that induce positive development practices. Floodplain management objectives can also be realized in conjunction with programs for urban revitalization and preservation, or through land acquisition by public bodies to control development.

Urban redevelopment and preservation

Renewal of cities is by nature a continuous process. It offers the opportunity to rectify many of the earlier development practices that have contributed to flood damage. In some cases land that is particularly vulnerable to flooding can be cleared by "down-zoning" it to open space uses. In many cases, however, economic constraints or the historical significance of a building or district make this impossible.

When renewal is to occur, design and development can make use of site design and floodproofing strategies to lessen the impact of flooding. Obviously, there is more latitude when working with a cleared site, but these strategies can also be applied to existing buildings.

Historical preservation is often a high priority in rehabilitation. Schemes to preserve important cultural artifacts require careful and creative use of damage reduction strategies to make a building safer from flood damage while respecting the integrity of the original design.

Acquisition and relocation

In many flood-prone areas existing development suffers repeated damage. Often such locations can be protected only by removing development, but this can rarely be accomplished without public ownership of the land. Public ownership is, likewise, the surest way to protect vacant land that is subject to development pressure.

A growing number of public bodies recognize the desirability of both acquiring such hazardous sites, either trough negotiation or eminent domain, and relocating existing uses to safer sites. This strategy mitigates recurring losses, helps to restore natural processes in the floodplain, and promotes open space uses such as agriculture of recreation.

Floodproofing

Despite floodplain management and related programs to remove structures from hazardous locations, buildings will inevitably continue to be located in such areas. It is necessary that these buildings be protected from flood damage. Floodproofing, working in conjunction with floodplain management, provides this kind of protection.

socially acceptable risk level

Floodproofing encompasses any technique intended to protect buildings from flooding, and typically includes elevating buildings above the flood hazard level, providing watertight closures for doors and windows, and using floodwalls around ground level openings or, alternatively, eliminating such openings. Also included are the use of water resistant materials, structural reinforcement to withstand water pressures and placement of mechanical elements in the upper parts of buildings. Floodproofing is applicable to historic buildings, to essential uses that are not suitable for alternative locations, and to areas in which the capital investments in the existing urban infrastructure requires continued occupation of a hazardous location. In these situations floodproofing can be indispensable. Floodproofing is especially suitable where moderate flooding with low stage, low velocity, and short duration is expected.

Forecasting, warning, and emergency preparedness
Forecasting, warning and emergency preparedness measures are integral parts of a well-balanced floodplain management system. For example adequate warning allows for the preparation of temporary floodproofing closures and the evacuation of people and building contents from the hazardous locations. This is, in part, a technical issue of concern to meteorologists and hydrologist and, in part, and administrative issue requiring a system of emergency planning, organization, communication, and public education.

Relief and rehabilitation
Relief and rehabilitation are, in first instance, not methods of reducing flood damage, but ways of dealing with damage after other measures have been insufficient. Relief and rehabilitation assistance can include direct clean-up operations as well as loans, grants, and tax reductions to facilitate rebuilding and relocating where necessary. Federal agencies are the primary source of this aid, with private support available from organizations such as the Red Cross.
Rehabilitation can provide important damage mitigation opportunities. When rehabilitation is necessitated by flood disaster, future flood losses can be reduced by ameliorating many of the problems that contributed to destruction. There are often strong local pressures to rebuild as quickly as possible, particularly where economic livelihood is involved. And such pressures are justified. Yet, just as often, there are long term economic and social reasons for breaking the cycle of repeated destruction and ensuring that earlier development mistakes are not duplicated.
Post-disaster rehabilitation is most effective if it responds to the needs of local residents but minimises future destruction. This requires that redevelopment proceed according to sound principles of floodplain management, taking advantage of the various methods for reducing flood damage that apply to new development.

Information on maps
In order to become eligible for Federal flood insurance, a number of basic maps has to be available. For the compilation of these maps, detailed surveys, funded by FEMA, have to be made. These studies identify pertinent information, such as base flood elevations, areas inundated by various magnitudes of flooding, floodway boundaries and coastal high hazard areas. The information is provided in the form of *Flood Insurance Rate Maps (FIRMs), Flood Boundary Maps* and *Floodway Maps*. Buildings in floodprone areas have to be constructed according to the design manuals provided by Fema [FEMA54, FEMA55, FEMA 114]. Also manuals are available of the planning these areas [FEMA15, FEMA 116].

2.3 determination of the design values

After having determined the risk level (and thus the allowable probability of failure), one can determine the strength of a structure and the design load on the structure. Unfortunately it is very difficult to determine the actual strength of an hydraulic structure. The only thing we can say is that we design a structure in such a way that under a given design load the probability of failure is very low. Around 1950 is was impossible to calculate the probability of failure of a structure under a given design load. In the following sections an overview is given of the history of the design values in the Netherlands.

2.3.1 the 1953 storm surge disaster

On 1 february 1953 it stormed. The water-level raised to a level of 0.6 m higher than the highest observed storm surge (of 1894), with as consequence that 1365 km^2 was inundated and 1835 people were killed. 47300 houses were damaged, the total damage to real estate was 160 million guilders (in 1953 1 U.S. $ was approx. 3.60 Dutch guilders). In comparison, in 1916 6874 km^2 and in 1894 306 km^2 was inundated. Because most of this area was also below mean sea level, after passage of the storm, the polders were still covered with water. Repairing 160 km of dike took more than a year and it cost 380 million guilders. The total direct costs of the disaster were 2000 million guilders, which was 14 % of the gross national product in 1952. The main conclusion was: this should never happen again, and a committee of specialists (the Delta-committee) was installed to find a solution

2.3.2 the solution of the Delta-committee

To overcome the problem that it was in those years impossible to calculate the inundation probability, it was decided to design dikes with a design load with a given probability of occurrence. Under this design load the construction has to be so strong that the probability of failure is extremely low.

Practically this means for the central part of the Netherlands that the design water level is the 1/10,000 years level. So we design a dike with a design water-level with a probability of 1/10,000 per year. This water-level is determined using extreme value statistics. Together with this design waterlevel we determine the accompanying other boundary conditions (like the wave-height, etc).

Then a dike is designed which is able to withstand these design conditions with a very low probability of failure. That means that it has to be fully stable. It has to have enough strength. Thus: geotechnically stable, no wave-overtopping, no erosion of the slope protection etc.

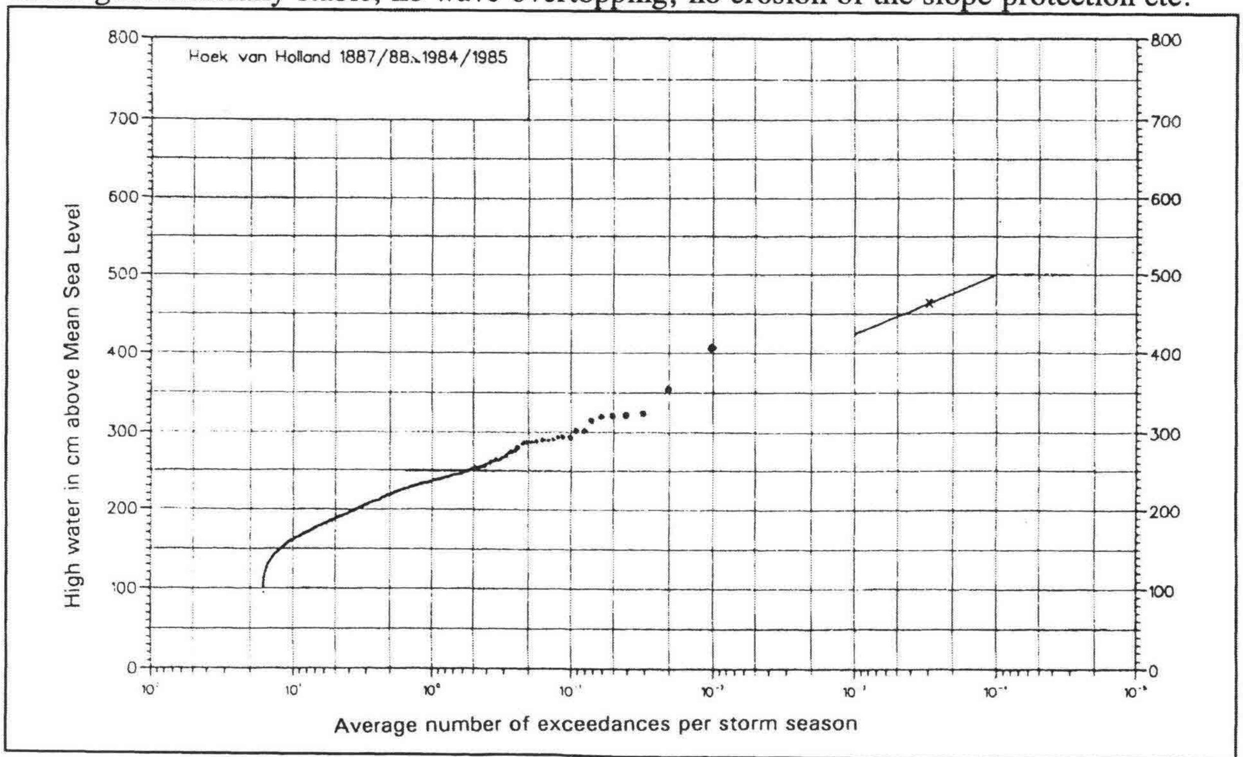


Figure 2.1: exceedance diagram for waterlevels at Hook of Holland

design values

In the previous figure the exceedance line of the waterlevels in Hook of Holland is presented. This line is the basic exceedance line in the Netherlands. All other stations along the coast are adapted from this line. In Hook of Holland the 1/10,000 waterlevel is 5.00 m above National Datum (N.A.P), which is approx. Mean Sea Level.

Using Hook of Holland as a basis, the 1/10,000 levels for other places along the Dutch coast can be determined. At other places the 1/10,000 level is quite different, because the both the tidal amplitude and the storm surge set-up is different.
An other criterion of the Delta-committee was that not every place in the Netherlands needed

Figure 2.2: Variation of design levels along the Dutch coast

to be protected against a 1/10,000 level. Places where less people live, or with less economic activity, may use a lower design value. For the south-western and northern part of the country a level of 1/4000 is adopted. The figure below gives an impression of the variation of the design levels along the Dutch coast.
This design method is common practice in the Netherlands since 1960, when the report of the so-called "Delta-committee" was published.

However, this method regards only the load on the dike. It is assumed that the dike will not fail under design conditions. Therefore the Delta-committee added some design rules. They advised that a dike should be designed in such a way that every cross-section can withstand this water-level, with the accompanying wave-run up in such a way that no serious damage to the dike will occur. The number of overtopping waves should be less than 2 %.

2.3.3 solution of the "Becht-committee" for river dikes

Around 1975 it was realized that also river dikes had to be improved, in order to guarantee an identical safety to the people living behind river dikes. The "Becht-committee" was formed to determine the height of river dikes. This committee concluded that inundation by rivers is less serious than inundation by sea water. This is caused by the fact that:

* inundation with fresh water causes less problems than inundation with salt water;
* the warning time for high-water run-off is longer than for a storm surge from the sea;
* the polders along the rivers are mainly above the normal water level, and consequently the water flows out of the polder after passing of the high-water in the river
* there is no tidal flow through the gap in the dike, and repair is therefore more easy.

Based upon these considerations the Becht committee decided that river dikes should be designed on a run-off with a probability of occurrence of 1/1250 per year. The "Becht-committee" was influenced by the fact that the public dit not like the works for raising dike-levels. In 1993 this work was reviewed by a new political commission [BOERTIEN, 1993], but this commission did not change the selected value of 1/1250.

2.4 elements of risk-analysis and probabilistic considerations

2.4.1 risk analysis and fault trees

The study of structural safety centres on the concept of failure and collapse. Although two terms are commonly used as having almost identical meanings, it is useful to draw a clear distinction.
A structure fails if it can no longer perform one of its principal functions. In the case of a dike (or other flood defence structure) this function is, in general, the prevention of inundation, i.e., preventing a protected region from being flooded, attended by loss of human life and/or damage to property.
A structure or a structural component collapses if it undergoes deformations of such magnitude that the original geometry and integrity are lost. In general, collapse will be attended by a greatly increased probability of failure. It is, however, quite conceivable that collapse occurs but not failure, e.g., slip affecting a dike during a long period of low water level. The opposite may occur in the event of overtopping: the dike fails, but does not collapse.

The word "risk" is sometimes used in various definitions:

"risk 1"	=	probability of an unwanted event
"risk 2"	=	consequences of an unwanted event
"risk 3"	=	the product of probability and consequences of an unwanted event
"risk 4"	=	risk 3 to the power N, in which N is the number of events per year.

In this lecture notes (and usually in risk-analysis) the third definition of risk is used.

Problems with the definition of risk

The purpose of the design of a flood defence structure is to obtain a structure which, during its construction ant throughout its intended service life, has sufficiently low probability of failure and of collapse. In order to achieve the best possible assessment of this, a risk analysis is performed (see figure below). The three elements of the risk analysis are: hazard - mechanisms - consequence.
A risk begins with the preparation of an inventory of the hazard and mechanisms. A mechanism is defined as the manner in which the structure responds to hazards. A combination of hazards and mechanisms leads, with a particular probability, to failure or collapse of the flood defence structure or of its component parts.
The boundary between failure and non-failure, or between collapse and non-collapse is generally called limit-state. A distinction is to be drawn between ultimate limit state (ULS) (failure or collapse with regard to the principal functions) and serviceability limit state (SLS) (failure with regard to other functions).
Finally, the consequences of failure or collapse must be considered. In the event of failure of the flood defence structure as a whole, the relevant inundation characteristics (inundation depth, inundation speed) must be ascertained and the material damage and non-material damage loss be estimated. The probability of failure multiplied by the damage or loss (=consequence) constitutes the risk. For optimal design it is essential to seek an appraisal in the sense of weighing the risk, on one hand, against the cost of constructing a flood defence structure, in the other.
In assessing the safety of flood defence structures it is very important to consider the system as a whole. Structures are composed of many components, each of which may be prone to many hazards and mechanisms. Collapse of component A may in turn pose a hazard to component B. The failure of some components may lead directly to failure of the system ("series connection"); in other cases components may compensate for one another ("parallel connection"). A useful aid to establish an ordered pattern in the many hazards, mechanisms and components is provided by diagrams such as fault trees and event trees.

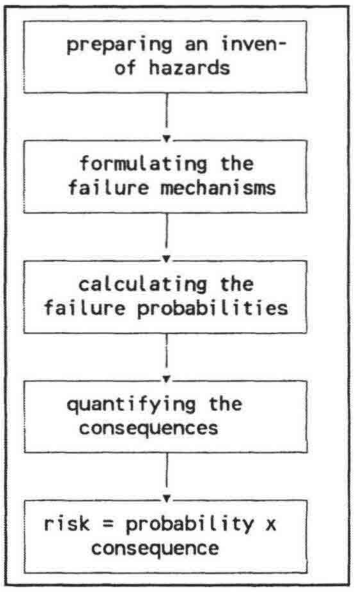
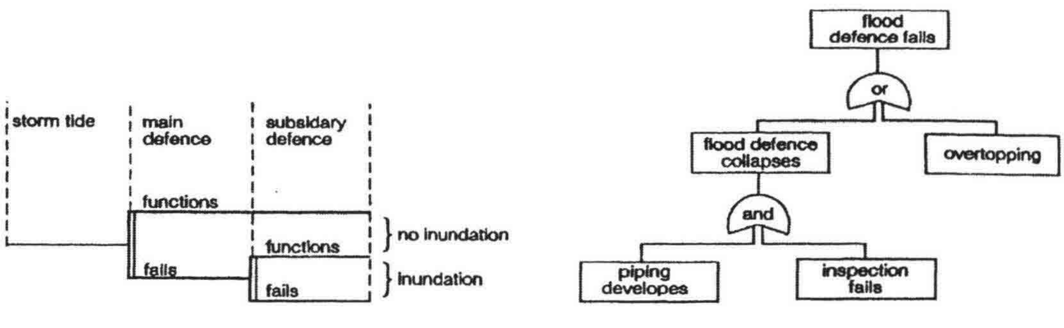


Figure 2.3: elements of the risk-analysis



a. Event tree

b. Fault tree

Figure 2.4: examples of an event tree and a fault tree

In case of an event tree, the procedure of going form an undesirable initial event (failure of a component, fire, human error) to the responses of the system and the consequences. A fault tree is based on the opposite procedure: starting from an undesirable event, it is analyzed how this may have been caused. In drawing a fault tree, symbols such as AND-gates and OR-gates are used. The AND-gate corresponds to the parallel system and the OR-gate to the series system. On the next page a fault tree for a dike or other flood defence system in general is given. For typical structures, like a lock in the dike, the problem is in general mot the structural failure, but the problem that the lock is not closed at the moment of occurrence of the flood. This can also be worked out in a fault-tree. As an example a fault-tree for the closure of the lock in the sea-dike near Vlaardingen is given. In this tree a single set of gates and one operating team is assumed.

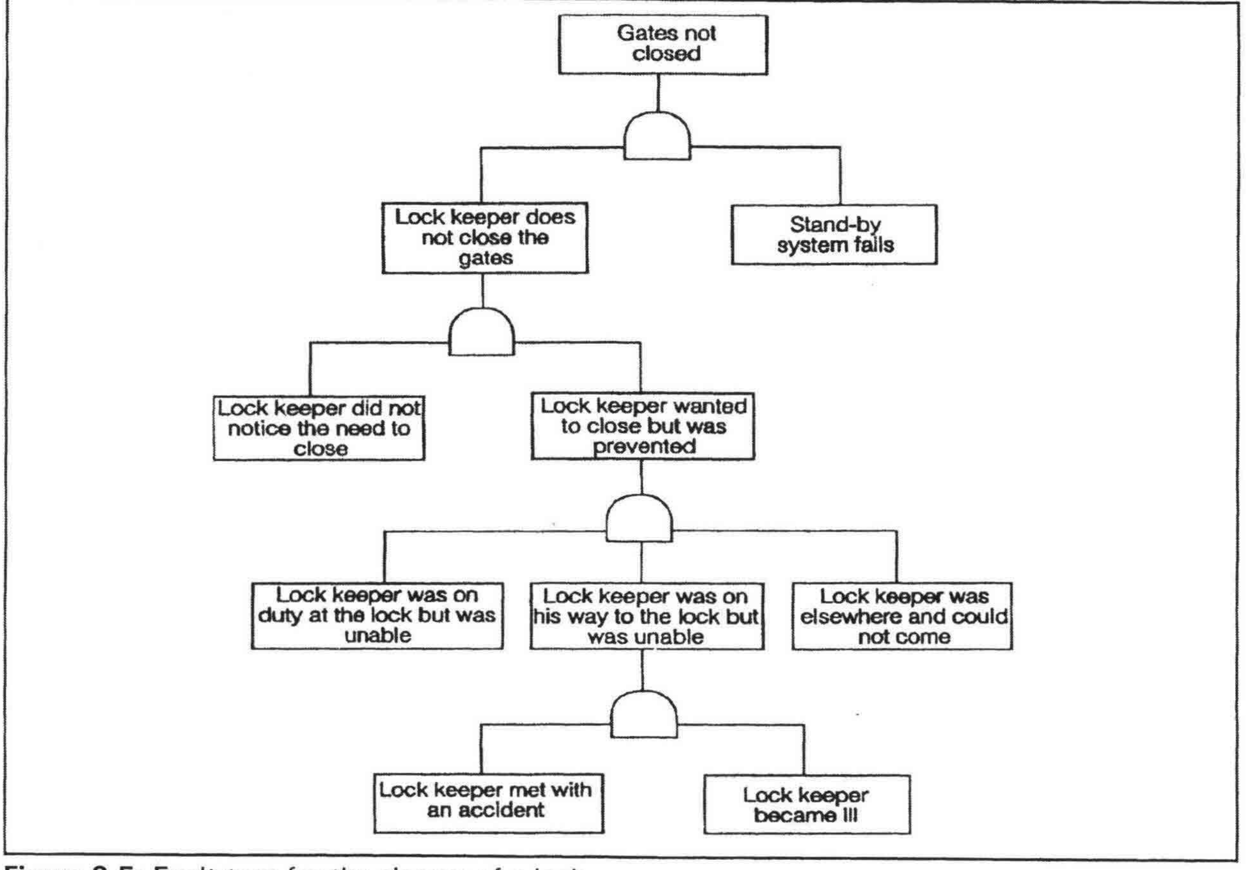


Figure 2.5: Fault tree for the closure of a lock

The drawback of event trees and fault trees is that they are rather strictly regulated. In an event tree it is, in principle, not permissible to combine branches, and in a fault tree, no dividing of branches is possible. Furthermore, the system is essential binary in character: an event occurs or it does not. In civil engineering, however, problems of a more

Design Philosophy

* A dike should prevent inundation
* Inundation is the unwanted event
* Inundation can be caused by several failure mechanisms
* All failure mechanisms are combined to a fault tree
* For every mechanism the probability of occurrence can be determined
* Using the fault tree the probability of the main event can be determined
* The main event should occur with a probability less than a given value. This value is a political choice

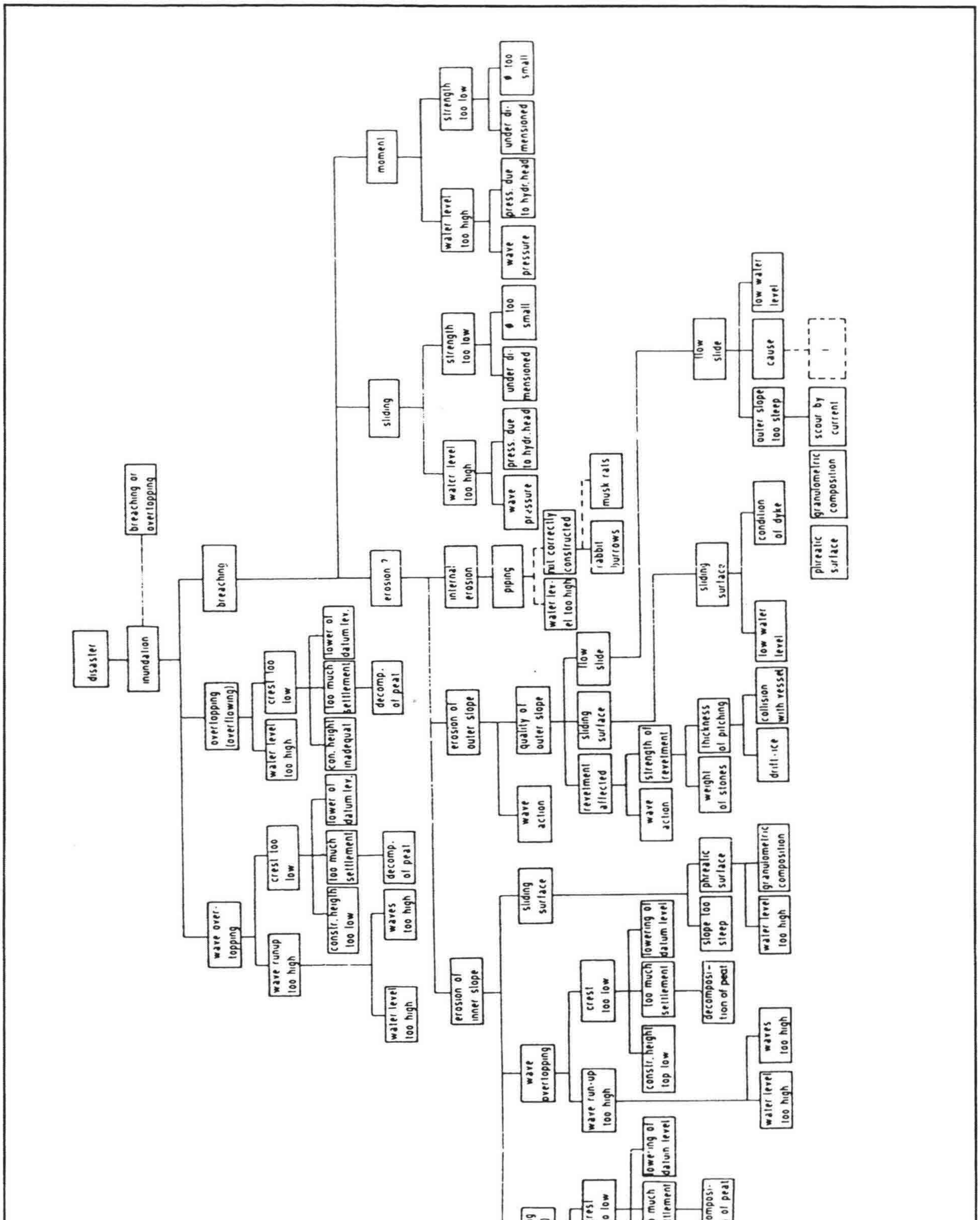


Figure 2.6: fault tree of a dike

continuous character are of fairly frequent occurrence. An alternative to event trees and fault trees which overcomes the said objections is provided by so-called cause-consequence-charts.

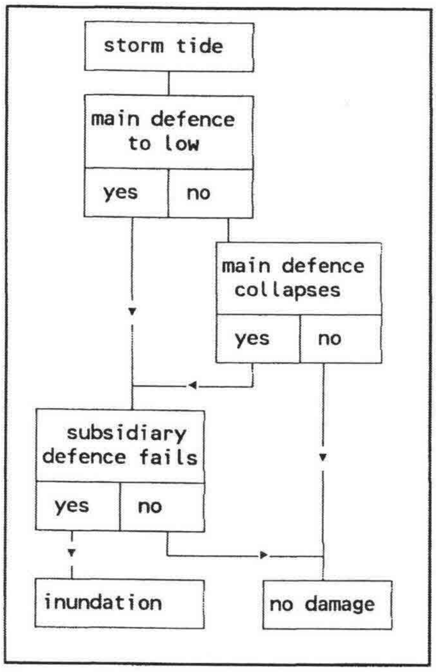


Figure 2.7: simple example of a cause-consequence-chart

It is noted that all the above mentioned techniques have a recording than rather a generating function. Conceiving what can go wrong, and how it can go wrong, remains the designer's responsibility - and a very important one. It is often considered that thinking of hazard or a mechanism is of greater importance that the whole analysis that follows.

Aids in preparing an inventory of causes of failure are:
* data-banks
* literature studies
* interviews
* study of actual instances of damage
* brainstorm sessions
* experience with similar structures

and so on. For commonly encountered structures most hazards are recorded in guidelines and manuals. A general problem is that mostly people are not willing on publishing data on failures, especially not when it are failures in their own jurisdiction. This is however, a fully misplaced feeling of honour. In principle, there are two approaches in ascertaining the probability of failure due to a particular mechanism. One approach is to make a direct estimate of the probability on basis of experience and intuition. Alternatively, a probabilistic calculation may be performed.

2.4.2 probabilistic considerations

For a probabilistic calculation it is necessary to have a computational model of the mechanism. On the basis of that model a so-called reliability function Z is established with regard to the limit state considered, in such a way that negative values of Z correspond to failure and positive values to non-failure.

Summarized: **Z = Strength - Load.**

The probability of failure can thus be represented symbolically as $P\{Z<0\}$. For simple processes (like the collapse of an overloaded plank over a ditch) the Z-function can be described easily. For the collapse and failure of a dike, this Z-function is very complicated, mainly because of the complicated interaction between water, soil, revetment, etc.
In these lecture notes therefore the probabilistic calculations will not be discussed. Referred is to the lectures on probabilism.

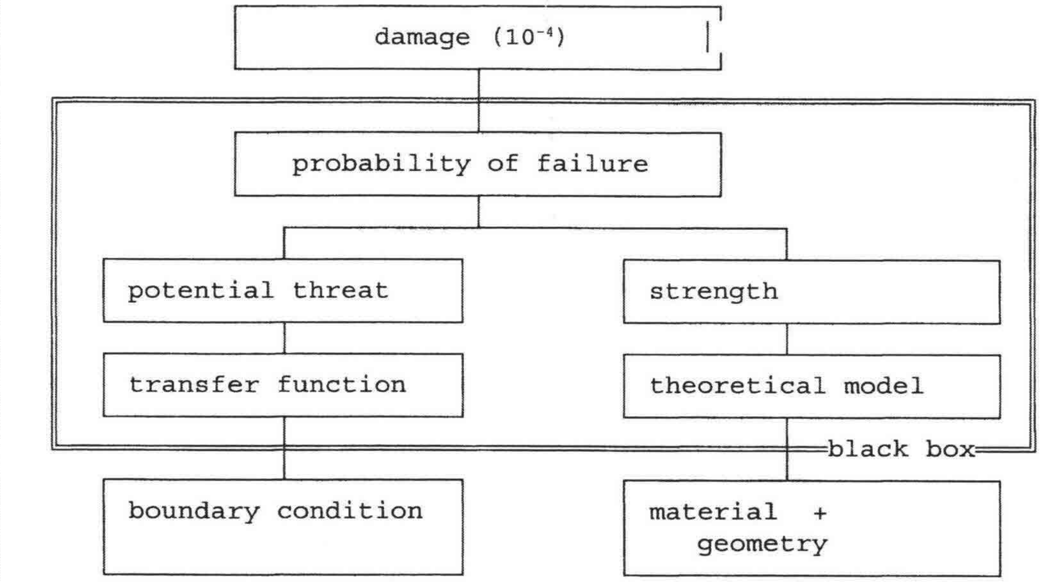
2.4.3 present status

In order to calculate a construction, three approaches were discussed in the above sections:
* deterministic approach.
  One calculates the average situation and add a appropriate safety factor.

- semi-probabilistic approach
  In calculations a characteristic value is used (for example the load which is not exceeded in 95 % of the cases, or the strength which available in 95 % of the construction material).
- probabilistic approach
  Take into consideration the full statistical distribution of all parameters.

At this moment scientific developments are going on in the probabilistic techniques. In this field nowadays a lot of research is done. Practical design at this moment is in general done using semi-probabilistic methods. In many cases even a lot is designed using a full deterministic approach.

As discussed above, in the Netherlands the design of a sea-defense was in all cases based upon a characteristic load (water-level and waves) with a defined probability of occurrence. The fact that there is also a variation in this load, as well as a variation in the strength of the sea-defense was neglected. Studies are performed in order to find a probabilistic method in which also these variations could be taken into account. With a probabilistic approach it is possible to achieve this, but for dikes this is still quite complicated.



general overview of calculation

For breakwaters a probabilistic calculation is quite possible at this moment (see "The use of rock in coastal and shoreline protection", page 47). Research is still going on in this field.
In the scheme above general overview of design calculations is presented. In a deterministic approach the "Black Box" is usually described very simple, with straightforward parameters. In a probabilistic approach for every step in the process the full statistical description is taken into account (variance in input data, variance in transfer functions, variance in models of strength, variance in material characteristics). A simple example is the wave run-up formula $R = 8\ H \tan \alpha$. In this formula H is a boundary condition, and α is parameter of the geometry. The whole Black Box is summarised in the above scheme.

The probabilistic approach requires much knowledge and much work.
Therefore often simpler methods are used, applying design rules.
A design rule is made in such a way that the result is a construction with an acceptable probability of failure, without quantifying that probability.
Mostly a construction is designed on one failure mechanism only, a margin is added, and it is supposed that the effects of other mechanisms are included in that margin.

In fact the approach of the Delta committee was that they concluded from a fault tree that there are 3 major failures for a dike:

* overflow
* damage due to overtopping waves
* damage due to geotechnical failure

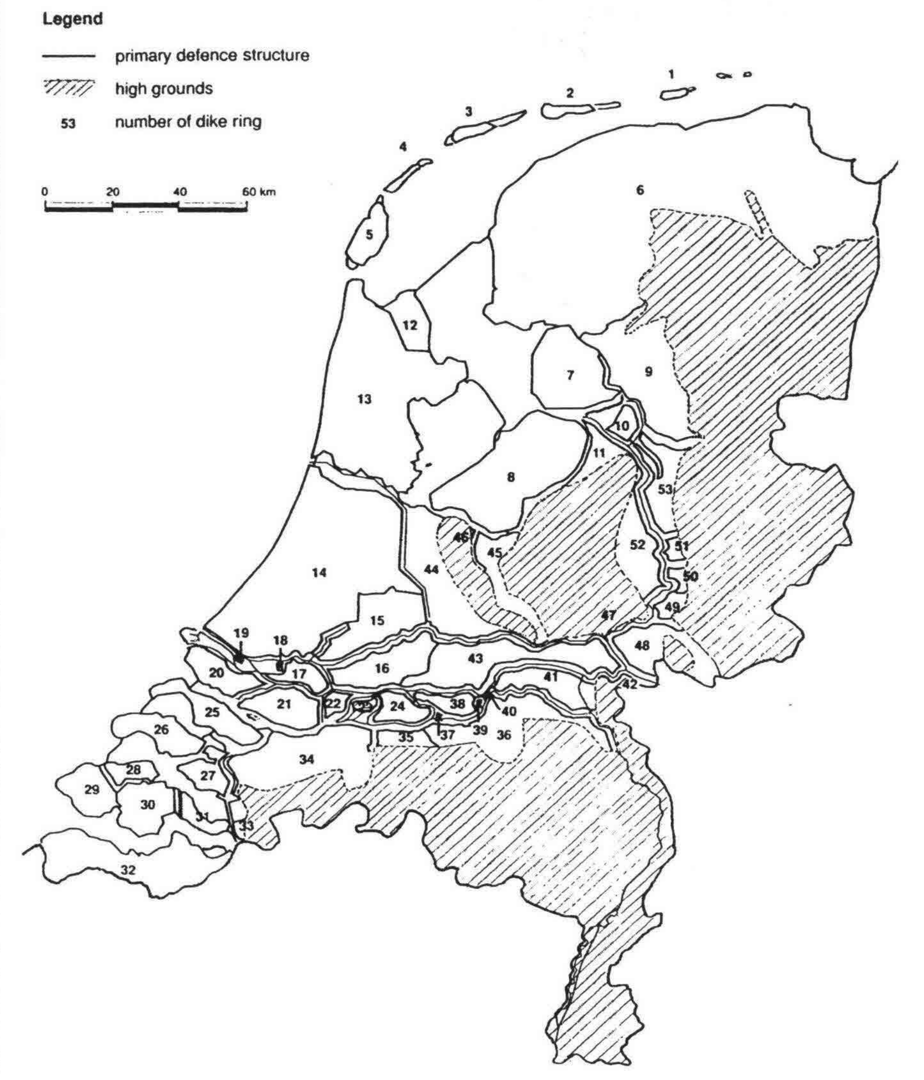
The probability of the last two items could not be determined at that time. But by over-dimensioning the dike, one could guarantee that the probability of occurrence of this damage was less than 1 % of the probability of overflow. This means that the total probability of failure of a dike is $10^{-4} + (<10^{-6}) + (<10^{-6}) = 10^{-4}$

2.4.4 the dike-circle

Using the probabilistic considerations, mentioned above gives a probability of failure of dike section. In fact a dike has also a given length. The inhabitant of a polder area it is not interested what is the probability of failure of a give dike section, but what is the probability of getting wet feet or worse. The low lying part of the Netherlands is divided in so-called dike-circles. A dike-circle is a low-lying area, which is surrounded by dikes, dunes and/or high grounds. Failure of one section of the sea-defense usually results in the inundation of a whole dike-circle. Each dike circle has a given allowable probability of inundation. In the following figure the area of the 40 dike-circles of the Netherlands are presented with the allowable inundation frequencies Te allowable inundation frequencies are laid down in the Law on Sea Defense. The choice of an allowable inundation frequency is fundamentally a political decision.

More recent studies are performed to find the optimal allowable inundation frequency, based upon the economic value of real estate and infrastructure in the dike circle. However, the values of human life, natural environment, historical and cultural values, etc. made it impossible to define the optimal value in an objective way.
It is interesting to mention that the results of these studies indicate that the economically optimal values are in the order of 10^{-4} to 10^{-5} per year. The frequencies proposed in the new Law on Sea Defense are a factor 10 higher (order 10^{-3} to 10^{-4}).

In this Law it is also stated that the boundary values (such as water-levels) have to be recalculated every five year, and that dike managing authorities have to certify every five years that their dike still fulfils the requirements. So, they have to check the height of the dike, the quality of the slope protection, etc. In this way it its tried to prevent that the effect of climatic changes causes surprises. The dikes have to be adapted to the new situations regularly. This is the main reason that design water-levels, etc., are not given in the regulations, but only their probabilities.



dike-ring	safety-level
1	1/2000
2	1/2000
3	1/2000
4	1/2000
5	1/4000
6	1/4000
7	1/4000
8	1/4000
9	1/1250
10	1/2000
11	1/2000
12	1/4000
13	1/10000
14	1/10000
15	1/2000
16	1/2000
17	1/4000
18	1/10000
19	1/10000
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35	1/2000
36	1/1250
37	1/1250
38	1/1250
39	1/1250
40	1/1250
41	1/1250
42	1/1250
43	1/1250
44	1/1250
45	1/1250
46	1/1250
47	1/1250
48	1/1250
49	1/1250
50	1/1250
51	1/1250
52	1/1250
53	1/1250

Figure 2.8: Safety levels in the Netherlands

2.5 basic difference with other type of constructions

All types of constructions will have to withstand certain design loads. In case of a dike the probability that the construction will meet its design load is quite low. The probability that a 1/10,000 dike will meet its design load in a lifetime of 100 years is only 1 %.
This makes that you have to be very careful with the design elements. For example, for the stability of the inner slope of a dike, one should prevent water flowing out of the slope. This can be achieved in two ways:

* by making a drain in the dike
* by making a low berm along the dike

Both solutions are given in the following figure.

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basic differences

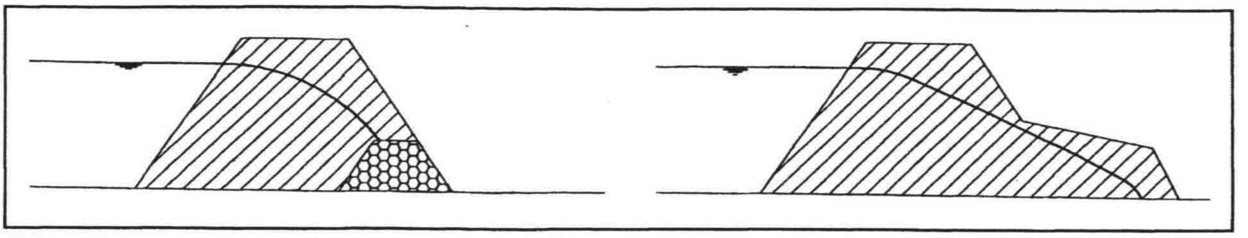


Figure 2.9: two alternatives to prevent outflow of water from the toe of the dike

In theory both solutions function very well. However, one has to be sure that the drain is not clogged. And there is no easy way to check that, because the drain functions only during extreme outside waterlevels. And they occur, by definition, very seldom. Also in 50 or 75 years no-one knows that there is a drain in the dike, and that functioning of this drain is essential during design conditions.
Therefore in dike-design one has always to choose for the simplest solution, for the solution requiring less maintenance, prevent hidden construction-elements and use materials which cannot deteriorate.

2.6 investing money

Building dikes is a costly business. The total value of the dikes (3700 km) and related structures in the Netherlands is 26 billion guilders. At this moment we spend approx. 300 million guilders per year for dike-improvement and related works. Maintenance costs of dikes are in the order of 85 million guilders per year. Maintenance of the sandy coast costs 45 million guilders.

extract from the Dutch National Budget, 1991

dike maintenance	85 million	
coastal maintenance	45 million	
artificial beach nourishment	80 million	
total maintenance		210 million
improvement of dikes in tidal regions	155 million	
Rotterdam storm surge barrier	105 million	
improvement of river dikes	55 million	
total improvement works		315 million
contract research		15 million
total cost flood control		540 million

The above figures are including the overhead from the personnel costs of the ministry of public works and are also including the expenditures made by lower authorities.

extract from the national budget of 1991

From these figures follows that maintenance of 1 m of river dike costs 25 guilders; 1 m of sea-dike costs 85 guilders and 1 m of sandy coast costs 150 guilders. The last figure is not including artificial beach nourishment. Because you have always to improve your dike system a yearly investment in "new" dikes of in the order of 100 million guilders is the minimum. This means that the flood defence system costs the inhabitants of the Netherlands approx. 300 million guilders per year after completion of the present improvement works.
Insurance premium for fire-insurance is in the order of 1.5 o/oo of the value of the assets to be protected. The value of the real estate protected by dikes in the Netherlands is in the order of 500 billion guilders. When there is a fire, also production stops. This can also be insured for approx. 1.5 o/oo of the value of the production. The yearly production in this area is in the order of 650 billion. So the total insurance premium to insure the lower part of the Netherlands against fire (1.5 o/oo insurance premium) is approx. 750 million guilders.
For a risk, comparable to flooding, like burning down, we are prepared to pay yearly 750 million guilders. To prevent the risk of flooding we pay only 300 million guilders. From these figures one has to conclude that flood defence is very cheap !!!
(By the way, we spend yearly 14206 million guilders on military defence.)

2.7 practical values

The values of 1/10,000 etc. are general values in the Netherlands. It is quite doubtful that in the rest of the world identical values have to be used. As indicated above, these values depend on the economic value of the activities in the area, the social acceptance of flooding, the amount of money people want to spent on flood protection, etc.

In general one can state that high-yield agricultural area should not be flooded too frequently, especially not by salt water. It also depends on the time of the year when flooding usually occurs. For high-yield agricultural areas which might be flooded by fresh water one should think of a flooding frequency of once every 10 years. When high investments have been done in irrigation and drainage, one might decrease the frequency to 1/25.
For salt-water flooding one should use frequencies in the order 1/50 to 1/100.

For individual houses a value of 1/50 - 1/100 is a good guideline. In this case is assumed that the flooding causes considerable damage to the construction of the house. If the flooding only causes some water in the house, the frequency might be in the order of 1/10.
For a complete village the values for damage-causing floods should be in the order of 1/500.
For big cities, industrial areas and areas vital for the functioning of the country (airports, railway stations, etc) the frequency of a damage causing flood should be in the order of 1/1000.

In the above it is assumed that the dikes only protect against a flood, and that after the flood or storm surge the water flows away naturally from the area. If the dikes protect a polder area with a surface level below normal high tide, then a tidal flow will remain in the dike breach, even after the storm surge. In that case the frequencies should be at least a factor 10 less.

Of course the above values are only first estimates. In many cases the selected safety will be a factor 10 more or less.

In any case one should compare the risk of flooding with the other risks in the area (problems with chemical industries, earthquakes, volcanos, etc).

In some cases a practical problem may arise with flooding frequencies in the range between 1/10 and 1/250. This return period is to short to regard this as "protected" by dikes, on the other hand the period is too short to maintain alertness by the inhabitants. In such areas the inhabitants develop a misplaced feeling that they are safe. In case of a flooding, they start complaining. People do not realise any more that they live in a vulnerable area.
Therefore it is advisable to avoid flooding frequencies for build-up areas between 1/10 and 1/250.

In Great-Britain a method is developed to calculate the return period, by calculating the "house equivalent" (HE). In terms of potential flooding damage, which is taken as a measure of the value of the asses at risk, an HE is defined as *the average annual cost of flooding damage suffered by an average house which is at risk of flooding*. For comparison other land use than houses, the following table is used:

Land use Features	Unit	House Equivalent
House	Number	1.0
Garden/allotments	Number	0.04
NRP - Manufacturing	Area (m^2)	0.030
NRP - Distribution	Area (m^2)	0.054
NRP - Leisure	Area (m^2)	0.032
NRP - Offices	Area (m^2)	0.033
NRP - Retail	Area (m^2)	0.035
NRP - Agricultural	Area (m^2)	0.01
C roads	Number	2.7
B roads	Number	6.3
A roads (non-trunk)	Number	15.9
A roads (trunk)	Number	31.7
Motorway	Number	63.5
Railway	Number	63.5
Forestry and scrub	Area (100ha, 0.01 km^2)	0.02
Extensive pasture	Area (100ha, 0.01 km^2)	1.3
Intensive pasture	Area (100ha, 0.01 km^2)	3.0
Extensive arable	Area (100ha, 0.01 km^2)	6.3
Intensive arable	Area (100ha, 0.01 km^2)	44.1
Formal parks	Number	0.6
Golf/race courses	Number	0.7
Playing fields	Number	0.1
Special parks	Number	9.2

NRP - Non-resident property

risk-analysis

All areas are divided into several Land Use Bands, each may have its own standard of protection.
The standard given in the following table is the possible target minimum standard (Return period in years) for tidal and sea defences.

Land Use Band	Range of house equivalents/km	Standard
A	>50	200
B	25 - 50	100
C	5 - 25	50
D	1.25 - 5	20
E	0.01 - 1.25	-

For many (especially) urbanised areas, this leads to low standards, because the standard is never more than 200. For rural areas, this can be a very attractive approach, however the given values for House Equivalents and the standard have to be adapted to local circumstances in every country.

3 Nomenclature

3.1 sea-dikes

The main characteristic of a sea dike is that it is attacked by varying waterlevels and a considerable wave attack. The period of high water is relatively short (a number of hours), and the water will rise very fast. The prediction of a storm surge is very difficult and only possible on short terms (on a time scale of hours). In general there are no big problems from a geotechnical point of view.
Therefore in general a sea dike has a relatively gentle outer slope and a berm on storm surge level (both to decrease the wave run-up) and a heavy revetment. A revetment is in general present at the height were the waves attack under normal conditions (to prevent erosion under normal conditions) and at the height where the waves attack during design conditions.
The inner slope of a sea-dike is in general steep. For practical reasons, slopes are designed not steeper as 1:2.5, because of problems with maintenance (on a 1:2.5 slope it is possible to walk and to mow the grass)

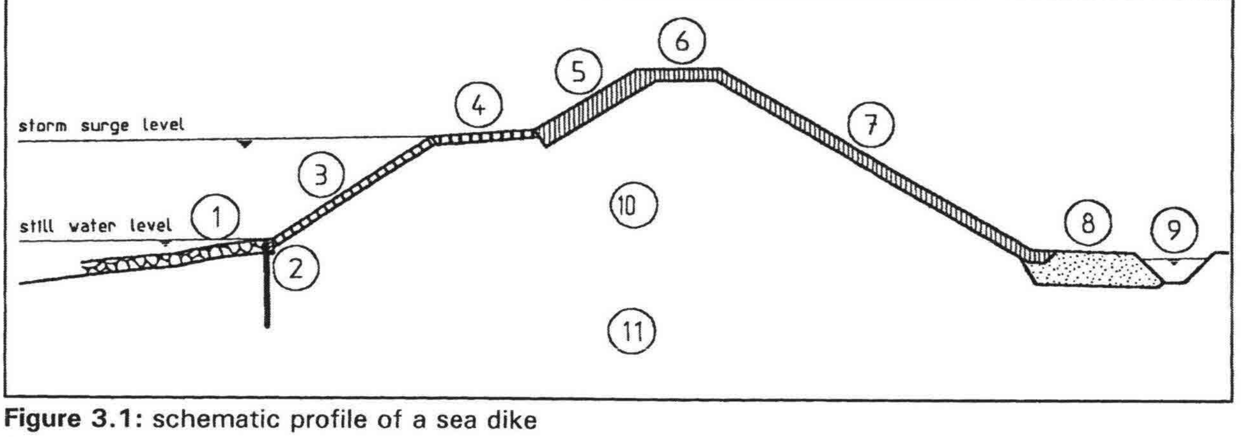


Figure 3.1: schematic profile of a sea dike

In the above figure one can distinguish the following elements:

1. <u>The toe-protection.</u> This protection is especially for situations during low water, so for the serviceability limit state. During a storm surge the waterlevel is much higher and waves will not attack the toe. A second important function of the toe-protection is to prevent scouring by currents (and waves) just in front of the dike. Very often the stones used for the toe protection are stones from old revetments. Between the stones and the subsoil some layer is in general required. This may be a geotextile. However it is very difficult to place such a geotextile without any ballast. Therefore very often a fascine mattress made of brushwood of bamboo is used. Also a flexible slab of bituminous material can be used (like open stone asphalt [Fixstone^{TM}] or asphaltic mastic.). In case of bituminous material one needs a gap, acting as valve to get rid of overpressure of water. Also some connecting construction is required to prevent the slab from sliding down into the river or tidal channel.
2. <u>The toe construction.</u> The toe is the foundation of the revetment. In general it is a row of wooden piles. Sometimes a concrete sheetpile is used. In general experience shows

that impregnated wooden piles or hardwood is very fitted for this function, so there is no need to use concrete or steel sheetpile.

3. The revetment. In the zone were wave attack is expected the slope needs protection. This can in the form of natural stone, concrete blocks, asphalt, concrete, and in cases where wave action is minor, even a grass cover is sufficient.
   The outer slope of a dike should be gentle in order to decrease wave run-up. However, a gentle slope require more space. For the Netherlands the optimum outer slope is in the order of 1:3.5. Outer slopes steeper than 1:3 are not recommended.
4. The berm. The main purpose of the berm is to decrease the wave run-up. But the berm is also very important for maintaining the revetment. Therefore it is in general not narrower than 5 m, and covered with asphalt or concrete blocks, to allow driving with trucks for maintenance. Experience has shown that a revetment with a length (measured along the slope) of more than 20 m is not serviceable. Making a berm in the middle improves the serviceability considerably. The slope of the berm is very gentle, in the order of 1:20 - 1:50. A horizontal slope is not advisable, because than water will remain on the berm.
   For decreasing wave run-up the berm should be designed approximately at design water level.
5. The upper outer slope. This section of the slope is in general only attacked by waves during design situations, and does not suffer from the wave impact, only from the wave run-up. Therefore a grass cover is in general sufficient. This crass cover should be maintained carefully.
   In the Netherlands it is necessary to have a clay-layer of approx. 0.8 m thickness under the clay in order to have a good grass cover (when the layer is thinner, the clay will dry out and the grass will die). In case of a design storm the waves will erode the clay slowly. A layer of 0.8 m clay is supposed to survive the storm (it is expected that after the design storm, so as an average, every 4000 - 10000 years, considerable repairs to the clay layer are required).
6. The crest. From a theoretical point of view the crest-width might be zero. Practically is a minimum of 2.5 m, otherwise the execution will be very difficult. The crest should be fully covered with clay or with the pavement of a road (the crest should be impermeable). In most cases there is a road on top of the dike. In that case the crest width is much more than 2.5 m. The minimum with is than determined by the minimum width of the road.
7. The inner slope. The minimum inner slope is 1:2.5 for serviceability. In that case one is still able to mow the grass. The inner slope needs no hard protection (no stone or asphalt) if the dike is high enough to prevent overtopping. In that case a clay layer of 60 cm is required.
   In those cases where more overtopping is allowed, it is necessary to make a more resistant slope protection.
   In some cases also an inner berm is designed. The only function of this inner berm is serviceability and allowing an easier construction.
8. Drainage. It is important that the water can flow out of the dike in a controlled way. Therefore in many cases some drainage is required. In general drainage can be made as a simple gravel layer.

9. Drainage and seepage ditch. The water that flows through the dike and is drained and the water that flows over the dike during overtopping should be able to flow away in a controlled way. Therefore in most cases (especially when the ground level is below high water) a ditch is made.
10. Dike body. The core of the dike is in general made of sand, because this is the cheapest material. A dike, fully made of clay, is more impermeable and better, but is too expensive. Therefore sand is used. The quality of the sand is of no importance.
11. The subsoil. The subsoil available in general has to be used. For sea dikes the quality of the subsoil is in general sufficient.

3.2 river-dikes

The main characteristic of a river dike is that it is attacked by a relatively slow varying waterlevel and only minor wave attack. The period of high water is relatively long (a number of days), and the water will rise slow. The prediction of a flood is very reliable on a timescale of days. Because of the low rising waterlevel, the whole dike-body will be saturated with water. The geotechnical problems are in general the most significant.

The composition of the subsoil below a river dike is in general highly variable. This is caused by the fact that a dike in general is build on the shore of a river. This makes the subsoil-composition at the river side completely different from the composition at the landward side.

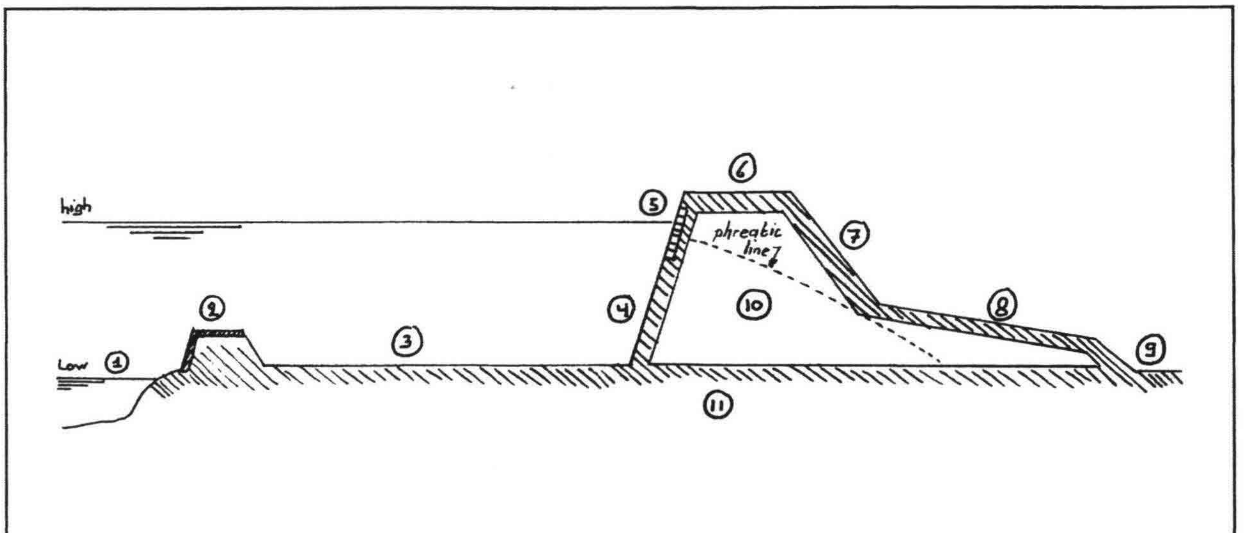


Figure 3.1: Schematic profile of a river dike

In the above figure one can distinguish the following elements:
1. The summer bed. This is the position of the river during the greatest part of the year.
2. The summer dike. This dike is to prevent the flood-plain (3) from flooding during small variations of the waterlevel in the river in the dry period. This dike has to be constructed in such a way that it may stand under water. A (relatively) thin cover of

clay and grass is sufficient in such cases, provided that the dike is not overflown. To prevent that, in several locations the summer-dike is made a little bit lower, to make sure that overflow will occur in these designed places. At these special places the summer dike is fully protected with a stone or asphalt cover (outer slope, crest and inner slope). The water can flow over such a point without causing damage (see figure)

3. Flood-plane. This space is required to guarantee a good run-off during high discharge. Therefore the hydraulic resistance of this area has to be low. Construction of buildings in this area is prohibited. Also it is not allowed to plant new bushes and trees in this zone.

4. Outer slope of the dike. The maximum steepness of the outer slope is 1:2.5. In general the slope can be covered with grass. In some cases it is necessary to make some provisions near the toe of the dike against scouring during high run-off. A good vegetation is therefore vital. Sometimes one find high trees in the floodplains near the dike. They cause some resistance. In cases where the floodplain is wide enough, this has the advantage that the current near the toe of the dike is somewhat less. A disadvantage is that because of shadow, the growth of grass on the dike is somewhat less (in Dutch conditions).

5. Revetment. At design level it is sometimes necessary to make a revetment. Because the fetch during high water may be considerable (very wide river with a depth of several meters) also waves up to 75 cm may occur. When these waves act several days on one place of the dike slope, they may cause erosion. In general the revetment can be a very light one.

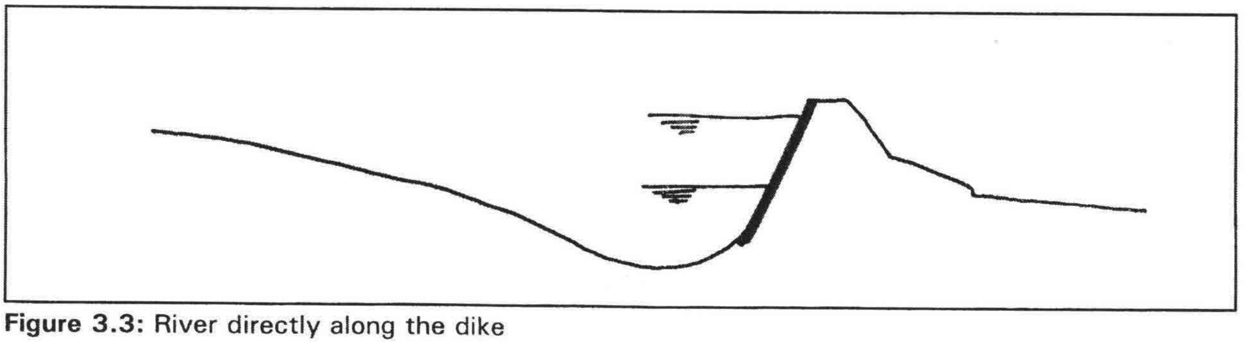


Figure 3.3: River directly along the dike

In some cases the river flows directly along the dike. This happens in general in the outer bends of meandering rivers or in the outer bends of tidal gulleys in estuaries and tidal rivers. In this case the revetment has to be continued much below the low water level. This is necessary to prevent uncontrolled scouring at the toe of the dike. Such scouring might endanger the stability of the dike.

6. Crest. Like for sea-dikes, the crest width for a river dike is also at least 2.5 m, in case of a road on the dike, it is much more. Also river dikes should have an impermeable crest.

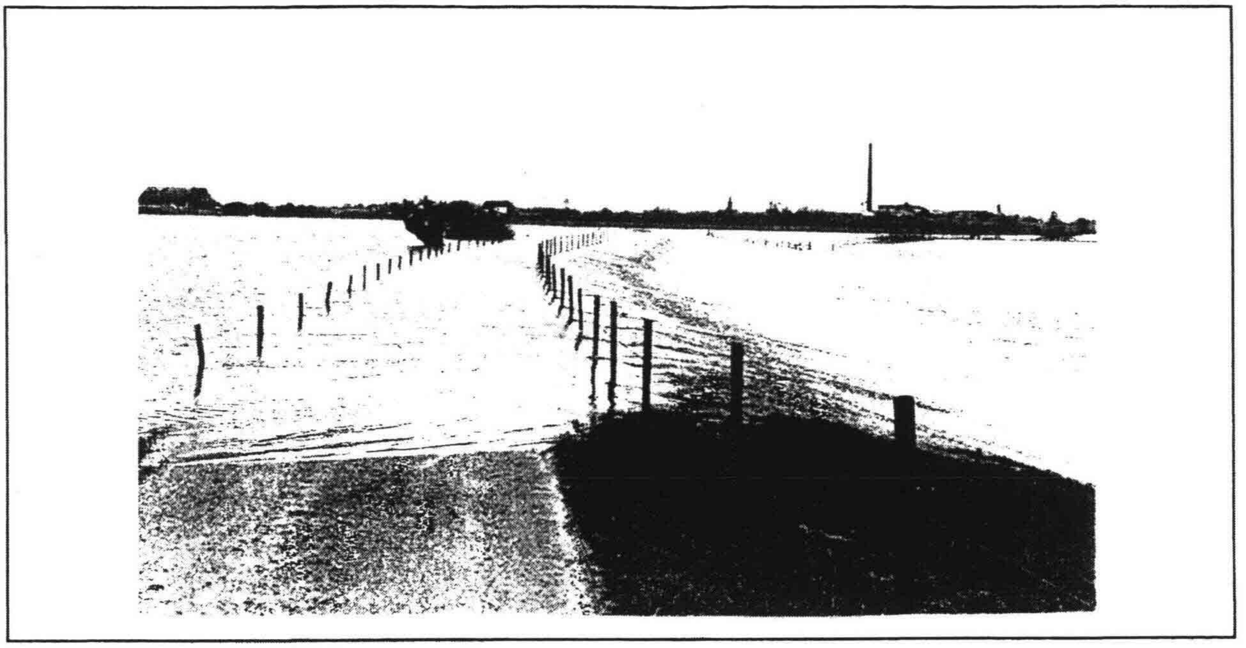


Figure 3.4: Overflow of a summer dike

7. Inner slope. The inner slope is in general more gentle than 1:3 in order to get a geotechnically stable slope. Because one will always try to keep the phreatic line inside the dike, the slope is made quite gentle.
8. Inner berm. To keep the phreatic line inside the profile, it is sometimes necessary to make a wide inner berm along the dike. This berm is in some cases also necessary to prevent piping.
9. Ditch. River dikes are designed for a no-overflow, no-overtopping condition. Therefore a ditch is not required. Sometimes it is even dangerous to make a ditch, because it might be the starting point for a sand boil.
10. Dike body. The dike body may consist of cheap material (sand). A clay dike is in some cases better, but is in general too expensive.
11. Subsoil. At some locations the subsoil is of such a bad quality that it is absolutely required to remove it. This is the case when the subsoil consists of very thick layer of soft peat layers. Then the consolidation is too much. The peat is removed and replaced by sand. This is done by hydraulic dredges. An example of such a removal are the dikes around the polders in the IJsselmeer area.

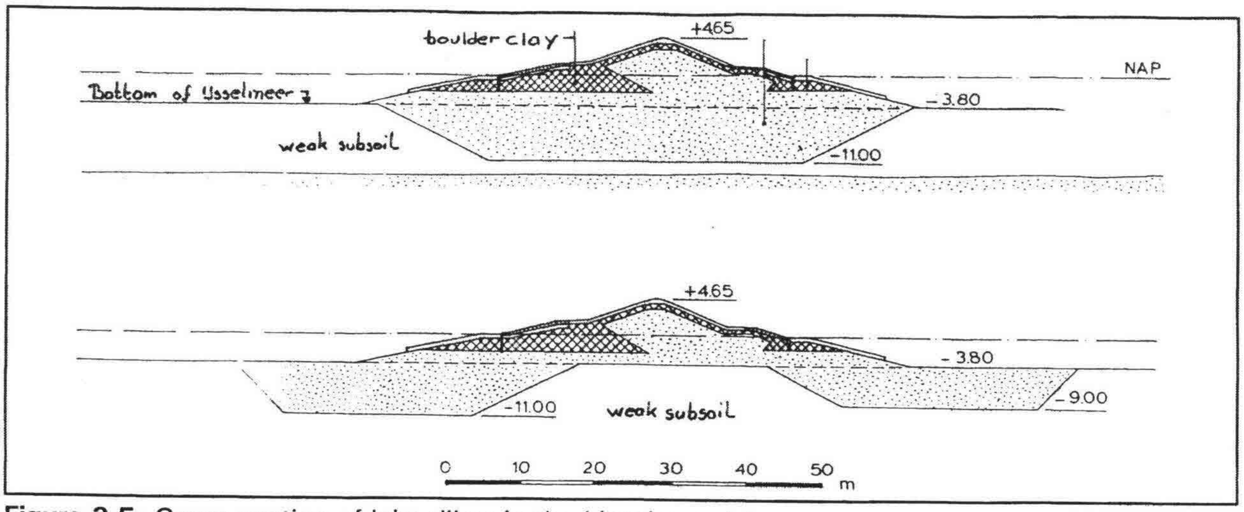


Figure 3.5: Cross-section of lake dikes in the IJsselmeer-lake

In figure 3.5 above two cross-sections are given of the dike around the Noordoostpolder. These dikes have been build in the IJsselmeer-lake. The local waterdepth was 3.8 meters. The peat and very soft clay was dredged away until a depth of 9 - 11 meters below Mean Sea Level. Afterwards the trench was filled with sand. This operation was carried out in the middle of the lake.
When these soft layers are not removed, the dike body will settle down into the subsoil. In the following figure an example is given from a river dike near Gouda. The dike at this location is a couple of hundred years old, and has been enlarged several times.

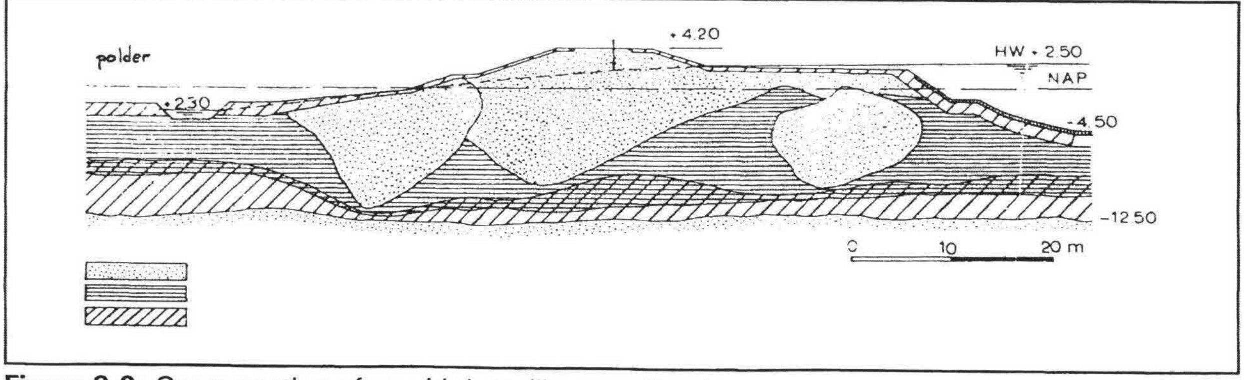


Figure 3.6: Cross-section of an old river dike near Gouda

3.3 dikes along canals and inland waterways

The third group of dikes consists of dikes along canals and inland water. Regions that border inland lakes and belt canals can be flooded by these lakes or canals, because the surface levels of the protected areas are below the water level. The results of failure are not so dangerous, as the water body that may cause an inundation is not so large, whereas the potential inundation areas are relatively small. Therefore, a relatively small design return period of 1:1000 years could be applied.

In the Netherlands, "belt" canals were excavated around the lakes to be drained. With the soil dug from these canals (often peat) a belt canal dike was made between the canal and the lake. These belt canals left part of the existing water course system intact, necessary for the transport of superfluous water to the sea or river, and for navigation. During the centuries of impoldering activities in the Western part of the Netherlands, an inter-connected network of belt canals and belt canal dikes was established. The subsoil in this area mainly consists of strata of peat and soft clay, causing severe subsidence of the dikes at a rate of up to 0.05 to 0.10 m per annum. As the water levels of the belt canal and the whole system of canals leading to the sea is kept constant, this subsidence has necessitated in frequent heightening of the belt canal dikes (up to once every 2 or 3 years), usually using locally available materials. This resulted in an inhomogeneous top layer of up to 4 to 5 m thickness of dredged mud, peat, clay, rubble, ashes and sometimes sand (see Figure 7.12). The additional weight often resulted in further subsidence.

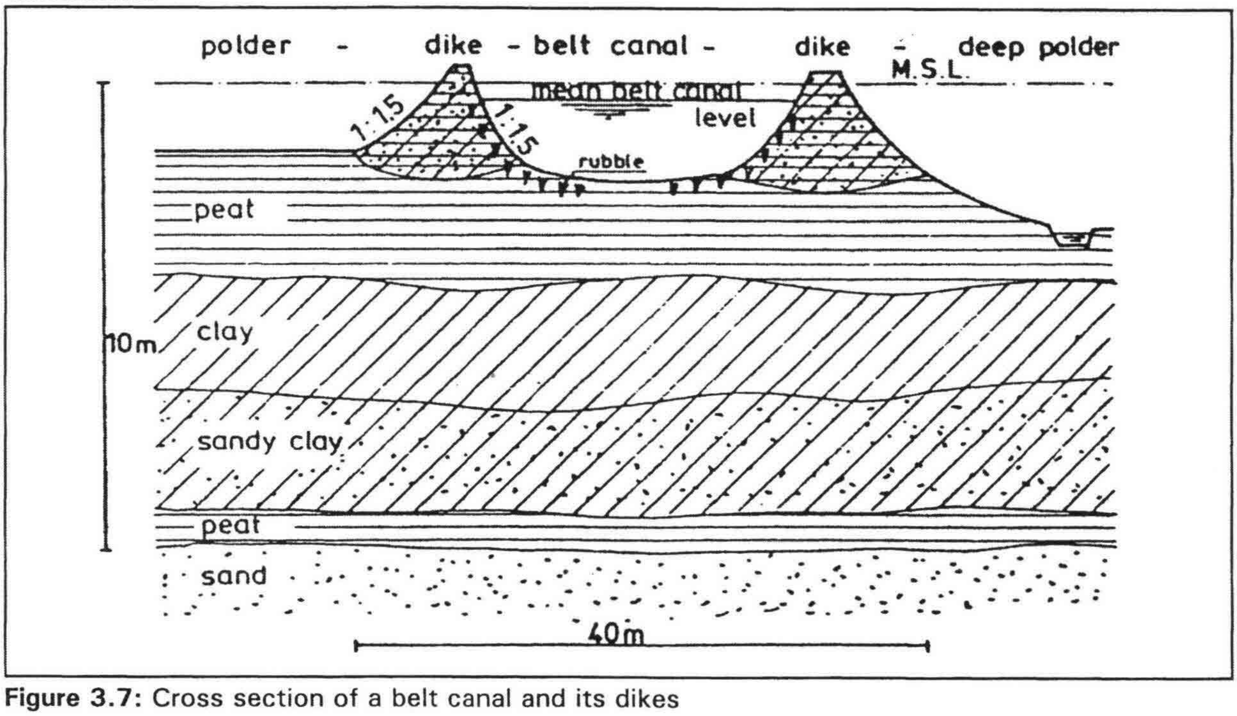


Figure 3.7: Cross section of a belt canal and its dikes

Safe belt canal dikes have become of increasing importance, since the growing population and industrialisation have necessitated building in the low level polders. The design of belt canal dikes has not changed very much over the years; maintenance is now the major activity. The crest must have a minimum width of 1.5 m, but a width of 3 m is recommended to cater for vehicular transport. The crest must be 0.5 to 0.8 m above the extreme belt canal water level, which is regulated by the pumping station of the polders itself, and by the pumping stations and sluices, which drain the belt canal water in the sea. The belt canal water level varies within certain limits.

In case a belt canal dike would burst, such a low-level polder would be inundated completely by the water stored in the belt canal system. Such a dike burst would cause a depression of the level of the adjacent belt canal, therefore also endangering the stability of the dikes along the canal. Division of the belt canal system by means of emergency weirs limits this effect to a restricted area. The problem of this solution is, however, that more local damage to the belt canal dikes would occur, because of the quick fall of the water level in the belt canal. The belt canal dikes are covered with turf and only protected against stream and wave attack at the water level, and therefore not sufficiently protected against such negative conditions.

A less obvious threat comes from animal activities: grazing cattle and especially musk-rats. Grazing cattle destroys the turf on the belt canal dikes, and makes a quagmire of the outer slope when drinking from the belt canal. Musk-rats are even more dangerous, because they burrow their holes in ground adjacent to open fresh water, which leads to instability problems. As the entrances lie below water level and the tunnel systems can be very extensive, the tracing of this threat is difficult. Catching the musk-rats is the only way to combat.

3.4 revetments

In general one may define a revetment as a sloping surface of stone, concrete or other material used to protect an embankment, natural coast or shoreline against erosion. Includied in this definition is that the protection of the seaward slope of a seadike (or the riverward slope of a levee) can also be called a revetment. However in those cases a revetment is often referred to as "slope portection".

The definition is somewhat vague. Often a (small) sheet-pile is part of a revetment, although this is according the definition impossible.

Types of revetments:

- I. sloping solutions
  - a. rip-rap (loose material, rock armour)
  - b. placed stones, pitched stones
  - c. constuctions with connected ballast
  - d. impermeable (bituminous) structures
- II. Vertical solutions
  - a. sheetpiling
  - b. piles
  - c. stones with pile-foundation
- III. Combinations
  - a. double row of piles (with reed)
  - b. piles with placed or dumped stones
  - c. bituminous structures

3.5 special constructions

A good dike is a wide construction. Suppose the height is approx. 5 m. With slopes of 1:3 on the outer side and 1:2.5 at the inner side, this gives a total width of 30 m. Additional there should be a restricted zone at both sides of the dike. In this restricted zone construction activities are forbidden. It is also forbidden to dig holes in the restricted zone. In build up zones this causes sometimes serious problems, because no space is available. Then special

constructions have to be designed, like cofferdams, movable constructions, etc. All these constructions can be build technically in such a way that they are completely safe.
However there are two major constraints of these special constructions:

a. Very often they consist of movable parts. Human action is required to bring these parts into position. Risk-analysis has proven that especially the human component in such special cases is determining risk factor. Also require these constructions a lot of maintenance.
b. Dikes are designed for an event with a very low probability. This is not very inspiring for maintaining the construction, especially when it is not visible. And because the life-time of the construction is very long (order of centuries), one needs to be certain that all vital components of the construction still function in many years. This is very difficult to guarantee, especially with hidden parts (steel sheetpile into the dike, who will check that it is not rusted away ??)

4 Limit states and boundary conditions

4.1 general

As stated in the previous chapters, the design should be based upon the functional requirements taking into account the environmental conditions in the project area and giving due regard to constructional aspects, operation and maintenance.

4.1.1 the functional design

The function of the coastal structure, as defined in the previous chapters, is mainly to protect the hinterland against the adverse effect of high water and waves. If high-water protection is required the structure should have a height well above the maximum level of wave-uprush during storm-surges. This normally calls for high crest elevations.
If, however, some overtopping is allowed in view of the character of the hinterland, the design requirement is formulated in terms of the allowable amount of overtopping. In Holland a value of 2 litres per second per running metre of dike is accepted for instance. Obviously crest-elevations can be reduced considerably in this case. For structures, such as breakwaters, where wave-reduction is the main objective a further reduction in crest-height can be applied. Wave-heights due to transmission and overtopping should be negligible during operational conditions but may reach values of 0.5 metres in extreme design conditions. Finally, training walls are mainly used to direct flow. The crest-elevation is mainly determined by constructional aspects, which implies that a minimum level of 2 meters above mean high water should be applied to guarantee an uninterrupted progress of work. Wave overtopping during operational and extreme conditions are of less concern in this case. Methods to compute the required crest-elevation will be given in the subsequent sections. As the computational method depends to a large extent on the structural-concept, this should be selected first.

4.1.2 the structural design

The selection of the structural concept depends on the function, the local environmental conditions and the construction constraints. The governing criteria are the technical and economic feasibilities of the project under consideration. Basically, a simple sheetpile wall will be sufficient to provide the required crest-elevation. Such a concept is feasible, only in small water depths with moderate wave action. In deeper areas the coffer-dam concept has to be applied, which is more complicated to build, especially in areas with frequent wave agitation. Another method to stabilize the sheetpile is the use of anchors. All these methods are particularly useful for slope and bank protection in waters which are well protected against waves.

Vertical-face structures can also be constructed by means of gabions, block type dams, or caissons. In this case the stability is derived from the weight of the structure, which is therefore called a gravity-type structure. Often, the performance of such structures in terms of overtopping, is improved by using a parapet or crownwall. Gravity-type structures may be used in moderate to large water depths, provided that no breaking waves occur. Due to the potential foundation problems as a result of dynamic wave-loading, such conditions should be at least twice the design wave height. Obviously, the construction, the transport and the positioning of caissons requires knowledge and experience, which often makes the method impractical. Especially in areas where the weather-windows for construction are small, the caisson concept could be a good solution. In many areas in the world wave-heights and foundation conditions are such that no gravity-structures or sheetpiles can be used. Sloping structures are a solution then, as wave loads on these structures are more easily accounted for. Moreover, foundation loads are more evenly distributed and differential settlements can, to a certain extent, be accepted.

The slope-type structure is most widely used since it is a versatile concept which can be constructed by less-experienced contractors as well. The structure can be used in moderate to deep water, and can be designed to withstand severe waves. With the increasing size of the structures, however, the limits of the application are more or less reached. For a general description of a dike or sloping seawall one is referred to the previous chapter.

4.1.3 design philosophy

Coastal defences are constructed to protect the population and the economically valueble areas against storm surges. However, absolute safety is nearly impossible to realize. Therefore it is much better to speak about the probability of failure (or safety) of a certain defence system. To apply this method, all possible causes of failure have to be analyzed and consequences determined. This method is actually under development in the Netherlands for dike and dune design. The "fault tree" is a good tool for this purpose. In the fault tree, one considers all possible modes of failure of elements which can eventually lead to the failure of a dike section and to inundation. (See the chapter on risk analysis.) They can also badly influence the behaviour of the revetment even if properly designed.

Although all categories of events which may cause the inundation of a polder are equally important for the overall safety, the engineer's responsibility is mainly limited to the technical and structural aspects. In the case of the seadike the following main events can be distinguished:

- overflow or overtopping of the dike
- erosion of the outer slope or loss of stability of the revetment
- instability of the inner slope leading to progressive failure
- instability of the foundation and internal erosion (i.e. piping)
- instability of the whole dike.

For all these modes of failure, one considers the situation where the forces acting are just balanced by the strength of the construction (the ultimate limit state). In the adapted concept of the ultimate limit-state, the probability-density function of the "potential threat" (loads) and the "resistance" (dike strength) are combined. The category "potential threat" contains basic variables that can be defined as threatening boundary conditions for the construction e.g. extreme wind velocity (or wave height and period), water levels, and a ship's impact (collision). The "resistance" of the construction is derived from the basic variables by means of theoretical or physical models (e.g. theoretical or semi-empirical stability-models of grains). The relations that are used to derive the potential threat from boundary conditions are called transfer functions (e.g. to transform waves or tides into forces on grains or other structural elements). The probability of occurrence of this situation (balance) for each technical failure mechanism can be found by applying mathematical and statistical techniques.

limit states

The safety margin between "potential threat" and "resistance" must guarantee a sufficiently low probability of failure. The 3 different philosophies currently available in construction practice are:
1. deterministic,
2. quasi-probabilistic and
3. probabilistic.

For fully probabilistic approach more knowledge must still be acquired concerning the complete problems associated with the use of theoretical models relating loads and strength.

The ultimate potential threat for the Dutch dikes is derived from extreme storm surge levels with a very low probability of exceedance (1% per century settlements or deformations, etc.). However, this deterioration of constructional resistance can cause an unexpected failure in extreme conditions. These are, so called, the serviceability- and fatigue limit states which can also be considered as inspection and maintenance criteria.

As already mentioned, the fully probabilistic approach for dikes based on the limit state concept is rather cumbersome because a theoretical description for various failure modes is not available yet. For a more detailed description of probabilistic approach for dikes design, one is referred to the lectures on probabilistic design.

4.2 ultimate limit states

4.2.1 ultimate limit state of failure

In the design process one is most interested in the ultimate limit state (ULS) of a failure mechanism. This state describes the situation wherein the acting extreme loads S are just balanced by the strength R of the construction. If the ultimate limit state is exceeded the construction will collapse or fail.

4.2.2 ultimate limit state of sustainability

Besides the Ultimate Limit State there are situations where the ever continuing presence of a load causes a deterioration of constructional resistance over time, without imminent danger of failure. In this case the mechanisms "erosion of the foreshore" and "settlement" are examples. This deterioration of constructional resistance may cause unexpected failure in extreme conditions. Hover the serviceability of the structure is often hampered before failure (excessive leakage due to piping). The Serviceability Limit State (SLS) is treated in the same way as the Ultimate Limit State.
A point of great practical importance is that a Serviceability Limit State, i.e. deterioration of constructional resistance over time can be improved in two ways:
1. increasing the resistance to guarantee sufficient strength during the service life;
2. the deterioration of the resistance can be controlled by inspection and maintenance procedures.

4.2.3 ultimate limit state during construction

In general most attention is paid to the behaviour of the structure after completion. But during construction there are also periods in which the construction may fail. During construction storms may also occur. But much more dangerous are the geotechnical failure mechanisms.

limit states and boundary conditions

In general, geotechnical failure occurs when the pressure on the soil particles decreases. During the construction this may happen when poorly drained soils are loaded. Then the water pressure increases, decreasing the pressure between the soil particles, leading to instability. After some time, due to drainage, the excess of water flows away, and the water-pressure decreases.
This makes it necessary that the construction process should also be analyzed on limiting states. Sometimes it is necessary to change the method of construction, and in very special cases it is even necessary to change the complete design, because it is not possible to build the original design. As mentioned before, this mainly happens with heavy dike constructions on soft soils.

4.2.4 overview of ultimate limit states (strength)

Good engineering practice requires that attention should be given to all possible modes of failure of the construction under design. This is a common approach in the design of steel and concrete structures, and also applicable for the design of dikes and levees and revetments. For an overview, refer to figure 4.1.

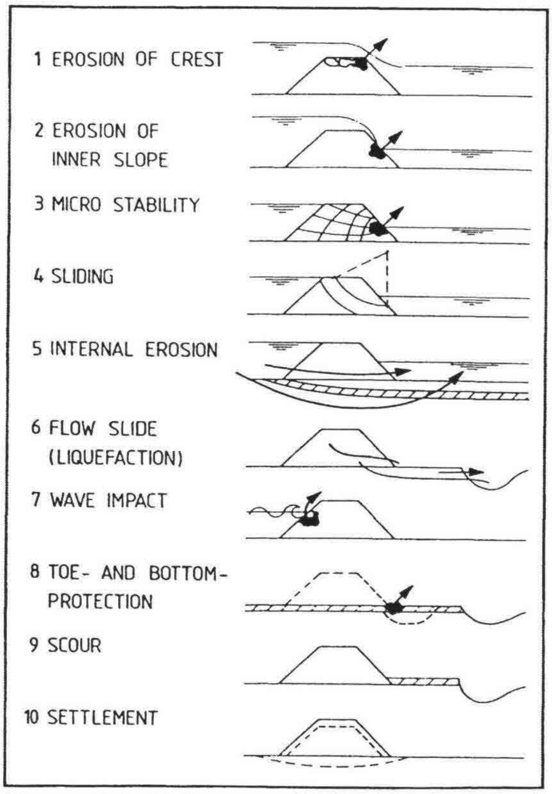


Figure 4.1: Failure mechanisms

1. erosion of the crest. This is caused by water flowing over the crest. The cause may be overflow of water, or overtopping by waves.
2. erosion of the inner slope. Usually caused by water flowing over the inner slope (either from overflow or from wave overtopping).
3. micro instability. Because of water seeping out of the slope, the individual grains are no longer stable, and start to move out of the slope.
4. sliding. Because the internal friction in the body of the dike decreases (usually by an increased water pressure) the friction along the slip circle decreases and the earth body is no longer stable.
5. internal corrosion. Groundwater flow under, or in the body of the dike may cause a movement of sand particles. This movement may lead to the formation of a pipe under the dike.
6. liquefaction. Because of an increased water pressure in the loosely packed sublayers, they loose their bearing capacities and a slide-flow will occur.
7. erosion of the outer slope. Because of the impact by wave and currents the armour units on the outer slope are no longer stable, and are moved away, creating a gap. Also collisions from ships and floating debris may lead to erosion of the outer slope.
8. damage to toe and bottom protection. Wave reflection and concentrated currents may remove the stones of the toe protection. Without its toe protection the revetment is no longer in a stable position.
9. scour. Currents may cause scouring in front of the bottom protection. The scouring holes may become that deep that the edge is no longer stable, and failure will be initiated. The effect of the current may be aggravated by wave action.
10. settlement. Because of the deadweight of the dike body, the subsoil will settle.

4.3 boundary conditions (loads)

4.3.1 general

It is clear that the boundary conditions for dikes follow mainly from water levels and waves. In general these are called the natural boundary conditions. The quality of the subsoil, important for the geotechnical stability, is sometimes also called a boundary condition. However in these notes the quality of the subsoil is treated as an separate item in the analysis of the limit states, because the quality of the soil is related more to the strength than to the loads.

Important for the construction of a dike are:

* The (maximum) water levels and the height of the waves;
* the magnitude of the water level difference between the inner side and the outer side of the dike;
* the magnitude of the water level difference along the dike (currents !!);
* the time between high and low water;
* the number of times that this water level difference occurs during the lifetime of the structure;
* the rising and falling of the water (how much time is taken the for water level to rise from normal to extreme).

A lot of relevant information for a seawall/dike design can be drawn from files and existing maps. In addition to this, a field reconnaissance and a land survey are indispensable, as well as photographic recording of the characteristic points in the area. Special attention should be paid to the position of the beach and/or onshore profiles, and the morphology of the area being considered (eroding/ accreting coast). The composition of the existing dike body and the geological structure of the subsoil are also very important. When these data are not available, soil investigations should be considered (soundings, borings etc.).

In view of the function of (coastal) water defences the loads will obviously be mostly due to the actions of long and/or short waves. In broad outline the following wave phenomena can be distinguished:

(a) low-frequency water levels changes, such as flood waves, tidal waves, wind set-up gradients and seiches;
(b) wind waves and swell;
(c) ship's waves in navigable waterways.

These water levels variations strongly influence the area which needs to be protected with hard revetment.

Water level variations on canals and water-storage channels are comparatively small; probably only caused by lockwater, seepage, drainage and wind effects. Water levels on lakes can vary as a result of wind set-up, inflow or outflow of water, and evaporation. Water levels in a reservoir can change markedly due to filling or emptying, but rainfall and wind set-up can also play a role.

Water levels on a river are determined by the river's discharge regime, and in addition for the lower reaches (estuaries) by tides and also wind set-up. For a coastal defence embankment water levels are governed by tides and winds.

It is clear from the above table that the differences in dike types can be found in the differences in natural boundary conditions. On of the most complex types of dikes (from a viewpoint of natural boundary conditions) is the sea dike. There we find all types of water level fluctuations:

* astronomical tide
* flood waves (tsunamis)
* squall oscillations (seiches) and gust bumps
* wind set-up
* climatological variations
* wind waves.

All these phenomena will be discussed in more detail in the following chapters. The boundary conditions for river dikes will also be treated. Any non-natural boundary condition is also very important and human acts have sometimes a considerable influence on the boundary conditions, by making dams, dredging rivers, etc.

Overview of various types of dikes

	water level	water level diff. across	duration	dh/dt	frequency	water level diff. along	wave height
canal dike	known	small	long	small	often	small	small
polder dike	known	small	long	small	often	small	small
lake dike	uncertain	variable	short	variable	seldom	small	moderate
reservoir	known	big	long	small	often	small	moderate
upper river	unknown	big	long	variable	seldom	big	small
tidal river	unknown	big	short	big	seldom	big	moderate
seadike	unknown	big	short	big	seldom	big	big

4.3.2 reference level, sea level rise, subsoil settlement

As indicated before, the design water levels should be related to a certain probability of occurrence. The value of this level is discussed in the next sections. However, in time there will be a change in the design level itself because the sea level is rising and the subsoil is settling. The quantity of rise in design level over time depends very strongly on the design period of the dike. In the Netherlands usually a design life of 50 years is used for dikes. For constructions which are not very easy to reconstruct completely (like a storm surge barrier) a lifetime of 100 - 200 years is used. The rationale behind this is that during such a period a lot of partial repairs will be done, but it is in general very difficult to change the complete layout of the structure. Also the values of sea level rise and bottom subsidence are very uncertain. It may be more than expected, and because it is in general not very expensive to make a barrier 1 m higher, why not do so and prevent a lot of problems in future.

boundary conditions (loads)

The settlement of the upper layers can be calculated quite accurately when enough geotechnical data are available. The settlement is caused by compression of sublayers due to the weight of the dike. This settlement may vary considerably over the route of the dike. The subsidence of deep layers is a more geological phenomenon and is much more difficult to calculate. Because it is quite constant over the whole route of the dike, it is generally combined with the sea level rise. The combination of sea level rise and subsoil subsidence is usually called relative sea level rise.

sea level rise

For many years it has been known that the sea level is rising. From geological evidence it has been observed that the sea level sometimes rose as fast as 5 m per century. However, in historical times this sea level rise was generally much more moderate. In the last century the relative sea level rise in the Netherlands was 20 cm per century. This relative sea level rise is the rise of the sea level itself plus the changes of the level of the land. In the Netherlands this is a subsidence, and consequently the relative sea level rise is more than the rise of the level of the sea itself. The relative sea level rise can be determined from long time series of water level observations. Long in this case means in the order of a century. Unfortunately for most of the places these long series are not available and an extrapolation has to be done from a much shorter time series. This is sometimes quite dangerous.
In the following figures the water level of two stations in the Netherlands are given. As can be seen from these figures there is a relative sea level rise of approx. 20 cm/century at this moment.

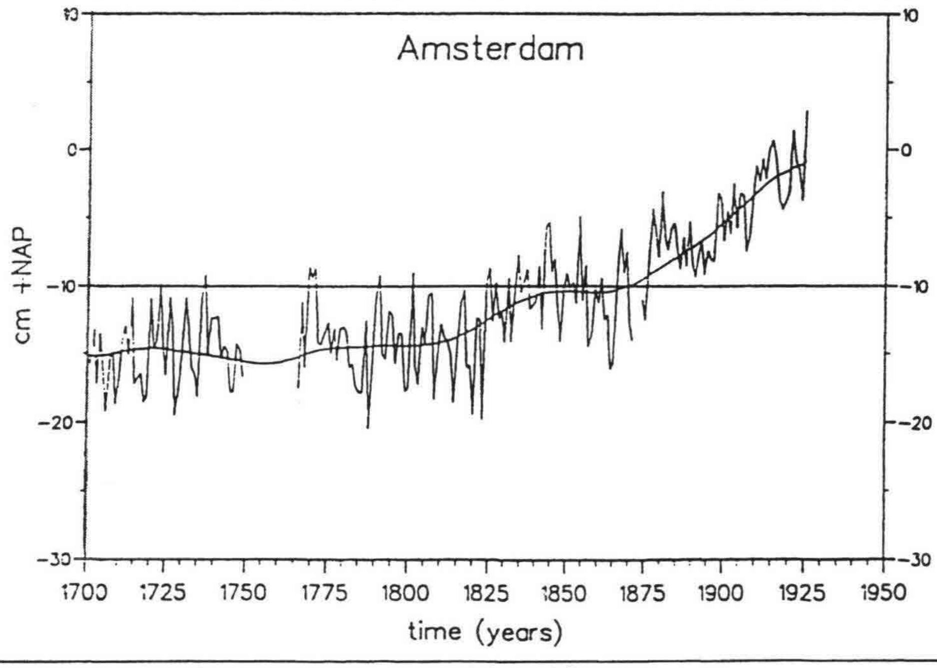


Figure 4.3: Average water level at Amsterdam 1700-1925

Since the beginning of the eighties it became clear that an increased sea level rise might be expected because of an increase in the greenhouse effect. High concentrations of CO_2 and

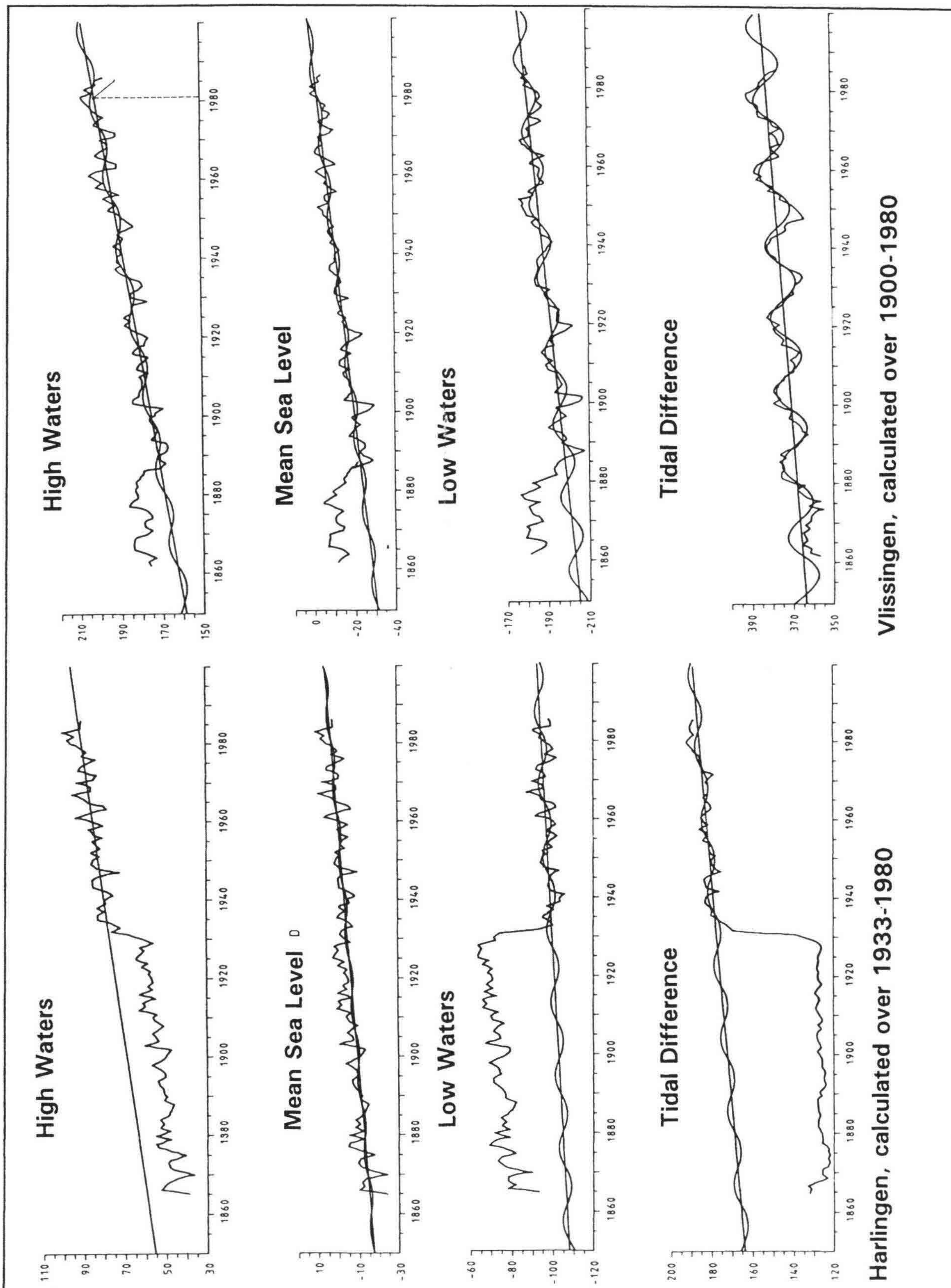


Figure 4.2: water level observations in Vlissingen and Harlingen

other greenhouse gasses cause higher global temperatures that cause an extra increase in sea level rise.
In the past, the sea level has never been constant. During the last Ice age the sea level in the North Sea was 140 m lower than at present. At that time a great part of the North Sea was land, and the Thames was a tributary of the Rhine river. Although the sea level in the last 1000 years has been rather stable, many fluctuations in the order of some decimeters occurred, see figure.

Since approx. 1850 there has generally been a relative sea level rise along the coast of the Netherlands of approx. 20 cm per century. The values per station vary from 15 cm near Terschelling to 26 cm near Hook of Holland. Recent research of the European Community shows that at this moment there is no statistical proof from the available data-series that there is an increase in the present rate of sea level rise. It can be proven that it will still take many years before such an increase in the rate of sea level rise can be proven from observations. However, investigators assume that the increased concentration of greenhouse gasses will lead to an increase rise in the sea level. Many figures have been mentioned. For the Netherlands the official estimate at this moment is a rise of 60 cm in the coming 100 years.
Publications of Hesselmans in 1993 and the IPCC supplementary report (1993) indicate that the rise of the temperature will be somewhat less because of the increase of soot in the atmosphere (2.5°C instead of 3.0°C). Based on this 2.5°C, Wigley (1992) computed a sea level rise of 48 cm until 2100. Including subsidence this means for the Netherlands a relative sea level rise of 52 to 56 cm .
Other studies, for example by Oerlemans (1993) indicate a sea level rise until 2100 of 58 cm. Because of the large unreliability in these figures, the "official" value of 60 cm/century is still used in the Netherlands. The standard deviation in this value is in the order of 25 cm.

<u>changes in the tide</u>
The tide along the coast of the Netherlands has changed considerably over the last 100 years, with a large variation along the coastline. The largest variations have occurred where the tide has been reduced considerably due to closing works. In areas where the morphology was altered considerably (Wadden Sea and Westerschelde), changes in the tide also occured.
A minor change in the position of the amphidromic points has also been observed. However, in points like IJmuiden and Helgoland (in Germany) changes in the tide have also been observed, which cannot be explained by civil works or morphological changes.

With reference to the average water level there is a clear increase in the high water levels and a decrease in the low water levels. For example for Terschelling the sea level rise for the period 1940-1990 is 15 cm per century, while the high water levels increased with 28 cm/century. The low water levels decreased with 1 cm/century. In Bath (along the Westerschelde) the high water level increased with 57 cm per century. For the period 1900-1990 this increase is 47 cm/century.

The stations in the table all give a higher rise of the average high water than of the mean water levels. Over the last 50 years (1940-1990) this difference is approx. 5 cm per century. In the Ems estuary in the north of the Netherlands this increases to 10 cm/century, in the Westerschelde it increases to 50 cm per century near Bath.

There are plans for increasing the depth of the Westerschelde by dredging. The consequence of this will probably be that the increased rise of the high water will continue.
It is therefore important to realise that not only will there be a change in the average water level, but also that due to the increase in tidal amplitude the high water levels will increase much more. This is especially true in cases where estuaries are deepened by dredging operations.

Relative sea-level rise for various stations in the Netherlands

	sea-level rise 1900-1990	sea-level rise 1940-1990	high-water rise 1940-1990	low-water rise 1940-1990	tidal difference 1940-1990
Bath	--	--	57	-6	63
Hansweert	16	25	44	20	24
Terneuzen	25	28	42	14	28
Vlissingen	23	19	29	15	14
Westkapelle	--	--	18	16	02
Hook of Holland	26	28	(44)	22	(22)
IJmuiden	21	20	32	16	16
Den Helder	17	20	22	12	10
Den oever	--	--	34	--	--
Kornwederzand	--	15	22	--	--
Harlingen	16	19	31	14	17
Texel	--	--	23	18	05
Terschelling	15	11	28	-1	29
Delfzijl	21	23	(49)	(-10)	(60)
Nieuw Statenzijl	--	--	32	--	--

(xx) means that there is not a clear linear trend

4.3.3 long duration extreme water levels (river floods)

For rivers it is relevant not to determine the once in N-years water level, but the once in N-years discharge of the river. This makes the hydraulic computations much simpler. In the Netherlands the design flood for the Rhine river has a probability of 1:1250 per year. For the Rhine river this is a discharge of 15,000 m^3/sec, for the Meuse-river it is 3650 m^3/sec [BOERTIEN, 1993].
Using these discharges at the places where the rivers enter the country (for the Rhine it is that near the village of Lobith, for the Meuse it is that near the village of Borgharen), one may calculate the water level along the river. One should of course take into account the effects of bifurcations, barrages, weirs, etc. In the Netherlands the general water level and discharge pattern is calculated using a one-dimensional computer model with many branches, the detailed water level is calculated with the use of a two-dimensional computer model. The accuracy of these models is in the order of a few decimeters for design circumstances (for those situations the models cannot be calibrated).
The shape of the floodwave is quite important, because this shape determine the time that a levee is loaded. Especially for the failure-mechanisms of piping and stability of the inner slope the saturation of the dike body is important. Full saturation takes several days, approx. the same time as a usual flood wave. When the waves are shorter, the risk of piping is less, when the floodwave is longer, there is considerably more risk of piping.

4.3.4 short duration extreme water levels (storm surges, wind set-up)

The water levels on the oceans also continue in shallow water. Some of the effects are amplified. But also typical nearshore effects may occur,like wind set-up. The friction of the wind causes a force on the water surface; this creates the ocean currents. But if the sea is relatively shallow, no return-current can be formed in deeper water, and the surface water will rise near the shoreline. This can happen during a storm, but it can also be caused by a very long lasting wind system, like a monsoon or a trade-wind. The effects are more signi-

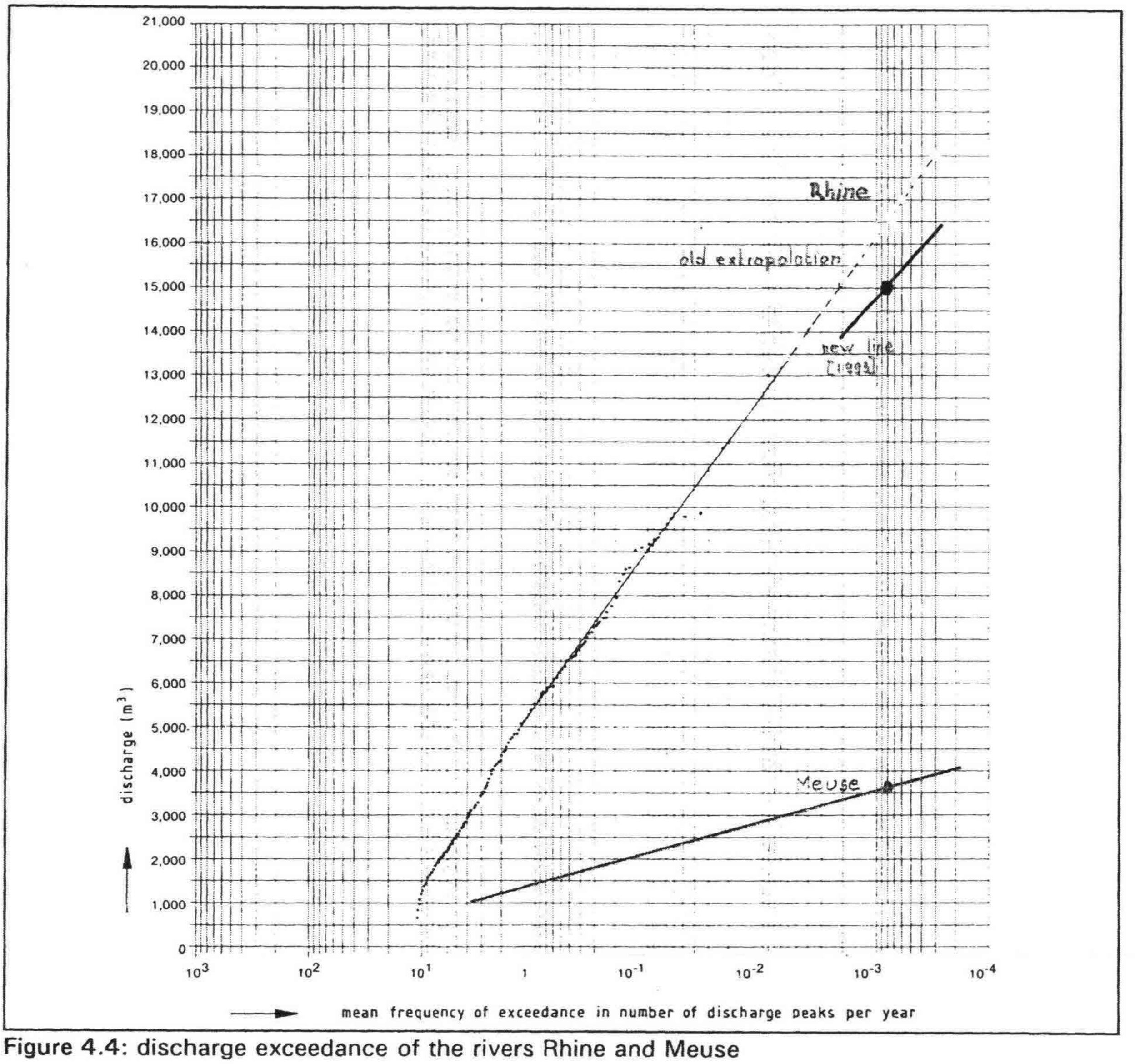


Figure 4.4: discharge exceedance of the rivers Rhine and Meuse

ficant in shallow seas like the South Chinese Sea, the North Sea or the northern Gulf of Bengal. In this last case set-up values of 6-9 m were observed. In the Netherlands the wind set-up along the coast of the North Sea can be in the order of 3.5 m (with a probability of 1:10000).

This effect can be determined in two ways:

* Analyze long time series of water level observations. In this way one gets a combined statistic of (astronomical) tide and storm surge. If time series are long enough, this is usually the best solution.
* Calculate the storm surge effect. There are several formulas for the computation of the set-up, they will be discussed in the end of this section.

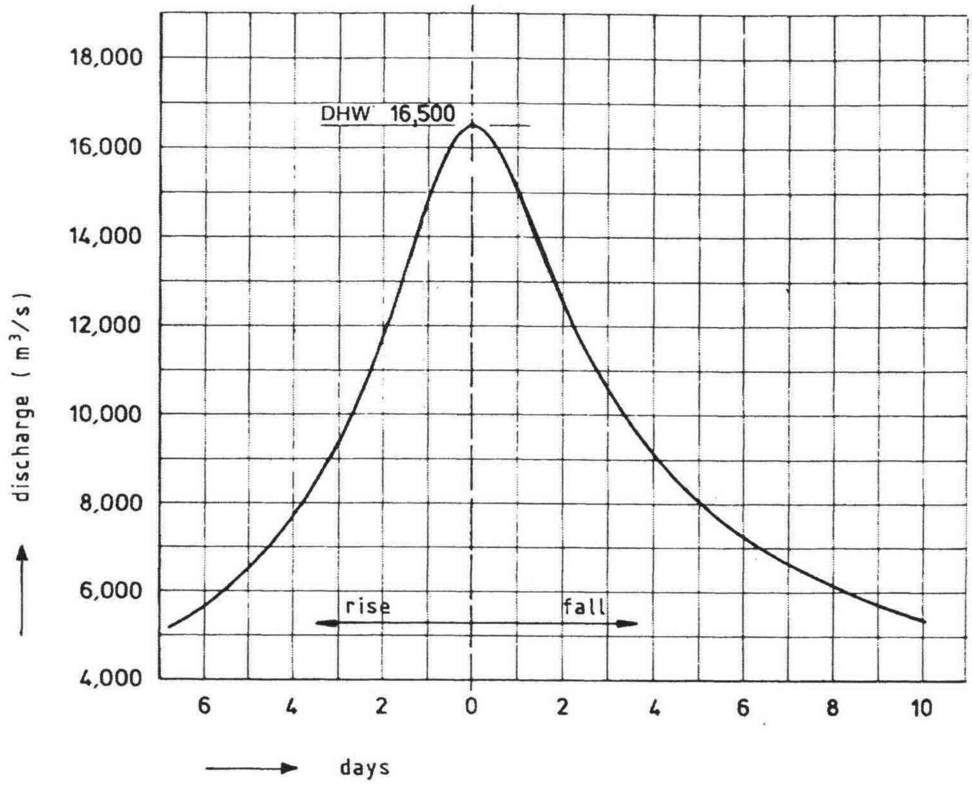


Figure 4.5: shape of the flood-wave in the Rhine river

These formulas are not very accurate. It is better to use observations of water levels, correct them for the tides (using the calculated astronomical tide) and put the observations in an exceedance diagram. From the diagram one may read the probability of occurrence of a given wind set-up.
For very low frequencies of occurrence, sometimes the tide itself is also included. In those cases one gets the probability of the water level itself. An example is the exceedance line derived by the Delta Committee in the Netherlands, see chapter 2.3., figure 2.1.

This procedure can be followed for each place along the coastline. The variation of this exceedance-line along the coast may be considerable. Extrapolation to other locations is therefore always a problem. In case of extrapolation one should always try to split up the water level in the astronomical component and the wind set-up, as it is given in the following figure.

computation of wind set-up

The wind causes a friction force on the water surface, resulting in a stress on the surface. This will result in either a current or a change in the slope of the water level. A current will occur when the water can flow away, a slope will occur when the water cannot flow away (e.g. in a lake). So, the water level change can be described as a differential equation in which parameters are included as: water depth, wind speed, fetch, geometry of the basin, etc.

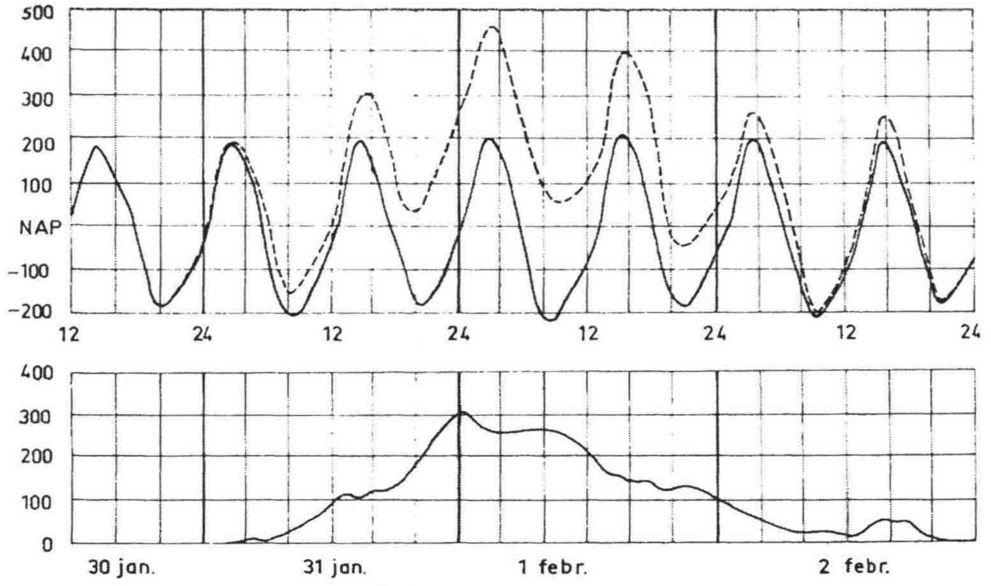


Figure 4.6: Water levels in Vlissingen during the 1953 storm surge

For irregular bottoms or irregular topographies the differential equation can be solved numerically. Calibration of these models is difficult, and therefore most of the models are not very reliable.
For a first estimate one can schematize the geometry. Then a standard solution of differential equation can be used.

For lakes and other nearly enclosed water bodies one can use the following formula:

$$\Delta S = \frac{1}{2} C_w \frac{\rho_{air}}{\rho_{water}} * \frac{U^2 F}{gh} \cos\phi$$

in which
- C_w = coefficient ($0.8*10^{-3}$ to $3*10^{-3}$)
- ρ_{air} = density of air (1.25 kg/m^3)
- ρ_{water} = density of water (1030 kg/m^3)
- U = wind velocity (m/s)
- F = fetch length (m)
- g = acceleration of gravity (m/s^2)
- h = water depth (m)
- ϕ = angle between wind direction and axis (degrees)

The factor 1/2 is caused by the fact that on the upwind side of the lake we have a set-down, and at the downwind side we have a set-up. So in fact we have to use only half the fetch length.

For a semi-enclosed bay one may use the same formula, but then one should omit the factor 1/2, because in that case there will be no set-down at the downwind side. There is a supply of water from the sea.

The solution of the differential equation for a tidal shelf is:

$$\Delta S=\sqrt{\frac{2C_w(\rho_a/\rho_w)\,U^2F}{g}\cos\phi+h^2}\;-h$$

Sometimes it is better to perform the computation in two steps. The first step for the deep part of the continental shelf, and subsequently a second step for the shallow part.
For a more detailed derivation, and other special cases refer to Ippen [1966].

4.3.5 combination of river floods and storm surges (the problem of tidal rivers)

Along tidal rivers there is a complicating factor. High water levels can be caused by high river run-off, but also by storm surges entering from the sea. In the following figures the local water level is presented for two stations at different points along the same dike circle. For the locations of the stations Jaarsveld(J) and Sliedrecht(S) see the inserts in the figures.
The water level is a function of the water level at sea (Hook of Holland) and the run-off (Rhine discharge at Lobith). Both the water level at Hook of Holland and the river discharge at Lobith have their own probability of occurrence. These variables are not fully independent. For example at station Jaarsveld the height of the dike is 6.0 meters above mean sea level. This water level can be achieved by a storm surge of 5.5 m above mean sea level. and a run-off of 8000 m^3/s or by a sea water levels of 3.3 m (once in ten years storm) and a run-off of 15000 m^3/s.
Using both the distribution of extreme storm surges as well as extreme river discharges, the exceedance-frequency for all stations can be calculated. Examples are presented in lower parts of the figures. From these diagrams follows that in Jaarsveld a water level of 6.0 m is exceeded with a probability of 1/4000. In Sliedrecht 3.75 m is exceeded with the same frequency.
When these diagrams are constructed for all stations along the dike, the water level with a probability of occurrence of 1/4000 per year can be determined for each dike section.
Unfortunately this does not mean that if the dikes are constructed in such a way, the probability of inundation of the dike-circle is also 1/4000 per year.
When a dike circle has only two section, both constructed accordingly, that they have each a probability of failure of 1/4000 per year, and they are attacked by fully uncorrelated dangers, the probability of inundation of the dike-circle is 2 times 1/4000 per year.

In reality a dike-circle has many sections, and they are attacked by partly correlated dangers. Mathematical techniques are available to also calculate the probability of inundation in those cases. This is in the order of three times the probability of failure of a single dike section.

4.3.6 very short fluctuations in water levels (waves and seiches)

<u>wind waves</u>

Wind waves have to be determined during design conditions. So one calculates the wind-waves which may occur during the design flood, as discussed in the previous section. For sea dikes with a windward location there is a very strong correlation between the wave-height and the water level. For dikes at the lee-side there is no correlation. For those dikes the maximum wave height may occur during a much lower water level.

Especially for dikes at the lee-side one should investigate several combinations of water levels and wave heights in order to find the leading combination.
When enough wave observations are available, it is possible to determine the design wave height directly from the data. Unfortunately in general not enough wave observations are

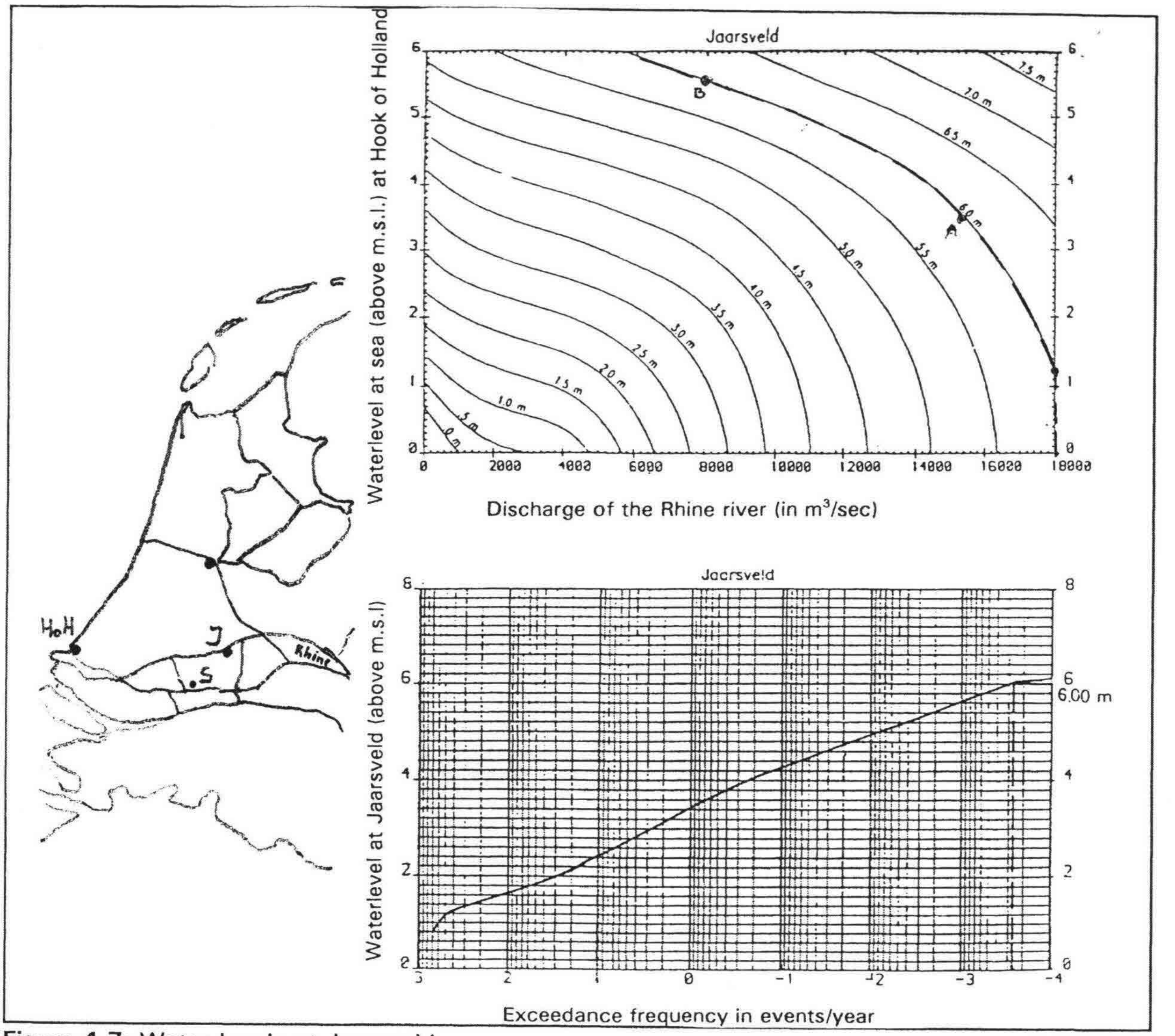


Figure 4.7: Water levels at Jaarsveld

available. In those situations it is necessary carry-out an intermediate step using wind-statistics.

First one has to determine if a correlation between windspeed and wave height exists for the location of the dike. In cases were waves are locally generated that is true. But when there is extensive ocean swell this is not true.

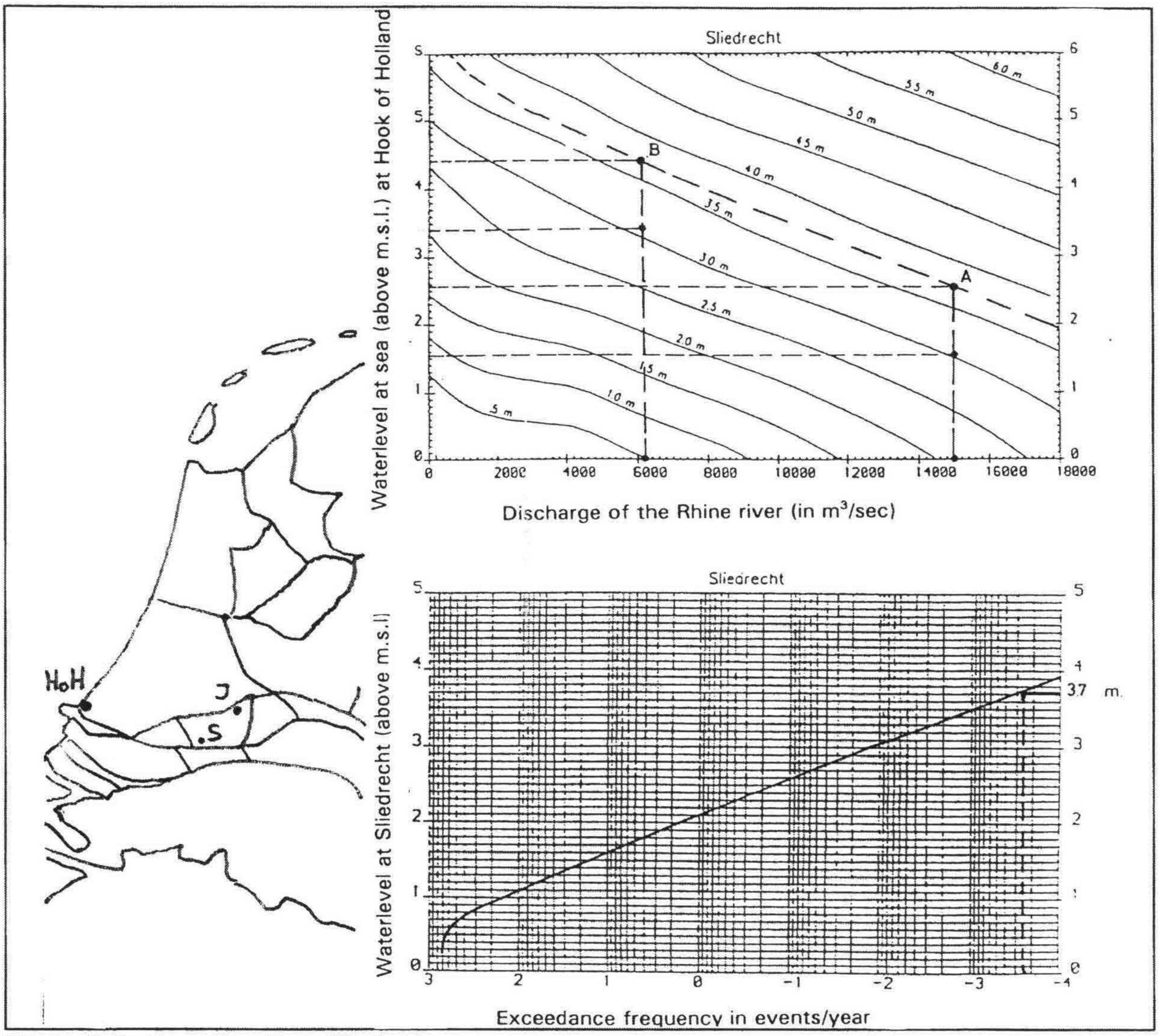


Figure 4.8: water levels at Sliedrecht

In case of ocean swell one should analyze storm patterns on the ocean. The ocean storm patterns are fed into a convenient computer program (or one uses the diagrams of Bretschneider) and the nearshore wave data (on deep water) are available. Finally the design wave is calculated near the coastline, using shoaling, refraction and diffraction calculation. This process is discussed in detail in a number of other lectures, and will not be repeated here.

In case of locally generated waves, one may determine the design wind speed (in the Netherlands, for the 1/10000-condition this is 33 m/sec, for the 1/4000-condition it is 31 m/sec). The wave height and wave period is calculated with the Bretschneider-method (This calculation can be performed with CRESS, routines 221 - 223; for details see for example the Shore Protection Manual).
This technique is also suitable for the river area.

In this method, wave height and wave period are determined under conditions where the wind blows over a water surface at a constant velocity and for such duration that the wave attains its maximum development. The time that is required for the wave to develop completely over the maximum existing fetch -about 5 km for most river and estuarine situations- ranges from three quarters of an hour to one hour, depending on the wind velocity. This corresponds well with the one-hour period over which the meteorological services present their wind-data. It can be assumed that the calculated wave heights are always able to develop fully when hourly wind velocity averages are used.

The significant wave height and the characteristic wave period are defined directly from the given wind velocity, the water depth and the fetch. This definition is obtained by means of empirically established wave development graphs and formulae. In the figures on the next pages, dimensionless parameters for wave height H_s, water depth d, and fetch F, at a given wind velocity u are presented.
For rivers and estuaries it is recommended that one value be determined for the water depth d to be adopted which is geared to the water depth for a dike skirting the river or the tidal channel, or else to the water depth above the (flooded) foreland. The zone immediately alongside the dike is of particular importance. The influence of local, deep channels can be ignored.

For relatively closed basins (bays, rivers, lakes, estuaries) it is recommended that an effective fetch F_e, which is geared to the configuration of the water surface in front of the dike be determined for the fetch F to be adopted. The effective fetch in a random situation is then equated with a weighted average of the projections on the wind direction of all the fetches. See the worked out example on the following pages. It is recommended to neglect fetches with a angle $\Theta > 45°$.

The effective fetch F_e, and with it the wave height, can be limited to a considerable degree by the presence of high-lying or flood-free areas, such as buildings in the floodplains, summer levees along the summer bed of a river etc., provided they are not too far below water levels and moreover, that there is certainty that these constructions will remain there during the design life of the dike. These aspects should be taken into account in order to prevent an unnecessarily high design.

<u>squall oscillations (seiches) and gust bumps</u>
Sometimes during a storm the water level will fluctuate. A distinction can be made between the surge elevation of the sea surface and the waves. There are also squall oscillations present (sometimes called seiches) which have periods of between a few minutes and more than an hour and which have half amplitudes of .2 to .3 m, as well as gust bumps which can create an additional elevation of up to .5 to .6 m and a duration of half an hour and more. These behave as slow waves which penetrate harbours and sheltered places in full force.
In determining the design height of a dike these squall oscillations and gust bumps must be taken into account.

The origin of the squall oscillations is not quite clear, they probably origin from the stochastic character of the air pressure in a storm. Squall oscillations occur very often, during 30 % of

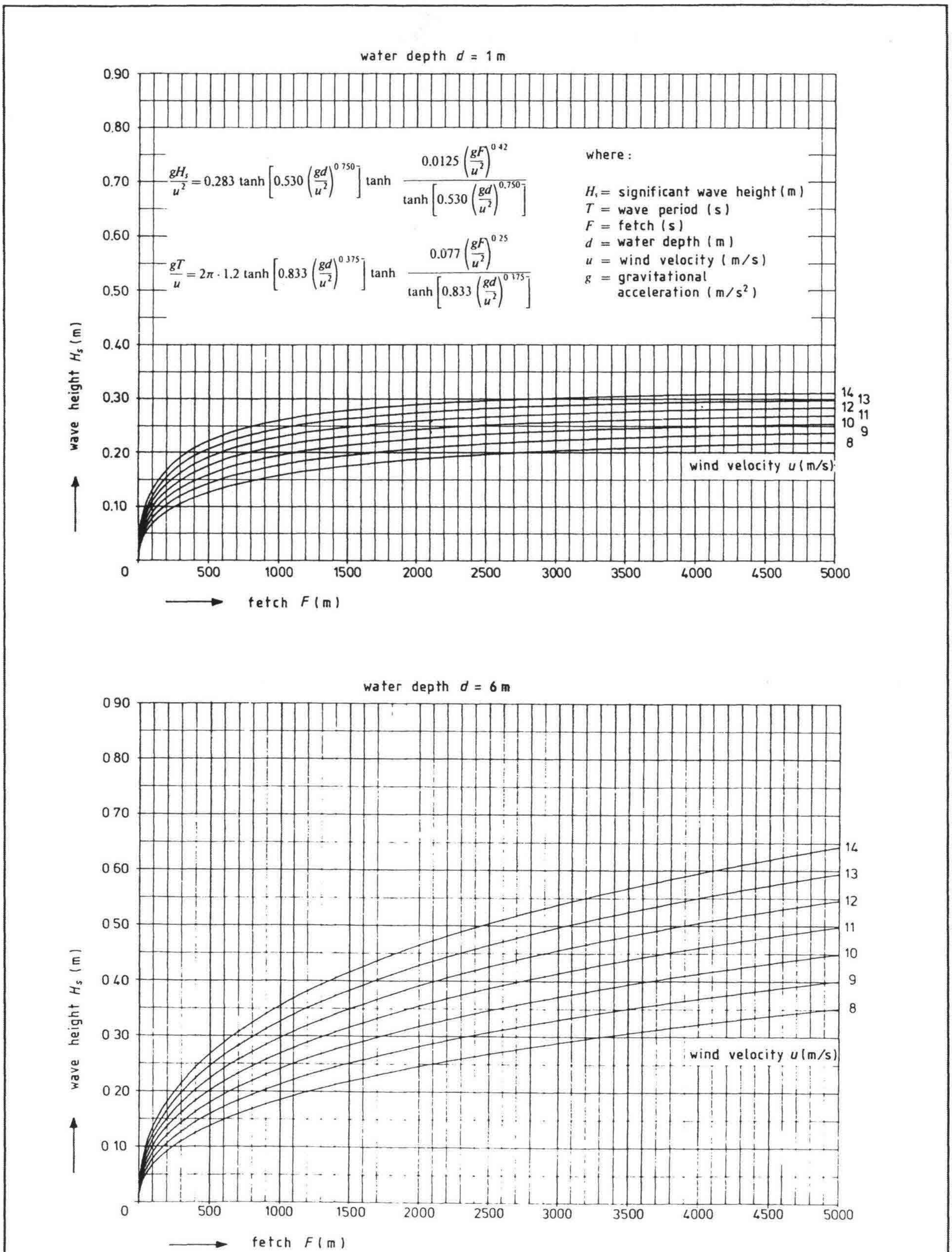
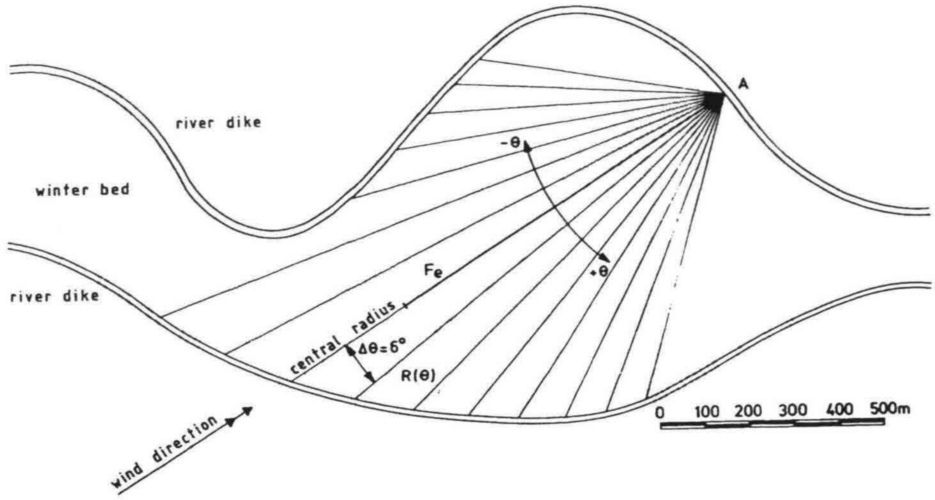


Figure 4.9: Wave height at a depth of 1 and 6 m



θ (degrees)	cosθ	$\cos^2\theta$	R(θ) (m)	$R(\theta).\cos^2\theta$
- 42	0.743	0.552	520	287
- 36	0.809	0.654	570	373
- 30	0.866	0.750	640	480
- 24	0.914	0.835	720	601
- 18	0.951	0.904	830	750
- 12	0.978	0.956	1340	1281
- 6	0.995	0.990	1240	1228
0	1.000	1.000	1140	1140
6	0.995	0.990	1050	1040
12	0.978	0.956	980	937
18	0.951	0.904	920	832
24	0.914	0.835	880	735
30	0.866	0.750	830	623
36	0.809	0.654	780	510
42	0.743	0.552	730	403

$\Sigma \cos\theta = 13.512$ $\Sigma R\theta.\cos^2\theta = 11220$

the effective fetch F_e follows from:

$$F_e = \frac{\Sigma R(\theta).\cos^2\theta}{\Sigma \cos\theta}$$

$$F_e = \frac{11220}{13.512} = 830 \text{ m}$$

Figure 4.10: determination of the effective fetch

the time, but during normal weather conditions they are of no importance. The period of these oscillations is as such that they may cause some resonance inside harbour basins. For example the resonance period of the harbour of IJmuiden is 35 minutes. So squall oscillations with a period of 30 - 40 minutes will resonate, and cause a strongly increased amplitude of the oscillation.

Figure 4.11 shows squall oscillations at several stations along the coast of Holland. It can be seen that the amplitude at IJmuiden and Scheveningen (measured inside the breakwaters) is much more than at Hook of Holland and Den Helder. The period of the oscillation is between 10 - 50 minutes, their half amplitude is in the order of 25 cm. In general one should add at the sea-coast the half-amplitude of the observed oscillations. Inside harbours one should make special calculations taking into consideration the effect of resonance. Along the tidal rivers and in estuaries one may reduce the values somewhat, proportional to what is given on gust bumps below.

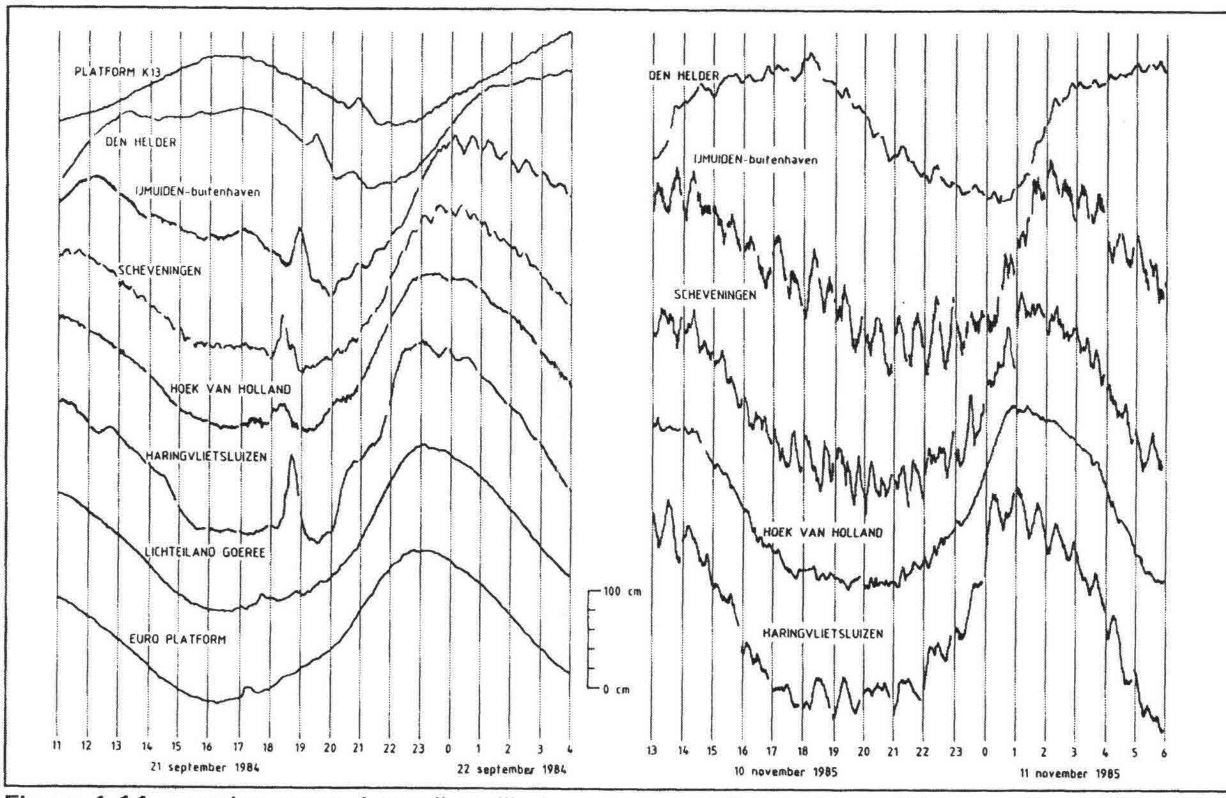


Figure 4.11: gust bumps and squall oscillations along the Dutch Coast. K13 is an oil rig 100 km off Den Helder, Lichteiland Goeree is 30 km off Hook of Holland and Europlatform is 60 km west of Hook of Holland.

Gust bumps are a different type of water level displacement. They are very pronounced, solitary waves. In figure 4.11 a gust bump is indicated. As can be seen this bump already existed at oil-rig K13, 100 km north of Den Helder. On can clearly see that it moves south and proceeded along the coast. The path of a gust bump can be followed quite clearly, contrary to the squall oscillation. The amplitude of the gust bump can be in the order of .4 - .6 m.

For the design of dikes in the Netherlands one has to take into account the following values:

at the coast	.6 m
in the mouth of estuaries	.3 m
5 - 15 km inside estuaries	.15 m
behind barrier islands	.10 m
20 km along tidal rivers	.05 m

However, the probability that these gust bumps occur simultaneously with the top of the storm is less than 1%. Therefore one is allowed to reduce the effect of a gust bump. The reduction is related to the wave run-up R. The surcharge for a gust bump (S_{gb}) can be determined as follows:

$$S_{gb} = B^2/(B+aR)$$

in which B is the amplitude of the gust bump, according to the above list, a is a coefficient with the value .25.

4.3.7 currents

In general the current is not a problem during design conditions. It is, however, a serviceability limit. Currents may cause erosion in front of the dike or may attack a grass slope. On the long term this will introduce failure. Therefore especially the toe should be protected against scouring by currents. The magnitude of the current can in general be determined using hydraulic computations. This is both valid for typical river dikes and for dikes along estuaries, where considerable tidal currents can be found.

4.3.8 other loads on a dike

Objects on the dike, such as for instance lampposts and trees, can transmit forces to the dike body that are caused by wind acting on these objects. These forces can have a detrimental influence on the strength an stability of the dike, especially under design conditions. However, it is difficult to give generally valid rules on this aspect. The magnitude of these forces as well as the effects can be checked in each particular case, assuming that the anchorage of the structures and object in the dike is such that their being blown over by the wind is reasonably excluded.

Apart from the question of closing the dike to traffic during high water levels, it is recommended that the traffic load on the dike should be taken into account in the design and the stability check. If a disaster should ever occur, the transport of heavy materials and equipment over the dike is necessary (it is than the only not-flooded road). It is recommended that a uniformly distributed design load be assumed of 15 kN/m^2 on a 2.5 m wide lane at the side of the critical slope, which is mostly the inner slope.
The load caused by the dead-weight of the dike must also be mentioned. This is particularly reflected in the surcharge of the subsoil in settlement analysis, as well as in the magnitude of the driving moment in the stability analysis of slopes. This load is determined by the geometry of the structure and the density of the soil types present in the dike; the degree of saturation of the material is also of importance.

The design shall take into account exceptional loads which can occur as a consequence of, for instance, (ship-)collisions, explosions, floating debris and ice on the river. However, these loads cannot or only scarcely be determined in the quantitative sense due to the unknown factors of size, effect, and probability of occurrence. Remarks on this subject can be made in a qualitative sense.
The danger of collision by a ship that has either swung out of line or broke from its moorings, occurs mainly in an outer bend on the river, when the foreland of the dike is either absent or situated far below the water level. Structures in the dike, such as retaining walls and lock gates are especially vulnerable. Experience has shown that considerable damage to a dike or levee can be caused by collision. Not only the revetment was damaged but also in several cases the water retaining earthbody itself.

Explosions can result in complete or partial weakening of the soil. Information on an insight into this phenomenon are as yet very limited. The possibility of explosions must be taken into account when explosives, such as ammunition and explosive gases (LPG) or fluids are stored -also in ships- in the vicinity of a dike.
Floating debris, such as trees, sleepers and ice can harm the revetment, in particular at locations outside the main current of the river or the tide, where there can still be a considerable wave action.
Floating ice can also result in ice-dams in the river, especially at location where the width of the river decreases. Such ice-dams will cause a considerable increase of the upstream waterlevel. In arctic rivers this rise may be in the order of several meters. The effect of ice is not included in these notes. For details see [PIANC, 1992]

4.3.9 biological effects

The design should also pay attention to loading as a consequence of biological activities: the burrowing of mice, moles, water-rats, coypus, rabbits, foxes and badgers. Particularly in dikes with a sand core, covered with clay it is very likely that this burrowing will result in serious harm or even undermine the covering clay revetment. When this take place on the river- or sea-side it may result in an elevated phreatic line in the dike body and it in not inconceivable that san from the core will be washed away, and that a dike breach will be the result. Attention is also drawn to the possibility of cavity formation caused by dying tree-roots or shrub-roots, and by trees that have been blown down. Tree and shrub vegetation also may cause shadow on the dike slope. In the Dutch climate this impedes the development of a proper grass cover.

4.4 the effect of ships

Banks and bed of inland waterways such as rivers and canals have to be designed taking into account the ship-induced water movements.
The definition of the relevant parameters for calculating the ship-induced primary and secondary ship waves are shown in de definition sketches of the following two figures and are concerning:

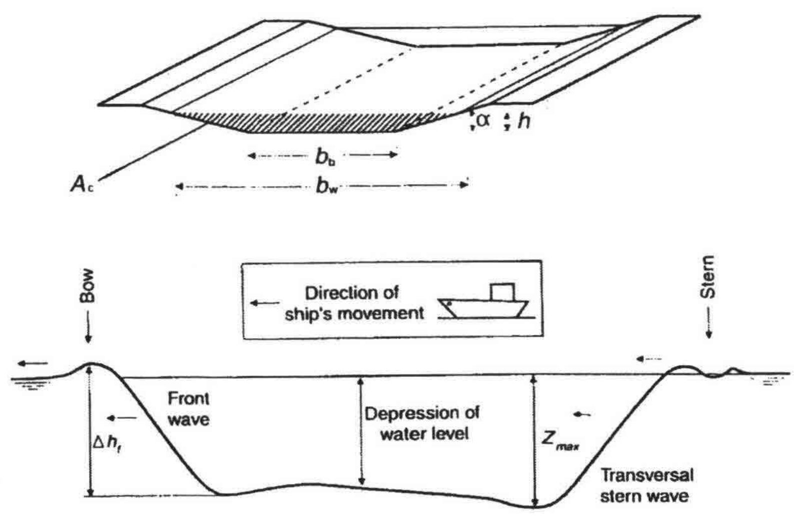


Figure 12: Definition sketch of ship induced water movements

- primary ship wave, consisting of:
  - front wave

- water level depression
- transversal stern wave;
- return current;
- secondary ship waves or interface peaks;
- outs jet.

Ship types, sailing behaviour (viz. ship speed and position in the waterway) and dimensions and geometry of the waterway determine the induced waver movements. Advanced calculation methods are available (e.g. see PIANC, 1987)which are implemented in the PC-program DIPRO (DImensioning PROtections). The relevant parameters for calculating the ship-induced primary and secondary ship waves are:

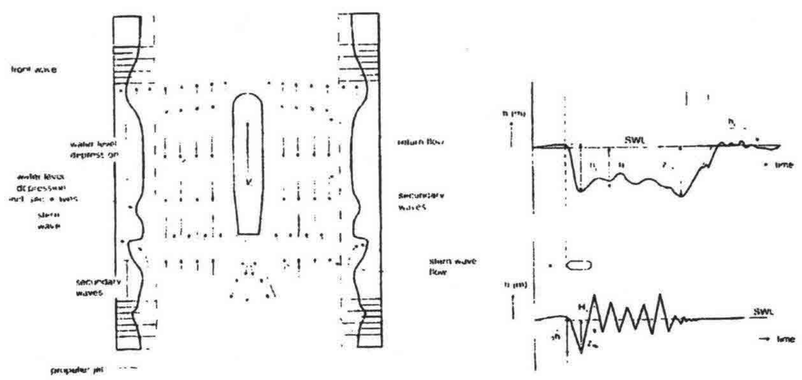


Figure 13: Characteristics of ship-induced water movements related to bank stability.

- ship length and beam width (L_s, B_s);
- ship sailing speed (V_s);
- loaded ship draught (or average empty draught);
- ship position, relative to the fairway axis (y) or bank (y_s);
- cross-sectional area of the waterway (A_c);
- waterdepth of the fairway (h);
- width of fairway at the bed (b_b) and at the waterline (b_w).

The waves give rise to run-up and run-down velocities, which may cause instability of rock used for bank protection.
A first estimate of the order of magnitude of the different water movement component can be obtained by the formulae presented in the calculation scheme in the following box.
For a more comprehensive discussion of ship-induced water movements in navigation channels is referred to PIANC [1987] and VERHEY [1983].

Calculation scheme of ship-induced water movements:

(1) determine vessel's submerged cross section (A_m):

$A_m = C_m\, b_s\, T_s$

b_s = width of ship;
T_s = draught of ship;
C_m = cross beam coefficient;
= 1 for push units and inland vessels;
= 0.9 for service vessels and tow boats;
= 0.7 for marine vessels;

==================================

(2) calculate limit speed of vessel (V_l):

$$F_L = \left[\frac{2}{3}\left(1-\frac{A_m}{A_c}+\frac{1}{2}F_L^2\right)\right]^{3/2}$$

with $F_L = V_L/\sqrt{g\, A_c/b_w}$

$$V_L = \sqrt{\frac{gL}{2*\pi}} \quad (for\ tugs)$$

(3) calculate actual speed (V_s) of vessel (or use admissible speed limit):
as a factor V_L:
$V_s = f_v\, V_L$ with

f_v = 0.90 unloaded ship
= 0.75 loaded ship

or (push/tow units):

$$V_s = 2.4\sqrt{\frac{A_c}{b_w}}\, e^{-2.9\frac{A_m}{A_c}}$$

==================================

(4) calculate mean water level depression (Δh):

$$\Delta h = \frac{V_s^2}{2g}\left[\alpha_s\left(\frac{A_c}{A_c^*}\right)^2 - 1\right]$$

with ($\alpha_s = 1.4 - 0.4\ V_s / V_l$
calculate net wet cross section of canal (A_c*) corrected for Δh
$A_c^* = b_b\ (h - \Delta h) + m(h - \Delta h)^2 - A_m$
calculate return flow (U_r):

$$\frac{U_r}{V_s} = \left(\frac{A_c}{A_c^*} - 1\right)$$

(5) calculate maximum water level depression ($\Delta\hat{h}$):

for: $b_w/L_s < 1.5$: $\quad \Delta\hat{h}/\Delta h = 1 + 2\, A_w^*$

for: $b_w/L_s \geq 1.5$: $\quad \Delta\hat{h}/\Delta h = 1 + 4\, A_w^*$

and return flow ($\hat{U}_r$):

for: $b_w/L_s < 1.5$: $\quad \hat{U}_r/U_r = 1 + A_w^*$

for: $b_w/L_s \geq 1.5$ $\quad \hat{U}_r\ /\ U_r = 1 + 3\, A_w^*$

where $A_w^* = y\ h/A_c$

==================================

(7) calculate front wave (Δh_t and i_t):

$$\Delta h_t = 0.1\Delta h + \Delta\hat{h}$$

$i_t = 0.03\ \Delta h_t$

(8) calculate stern wave (z_{max}, i_{sw}, i_{max} and u_{max}):

$$z_{max} = 1.5\Delta\hat{h}$$

$$i_{sw} = 0.64\left[\frac{V_s}{\sqrt{gh}}\right]^5$$

$$i_{max} = z_{max}/z_o$$

$$u_{max} = V_s\left[1 - \frac{\Delta D_{50}}{z_{max}}\right]$$

where:

$$z_o = 0.16\ (0.5b_w - 0.5b_s - y) - c_2$$

and coefficient

$$c_2 = 0.2\ to\ 2.6$$

(9) calculate secondary ship waves (H_i, L_i, T_i):

$$\frac{H_i}{h} = \alpha_i\frac{V_s^2}{gh}\left[\frac{s}{h}\right]^{-1/3}$$

$$L_i = \frac{2}{3}2\pi\frac{V_s^2}{g}$$

$$T_i = 5.1\ V_s/g$$

where
s = distance along the wave crests between slope and ship, s = y_s / sin (19.3°)
α_i = coefficient
= 1 for tugs and recreational craft and loaded conventional ships
= 0.35 for unloaded conventional ships
= 0.5 for unloaded push units

water level depression, front and stern waves
The average height of water level depression (Δh), front wave (Δ_f) and transversal stern wave (z_{max}) varies at the bank between 0.3 and 0.5 m; occasionally heights of 1.0 m will occur. The duration of the water level depression varies between 20 and 60 seconds depending on the type of ship speed. The period of the front and transversal stern is about 2 to 5 seconds.

secondary ship waves
In most cases push and tow-units or loaded conventional motor vessels are responsible for the severest primary wave (z_{max}) and tugs, survey vessels and recreational ships for the severest secondary waves (H_i). Secondary ship waves or interference peaks (H_i) will vary between 0.25 and 0.5 m. Maximum values of H_i are 1.0 m, generated by small, fast sailing ships. The wave period (T_i) is 2 to 4 seconds. The angle of wave incidence (β) relative to the normal of the bank is 55 degrees (approx.).
It should be noted that the characteristics of secondary ship waves have some proven similarity with wind waves regarding to the application of response to embankment structures and therefore the basic parameters of wind waves apply. The largest wave in a wave train of secondary waves have a height H_i and a length L_i, which can be approximated (for $V_s/\sqrt{(gh)} < 0.8$) with:

$$H_i = 1.2\alpha_i h\left(\frac{s}{h}\right)^{1/3}\frac{V_s^2}{gh}$$

$$L_i = 4.2\frac{V_s^2}{g}$$

where α_i is a coefficient depending on the type of ship (see scheme in box).

wave run-up and run-down velocities
Run-up (R_u) and Run-down (R_d) of ship induced waves may be approximated by a set of empirical relations. The formulae have been calibrated with typical vessels sailing on the Dutch inland waterways and should yet be regarded as very specific. Similar ship-wave parameters have been used as for wind waves, so ship-induced run-up (R_u) are written in terms of the similarity parameter ξ for ship waves:

$$\xi < 2.6: \quad R_u / H = \xi$$

$$2.6 < \xi < 3.0: \quad R_u / H = 6.5 - 1.5\,\xi$$

$$\xi > 3.0: \quad R_u / H = 2.0$$

Given the specific character of the above given formulae the reliability for an arbitrary case may be limited.
The highest run-up values occur due to the interference peaks or secondary ship waves (incidence β) and can be estimated with:

$$R_u / H_i = 2.0\,\xi\,\sqrt{\cos\beta}$$

This formula is valid for plane smooth surfaces and in order to obtain the effective run-up it should be multiplied by a roughness reduction factor (v_r) and (when relevant) for the presence of a berm (v_B).

Similarly, a set of specific formulae has been found for run-down (R_d) of ship-induced waves. As an approximation of the maximum run-down should be taken the maximum of either one of:

- transversal stern wave: $$\frac{R_d}{D_{50}} = 4.4 \left(\frac{Z_{max}}{\Delta D_{50}} - 0.3\ cotg\alpha\right)$$

and

- interference peaks: $$\frac{R_d}{D_{50}} = 1.0 \frac{H_i}{\Delta D_{50}} (10 \tan\alpha + 0.5) \sqrt{\cos\alpha}$$

For D_{50} the value of the material situated lower on the slope should be substituted.

Maximum run-up and run-down velocities are denoted by u_{Ru} and u_{Rd} respectively and can be expressed as a function of z/R_u and/or wave steepness (s_m).

Run-up velocities:

$u_{Ru}/\sqrt{(gH)} = 0.27\ s_m^{-0.5} (1 - z/R_u)^{0.5}$; for $z/R_u > 0$

$u_{Ru}/\sqrt{(gH)} = 0.2\ s_m^{-0.5}$; for $-0.4 < z/R_u < 0$

$u_{Ru}/\sqrt{(gH)} = 0.3\ (z/R_u)^{-2}$; ; for $z/R_u < -0.4$

Run-down velocities:

$u_{Ru}/\sqrt{(gH)} = 1.1\ (1 - z/R_u)^{0.5}$; for $z/R_u > -0.4$

$u_{Ru}/\sqrt{(gH)} = 1.18\ (z/R_u)^{-2.3}$; for $z/R_u < -0.4$

where: z = vertical coordinate relative to still water;
s_m = wave steepness = $2\pi H\ (gT^2)$.

Propeller jet velocities

The propeller jet induces water velocities at the bed. These are in particular of importance if a ship is manoeuvring, thus in and next to locks and near quay walls. The water velocities in the propeller jet of a sailing ship can for most situations be ignored. A relationship to assess the velocities at the bed yields (for ship speed V_s = 0, otherwise U_{max} is relative to the sailing-ship):

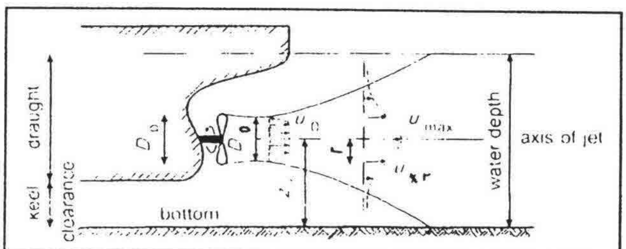


Figure 14: water movement due to propeller thrust

$$U_{max} = 1.15\alpha_p\ (P/\rho_w D_p^2)^{0.33} D_p/z_p$$

where:

P = installed power (W);
D_p = effective diameter of propeller, D_p = 0.7 to 1 times the real diameter (m);
z_p = distance between the propeller axis and the bed (m);
α_p = a coefficient (-); depending on the type of propeller, $0.25 < \alpha_p < 0.75$.

A rough estimate of the values to be expected at the bed is 2.5 m/s (keel clearance under the ship approximately equal to the diameter of the ship's propeller).

4.5 geotechnical conditions

For major structures a good geological analysis, based on the overall geological structures of the country, is of the utmost importance for an understanding of the geophysical and geohydrological conditions. The most important geological aspects are:
- geological stratification, formation and history
- groundwater regime
- seismicity

The main questions which a geotechnical investigation has to answer are:
- what kind of soil is found and at what depth, i.e. soils such as sand, clay and peat or sof rock such as limestone and calcareous sandstone, or very hard rock such as quartzite and basalt,
- what are the mechanical properties of the various soils with respect to their strength and deformation characteristics,
- is the soil fissured or weathered,
- will the soil degrade in (short) time.

The first step is to organize and design site investigations. The field programme forming part of the site investigation is complemented by laboratory testing and geotechnical calculations. The last and perhaps most difficult step is the integration of the result of the investigations and structural design, resulting in the final foundation design.

At the set-up and organization of the soil-investigation program the geotechnical engineers is confronted with the following questions:
- which soil data have to be collected,
- at what locations (number and depths),
- which site-investigation techniques and laboratory test should be performed,
- when is the programme to be carried out and,
- who will take care of the contracting work in the field, the laboratory tests and the interpretation of the results.

The answer of these questions will depend among others on:
- the boundary conditions stipulated by the client (time and money schedule);
- the knowledge, judgment and experience of the geotechnical engineer;
- the availability of existing data, for example topographical, geological and geotechnical maps;

- the phase of the design: for a preliminary design only global information over a wide area is needed to recognize the main geotechnical problems; in the final design phase or during the construction period detailed information on engineering soil parameters is needed;
- the type of geotechnical failure mechanisms involved;
- the availability and restrictions (including the terrain accessibility) of the investigation tools and the quality of the personnel to handle these tools.

A high quality investigation must be economically efficient in the sense that the cost of the investigation must be money well spent. The investigator must be able to justify each and every item in the site investigation in terms of the value of that item in building up the geotechnical model. The investigator should be able to show good and sufficient reason for undertaking each part of the investigation.

It is emphasized that there is no standard form of site investigation for a particular engineering work. Each site investigation should be regarded as a completely new venture. Several standardized investigation techniques have, however, been developed, of which the geological and geotechnical engineer can make use for obtaining the relevant data for his basic calculations and design criteria. Roughly spoken four types of site investigation methods can be distinguished:
- geophysical measurements from the soil surface;
- penetration tests, such as cone penetration and standard penetration tests;
- borings, including sampling and installation of observation wells;
- specific measurements, such as plate loading tests and nuclear density measurements.

Depending on the functional requirements coastal structures have to withstand a combination of actions induced by waves, currents, differences in water levels, seismiticy and other specific loadings (such as ship collisions or surcharges). These actions, including the dead-weight of the structure, have to be transferred to the subsoil in such a way that:
1. the deformations of the structure are acceptable and
2. the probability of instability is sufficiently low.

The transfer of actions through the structure to the subsoil involves changes in soil stresses (pore water pressures and effective stresses) in the soil layers and, in case of sloping structures, also in the structure itself. Particularly in soft soils the stress changes will gradually develop during a long period of time. Due to these changes in soil stresses the underlying and adjacent soil layers will deform vertically and horizontally while the shear strength of the soil may be reduced. As a consequence any structure built on top of the soil layers will deform too or may even lose its stability. This applies to both the design conditions (i.e. under extreme loadings) and the loadings during the construction period of the structure.

The changes in soil stresses and the associated deformations not only depend on the (hydraulic) loadings, but on the geometry (e.g. slope steepness), the structure weight and on the permeability, stiffness and shear strength of the subsequent structure and soil layers as well. For this reason the design of coastal structures has to be based on an integral approach of the interaction between the structure and the subsoil. A good knowledge of the main geotechnical properties of the soil layers and the construction materials is therefore needed. The main geotechnical limit-states that should be evaluated in the design of the sloping structures are:

- macro-instability of slopes due to failure along circular or straight sliding surfaces;
- settlements (and horizontal deformations) due to the self weight of the structure;
- micro-instability of slopes caused by seeping out of groundwater;
- piping or internal erosion due to seepage flow underneath the structure;
- liquefaction caused by erosion (flow sides) or by cyclic loading (wave actions or earthquakes);
- erosion of revetments at the outer slopes (or under water slopes) due to instable filters or local failure of top layer elements.

5 Design calculations

The degree of wave attack on a dike or other defence structure during a storm surge depends on the orientation in relation to the direction of the storm, the duration and strength of the wind, the extend of the water surface fronting the sea-wall and the bottom topography of the area involved. For coastal areas there is mostly a certain correlation between the water level (tide plus wind set-up) and the height of the waves, because wind set-up and waves are both

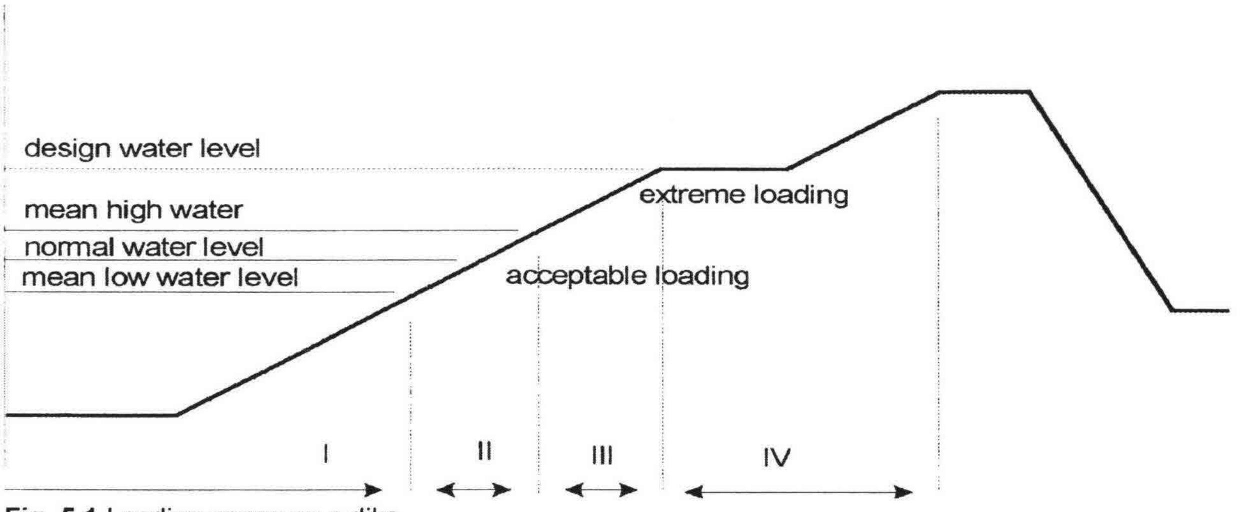


Fig. 5.1 Loading zones on a dike

caused by wind. Therefore, the joined frequency distribution of water levels and waves seems to be the most appropriate for the design purposes (also from the economical point of view).

For sea-walls in the tidal region, fronting deep water, the following approximate zones can be distinguished:

I the zone permanently submerged (not present in the case of a high level "foreshore");

II the zone between MLW and MHW; the ever-present wave-loading of low intensity is of importance for the long-term behaviour of structure;

III the zone between MHW and the design level, this zone can be heavily attacked by waves but the frequency of such attack reduces as one goes higher up the slope;

IV the zone above design level, where there should only be wave run-up.

A bank slope revetment in principle functions no differently under normal circumstances than under extreme conditions. The accent is, however, more on the persistent character of the wave-attack rather than on its size. The quality of the sea-ward slope can, prior to the occurrence of the extreme situation, already be damaged during relatively normal conditions to such a degree that its strength is no longer sufficient to provide protection during the extreme storm. The division of the slope into loading zones has not only direct connection with the safety against failure of the revetment and the dike as a whole, but also with different application of materials and execution- and maintenance methods for each zone.

Alternatives have to be generated during the conceptual design phase, the preliminary design phase and the detailed design phase in order to select the most suitable design. It is emphasized that for each design phase these alternatives should be worked out at a comparable level of detail. The same applies to the construction alternatives which may have a great influence on the total structure costs.

5.1 alignment and profile (effect of berms)

The interaction between waves and slopes is dependent on the local wave height and period, the external structure geometry (water depth at the toe), slope with/without berm, the crest elevation and the internal structural geometry (types, size and grading of revetments and secondary layers). The type of structure wave interaction is defined by the surf similarity number, sometimes called Iribarren-number or breaker parameter:

$$\xi = \frac{\tan\alpha}{\sqrt{H_s / L_o}} = \frac{1.25}{\sqrt{H_s}} T \tan\alpha$$

where:

ξ = surf similarity number
H_s = incident wave height
L_0 = wave length at deep water (= 1.56 T^2 in metric units)
T = wave period
α = slope angle of the front face

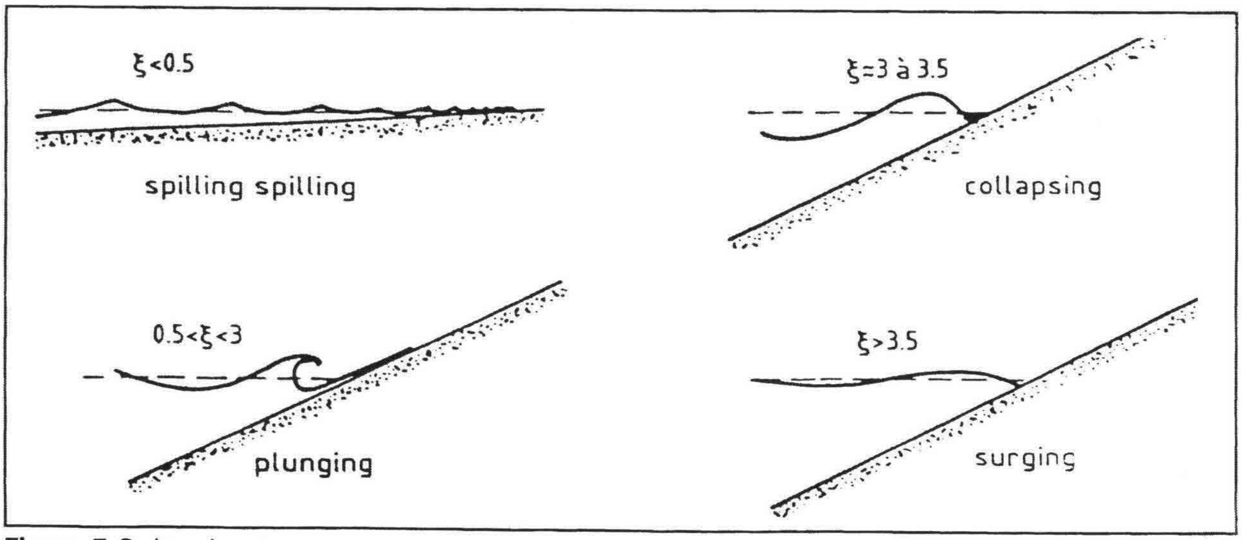


Figure 5.2: breaker types

For large values of the wave length or for large values of α (steep slopes), the wave behaves like a long wave, which reflects against the structure without breaking - a so called surging wave. For shorter waves and medium slopes waves will short and break, causing plunging breakers for ξ values in the range of 1 to 3. This figure is common along the Dutch coasts with slope angles of 1:3 to 1:5, wave periods 6 to 8 s and wave heights of 3 to 5 m. For mild slopes wave breaking becomes a more continuous process, resulting in a more gradual dissipation of wave energy. This type of breaking is called spilling. For the design of structures, surging and plunging breaker are of main importance. The area which suffers from wave-loading is bounded by the higher uprush and the lowest downrush point. Obviously this zone is varying with the tide. The value of maximum up and downrush is shown the following figure, both for impervious and pervious slopes.

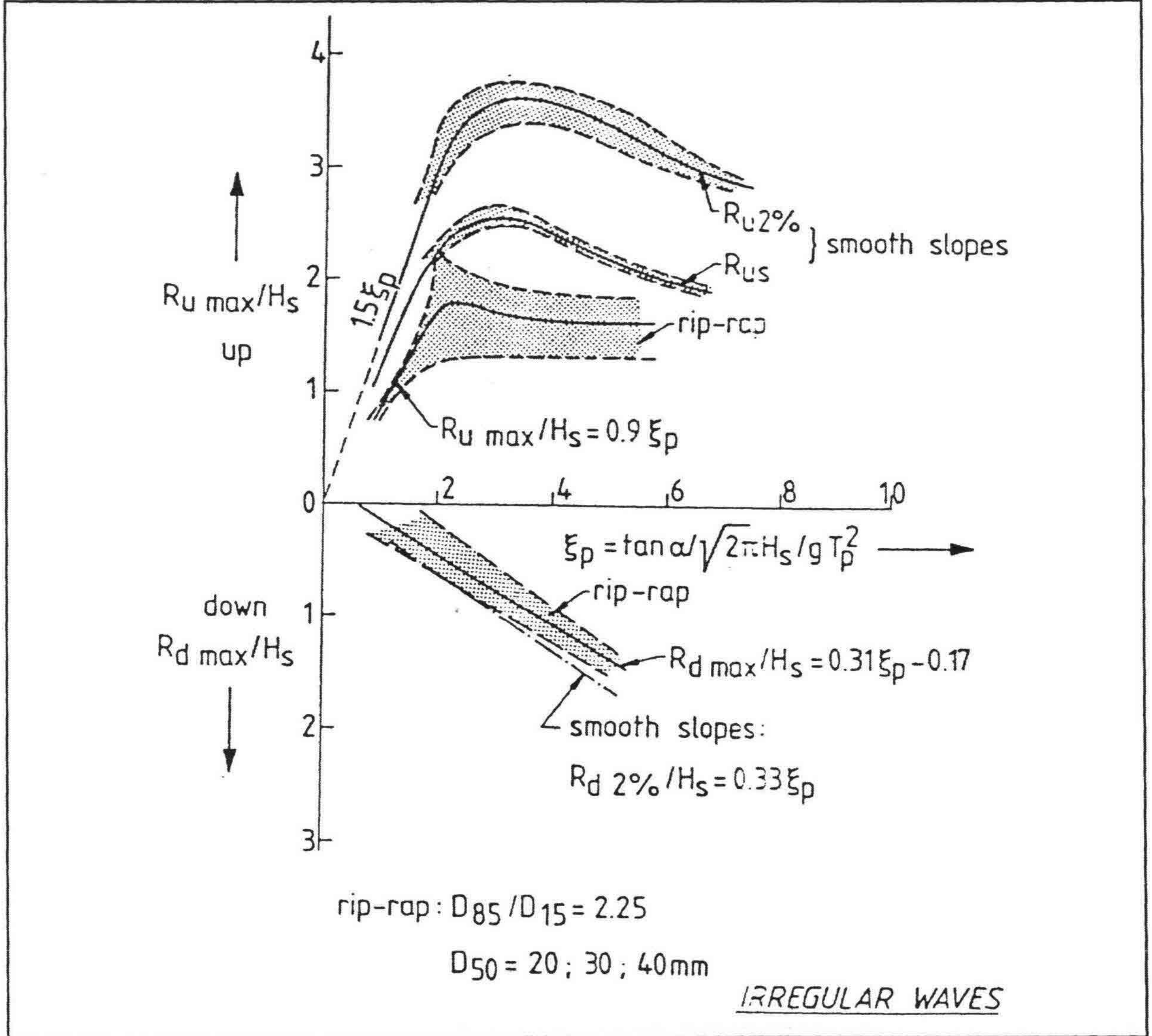


Figure 5.3: Run-up and run-down for smooth and riprap slopes

If the uprush exceeds the crest level, figures are no longer applicable.

No reliable formula are available to predict the maximum velocities during uprush and downrush. For surging and spilling breaker, numerical solutions have been obtained, which are, however, not yet operational. As first approximation, the maximum velocity, U_{max}, on a smooth slope can be computed by the following formula [PILARCZYK, 1990; CRESS-routine 241]:

design calculations

$$U_{max} = a \sqrt{gH_s}\, \xi^b$$

where:
H_s = significant wave height
g = gravity
a = coefficient equal to about 1 for irregular waves and
b = exponent equal roughly to 0.5

The gradient of the bank may not be so steep that the whole slope or the revetment can lose stability (through sliding). These criteria give, therefore, the maximum slope angle. More gently (flatter) slope leads to a reduced wave-force on the revetment and less wave run-up; wave energy is dissipated over a greater length. By using the wave run-up approach for calculations of the crest height of a trapezoidal profile of a dike for different slope gradients, the minimum volume of the embankment can be obtained. However, this does not necessarily imply that minimum earth-volume coincides with minimum costs. An expensive part of the embankment comprises the revetment of the waterside slope and the slope surface (area) increases as the slope angle decreases. The optimum cross-section (based on costs) can be determined when the costs of earth works per m^3 and those of revetment per m^2 are known. Careful attention is needed however, because the revetment costs are not always independent of the slope angle, e.g. for steep slopes the heavy protection is necessary while for the mild slopes the (cheaper) grass-mat can often provide a sufficient protection. Another point of economic optimization can be the available space for dike construction or improvement.
The common Dutch practice is to apply the slope 1 on 3 on the inner slope and between 1 on 3 on 5 on the outer (seaward) slope. The minimum crest width is 2.5 m.
The water-side berm is a common element in the Dutch dike construction. It could in the past lead to a reduction in the expenditure on stone revetments (on a very gently sloping berm a good grass-mat can be maintained) and it produced an appreciable reduction in wave run-up. Present practice in order to obtain a substantial reduction in wave run-up, is to place the outer berm at (or close to) water level of the design storm flood. If the berm lies too much below that level, the highest storm flood waves would not break beneath or on the berm and the run-up will be inadequately affected, and the grass-mat on the upper slope too heavily loaded by waves leading to possible erosion. For the storm flood berms at high design levels as in the Netherlands (freq. 10^{-4}) there are in general no problems with the growth of grass on the berm and the upper slope. However, there can be circumstances which require also the application of a hard revetment on the berm and even on a part of the upper slope i.e. when higher frequency of water level is applied, leading to more frequent overwash of the upper part by salt water due to the run-up or wave-spray (a common grass-mat can survive only a few salty events a year).

An important function of the berm can be its use as an access road for dike maintenance. In general care should be taken to prevent erosion of the grass-mat at the junction with the revetment. The abrupt change in roughness may lead to increase of surface turbulence and more local erosion. It is advisable to create a transition zone by applying the cell-blocks, geogrids or other systems allowing vegetation.

5.2 freeboard

General considerations on the height of a dike

The height of a dike was for many centuries based on the highest known flood level that could be remembered. It is evident that in this way the real risk of damage or the probability of flooding were unknown. Little was known about the relation between the cost to prevent flooding and the cost of the damage that might result from flooding. In the 20th century it was found that the occurrence of extremely high water levels and wave heights could be described adequately in terms of frequency in accordance with the laws of probability calculus. However the curves of extreme values, based on a relatively short period of observations, mostly have to be extrapolated into regions far beyond the field of observations with the risk for some uncertainties. After the 1953 disaster, the frequency of the risk of flooding was studies in the Netherlands in relation to the economic aspects. Finally it was decided to base the design of all sea dikes fundamentally on a water level with a probability of exceedance of 10^{-4} per annum. In the Netherlands the wind set-up is mostly incorporated in the estimated storm-surge level. If it is not a case, the wind set-up should be calculated separately and added to design water level.

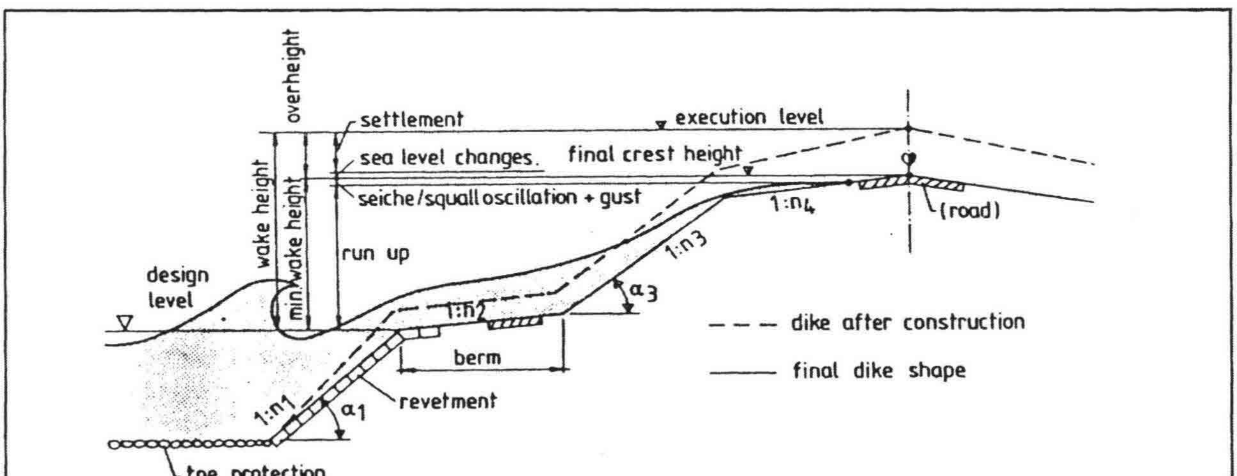


Figure 5.4: Determination of dike height

Besides the design flood level several other elements also play a role in determining the design crest level of a dike:

- Wave run-up or overtopping level (see Note) depending on wave height and period, angle of approach, roughness and permeability of the slope, and profile shape, Note:

  Wave run-up or wave overtopping criteria

  > Traditionally for sea-dikes the 2% criterion is used. This means that the height of the dike should be such high that (within one storm, determined by only one H_s) less then 2% of the wave-runup tongues should reach the crest of the dike. The background of this idea is that the quantity of water is in that case small enough to guarantee that the overtopped discharge will not cause any damage to the inner slope. The wave run-up formulas give always this 2% run-up value.

Nowadays there is a strong tendency to use the wave-overtopping as a criterion. It that case it is determined that during the storm the discharge over the crest should be less than q litres/second per meter of dike. The allowable value of q depends on the quality of the inner slope. At this moment the following values are used:

for an inner slope of undefined quality	q_{max} = 0.1 l/s/m'
for a good designed inner slope, 1:3	q_{max} = 1 l/s/m'
for a inner slope with hard protection	q_{max} = 10 l/s/m'
in very exceptional cases	q_{max} = 75 l/s/m'

The definition of a "hard protection" is a protection with asphalt, stones, concrete blocks, etc. In an exceptional case, the inner slope needs to be protected very carefully with hard pavement, there have to be utilities for the water to flow away, there has to be sufficient storage capacity landward of the dike and the water should not flow into houses and public buildings. The time that an overflow of more than 10 l/s/m' is allowed should never be longer than 3 hours.

- An extra margin to the dike height to take into account squall oscillations (seiches) and gust bumps (single waves resulting from a sudden violent rush of wind); this margin in the Netherlands varies (depends on location) from 0 to 3 m for the seiches and 0 to 0.5 m for the gust bumps,
- A change in chart datum (NAP) or a rise in the mean sea level (in the Netherlands: till now assumed roughly 0.25 m per century),
- Settlement of the subsoil and the dike-body during its lifetime.

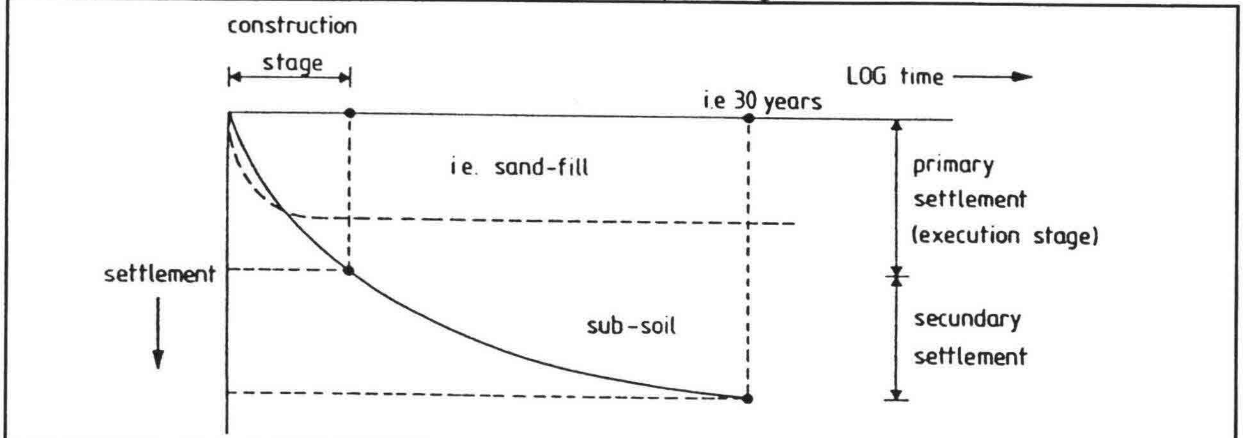


Figure 5.5: Settlement as a function of time

The combination of all these factors mentioned above, defines the freeboard of the dike. The recommended minimum freeboard is 0.5 m.

The height of a dike is:
Design water level
+ wave run-up or overtopping level
+ margin for * seiches
 * gust bumps
+ sea level rise + increase of tidal amplitude
+ settlement of subsoil

Note: "freeboard" (= total of all extras) should be at least 50 cm.

Design of dike height

5.3 wave run-up and overtopping

5.3.1 Wave run-up

In all designs it is common use to apply the 2% run-up. There is no real physical justification of the number 2, but all other design criteria are based upon this 2%. Therefore it is not advised to use other values, except in very special cases.
For an other run-up exceedance than 2%, one may use the Rayleigh distribution:

$$\frac{R_n}{R_{2\%}} = 0.71\sqrt{-\frac{1}{2}\ln(n)}$$

where
n = exceedance percentage (e.g. 0.01 for the 1 % run-up).
The value 0.71 come from the relation between the 2 % run-up and the significant run-up (13.5 % run-up).

In the period 1990-1993 research has provided new formulas for the calculation of run-up. In the following section the new values are presented.
The effective run-up (R), on an inclined structure can be defined as

$$R = R_{2\%}\ \gamma_R\ \gamma_B\ \gamma_\beta\ \gamma_h$$

where

$R_{2\%}$	=	run-up on smooth plane slopes, defined as the vertical height above still water level, which is exceeded by 2% of the waves in a wave field,
γ_R	=	reduction factor due to slope roughness and permeability,
γ_B	=	reduction factor due to berm
γ_β	=	reduction factor due to oblique wave attack
γ_h	=	reduction factor due to shallow water

For random waves $R_{2\%}$ can be expressed by [VAN DER MEER, 1993]

$$R_{2\%}/H_s = 1.6\ \xi_p \qquad \text{for } \xi_p \leq 2$$

and

$$R_{2\%}/H_s = 3.2 \qquad \text{for } 2 < \xi_p < 4$$

where

H_s	=	significant wave height,
ξ_p	=	breaker parameter for the peak-period (research has indicated that run-up can better be described using the peak-period instead of using the mean period; usually the peak period is 1.1 to 1.25 times the mean period and also the peak period is nearly equal to $T_{1/3}$).

For very narrow-banded spectra, one may reduce the total wave run-up with approx. 20 %.

design calculations

For a wave steepness of about 5% (typical storm value for the North Sea Coast) and a typical North Sea spectrum, one obtains the so called "Old Delft Formula" commonly used in the past for calculation of 2% run-up ($R_{2\%}$) on the Dutch sea dikes, viz.

$$R_{2\%} = 8\ H_s\ \tan\alpha$$

which is valid for $\cot\alpha \geq 3$ and relatively smooth revetments.

example:

*We have a wave with a H_s of 2.5 m, a T_p of 8 seconds, and a smooth slope of 1:4. The Surf-Similarity Parameter ξ is 1.58. Thus R = 2.5 * 1.6 * 1.58 = 6.32 m. With the Old Delft Formula R = 8 * 2.5 /4 = 5.0 m. This is less. With the New Formula a run-up of 5 m can be calculated when the peak period is T_p = 6.3 seconds. It is clear that especially in case of swell waves, the Old Delft Formula is not very reliable and underestimates the run-up.*

The constants 1.6 and 3.2 in the run-up formula do not describe the average of the observations, but they describe a line describing a run-up which is not exceeded by 90 % of the observations. This is the value which is recommended for design purposes. When one uses a probabilistic approach, one should use the average and the standard deviation of the constant. The average values are 1.5 and 3.0; the variance for these "constants" is $V = \sigma/\mu = 0.06$.

example:

*Suppose I want to know the 2%-run-up which is not exceeded in 98 % of the cases in the example given above. The average run-up is 2.5 * 1.5 * 1.58 = 5.92 m. The standard deviation of the constant is 0.06 * 5.92 = 0.35 m. So 98 % can be found using the Normal Distribution: 5.92 + 2.88 * 0.35 = 6.93 m*

varying slope

In case of a non-constant slope angle (for example when the slope below a berm is different from a slope above a berm), one has to calculate an equivalent slope. This can be done with the following procedure:

* determine a point on the slope at a depth H_s below the Still Water Line.
* determine a point on the slope at a height H_s above the Still Water line.
* determine the horizontal distance between these two points.
* if the berm is included in the found distance, subtract the berm width from the found distance.
* the found distance divided by $2H_s$ gives the cotangent of the slope.

Example:

*Suppose a dike has a 1:4 slope below the berm and a 1:3 slope above the berm. The berm is at 1 m above SWL, the H_s is 2.5 m. The first point is 4 * 2.5 = 10 m from the water line. The upper point is (1.0 * 4 + 1.5 * 3) = 8.5 m from the water line. So the total distance (not including the berm width) is 18.5 m. 18.5/(2*2.5) = 3.7. So the equivalent slope is 1:3.7.*

slope roughness

The reduction factors for surface roughness and permeability, γ_R can be roughly estimated as follows:

Type of slope (covering layer)				Reduction factor γ_f	Older factors
Smooth concrete or asphalt				1.0	1.0
Smooth and closed placed blocks				1.0	0.9
Gras(3cm) and rough/permeab.pitched stones				.90-1.00	0.85-0.90
1 layer of rock (H_s/D = 1.5 to 3)				.55-0.60	0.80
2 or more layers of rock (H_s/D = 1.5 to 6)				.50-0.55	0.50-0.55
Blocks as roughness elements: Height = f_h, width = f_b					
Name of system	f_h/f_b	f_b/H_s	Coverage ratio		
1/25 block	0.88	0.12-0.24	1/25	.75-0.85	
1/9 block	0.88	0.12-0.19	1/9	.70-0.75	
half block height	0.44	0.12-0.24	1/25	.85-0.95	
only above SWL	0.88	0.12-0.18	1/25	.85-0.95	
wide block	0.18	0.55-1.10	1/4	.75-0.85	
Ribs: f_b/H_s = 0.12-0.19 and f_L/f_h = 7 (optimum) where f_L = distance between the ribs				.60-0.70	

Table: Reduction factors γ_f for different cover layers

It is also possible to place artificial roughness elements on the slope, so called ripples. The width of a ripple is given by f_b, the height by f_h and the distance between the ripples with f_L. For ripples with a $f_b/H_s = 0.12 - 0.19$ and a $f_L/f_h = 7$ the reduction-coefficient is the order of 0.6 to 0.7. This is the optimum ripple lay-out.

berm reduction

In case of slopes with a berm (see Figure) the run-up will be reduced by a factor γ_b . The effect of a berm with a constant width (B) is maximum when the berm is situated approximately at the average water level ($d_B < 0.5 H_s$, see definition scheme in the figures). It has furthermore been found that the run-up diminishes with increasing berm-width although the reduction rapidly falls off once a certain minimum width is exceeded, i.e. $B = 0.25 L_0$ for non- and weak-breaking waves, and $B = 4H_s$ for strong breaking waves.

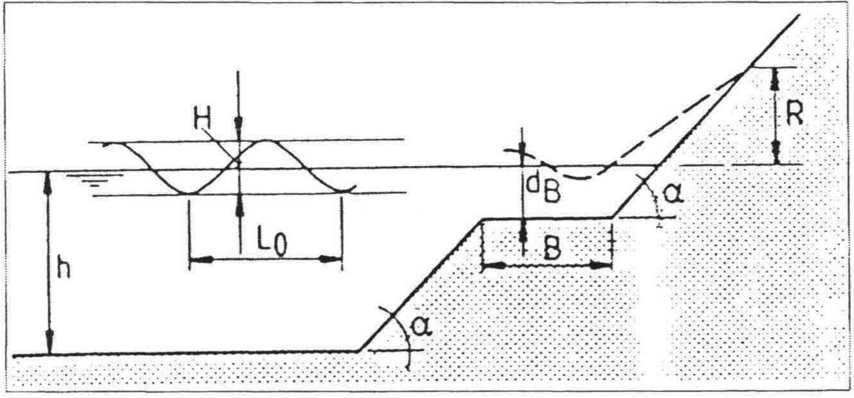


Figure 5.6: berm reduction

VAN DER MEER [1993] presented a computational rule for berm width reduction for a horizontal berm:

$$\gamma_b = 1 - r_b + 0.5 r_b \left(\frac{d_B}{H_s}\right)^2 \qquad (\text{min } 0.6 \textit{ and } \text{max } 1.0)$$

$$r_b = \frac{B/H_s}{2\,COT\alpha + B/H_s}$$

in which d_B is the depth of the berm below design water level and B is the berm width.
In case of a non-horizontal berm one should calculate the parameter r_b with the following formula:

$$r_b = 1 - \frac{\tan\alpha_{eq}}{\tan\alpha}$$

in which the equivalent berm-slope is determined by drawing a line from a point H_s below the berm to a point H_s above the front-level of the berm. Tan α_{eq} is therefore not related to the Still Water level. See figure.

The effect of a berm becomes nearly zero when it lies more than $\sqrt{2}$ H_s above or below the Still Water Line.
In those case when the berm lies approx. $\sqrt{2}$ H_s above the SWL, the computed run-up can be higher than the height of the berm. In those cases the real run-up will never exceed the height of the berm, especially when the berm has a substantial width.

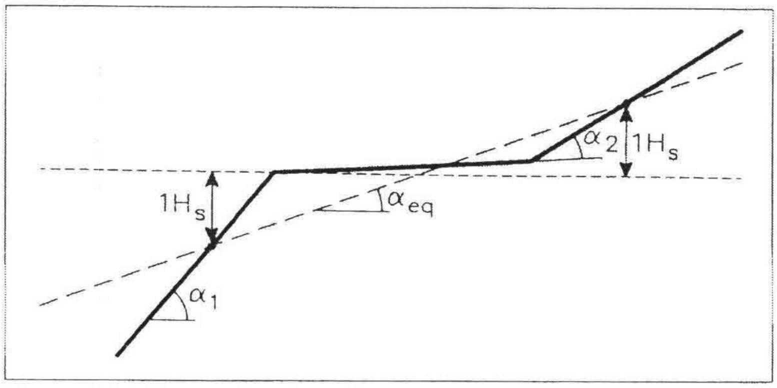


Figure 5.7: Determination of the equivalent berm slope

Oblique wave attack
Oblique wave attack, under angle β can be roughly taken into account by γ_β. For longcrested waves this can be computed with:

$$\gamma_\beta = \cos(\beta - 10^\circ) \qquad \textit{for} \qquad \beta < 65^\circ$$

For $\beta > 65^\circ$, $\gamma_\beta = 0.6$. This parameter is a non-linear function of the angle of incidence of the waves. There is a clear difference between short-crested and long crested waves. In case of real swell it is advised to use a value of γ_β of 1.05 for $10^\circ < \beta < 30^\circ$. In all other cases one should apply the line for short-crested waves. For the magnitude of γ_β is referred to the figure.

For shortcrested waves, the reduction can be calculated with:

$$\gamma_\beta = 1 - 0.0022\beta$$

In this formula one should enter the value of β is degrees. The formula is valid from 0° to 90°.
For $\beta > 65^\circ$, R_n should never be less than H_s !

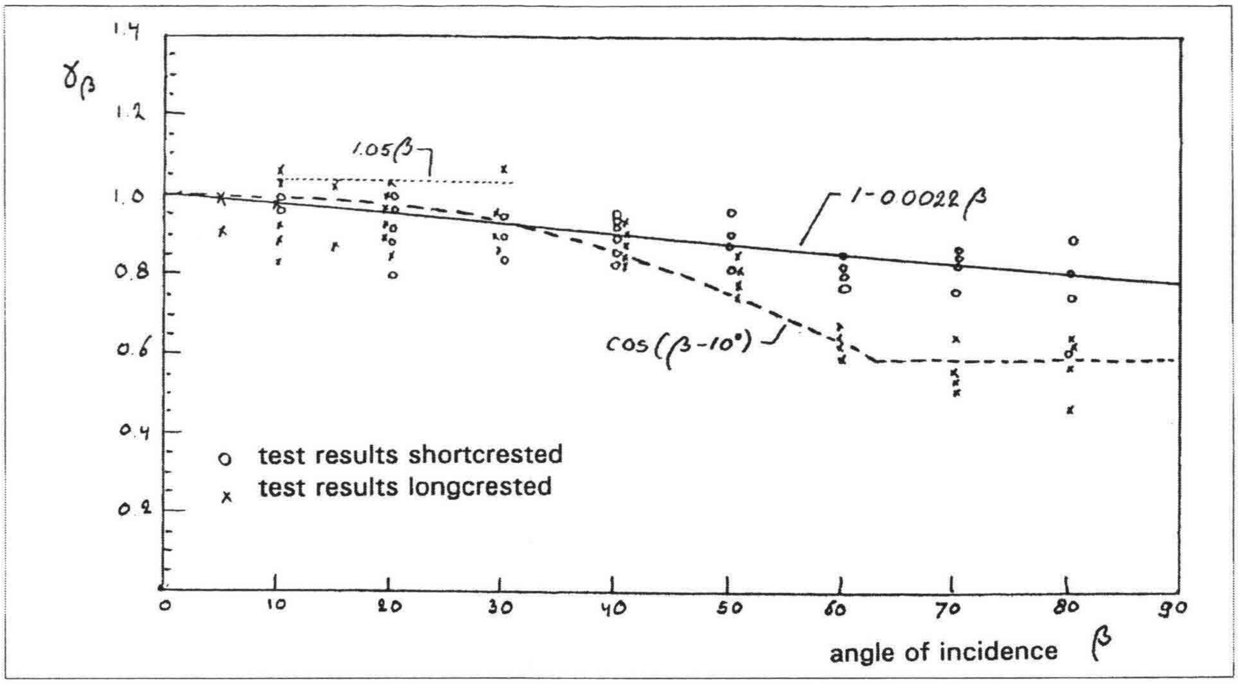


Figure 5.8: run-up reduction for oblique wave attack

the effect of shallow water

The reduction for depth limited situation (shallow water on a gentle sloping foreshore) is given by:

$$\gamma_h = \frac{H_{2\%}}{1.4H_s} = 1 - 0.3\left(4 - \frac{H}{H_s}\right) \qquad for \quad \frac{h}{H_s} < 4$$

$$\gamma_h = 1 \qquad for \quad \frac{h}{H_s} \geq 4$$

where $H_{2\%}$ is the wave height at the toe of the structure, exceeded by 2% of the waves, and H_s is the significant wave height at the toe of the structure.

Shape of the wave spectrum

Depending on the wave spectrum, i.e. the anticipated maximum wave height, and the type and permeability of revetment, type of subgrade, the run-up can vary reasonably and thus, the slope protection has to be more or less extended. For particular cases model investigation may give a proper answer.

In case of a spectrum with clearly two peaks, one may determine an equivalent period T_{peq}. This equivalent period can be used in the run-up and overtopping formulas. The method is explained in the figure.

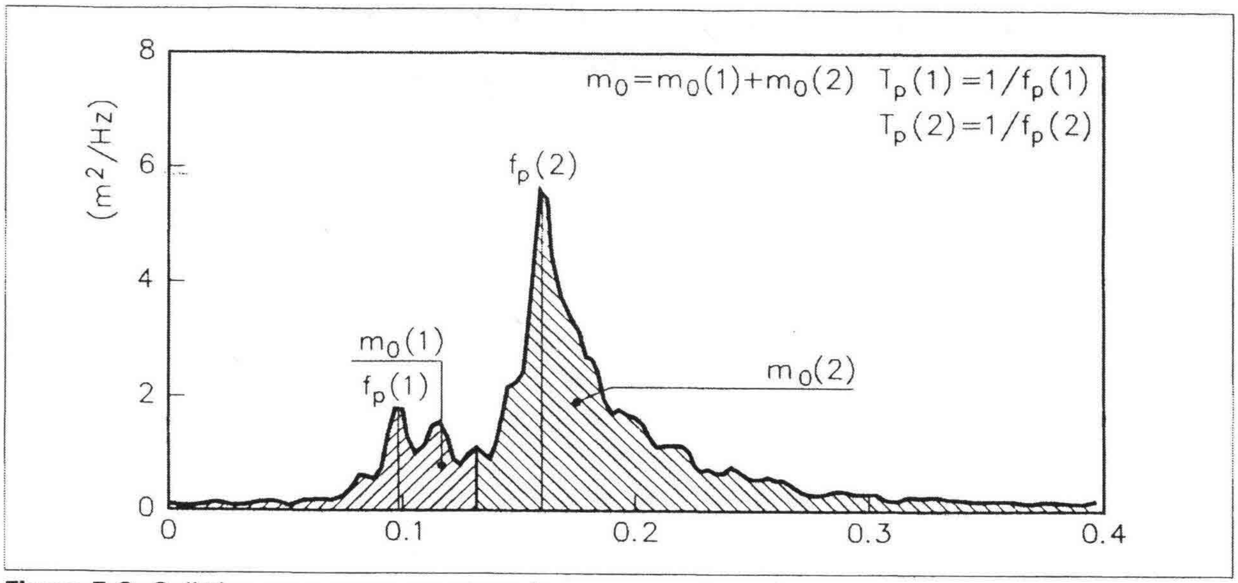


Figure 5.9: Splitting up a wave spectrum in two separate parts

The two-peaked spectrum is divided into two parts, from both parts (and from the whole spectrum) the zero-order moment is determined. Both wave-height and wave period of both components determine the equivalent peak period, because of the direct relation between wave-height and moment ($H_s = 4\sqrt{m_0}$). The formula to be used is:

$$T_{peq} = \sqrt[4]{\frac{m_0(1)}{m_0} T^4{}_p(1) + \frac{m_0(2)}{m_0} T^4{}_p(2)}$$

For the parameters, see above figure.

The lower limit of slope area attacked by waves (where a primary protection is necessary) can be roughly defined by

$$\frac{R_d}{H_s}(down) = 0.8\xi + 0.5 \qquad for \quad \xi < 2.5$$

and

$$\frac{R_d}{H_s}(down) = 2.5 \qquad for \quad \xi \le 2.5$$

Remark: R_d is not the point until where the water runs down the slope, but this is the point until where one has to take wave attack into account.

Below this limit, if necessary, slope protection has to be designed on the base of occurring return flow (shipwaves) or on the base of longshore current or (orbital-) velocities of wind waves.

The above method for calculation of wave runup can be found in CRESS-routine 241.

5.3.2 Wave overtopping

The amount of wave overtopping is of importance for the stability of the protection structure (crest and inner slope) and for the prediction of inundation. Thus, the amount of overtopping can be decisive for the choice of the crest-level of the structure.

The independent parameters which determine the overtopping of waves coincide entirely with those for wave run-up. The dependent parameter is wave overtopping, which may be expressed quantitatively in a variety of ways.

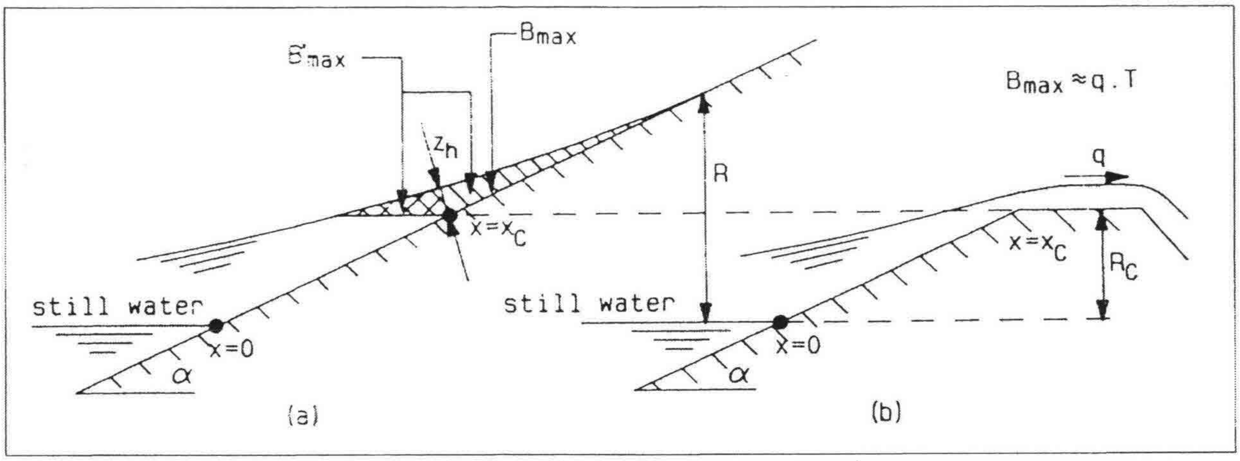


Figure 5.10: Definition sketch for the hypothesis of wave overtopping

Following the definitions in above figure, one may define the momentary overtopping volume per wave. The maximum quantity of water stored above a certain location on the slope per wave (B_{max}), as measured in the run-up experiments without overtopping, will overtop per wave (T) if the crest of the dike (R_c) would be situated below the level of max. run-up (R). Based on the tests with regular waves, the storage area B can be defined as:

$$B_{max} = 0.1(\cot \alpha)^{1.5} \; (R\text{-}R_c)^2 \quad \text{for } R \geq R_c$$

and $\quad q = B_{max}/T \quad$ = overtopping discharge per wave

This equation can be of use for dimensioning of the splash area. For the maximum discharge $R = R_{2\%}$ may be introduced and for the significant discharge $R = R_{2\%}/1.4$.

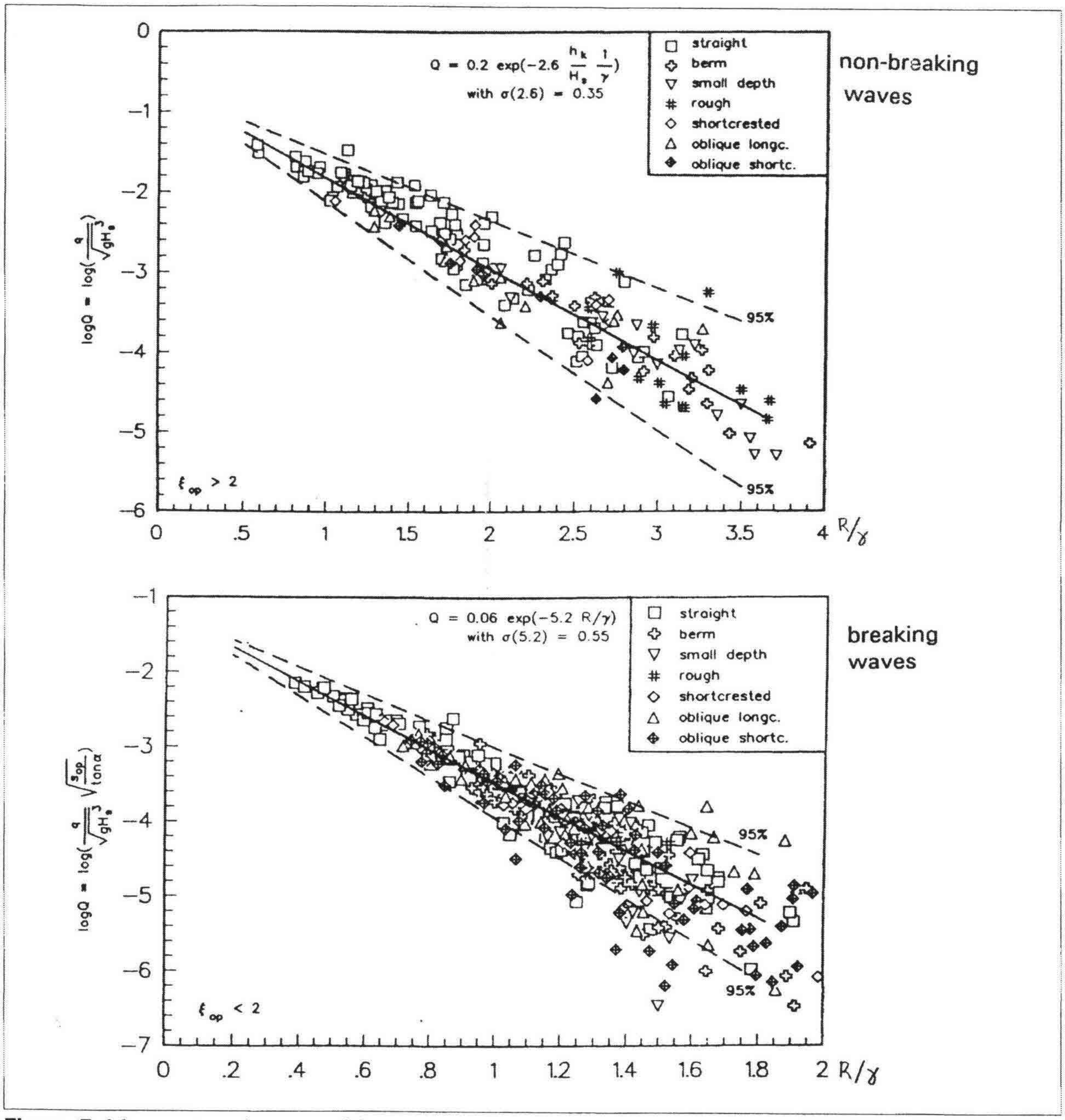


Figure 5.11: overtopping quantities

Recent research commissioned by the Dutch Technical Advisory Committee on Water defences and undertaken by Delft Hydraulics [VAN DER MEER, 1993] resulted in some data on the quantity of overtopping discharge.
Wave overtopping can be described by:

$$Q = a \exp\left(b \frac{R}{\gamma}\right)$$

in which Q is the dimensionless discharge, a and b are coefficients, R is a dimensionless freeboard and γ is the total reduction for the influence of berms, roughness, depth limitations and oblique wave attack.

Wave overtopping on dikes has to be divide in overtopping by plunging (breaking) and surging (non-breaking) waves. The transition between plunging and surging has been defined as ξ = 2.

For plunging (breaking) waves the above equation becomes:

$$Q = 0.06 \exp(-4.7 \frac{R}{\gamma})$$

with:

$$Q = \frac{q}{\sqrt{gH_s^3}} \sqrt{\frac{h/L_o}{\tan\alpha}} \quad and \quad R = \frac{h_k}{H_s} \frac{1}{\xi}$$

where:

q = the overtopping rate (m^3/s per m)
h_k = the crest freeboard (m)

For surging (non-breaking) waves this equation becomes:

$$Q = 0.2 \exp(-2.3 \frac{R}{\gamma})$$

with:

$$Q = \frac{q}{\sqrt{gH_s^3}} \quad and \quad R = \frac{h_k}{H_s}$$

The total reduction-coefficient is $\gamma = \gamma_b \gamma_f \gamma_\beta \gamma_h$; which are the reduction components for berms, roughness, oblique wave attack and for depth limited wave attack. These components are identical to those computed for the run-up formula.

The reduction for the roughness and the oblique angle of incidence are identical to the reduction-factors for run-up calculation. Only in case of shortcrested waves one should use:

$$\gamma_\beta = 1 - 0.0033\beta$$

The reduction for striking waves (β = 90°) becomes 0.7.

Like in case of run-up, also in the overtopping formula the values 4.7 and 2.3 are 90% exceedance values. The real averages are 5.2 and 2.6. The standard deviation of the first constant is σ= 0.55, for the second constant the standard deviation is σ= 0.35; see also the figure.

For the computation of overtopping is referred to CRESS-routine 242.

5.4 Example of a dike height calculation

5.4.1 Sea dike

Design a dike for a design frequency of 1/100 per year and of 1/1000 per year. The dike will be located along a shallow sea.

<u>Given</u> *(data are taken from Hook of Holland, The Netherlands; MSL is used as reference level):*

- *Exceedance line of waterlevels (=astronomical tide + storm surge)*
  - *1/100 ==> 3.5 m above MSL*
  - *1/1000 ==> 4.25 above MSL*
- *Exceedance line of wind-speeds*
  - *1/100 ==> 22 m/s*
  - *1/1000 ==> 25 m/s*
- *Wave height on deep water (wave period is 10 seconds)*
  - *1/100 ==> 6.5 m*
  - *1/1000 ==> 7.5 m*
- *Depth in front of the dike: 5 m below MSL.*
- *Slope of the dike 1:4 (1 vertical, 4 horizontal), berm located at Still Water Level (SWL) with a width b = 10 m.*

The dike is near to the coast, so the maximum gust-bump should take into account. For the Netherlands the gust bump is 0.5 m. The dike is not located along a harbour basin, so seiches are not relevant.

The design life of the dike is 50 years. The current sea-level rise is 20 cm/century. The anticipated sea-level rise is more. However, it is not included in this design, but the design has to be in such a way that it is possible to increase the crest level of the dike after 25 years without many difficulties. In this area, the amplitude of the tide is also increasing with 10 cm per century. So for 50 years this is approximately 5 cm.

Summary of levels

Frequency	*1/100*	*1/1000*
Design level	*3.50 m*	*4.25 m*
Gust bump	*0.50 m*	*0.50 m*
Seiches	*0.00 m*	*0.00 m*
Sealevel rise	*0.15 m*	*0.15 m*
Design water level	*4.15 m*	*4.90 m*

The design wave height is depth-limited; in this situation a breaker index $\gamma = 0.5$ may be used.

Frequency	*1/100*	*1/1000*
Waterdepth	*5+4.15= 9.15 m*	*5+4.90= 9.90 m*
Design wave	*4.45 m*	*4.95 m*

The wave run-up can be calculated with the "New Delft" run-up formula. It is also possible to calculate the dike height for a given allowable overtopping. In this case an allowable overtopping of 1 l/s/m dike is selected and a friction $\gamma = 0.95$ is used. The calculation results are:

Frequency	*1/100*	*1/1000*
Total run-up	*6.92 m*	*8.28 m*
Freeboard ($q = 1$ l/s/m)	*7.88 m*	*8.54 m*

From the above results, it is clear that for this case the overtopping criterion is more strict than the run-up criterion. Especially for the 1/100 Frequency, the difference is considerable. However, because all design criteria are based on 2% run-up, these values will be used. One has to realise that the amount of overtopping during the heavies part of the storm is much more than 1 l/s/m.

The design crest level can be found from:

Design crest level = design water level + freeboard + expected settlement of the subsoil.

The expected settlement of the subsoil depends very much on the type of subsoil at the location. For seadikes in the Netherlands which are build on top of good quality clay subsoil, one may expect a settlement during the design life time of the dike (50 years) of approximately 0.5 m.

Frequency	*1/100*	*1/1000*
Ultimate crest level (above MSL; using run-up)	*4.15 + 6.92 = 11.10 m*	*4.90 + 8.28 = 13.20 m*
Construction crest level (above MSL; including settlement)	*11.5 m*	*13.7 m*

5.4.2 Lake dike

In case of a lake, the computation is somewhat different. Suppose the dike is located along a lake of 40 km width, and a depth of 10 m. Near the shore there is a shallow zone of 1 km wide and 3 m deep (the Average Lake Level, A.L.L.) is used as reference level). Of course, the waterlevel depends very much on the wind-set up. The following wind set-up can be calculated:

Frequency	1/100	1/1000
Wind speed	22 m/s	25 m/s
Wind setup	0.27 m	0.35 m

For both cases, the wave conditions at the transition of the deep and shallow zones are approximately $H = 2$ m and $T = 5$ s (Bretschneider formula). This results in the following run-up results (for a dike with an outer slope of 1:4 and a berm width of 10 m at SWL):

Frequency	1/100	1/1000
Run-up	1.70 m	1.73 m
Freeboard (q = 1 l/s/m)	1.59 m	1.63 m

In this case, the overtopping criterion is less strict than the run-up criterion.

In The Netherlands, the subsoil below a lake-dike is usually very soft. Therefore, a settlement of 1 m for the coming 50 years is not a bad guess. However this value may vary considerably (up to 3 m).

Frequency	1/100	1/1000
Wind setup	0.27 m	0.35 m
Run-up	1.70 m	1.73 m
Settlement	1.00 m	1.00 m
Construction crest level (above A.L.L.)	3.0 m	3.1 m

5.5 Example of a berm width optimization

It is clear that making a wider berm will decrease the required dike height. But what is the optimum berm width? This depends on the const of earthomoving, the cost of land aquisition and the cost of the revetment. In the following example this is worked out. The prices used are realistic prices used in the Netherlands in 1995.

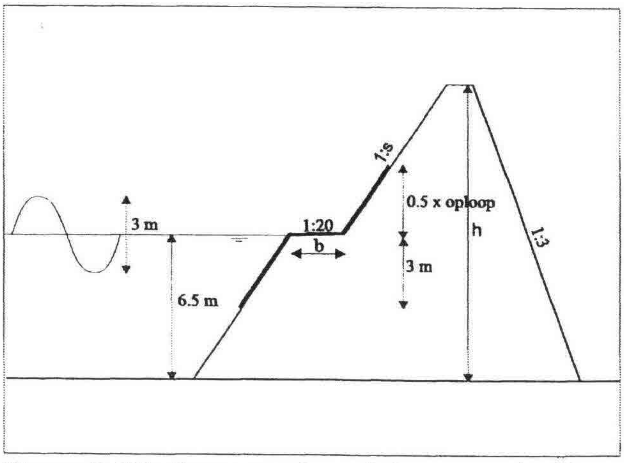


Figure 5.12: Cross section of the example dike

Given is a wave condition near the toe of the dike with shortcrested waves ($H = 3$ m and $T = 8$ seconds). The slope of the berm (at

Still Water Level) is 1:20, the slope protection is smooth, the depth in front of the dike (below SWL) is 6.5 m. (in case of deep water in front of the dike, overtopping would be more). The crest width has to be 2 m, the inner slope is 1:3. The ground level at the inner side is identical to the bottom level at the outer side.

The dike crest level has to be determined for the q = 0.1, 1 and 10 l/s/s criteria. This results in the following values of dike heights (in m above the bottom):

Slope / Overtopping / Berm width	1 : 3			1 : 5			1 : 8		
	0.1 l/m/s	1.0 l/m/s	10 l/m/s	0.1 l/m/s	1.0 l/m/s	10 l/m/s	0.1 l/m/s	1.0 l/m/s	10 l/m/s
0 m	17.9	15.4	12.9	13.2	11.7	10.2	10.4	9.5	8.6
5 m	15.7	13.7	11.7	12.1	11.1	9.8	10.2	9.3	8.5
10 m	14.3	12.6	10.8	11.9	10.7	9.5	10.0	9.2	8.4
15 m	13.4	11.9	10.4	11.4	10.3	9.2	9.8	9.0	8.3

Note: For the slopes 1:3 and 1:5 a roughness coefficient of 1 is selected (e.g. for Basalton), for the 1:8 slope a roughness-coefficient of 0.95 is used (grass).

*Simple geometry [Vol = 6.5 b + (s/2 +1.5)*h² + 2h] leads to the following volumes of the dike (in m³). Using a price of ƒ15 per m³ leads to following cost figures:*

Slope / Overtopping / Berm width	1 : 3			1 : 5			1 : 8		
	0,1 l/m/s	1,0 l/m/s	10 l/m/s	0,1 l/m/s	1,0 l/m/s	10 l/m/s	0,1 l/m/s	1,0 l/m/s	10 l/m/s
0 m	997 (ƒ 14955)	742 (ƒ 11134)	525 (ƒ 7875)	723 (ƒ 10850)	571 (ƒ 8564)	437 (ƒ 6548)	616 (ƒ 9235)	515 (ƒ 7731)	424 (ƒ 6360)
5 m	803 (ƒ 12051)	623 (ƒ 9345)	467 (ƒ 6999)	642 (ƒ 9635)	548 (ƒ 8213)	436 (ƒ 6544)	625 (ƒ 9377)	527 (ƒ 7902)	447 (ƒ 6703)
10 m	707 (ƒ 10606)	567 (ƒ 8497)	437 (ƒ 6548)	655 (ƒ 9829)	544 (ƒ 8165)	445 (ƒ 6675)	635 (ƒ 9525)	549 (ƒ 8234)	470 (ƒ 7048)
15 m	663 (ƒ 9945)	546 (ƒ 8192)	443 (ƒ 6642)	640 (ƒ 9602)	543 (ƒ 8136)	455 (ƒ 6817)	645 (ƒ 9680)	561 (ƒ 8415)	493 (ƒ 7395)

*For this dike, polderland has to be bought. The amount of land to be purchased depends on the width of the dike. In the following table the total width of the dike is given [W = b + 2 + (3+s)*h], as well as the price of polderland to be purchased, assuming a price of ƒ 7.5 per m².*

Slope / Overtopping / Berm width	1 : 3			1 : 5			1 : 8		
	0,1 l/m/s	1,0 l/m/s	10 l/m/s	0,1 l/m/s	1,0 l/m/s	10 l/m/s	0,1 l/m/s	1,0 l/m/s	10 l/m/s
0 m	109 (ƒ 820)	94 (ƒ 708)	79 (ƒ 596)	108 (ƒ 807)	96 (ƒ 717)	84 (ƒ 627)	116 (ƒ 873)	107 (ƒ 799)	97 (ƒ 724)
5 m	101 (ƒ 759)	89 (ƒ 669)	77 (ƒ 579)	104 (ƒ 778)	96 (ƒ 718)	85 (ƒ 640)	119 (ƒ 894)	109 (ƒ 820)	101 (ƒ 754)
10 m	97 (ƒ 734)	88 (ƒ 657)	77 (ƒ 576)	107 (ƒ 804)	98 (ƒ 732)	88 (ƒ 660)	122 (ƒ 915)	113 (ƒ 849)	104 (ƒ 783)
15 m	97 (ƒ 730)	88 (ƒ 663)	79 (ƒ 596)	108 (ƒ 812)	99 (ƒ 746)	91 (ƒ 680)	125 (ƒ 936)	116 (ƒ 870)	108 (ƒ 812)

On the slope, one has to place a slope protection. It is assumed that only half the run-up height has to be protected with a slope protection. The slope protection has to be continued to a value of 3 m below the berm. Assumed is that the price for the protection is:

ƒ 120 per m² for a 1:3 slope
ƒ 100 per m² for a 1:4 slope
ƒ 80 per m² for a 1:5 slope

For the 1:8 slope no slope protection is foreseen, so only the cost of the berm are included. The cost of the berm cover is assumed to be ƒ 50 per m².
This leads to the following values (m² of slope protection and cost)

Slope / Berm width	1 : 3	1 : 5	1 : 8
0 m	8.3 m 22.6 m2 (ƒ 2713)	5.0 m 28.0 m2 (ƒ 2244)	3.0 m 0 m2 (ƒ 250)
5 m	6.7 m 19.8 m2 (ƒ 2630)	4.4 m 26.3 m2 (ƒ 2351)	2.8 m 0 m2 (ƒ 250)
10 m	5.6 m 17.8 m2 (ƒ 2641)	4.0 m 25.0 m2 (ƒ 2500)	2.6 m 0 m2 (ƒ 250)
15 m	5.0 m 16.6 m2 (ƒ 2747)	3.6 m 23.7 m2 (ƒ 2648)	2.5 m 0 m2 (ƒ 250)

Adding all the cost from the above tables leads to the following total cost table:

Slope	1 : 3			1 : 5			1 : 8		
Overtopping / Berm width	0.1 l/m/s	1.0 l/m/s	10 l/m/s	0.1 l/m/s	1.0 l/m/s	10 l/m/s	0.1 l/m/s	1.0 l/m/s	10 l/m/s
0 m	ƒ 18,489 (100%)	ƒ 14,555 (79%)	ƒ 11,184 (60%)	ƒ 13,901 (75%)	ƒ 11,525 (62%)	ƒ 9,419 (51%)	ƒ 10,108 (55%)	ƒ 8,529 (46%)	ƒ 7,084 (38%)
5 m	ƒ 15,439 (84%)	ƒ 12,643 (68%)	ƒ 10,207 (55%)	ƒ 12,765 (69%)	ƒ 11,283 (61%)	ƒ 9,536 (52%)	ƒ 10,521 (57%)	ƒ 8,972 (49%)	ƒ 7,707 (42%)
10 m	ƒ 13,980 (76%)	ƒ 11,795 (64%)	ƒ 9,765 (53%)	ƒ 13,132 (71%)	ƒ 11,397 (62%)	ƒ 9,835 (53%)	ƒ 10,940 (59%)	ƒ 9,583 (52%)	ƒ 8,331 (45%)
15 m	ƒ 13,422 (73%)	ƒ 11,602 (63%)	ƒ 9,984 (54%)	ƒ 13,062 (71%)	ƒ 11,530 (62%)	ƒ 10,144 (55%)	ƒ 11,366 (61%)	ƒ 10,035 (54%)	ƒ 8,957 (48%)

If one disregards the no-berm+very-steep alternatives, the conclusion is that for all dikes with a revetment all alternatives are in the same price range. For example, if one looks to the 1 l/s criterion (and excluding the no-berm-extreme-steep-slope case and the 1:8 slopes, the price difference is only ƒ 1360, which is in the order of 10 % of the total costs. In case one can avoid a revetment by using a 1:8 slope, in the given case this is clearly the cheapest solution. Lowering the 0.1 l/s criterion to a 10 l/s criterion lowers the total cost in the order of 25 to 30%.
The general conclusion from this example is that it is impossible to tell of beforehand what is the (economically) optimal berm with and slope.

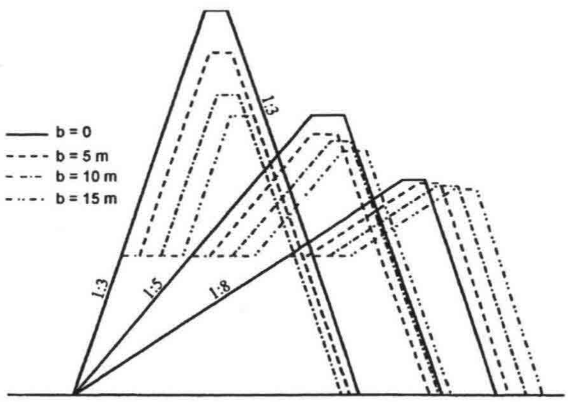


Figure 5.13 Relation between slope and berm width on dike height (for overtopping q = 0.1 l/s/m)

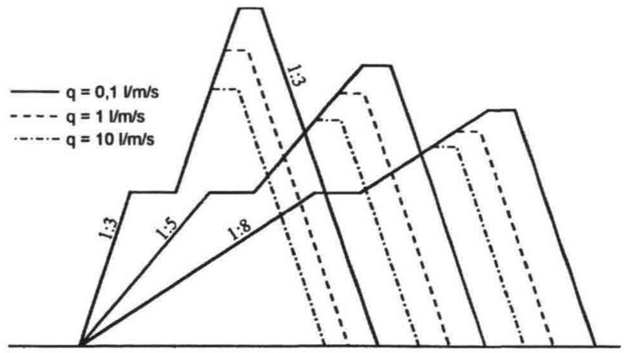


Figure 5.14 Influence of slope and overtopping on dike height (berm width b = 10 m)

6 Revetments and slope protection

By definition, a revetment is a slope protection designed to protect and stabilize a slope that may be subject to action by currents and waves. To fulfil this function, the following aspects have to be taken under consideration in the design process:

a. stability (toplayer, sublayer, subsoil, foundation)
b. flexibility (following the settlement without influencing the stability)
c. durability (toplayer, asphalt, concrete, geotextile, cables)
d. possibility of inspection of failure (monitoring of damage)
e. easy placement and repair (local damage)
f. overall safety (primary or secondary defence, geometry of foreshore, etc.)
g. additional functional requirements, i.e. special measures for reduction of run-up and/or roads for maintenance activities (berm requirements, etc)

The best revetment is one which combines all these functions.

There are numerous types of revetments. They can be distinguished in several groups:

- rip-rap or uniform rock is applied in many projects.
- concrete armour units; often applied where rip-rap was not possible.
- regular placed stones or concrete blocks; applied in cases where wave attack was not very extreme, but a very stable, and flexible revetment is required.
- other types of revetments, like:
  - sand-bags or sand sausages filled with sand, gravel, cement, rock, etc.
  - gabions (wire mesh containers with relatively course material.
  - gravel
  - asphalt
  - grass on a clay layer
  - geotextiles

6.1 choice of a revetment

The strength of the various materials relate to the various loads. In the following table a overview is given of the importance of the loads for each type of revetment:

Revetment type	Waterlevel fluctuations (hours)	Waves (seconds)	Wave impacts (seconds/100)
stones, rip-rap,etc	o	xxx(velocity)	o(except subsoil)
placed blocks	o	xxx(pressure)	o(except subsoil)
impervious layers	xxx	o	xx
grass on clay	x	xxx(velocity)	x

o means almost unimportant, while the number of crosses is a measure of the relative importance. Loose stones are too permeable for slowly varying waterlevel fluctuations, while wave impacts will do no harm to the stones (they are already broken); the subsoil can liquefy when it is not compacted. For placed blocks the situation is more or less the same, only the mechanism of failure in wave attack is completely different. Impervious layers are sensitive to waterlevel fluctuations; the phreatic level cannot follow a sudden drop. This would be true all the more for waves, but these are again too fast to lift the layer, because there can be only a very limited flow to where the layer is lifted. The very fast wave impacts now can damage the layer due to stresses in the material. Grass is sensitive to all wave action on the slope and is not used in the zone with the maximum wave-attack, while a clay layer may be relatively imperious, so waterlevel fluctuation can cause some problems.

These differences depend on the local load situation and result in comparative designs with different costs. Besides costs there are other reasons to choose or reject a revetment-type. Some of them are summarized in the following list.
a. strength: Every construction can be designed strong enough, but a material with a steep damage curve is less favourable because of the possibility of progressive failure.
b. flexibility: When settlement or scour plays an important role, a construction that preserves its strength while it follows the changes can influence the choice.
c. construction: Easy and fast construction can mean lower costs; sensitivity for tolerances can play an important role here.
d. maintenance: How easy is it to inspect or to repair a (part of a) construction ?
e. sustainability: How does the material react to physical, chemical or biological processes; it should not age too easily.
f. environment: How well does the protection fit into the environment, what other functions have to be fulfilled ?
g. recreation: A concrete slab is not the ideal place for a picnic.

6.2 Stability of the top layer

6.2.1 Loose Material (rock armour layers)

Many methods for the prediction of rock size of armour units designed for wave attack have been proposed in the last half century. Those treated in more detail are the Hudson formula as used in the Shore Protection Manual [CERC, 1984] and the formula derived by VAN DER MEER [1988]. The original Hudson formula is written as:

$$M_{50} = \frac{\rho_s H^3}{K_D \Delta^3 \cot\alpha}$$

K_D is a stability coefficient taking into account all other variables. K_D values suggested for design correspond to a "no damage" condition where up to 5 % of the armour units may be displaced. In the 1973 edition of the Shore Protection Manual the values given for K_D for rough, angular stone in two layers on a breakwater trunk were:

$K_D = 3.5$ for breaking waves
$K_D = 4.0$ for non-breaking waves.

The definition of breaking and non-breaking is different from plunging and surging as described in chapter 5.1. A breaking wave in the Hudson formula means that the wave breaks due to the foreshore in front of the structure directly on the armour layer. It does not describe the type of breaking due to the slope of the structure itself.
The main advantages of the Hudson formula are its simplicity and the wide range of armour units and configurations for which values of K_D have been derived. The Hudson-formula has also many limitations. Briefly they include:

- Potential scale effects due to the small scale at which most of the tests were conducted;
- The use of regular waves only;
- No account taken in the formula of wave period or storm duration;
- No description of the damage level;
- The use of non-overtopped and permeable core structures only.

The use of $K_D\cot\alpha$ does not always best describe the effect of the slope angle. It may therefore be convenient to define a single stability number without $K_D\cot\alpha$. Further, it may often be more helpful to define in terms of a linear armour size, such as the typical or nominal diameter. The Hudson formula can be re-arranged to:

$$\frac{H_s}{\Delta D_{n50}} = N_s = (K_D \cot\alpha)^{1/3}$$

Using this structural parameter, based on an extensive series of model tests, Van der Meer developed two formulae for the stability of armour for plunging and surging waves respectively. These formulae are:

For plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2 P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$

For surging waves:

$$\frac{H_s}{\Delta D_{n50}} = 1.0 P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha}\; \xi_m^{P}$$

The transition from plunging to surging waves can be calculated using the critical value of ξ_m:

$$\xi_{mc} = \left[6.2 P^{0.31} \sqrt{\tan\alpha}\right]^{1/(P+0.5)}$$

For $\cot\alpha \geq 4.0$ the transition from plunging to surging does not exist and for these slope angles only the plunging wave formula should be used. The permeability factor P for various structures is given in figure 6.1. This factor should lie between 0.1 and 0.6. Both the Van der Meer formulae and the Hudson formula can be found in the CRESS-package.
Design values for the damage level S are shown in the table. The level "start of damage", S = 2-3, is equal to the definition of "no damage" in the Hudson formula. The maximum number of waves N which should be used is 7500. After this number of waves the structure is more or less in equilibrium.

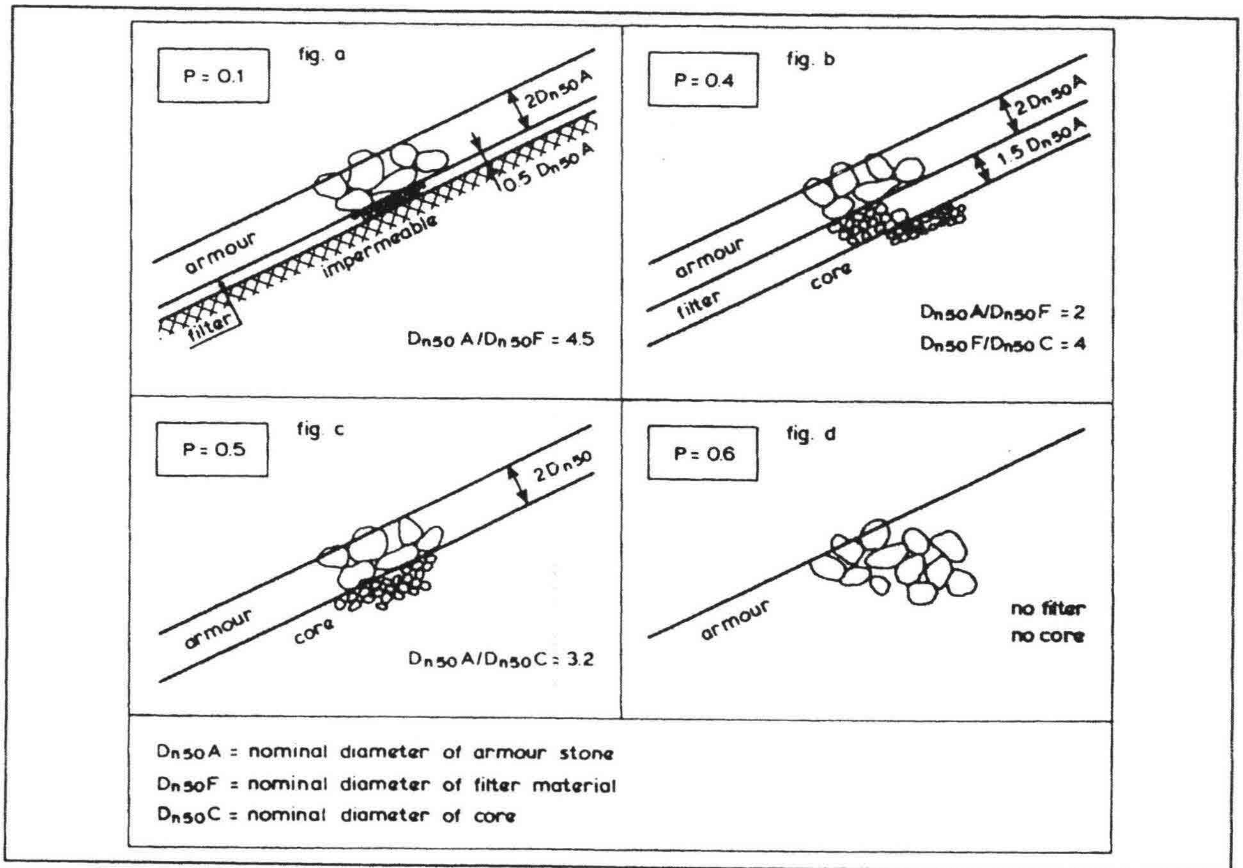


Figure 6.1: Permeability factor P for various structures

Slope	Initial damage	Intermediate damage	Failure
1:1.5	2	3-5	8
1:2	2	4-6	8
1:3	2	6-9	12
1:4	3	8-12	17
1:5	3	8-12	17

The wave steepness should lie between $0.005 < s_m < 0.06$ (almost the complete possible range). The relative mass density varied in the tests between 2000 kg/m^3 and 3100 kg/m^3, which is also almost the possible range of application.
The N_s in the Hudson formula is only related to the slope angel $\cot\alpha$. Therefore a plot of N_s versus $\cot\alpha$ shows only one line for the Hudson formula. The Van der Meer-formulae take into account the period (or the wave steepness), the permeability of the structure and the storm duration. Especially for revetments the permeability is an important factor, because in a revetment the permeability is usually much less than in a breakwater.
Figure 6.2 shows a sample calculation, indicating the difference between the Hudson and Van der Meer formulae.

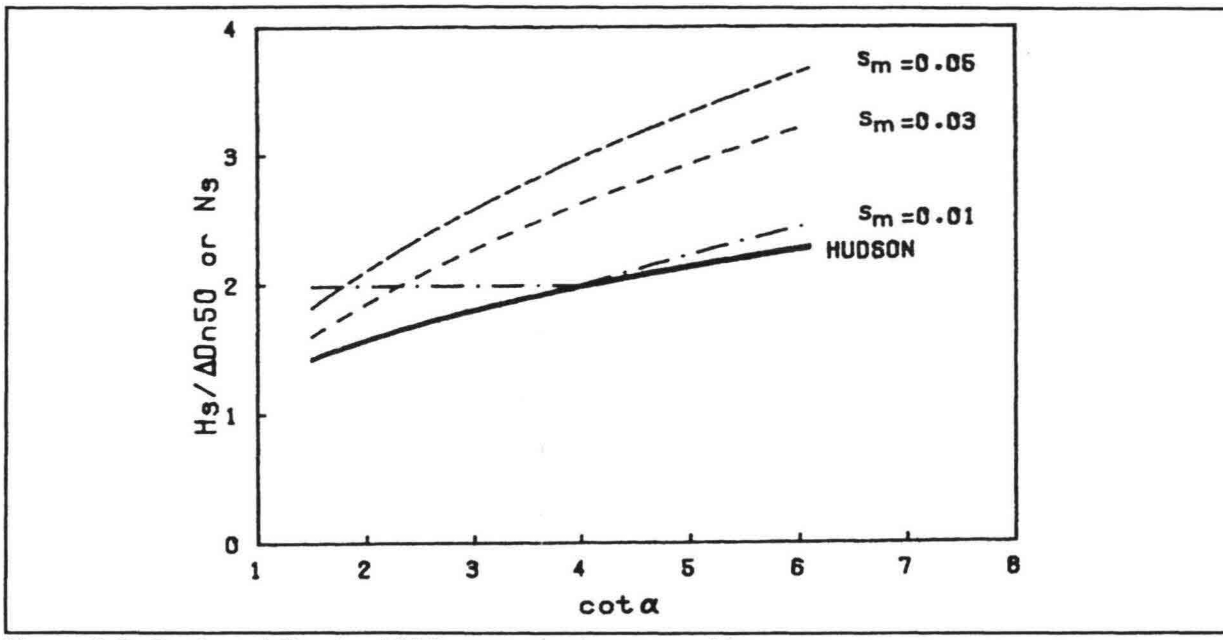


Figure 6.2: Rock stability for 1000 waves and a permeable structure

In these lecture-notes the stability of other type of armour units (concrete blocks, tetrapods, etc.) is not discussed, because their application for revetments is very limited. That topic is discussed in detail in the lectures on Breakwater design.

6.2.2 placed blocks

Placed blocks come in many shapes; the human fantasy is the only limiting factor. The variation is in the coherence; pitched, pinched, connected with cables or geotextile, or interlocked. Another variation is the shape of the upper side of the blocks which is meant to reduce the wave run-up.

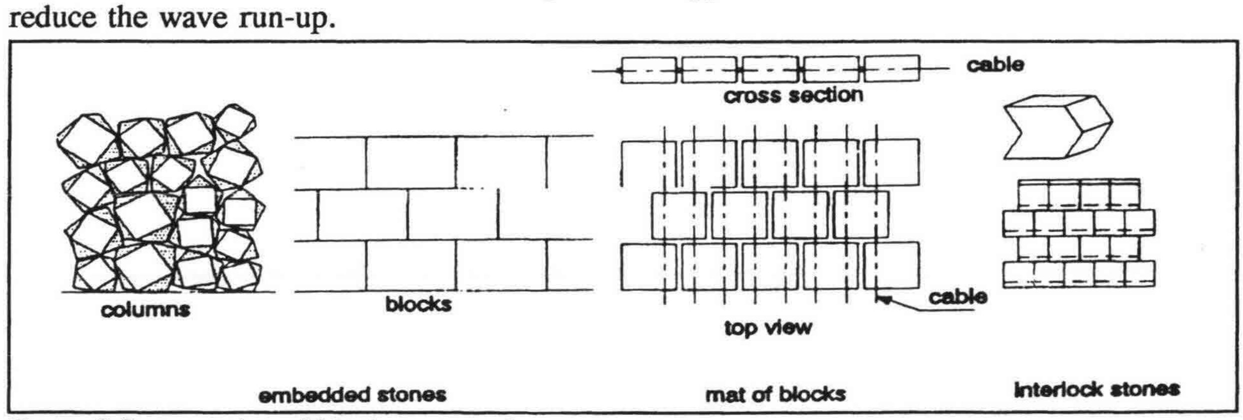


Figure 6.3: examples of block systems in revetments

The transition between the blocks and the underlying soil is another variable. In the chapter on filters this topic will be worked out in more detail. Some examples are given in figure 6.4.

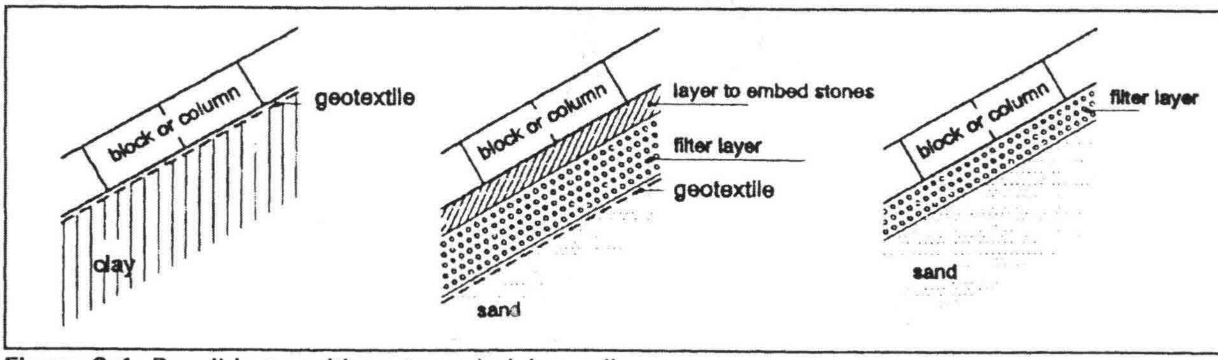


Figure 6.4: Possible transitions to underlying soil

Sometimes the blocks are placed directly on the clay; this requires a high standard during execution, and is therefore only recommended when execution can be controlled very effectively, and the weather does not cause problems. When the blocks are not of identical height, a layer to embed the blocks is necessary to correct the height differences. This is specially the case when natural material like basalt columns are used. It is sometimes combined with a filter layer.

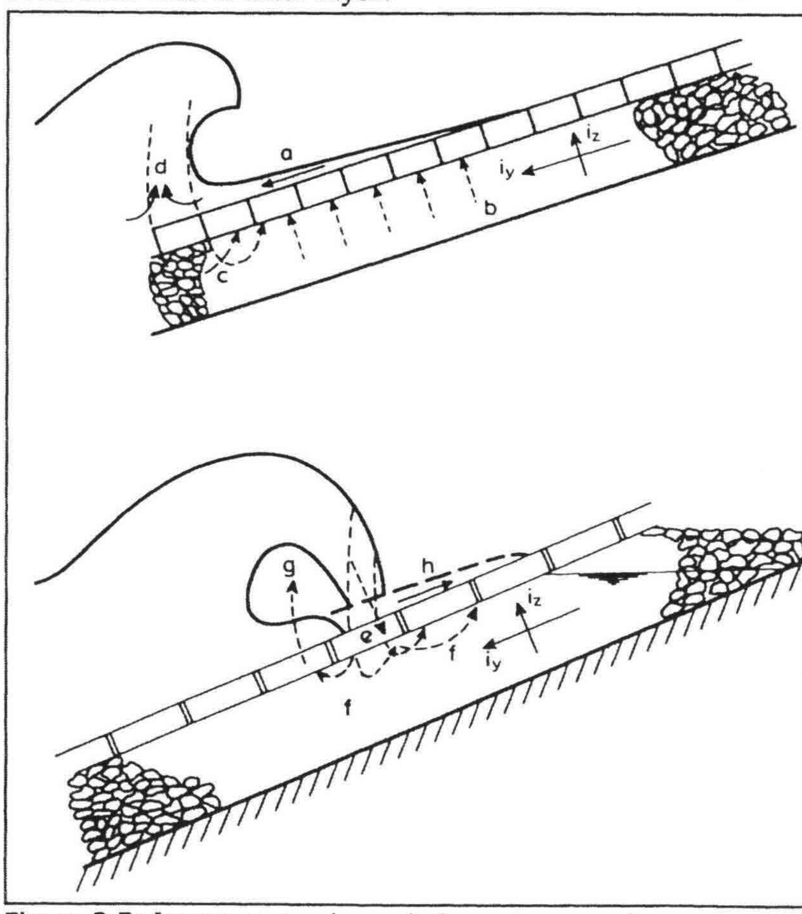


Figure 6.5: forces on top-layer during a wave cycle

a: forces due to backwash
b: quasi-stationary pressures due to wave set-up
c: pressures due to wave front
d: velocity-field in wave
e: wave shock (impact)
f: pressures due to wave breaking
g: low pressure due to air in water
h: forces due to uprush

i_z: gradient perpendicular to the slope
i_y: gradient parallel to the slope

Figure 6.5 shows the phenomena that play a role in the stability of placed blocks during a wave cycle. For loose grains velocity, friction and gravity are not enough to describe the stability. At least inertia and porosity should also be taken into account. For placed blocks the situation is all the more complex. From tests, calculations and reasoning it was established that the phenomena b and c are dominant in the process. In fact, the wave action on and under the blocks cannot be separated and porous flow phenomena have to be taken into account.

Figure 6.6 shows the dominating situation at maximum downrush; the pressure on the blocks is low in front of the wave, while under the blocks it is high, due to the propagating wave pressure in the filter layer and to the relatively high phreatic level in the slope. This causes an uplift force on a block. Very important for the magnitude of this uplift force is the relation between the permeability of the top layer and the filter layer, usually expressed in a leakage factor, Λ. This will be discussed in the following section.

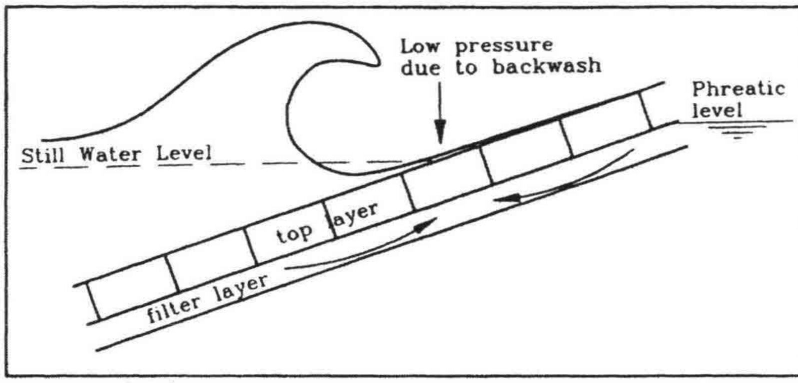


Figure 6.6: Critical situation for placed blocks during a wave cycle

the leakage factor

To demonstrate the relative importance of permeabilities, the porous flow in the revetment is simplified as follows: the flow through the filter layer is assumed to be parallel to the slope while the flow through the top layer is supposed to be perpendicular to it (y is the coordinate along the slope). The flow in the filter (q_F) and the top layer (q_T) can be expressed as:

$$q_F = -k_F \frac{d\Phi_F}{dy}$$

$$q_T = k_T \frac{(\Phi_S - \Phi_F)}{d} = k_t \frac{\Delta\Phi}{d}$$

In these formulas Φ is the pressure head and k is the permeability. The thickness of the top layer is d, the thickness of the filter is b. From continuity ($\Delta q_F.b = q_T.\Delta y$) follows that $q_T = b.d.q_F/dy$, which leads to:

$$\frac{d^2\Phi_F}{dy^2} = \frac{k_T(\Phi_F - \Phi_s)}{k_F bd} = \frac{\Phi_F - \Phi_S}{\Lambda^2} \rightarrow$$

$$\Delta\Phi = \Lambda^2 \frac{d^2\Phi_F}{dy^2} \qquad (\Lambda = \sqrt{\frac{k_F bd}{k_T}})$$

From this equation can be seen that the head difference over the top layer depends directly on Λ. A relative thick and permeable filter layer and/or a relative thick and impermeable top layer give a large Λ and hence, a large head difference over the top layer. So a large leakage length Λ is unfavourable for the stability of the blocks.

This equation can be solved analytically for highly schematized boundary conditions and when laminar flow (or more precise: a linear relation between velocity and pressure) is assumed. Further description is outside the scope of this lecture and the reader is referred to the contribution of Bezuijen, Klein Breteler and Burger in PILARCZYK [1990].

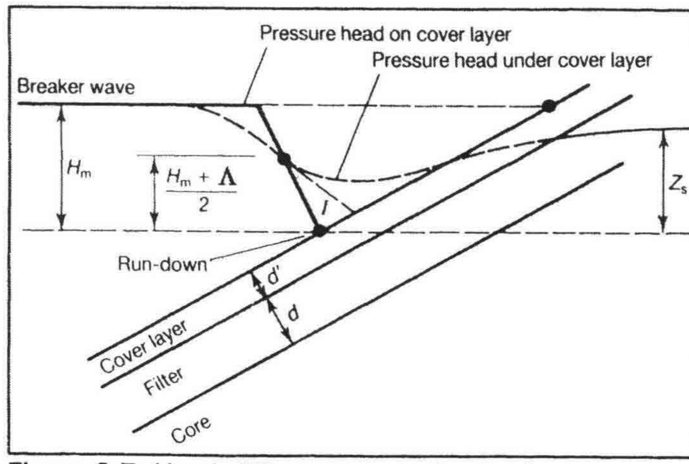


Figure 6.7: Head difference over the top-layer

Whether a block is pushed out or not depends further on the strength of the revetment in which two factors play an important role. In the first place of course the coherence of the blocks, which in the case of loose blocks is the friction between them. The second factor is the flow towards a stone when it is pushed out. With a relatively small permeability of the filter layer, the block is sucked onto the slope because only very little water can flow into the growing hole leading to a sudden decrease of the pressure under the block. From all this it becomes clear that a permeable top layer and an "impermeable" filter layer lead to the most stable construction. This means that filter layers should be kept as thin as possible. That is why the idea of blocks placed directly on clay is a very attractive one from a theoretical point of view. However, in practice it is virtually impossible to prepare a good clay bed to place the blocks on.

By equating the formulae for the load and for the strength as given above, it is possible to determine the stability of a slope protection. However it is quite difficult to solve these equations, even in a numerical way. Especially because of the quick variations of the pressure as a function of time, a solution is difficult to obtain. The computer program STEENZET, developed by Delft Geotechnics and Delft Hydraulics is able to do so. Handling this program is far from easy, and much experience is required. Therefore this program is not very well suited to the standard design of slope protections.
A somewhat more simple version of this program, called Anamos, is available for design of block-revetments (both square blocks as well as Basalton). This program is only available in Dutch and can be obtained from the Ministry of Public Works in Delft (P.O. Box 5044; 2600 GA Delft; *f* 470.--).
However, for a smaller areas also the simplified method of PILARCZYK [1993] (or an other recent paper of Pilarczyk) can be used. This method is discussed in the following section.

<u>design of a block protection (and related types)</u>
For practical design one can use the general empirical (approximate) formula derived by Pilarczyk (1990):

$$\frac{H_s}{\Delta_m D} \leq \Psi_u \Phi \frac{\cos\alpha}{\xi^b} \qquad for \qquad \cot\alpha \geq 2$$

or

placed blocks

$$\Delta_m D = \Psi_u^{-1}\, \Phi^{-1} \cos^{-1}\alpha\; H_s\, \xi^b$$

with:

- ξ = surf similarity index on a slope
- Ψ_u = system-determined (empirical) stability upgrading factor (Ψ_u = 1.0 for riprap as a reference and $\Psi_u \geq 1$ for other revetment systems) [-],
- Φ = stability factor or stability function for incipient of motion, defined at $\xi = 1$ [-],
- H_s = significant wave height [m],
- T = average wave period [s],
- L_o = wave length [m],
- D = specific size or thickness of protection unit [m],
- α = slope angle [°],
- Δ_m = relative density of a system-unit [-],
- b = exponent related to the interaction process between waves and revetment type (roughness, porosity/permeability etc), $0.5 \leq b \leq 1$. For rough and permeable revetments as riprap, b = 0.5. For smooth and less permeable placed-block revetments it can be close to b = 1. The value b = 2/3 can be treated as a common representative value for other systems (i.e. more open blocks and block-mats, mattresses of special design etc.).

The effect of composition (permeability) of revetment is illustrated in fig. 6.1

D and Δ_m are defined for specific systems as:

- rock: $D = D_n = (M_{50}/\rho)^{1/3}$ and $\Delta_m = \Delta = (\rho_s - \rho_w)/\rho_w$
- blocks: D = thickness of block and $\Delta_m = \Delta$
- mattresses: D = d = average thickness of mattress and $\Delta_m = (1-n)\Delta$, where n = bulk-porosity of fill material and Δ = relative density of fill material. For common quarry stone $(1-n)\,\Delta \approx 1$.

For rock and $\xi > 3$, the sizes calculated at $\xi = 3$ can still be applied.

The wave attack on a slope can be roughly transformed into the maximum velocity component on a slope during run-up and run-down, U_{max}, by using the following formula:

$$U_{max} \approx a\sqrt{g\, H_s\, \xi}$$

(for irregular waves and smooth slopes $a \approx 1$; in other cases $a < 1$).

The stability factor Φ for loosely aggregates can be more generally defined using the Van der Meer's formula (see previous chapter and figure 6.8).In the case of relatively impermeable core (i.e. sand or clay, $P_b \approx 0.1$) and limited number of waves ($N \approx 3000$) the following indicative Φ-values can be determined:

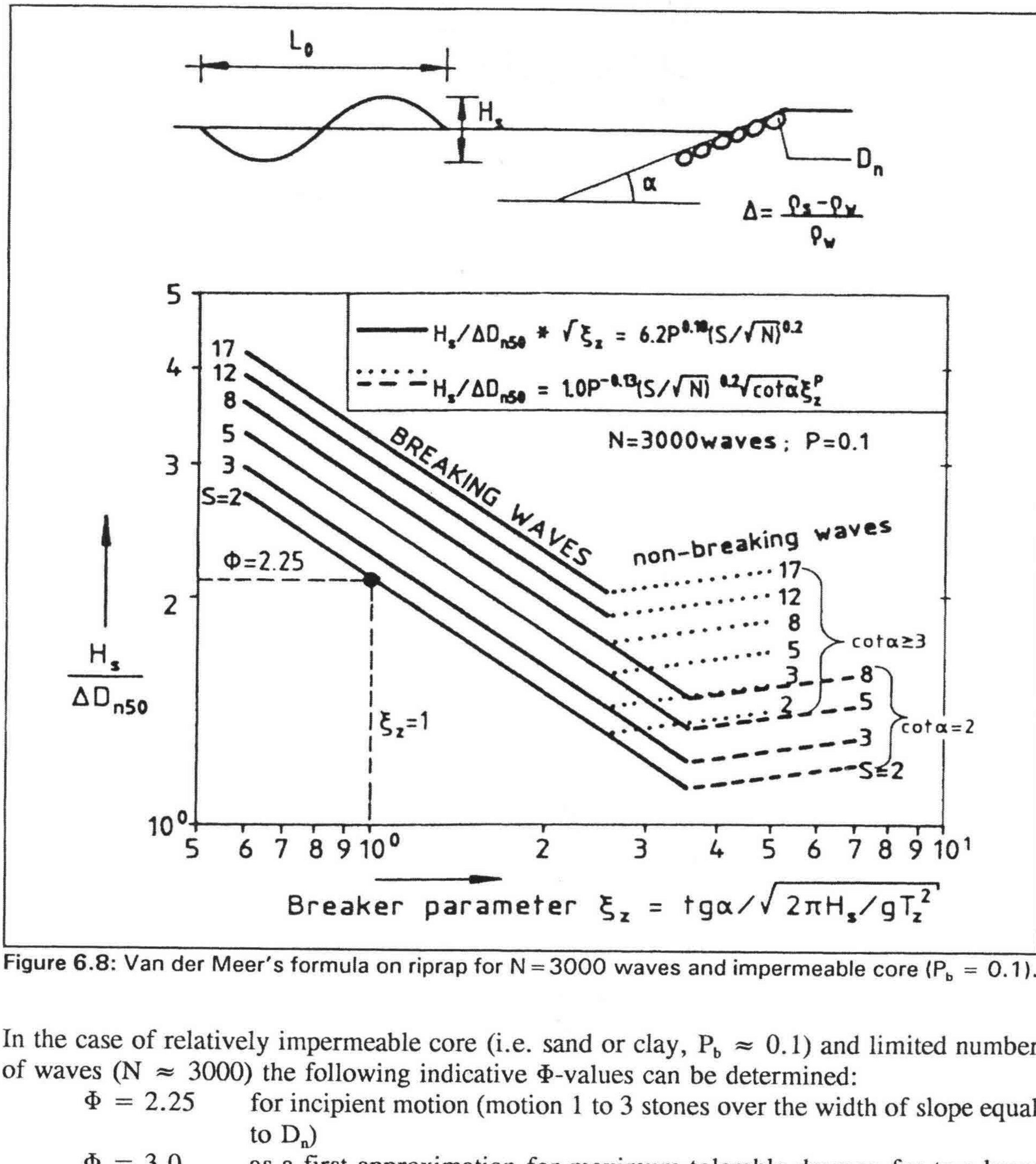


Figure 6.8: Van der Meer's formula on riprap for N = 3000 waves and impermeable core (P_b = 0.1).

In the case of relatively impermeable core (i.e. sand or clay, $P_b \approx 0.1$) and limited number of waves ($N \approx 3000$) the following indicative Φ-values can be determined:

- $\Phi = 2.25$ for incipient motion (motion 1 to 3 stones over the width of slope equal to D_n)
- $\Phi = 3.0$ as a first approximation for maximum tolerable damage for two-layer system on granular filter (i.e. $S_b = 8$ and damage-depth less or equal to $2D_n$)

These conditions are close to the average test conditions in the past when the rock and other alternative systems were examined based on the Hudson's stability equation.

The Φ-value equal to 2.25 will be used further as a reference value for the stability comparison with other alternative systems. The difference with stability of rock due to the improving measures will be expressed by the upgrading factor Ψ_u.

placed blocks

The important difference between the loosely packed rock and the alternative systems concerns the behaviour of the systems after the initiated movement (damage). Due to the self-healing effect of the loosely packed rock a certain displacement of rock units can be often accepted (up to $\Phi \approx 3$). In the case of alternative systems i.e. block revetment, the initial damage (i.e. removing of one block) can easily lead to a progressive damage; there is no reserve-stability. In all cases, experience and sound engineering judgement play an important role in applying these design rules, or else mathematical or physical testing can provide an optimum solution.

The comparison of stability of various systems (parameter Ψ_u) and the necessary parameters for calculation purposes are comprised in the following box.

<u>friction</u>

It is very difficult to determine the amount of friction between blocks. The friction can be very high, but sometimes there might be not any friction at all (a loose block). During some field tests, blocks were pulled out of the slope. The forces required were sometimes extremely high, but sometimes, a force only a little more that the deadweight of the block was enough.

type of block	average weight (kg)	pulling force average (kgf)	pulling force stand. dev (kgf)
Basalt type 1	17	1763	1282
Basalt type 2	32	2178	1248
Basalt type 3	35	1528	1037
Basalt type 4	50	8874	3324
Blocks .5x.5x.2	180	3764	2194
Round type stone	16	668	369

As can be seen in the table the standard deviation of the pulling force is extremely high. But it is also clear that the pulling force is sometimes 50 times deadweight.

<u>Comment for users</u>

- block revetments and block-mats

The use of Ψ_u -values higher than 2.5 is not advised except when supported by mathematical models and/or large-scale tests incorporating geotechnical stability. For older revetments some increase of stability is often observed due to the increase of natural friction and/or inter-locking. However, permeability may decrease which acts adversely.

The edges of the adjacent block-mats, if not properly connected to each other, should be treated as free blocks (Ψ_u = 1.33 to 1.50). For slopes steeper than 1 on 3 the geotechnical (in-)stability (i.e. sliding) can be a decisive factor and it should be examined properly. In the case of toplayer placed directly on (compacted-)sandy subsoil and geotextile the impinging

Criterion	Limits
$\frac{H_s}{\Delta_m D} = \Psi_u \cdot \phi \frac{\cos\alpha}{\xi_p^{\ b}} = \psi_u 2.25 \frac{\cos\alpha}{\xi_p^{\ b}}$	$\phi_{(rock)} = 2.25$ $ctg\alpha \geq 2$ $\xi_p < 3$

System	ψ_u	Description	Sublayer
Ref.	1.0	Riprap (2 layers)	Granular
Rock	1.33	Riprap (tolerable damage)	Granular
Pitched	1.00	Poor quality (irregular-)stone	Granular
Stone	1.33	Good quality (regular-)stone	Granular
	1.50	Natural basalt	Granular
	1.50	Loose closed blocks; H_s < 1.5 m	Geotextile on sand
Blocks/	1.50	Loose (closed-)blocks	Granular
Block-	1.50	Blocks connected to geotextile	Granular
mats	2.00	Loose closed blocks	Geotextile on clay
	2.00	Cabled blocks/Open blocks (> 10%)	Granular
	>2.50	Grouted (cabled-)blocks/Inter-locked blocks adequately designed	Granular
Grout	1.50	Surface grouting (30% of voids)	Granular
	1.50	Pattern grouting (60% of voids)	Granular
Open Stone Asphalt	2.00	Open stone asphalt; $U_p \leq 6$ m/s	Geotextile on clay
	2.50	Open stone asphalt; H_s < 4 m	Sandasphalt
Gabions	2÷3.0	gabion/mattress as a unit, H_s < 1.5	Geotextile on sand or Geotextile on clay
	2÷2.5	stone-fill in a basket; $d_{min} = 1.8\ D_n$	
Fabric	1.00	P_m < 1 less permeable mattress	Sand or Clay and ev. Geotextile
Con-	1.50	$P_m \approx 1$ (P_m=ratio permeab.top/$_{sublayer}$)	
tainers	2.00	$P_m \geq 2$ permeable mattress of special design	
Grass	-	Grass-mat on poor clay; U_p < 2 m/s	Clay (U_p= permiss. velocity)
		Grass-mat on proper clay; U_p < 3 m/s /s	

Indicative categories for protective systems

and/or liquefaction. It should be noted that for practical reasons the minimum thickness of loose blocks is about 0.10 m and for blocks grouted with granular material and blockmats is loose blocks is about 0.10 m and for blocks grouted with granular material and blockmats is 0.08 m. More sophisticated approach to stability aspects of these systems can be found in the guidelines on dimensioning of block revetments.

- grouted stone

Surface grouting is not advised in the case of highly permeable sub-layers. Creation of a completely impermeable surface should be avoided because it may introduce extra lift forces (blasting effect). In the case of pattern grouting about 50 to 70% of the total surface is filled. The upgrading factor is very dependent on the execution and care must be taken to ensure that the grout does not remain in the surface of the stone-layer only, or sags completely through the layers. In the area of high wave impact, the grouted lumps themselves can be split by dynamic actions, therefore this type of construction should be applied up to $H_s = 3$ m (frequent loading) and $H \leq 4$ m (less frequent loading). In the later case, for safety reasons, it is recommended to use of three layers of broken stone. If a lump of grouted stones is split and washed away, the third layer will still protect the core since it is held by the overlying grouted lumps.

- bituminous systems

In the case of open stone asphalt on sand asphalt filter, the thickness of the system may be defined as the total thickness of both layers. For the edges of all bituminous systems the $\Psi_u = 2$ should be applied. Because of possibility of liquefaction, the open stone asphalt on geotextile and sand is recommended only up to $H_s = 2$ m. For $H_s > 2$ m the sand-bitumen filter under the toplayer of open stone-asphalt is recommended. Due to the limited resistance of open stone-asphalt against surface erosion (max. velocity, u = 7 m/s) this system can be applied up to $H_s = 3$ m, and, for a less frequent wave loading, up to $H_s = 4$ m.
For practical reasons, the minimum thickness of open stone asphalt is 0.08 m if prefabricated and 0.10 m if placed in-situ. However, the more common thickness are respectively 0.10 and 0.15 m. Bituminous plate-systems (especially impermeable ones) should also be examined concerning the allowable stresses and strains (bending moments) and the uplift criterion. The calculation methods can be found in (see next chapter). The example of thickness of various asphalt revetments related to the allowable stresses is given below for compacted sandbed with slope 1 on 3:

H (m)	asphalt concrete	open stone asphalt	sand asphalt
2	0.10 m	0.20 m	0.40 m
3	0.20 m	0.40 m	(0.80 m)
4	0.30 m	0.65 m	
5	0.40 m		

In general, the resistance of the sand-asphalt is limited to the velocity of 3 m/s and the wave height of 1.5 m (or $H_s \leq 2$ for less frequent loading). Currents are not usually a determining load in the design of asphalt concrete.

- gabion baskets and mattresses

The primarily requirement is that the gabion or mattress of thickness "d" will be stable as a unit. The thickness of the mattress can be related to the stone size D_n. In most cases it is sufficient to use two layers of stone in a mattress ($d = 1.8\ D_n$) and the upgrading factor can be recommended in the range $2 \leq \Psi_u < 3$ (max). The secondary requirement is that the movement of stones in the basket should not be too high because of the possible deformation of baskets and the loading on the mesh-wires. To avoid the situation that the basket of a required thickness "d" will be filled by too fine material, the second criterion, related to D_n, have been formulated. The choice of $\Psi_u = 2$ to 2.5 related to D_n means, that the level of loading of the individual stones in the basket will be limited roughly to twice the loading at the incipient motion conditions. Thus:

D_n (dynamic stable) when $\Psi_u \leq 2.5$, and
d (stable) when $d \geq 1.8\ D_n$.

In more than 2-layer systems it is preferably to use a finer stone below the toplayers (i.e. up to 1/5 D_n) to create a better filter function and to diminish the hydraulic gradients at the surface of the subsoil. The formulations for gabions and mattresses are only valid for waves with a height up to $H_s = 1.5$ m, or for less frequent waves up to $H_s = 2.0$ m. In either case it is important that both the subsoil and the stone infill are adequately compacted. When the current exceeds 3 m/s or the wave height exceeds 1 m then a fine granular sublayer (about 0.2 m thick) should be incorporated. In other cases it is satisfactory to place the mattress directly onto the geotextile and compacted subsoil. For practical reasons, the minimum thickness of mattresses is 0.15 m.

- fabric and other containers

The stability criterion for fabric mattresses of thickness "d" filled with sand, sand-cement or other materials attacked by waves is derived from some (limited) tests and recent knowledge of revetment principles. The value of upgrading factor (Ψ_u) depends on the ratio of the permeabilities of the mattress and the subsoil, P :

- for $P_m < 1$: $c = 1.0$ (less permeable mattresses)
- for $P_m = 1$ to 2 : $c = 1.5$
- for $P_m >> 1$ (i.e. ≥ 5): $c = 1.75 - 2$ (permeable mattresses of special design).

The permeability of the mattress should be treated as an integrated permeability of all the components i.e. geotextile container and fill-material together. For wave heights $1\text{ m} < H_s < 2\text{ m}$ and a sandy subsoil special measures against sliding and/or liquefaction should be taken: extra compaction, extra thickness (50 to 100%), eventually a fine granular sublayer 0.2 m thick (broad graded) etc. In the case of permeable mattresses (i.e. gravel-fill) on sandy subsoil the underneath part of the container should be preferably made of the sand-tight geotextile (filter function). For slopes steeper than 1 on 3, to avoid sliding, special attention should be paid to the anchoring at the top of the mattress and to the adequate toe support. Special measures should be taking concerning the transitions (avoid exposed edges), scour protection at the toe and the protection in the splash area due to the overtopping. Sand-mattresses, even properly compacted, are very susceptible to deformation. Therefore, their permanent use should be limited to relatively mild wave attack ($H_s < 1.5$ m). In general, the use of fabric containers of various (specific) designs for wave heights higher than

2 m and application of Ψ_u-values different from the above-mentioned is only recommended when supported by the large-scale or prototype tests, including geo- technical stability. For practical reasons, the minimum thickness of fabric containers/mattresses is 0.15 m. For promoting vegetation through the geotextile mattress the sand and/ or fine gravel, mixed with cohesive additives and seed, are very suitable as fill material.

effect of storm duration

In the Pilarczyk-formula the duration of the wave attack is not included. The values given for the parameters are based on the most common situations. However, sometimes the wave attack at a certain level of the dike is quite constant, and attack may persist for many hours. It is clear that the probability of failure during a storm increases when the storm lasts longer. On revetments with a big tidal difference this is not so important, because there the attack is shifting from one level to another during the storm.
In cases of no tide, or a very limited tidal difference, this aspect can be very important.
In order to investigate this problem, a number of full-scale long duration tests were performed in the Delta flume of Delft Hydraulics. This resulted in the following two graphs for the probability of failure of a revetment as a function of the duration of the wave attack.

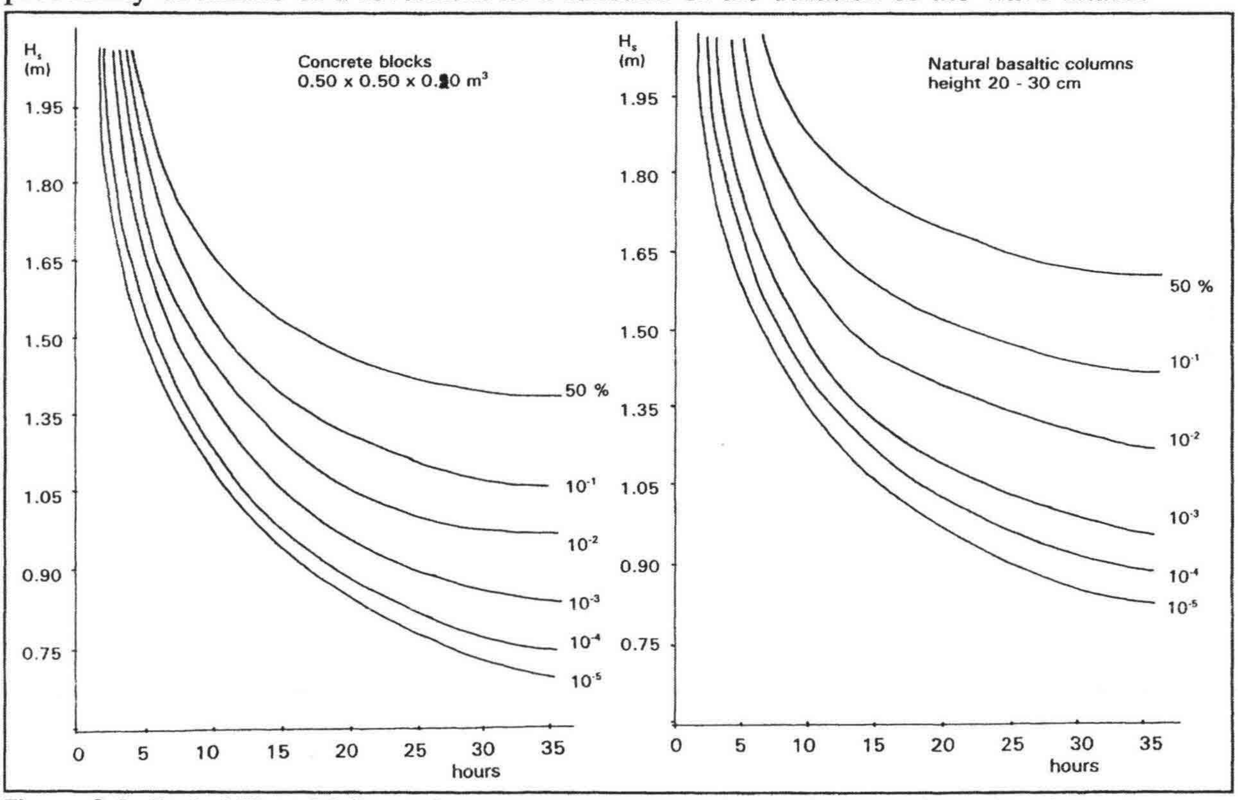


Figure 6.9: Probability of failure of a slope-protection during a storm of given duration.

6.2.3 asphalt revetments [adapted from Rijkswaterstaat, 1985]

The most important mix types applied in hydraulic structures are:
1. Asphaltic concrete
2. Asphalt mastic
3. Grouting mortars
4. Dense stone asphalt
5. Open stone asphalt
6. Lean sand asphalt
7. Asphalt membranes

Mixes are defined by the constituents, the nature of asphalt mix and the binder material.

Asphaltic concrete
Asphaltic concrete is probably the best known mix type. It is a mixture of crushed stones or gravel, sand and filler in which the pores are practically completely filled with bitumen. The voids ratio is 3 to 6%.In general the material must be compacted and is <u>unsuitable for application under water or in the tidal zone</u>. In view of the small voids ratio required, asphaltic concrete can be considered to be impermeable. Asphaltic concrete is applied as a watertight dike revetment above the mean high water level, and as a lining for canals, reservoirs etc.

Mastic
Mastic is a mixture of sand, filler and bitumen. There is more bitumen available than necessary for filling the voids in the sand filler mixture. The mix, therefore, is naturally dense and need not be compacted. Mastic can be poured at working temperatures and is used, for asphalt slabs above and <u>under water</u> for lining or as bed and toe protection. When cold, mastic forms a viscous quasi-static mass.

Grouting mortars
Grouting mortars are hot-type mixes of sand, filler and bitumen of which there is more than required to fill the voids in the mineral; stone and gravel can be added if necessary. These mortars are used for grouting stone revetments <u>above and below</u> waterlevel and also for slab construction.

Dense stone asphalt
Dense stone asphalt is a gap-graded mixture of stone, sand, filler and bitumen. The amount of bitumen slightly overfills the mixture. The material is, therefore, water impermeable. It is used as bottom and slope protection and also in toe construction.

Open stone asphalt
Open stone asphalt is a gap-graded mixture of mastic and stone. A stone frequently used is limestone 20/40 mm. Mixing is carried out in two stages. First mastic is prepared and secondly it is mixed with limestone. The mastic binder only coats and connects the limestone particles together. It is an 'underfilled' mix and, because of its open structure, should not be placed under water except in the form of prefabricated mattresses.

Lean sand asphalt
Lean sand asphalt is a mixture of sand, often locally obtained, with 3 to 5% bitumen. It is a greatly 'underfilled' mix and the function of the bitumen is simply to coat the sand grains and bind them together. After some time the permeability is very similar to the sand from which it is made. It is used as a core material for reclamation bunts, filter layers and as permanent or temporary cover layer above and below water-level.

Membranes
Membranes are thin impermeable watertight layers of bitumen which are prepared in-situ or prefabricated. Membranes are used as impermeable linings for canals, banks and water courses.

The functional requirements of an asphalt revetment
The function of a dike is to protect the area behind from floating. In order to protect the dike body, which is generally built up from soil, against erosion it is often provided with a revetment. Since the revetment is, at the same time, a part of the dike it can also fulfil other functions as, for example, watertightness. In general, for design purposes, the revetment may not be used to increase the safety of the dike body, in itself, against sliding.

An asphalt revetment must, in order to function satisfactorily, fulfil various requirements; requirements which stem partially from the loads which can be exerted. These are:

1. The revetment must be so that material from the dike body cannot pass through it. Sometimes the requirement is that the revetment should be watertight.
2. The revetment must be able to withstand:
   -- waves generated by wind and ships;
   -- currents and material carried with it such as sand, stones and driftwood (abrasion);
   -- uplift water pressures (only applicable to a relatively watertight revetment);
3. The revetment must be able to adjust to settlements and scouring, within limits, and must remain in contact with the subsoil.
4. The revetment must be stable. It should not slip down from the slope, also, during construction. It should, also, be stable, as a whole, so that it cannot be carried away by the loads which act upon it. This implies a certain dimension and weight.
5. The revetment must be weather and water resistant to erosion, corrosion, light, wind, temperature, and ice. The environment should not be able to damage the revetment or vice versa.
6. The revetment must be durable, that is, it must continue to function throughout its design life.
7. The revetment should, preferably, be aesthetically acceptable. This condition, however, if it is contrary to other requirements, should never be overriding.
8. In addition the following should be taken into account and guarded against:
   -- biological damage by plants, animals and sea organisms;
   -- chemical damage by polluted or salt water;
   -- possible land traffic, during construction and when completed;
   -- vandalism;
   -- recreational activities;
   -- vessels and anchors.

The above mentioned requirements must, in principle, be satisfied. This is possible by composing the revetment of one or more materials. Often, because the various requirements demand different revetment properties, a compromise must be sought.

The combination of the revetment and the dike body determine whether or not the dike functions satisfactorily, also in the coarse of time. Also it should be possible to construct the revetment under to local conditions. The dike body itself should be such that a reasonable cheap, well-functioning revetment can be achieved.

The dike body
An important factor to take into consideration when dimensioning and constructing a dike revetment is the nature of the dike body, that is, the sandbed. The following aspects are important:

a. The bearing capacity of the dike body determines among others the performance of a revetment under wave attack and other forces, and, therefore, plays a large role in the dimensioning. If the bearing capacity is large then often the thickness of the revetment can be reduced. The properties of the soil such as the modulus of elasticity, the modulus of subgrade reaction and Poisson's ratio are important. They themselves are influenced by the amount of compacting of the dike body. These properties can be determined from, for example, plate bearing tests.
b. A high degree of compaction can, amongst other things, avert the softening of a saturated or almost saturated soil by impact loads, for example wave attack, which can cause loss of bearing capacity. A relative proctor density of 95-100%, down to a depth of about 2 m, can, in general, in sand reduce the possibility of softening to an acceptable minimum. Bad, permeable, wet soil is prone to weakening; the presence of mud, in this context, is undesirable.
c. The permeability of the sandbed is important in connection with groundwater flow in the dike body and, consequential uplift pressures under a relatively watertight revetment. It is also important in connection with likely softening of the sand body.
d. The placing above water of an open asphalt mix on a saturated sandbed can, through the influx of water, result in the early development of stripping. Vibration compaction can soften a loosely packed saturated sandbody. Under impermeable mixes, as asphaltic concrete, uplift pressures can develop while the asphalt is still soft. This situation can, for example, occur by the delivery water from hydraulic filling work.
e. The dike body should have sufficient bearing capacity to support construction activities. If the sandbed has little resistance to deformation it is difficult to compact and, construction equipment can cause track impressions ('rutting'). After the sandbed has been compacted and made smooth it should not be driven over or care should be taken to ensure that it is not disturbed. The dike body is often formed of sand which is reasonably easy to compact. A good compacted subsoil gives fewer problems while compacting the revetment initial cracking is limited and the voids ratio required easier to achieve.

   In some areas in the Netherlands it has been common practice to only smoothen the sandbed with a bulldozer. This, however, only produces a limited improvement on the original density. On several dike projects, tests have shown that, even at considerable depth, the compaction was low. It is, in fact, better to build up the sandbody in thin

layers, with bulldozers. The dike face can then be profiled accurately, also with bulldozers. If this does not produce the compaction required, then a vibration roller should be used. For this process it is recommended that the sandbed is first sprayed with water.
The best dike body construction method, however, is to dump an excess of material, and then, after this has ben compacted, to produce the profile required by grading. The application of a clay underlayer is not recommended because of its weak consistency and the possibility of "frost heave". If an asphalt revetment is to be built on an existing clay revetment this should be of good quality, otherwise it should be removed. Investigations of the quality are desirable.
From a structural point of view there are generally no problems with underlayers of inert mine waste or lean sand asphalt. There can be some problems when laying an asphalt revetment on mine waste in connection with obtaining an equal revetment thickness.
Subsoil can be improved by:

-- mechanical compaction;
-- physical/mechanical treatment; an improvement of the particle skeleton in combination with mechanical compaction. This method has not yet, generally been used in hydraulic structures, but perhaps it will be in the future.

f. After constructing the revetment the dike body will tend to settle. If it has not been well compacted or if there are clay or peat layers in the subsoil, the settlement can be large and irregular. If the bed, is at the same time highly permeable then it is possible that the grain stress only recovers slowly and that the bearing capacity of the bed temporarily appears to be insufficient. This effect must certainly be taken into account with clayey subsoils; good drainage in this case is essential. With very permeable material the situation does not develop.

The bearing capacity of a ground mass can be measured using C.B.R. tests, soundings, or plate-bearing tests. It is recommended that laboratory investigations are carried out to determine values of permeability, proctor density, friction factors, etc. Sufficient measurements should be made to obtain representative values. The following recommendations are made for compacting subsoil: The minimum compaction should be 95% of the maximum proctor-density. The average compaction should be 98%.

Design methods
In the following sections design methods are given for the most common applications of asphalt revetments, including:

-- A dense asphalt revetment against hydraulic uplift pressures resulting from quasi-static conditions.
-- A plate-type asphalt revetment against wave impacts.
-- A dense underwater bed protection against uplift pressures caused by currents and waves.
-- A surface- or pattern-grouted crushed stone layer against wave action.
-- An asphalt revetment against currents.
-- An asphalt revetment adapting to irregular settlement and scouring. Determination of the maximum dike face slope is also discussed. Design methods are not given for all

Determination of the maximum dike face slope is also discussed. Design methods are not given for all types of loading situations. The methods described can probably also be applied to loadings other than those discussed.

Dense asphalt revetment designed to resist hydraulic uplift pressures
Hydraulic uplift pressures develop under a sealed, absolutely or relatively non-permeable dike revetment as a result of differences in water-level inside and outside the dike body. These pressures can force the revetment off the dike face. To prevent this and to ensure that the revetment continues to function it is necessary to take the phenomenon into account in the design. Air pressure can develop under an airtight dike crest revetment as a result of rising groundwater levels. This pressure can cause the revetment to crack. The effect can be avoided by ensuring that there is good ventilation. Air pressures act over the whole width of the crest whereas water pressures tends to act locally.

Hydraulic uplift pressures
Hydraulic uplift pressures can be caused by:
-- Quasi-static conditions:
- The groundwater level in the dike lags behind the ebb and flood of the tide. As soon as the groundwater level is higher than the water-level outside the dike body there is an hydraulic uplift pressure under the revetment.
- The largest uplift pressures can be expected after a storm surge when the water-level outside the dike body falls rapidly and the groundwater in the dike falls more slowly.
- Uplift pressures can develop during construction and for some considerable time afterwards if sand is moved by means of hydraulic transport in the vicinity of the revetment.

-- Dynamic conditions:
- Uplift pressure develops in the dike body when the water-level outside is lowered locally, over a very short time period, by a passing ship.
- Uplift pressures develop when wind waves produce changes in water-level on the dike face.

Dynamic conditions which can cause uplift water pressures are not discussed further here. Often the time interval in which they act is so small that insufficient water can flow to develop pressures of any consequence.

A large number of factors influence the development of hydraulic uplift pressures:
-- the height, duration, and form of time-dependent boundary conditions such as storm surges and tides in front of the dike, and the potential on the rear side of the dike: the polder level, the drainage ditch level, etc.;
-- the permeability and the differences in permeability of the soil in and under the dike body;
-- the dike geometry: dimensions; dike face slope; berms; toe level;
-- the water storage capacity of the subsoil;
-- the level of the foreshore in front of the dike;
-- the level of any impermeable layers, for example, clay, in the subsoil;
-- the length of any sheetpiling in the toe;

-- the presence and type of any drainage systems in the toe or elsewhere in the dike body.

A drainage system is sometimes installed in the dike to control groundwater movements. By using a toe drainage it is possible to completely or partly prevent the build-up of uplift pressure. It is important that such drainage systems continue to function throughout the lifetime of the structure.

Because many of the above factors are variable it is not possible to give general rules for determining the amount of uplift pressure which can develop. It is always advisable, certainly for large projects, for sites where the subsoil is not homogeneous, when a permeable layer is present under the revetment, or if the revetment design is very different from that schematised in the following figure to determine the uplift pressures using electrical analogues or a finite element method of calculation.

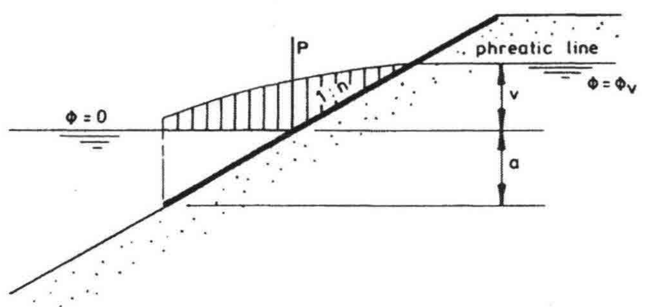


Figure 6.10: Schematization of the revetment for the Van der Veer formula.

The Van der Veer method can be used to obtain a very preliminary estimation of uplift pressures. This method has, however, drawbacks, the most important being:

-- The subsoil must be homogeneous, a situation which rarely occurs in practice.
-- The groundwater level has to be estimated.
-- The revetment must be schematized as shown in figure 6.10.

The Van der Veer method often gives values which appear to be too low in comparison with electrical analogue results.

The electrical analogue

A good method for determining the hydraulic uplift pressures under an asphalt revetment is the electrical analogue. This method, which has been developed extensively, simulates the groundwater flow by electric current. However, these models are nowadays mainly replaced by finite element computer programs.

The van der Veer formula

This formula is suitable for a preliminary estimate of uplift pressures. It is based on two-dimensional groundwater flow in a homogeneous subsoil and the presence of the maximum uplift pressure at the location of the outside water-level. The latter occurrence is valid if more than 20% of the revetment lies under water, measured from the bottom edge of the revetment to the level of the phreatic line. The height of the phreatic line must be estimated.

The maximum uplift pressure is determined for stationary flow, --constant horizontal supply of groundwater --and non-stationary flow. The maximum uplift pressure, P, develops at the waterline:

when $v/(a+v) < 0.8$ to 0.085

and is given by $p = c * \phi_v$ where ϕ_v is the difference between the water-level in front of the dike and the groundwater level in the dike body in the stationary case.
The coefficient c is given by:

Stationary flow: $$c = \sqrt{1-\left(\frac{v}{a+v}\right)^{\pi/\theta}}$$

Non-stationary flow: $$c = \frac{1}{\pi} \arccos \left[2\left(\frac{v}{a+v}\right)^{\pi/\theta} -1\right]$$

with: $$\theta = arctg\ (n) + \frac{\pi}{2}$$

where the dike face slope is l: n.

In the stationary case, in which the phreatic line remains at a constant level, the uplift pressure is notably bigger than the value obtained with the formula for non-stationary flow. The affects of a sheet pile wall or a toe protection are shown in figure 6.11

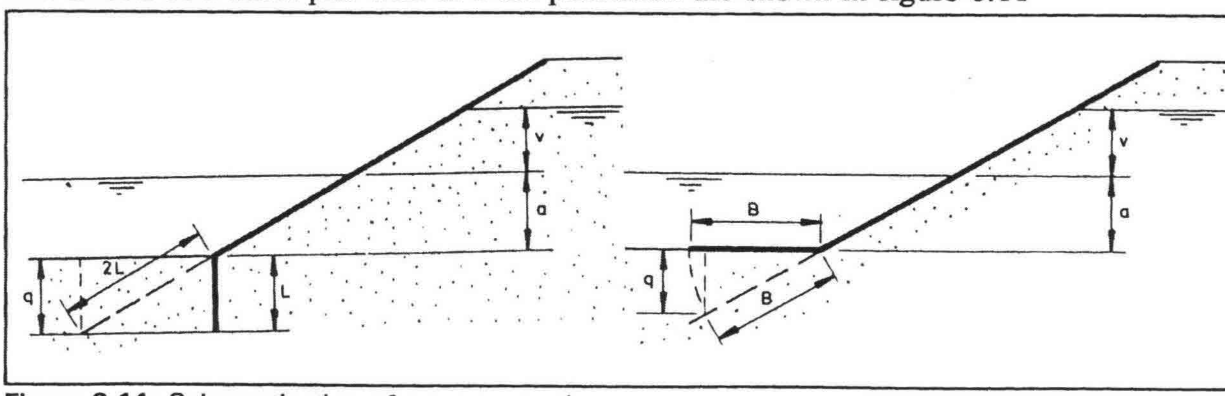


Figure 6.11: Schematization of a toe protection

The coefficient c then becomes:

stationary flow: $$c = \sqrt{1-\left(\frac{v}{a+q+v}\right)^{\pi/\theta}}$$

non-stationary flow: $$c = \frac{1}{\pi} \arccos \left[2\left(\frac{v}{a+q+v}\right)^{\pi/\theta} -1\right]$$

When making a preliminary estimate for tides and storm surges the parameter v, can be taken as 50% of the difference between the highest and the average outside water-levels. For long-term water-level differences, such as in reservoirs and in high water conditions in rivers v is taken as 100% of the difference.

Design criteria

What is the effect of hydraulic uplift pressures on a dense asphalt revetment?

a. If the weight component of the revetment down the dike face is greater than the frictional resistance then the revetment will tend to slide locally. The revetment will hang on higher parts and rest on lower parts where the frictional resistance is still sufficiently large. As a result deformation --stress and strain-- will develop.
Because the asphalt is viscous the deformation will be permanent. With a series of loading cycles the deformation can become so large that the material will yield. In addition signs of fatigue will appear.
The area of greatest uplift pressure will move together with the outside water level down the dike slope. The section of revetment which originally supported the section above will, at a certain condition, also slide. With successive high waters the revetment will, thus, tend to creep like a caterpillar down the slope.

b. If the hydraulic uplift pressure is larger than the weight component normal to the dike face then upward forces will develop which can lift the revetment. In the resulting cavity underneath the revetment, sand movements can take place which prevent the revetment from returning to its original profile. Because these sand movements are downwards there is a tendency for bulges to develop down the slope and subsidence on the upper parts. In view of the characteristics of asphalt the bulging can on the long-term or with regular loading, be considerable and continuing.

The following design criteria can be set out:

1. Sliding criteria
The revetment should be designed so that it does not slide under frequently occurring loading situations such as spring tides.

2. Uplift criteria
In loading situations which occur rarely, such as storm surges, the component of the weight of the revetment, normal to the dike face should be greater than the uplift pressure caused by the water.

3. Equilibrium criteria
The revetment must be in equilibrium as a whole.

Remarks:

In order to limit the uplift pressures an open revetment can be used in the area on the dike face where the greatest pressures occur, mostly in the tidal zone. The uplift water pressures can be schematized as shown in the following figure.

The uplift pressure must be determined by the designer. Determination of the maximum uplift pressure σ_{wo} is treated earlier. The variation of the pressure can also be determined using an electric analogue or a finite element calculation. The term σ_{wo}, used in the following formulae represents the maximum hydraulic uplift pressure. The Van der Veer method and the electrical analogue give the maximum potential difference, p, in metres of water, at the surface of the revetment. The uplift water pressure, σ_{wo} is then obtained by adding h cosα to the value of p and multiplying the whole by ρ_wg.

$\sigma_{wo} = \rho_w g\ (\rho + h \cos \alpha)$.

Which indicates that when p = 0, the uplift pressure $\sigma_{wo} = \sigma_w$ gh cos α.

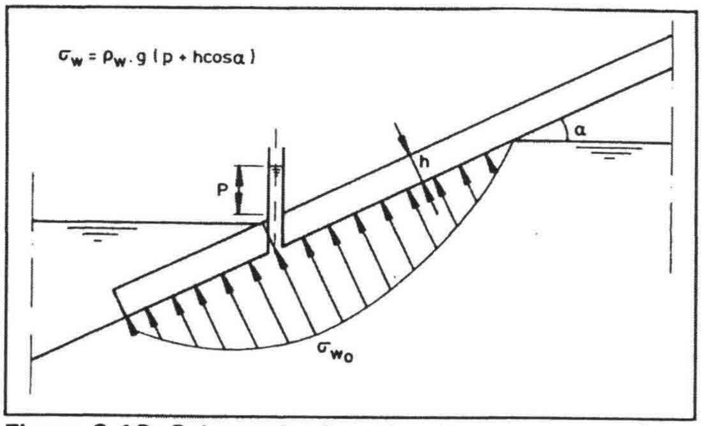


Figure 6.12: Schematization of water pressure under a sealed revetment

The dimensions of the revetment can be obtained using the following formula

1. Sliding criterion

$$h \geq \frac{f\,\sigma_{wo}}{\rho_a\, g\,(f\cos\alpha - \sin\alpha)}$$

2. Uplift criterion

$$h \geq \frac{\sigma_{wo}}{\rho_a\, g\cos\alpha}$$

If the revetment is not supported below, for example, by a toe construction or by another revetment then a check must be made to ensure that the tensile strength in the asphalt is not exceeded. If there is a possibility that it would be exceeded then the layer thickness must be increased and/or the dike face slope reduced. In this case the maximum value of the hydraulic uplift pressure which would be reached in the most extreme conditions can be used.

Symbols used:

h	=	revetment thickness (m)
σ_{wo}	=	maximum uplift pressure (N/m^2) For the sliding criterion σwo is determined for frequently occurring conditions; for the uplift criterion, conditions which occur very rarely are used
α	=	slope of dike face (dgr)
ρ_a	=	asphalt bulk density (kg/m^3)
ρ_w	=	density of water(kg/m^3)
g	=	acceleration due to gravity
f	=	coefficient of friction: f = tg ϕ if $\phi > 0$, else f = tgΘ
ϕ	=	angle of internal friction of the subsoil (dgr).
Θ	=	angle of friction between the revetment and the subsoil (dgr).

3. Equilibrium criterion
Will be discussed later.

Specific constructional features

Toe protection

Mastic, in the form of a horizontal or almost horizontal slab, can be used in front of the toe of a dike, see figure. If a mastic slab is sealed directly onto the toe of a dense revetment very high uplift water pressures can develop under the whole construction and a very thick revetment would be required. To avoid this an open 'berm' can be used between the slab and the toe or good drainage must be provided.
The uplift criterion applies to the slab. In the first approximation:

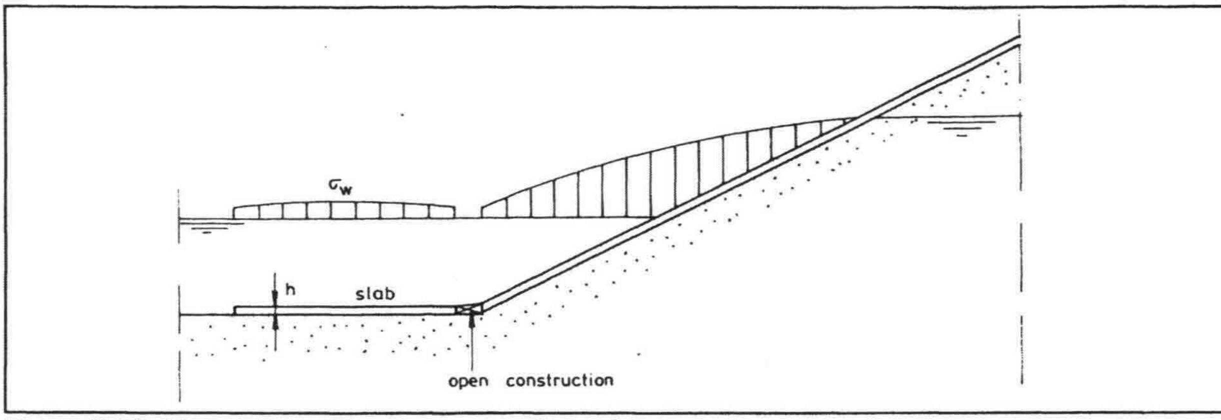


Figure 6.13: Hydraulic uplift pressures under a slab

$$h \geq \frac{\sigma_w}{\rho_a\, g \cos \alpha}$$

In which:

σ_w	=	maximum uplift water pressure under the slab (N/m^2)
h	=	slab thickness (m)
ρ_a	=	bulk density of mastic (kg/m^3)
g	=	acceleration due to gravity (m/s^2)
α	=	angle of inclination of the slab (dgr)

If it is likely that scouring occurs and the slab has to adjust to this the following checks must be made:

a. The uplift criterion in the new situation should not be exceeded. N.B.: the uplift pressures will be different in the new situation.
b. The slab should not slide.

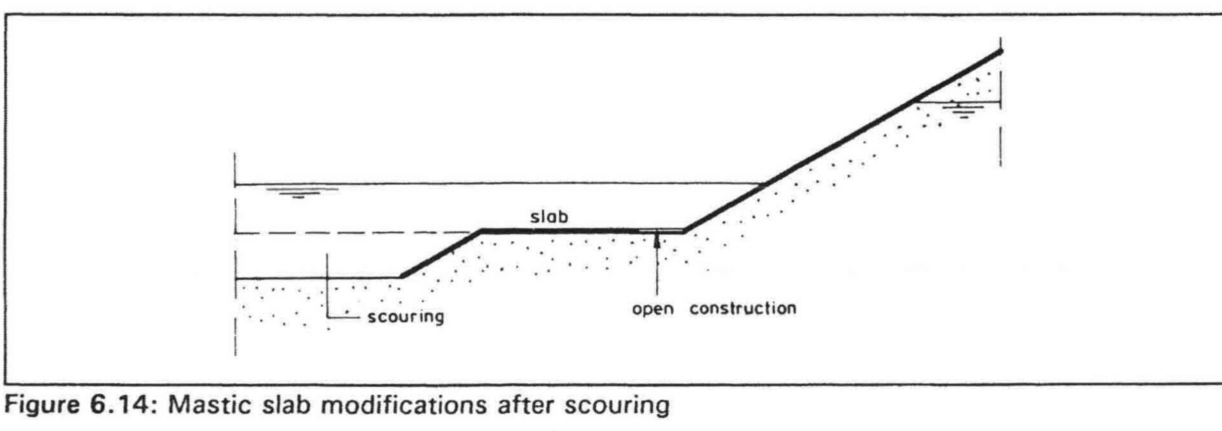


Figure 6.14: Mastic slab modifications after scouring

Grouted rip-rap (crushed stone)
In the past it was often recommended not to use revetments like asphaltic concrete in that part of the dike face where the largest hydraulic uplift pressures would occur and where the sliding criterion would be exceeded (in the Netherlands, in the tidal zone). Material with a more definite skeleton structure such as (grouted) rip-rap was recommended. With such a material the normal stresses are transferred better to the toe. An asphalt revetment with a definite skeleton structure will behave in a less viscous manner than a mix in which all the particles are coated with bitumen. (Another solution, if it is possible, is a water-permeable revetment.) A fully grouted stone layer has relatively large internal stability. Therefore, in the past, the design was never based on the sliding criterion; under extreme conditions the design was based simply on uplift. It is essential that the revetment is well supported by, for example, a toe construction. A fully grouted stone layer is, in principle, impermeable, although due to lack of adhesion between the stone and the grouting mortar, there will be a certain amount of permeability. No account of this should be taken in the design.

Lean Sand Asphalt
The permeability of lean sand asphalt should be similar to or larger than that of the underlying sand bed in order to prevent the development of hydraulic uplift pressures.

Asphaltic Membranes
Asphaltic membranes must be watertight and must remain so under the water pressures which develop. The value of the limiting pressure can be found from a permeability test; or derived from the manufacturer's specifications. It should not be possible for uplift water pressures to lift a membrane, that is, the protection layer should be sufficiently heavy. The dimensions of the protection layer can be found using uplift criteria:

$$h \geq \frac{\sigma_w}{\rho g \cos \alpha}$$

In which:

h	=	thickness of the protection layer (m)
σ_w	=	uplift water pressure (N/m^2)
ρ	=	bulk density of the protection layer (kg/m^3)
g	=	acceleration due to gravity (m/s^2)
α	=	slope angle (dgr)

If the membrane is applied on a slope and covered with an asphalt mix, the sliding criterion must also be applied. In addition, tensile forces in the membrane should not be too excessive.

Design of a plate-type asphalt revetment against wave impacts
The largest forces that waves can exert on a plate-type revetment are impacts caused by plunging breakers. Plate-type revetments include asphaltic concrete plates, mastic slabs, fully grouted stone layers, open and dense stone asphalt and layers of lean sand asphalt.

Wave impact loads
A wave impact occurs when a mass of water from a plunging breaker strikes the slope at great speed. A wave impact is, in fact, regarded as a pressure which acts over a certain width. To obtain the appropriate dimensions the impact is schematized as a line load.

$$P = p \cdot b$$

where:

P = the size of the wave impact (N/m^1)
p = the maximum pressure (N/m^2)
b = the width over which the pressure must act in order to represent the complete wave load (m)

Wave forces are dependent on a large number of factors such as, for example, wave height and steepness, and slope angle. Preferably they should be determined for each particular situation by observations and investigations. If this is not possible use can be made of the data given below. These data have been developed from the results of an investigation carried out by the Delft Hydraulics.

The various parameters are given by the following relationships:

-- Maximum pressure

$$p = \rho_w \, g \, q \, H$$

in which:
ρ_w = density of water (kg/m^3)
g = acceleration due to gravity (m/s^2)
H = wave height (m)
q = a factor related to the slope (see table below)

slope	q
1 :2	2.3
1 :3	2.7
1 :4	2.3
1 :6	2.0

-- The schematized width over which the maximum pressure is considered to act: $b = 0.4\ H$
-- The duration of the pressure effect, t sec, is, depending on the particular model:
slope $\leq$ 1: 3; $t = 0.06\ H^{1/2}$
slope $\geq$ 1: 4; $t = 0.18\ H^{1/2}$
-- The length of the wave impact is dependent on the angle of wave approach to the slope and the speed of propagation of the wave. The larger these factors are, the shorter is the impact length.
-- A breaking wave hits the slope at a distance Δh below still water level (SWL), see Figures.

In the above the parameter H is the height of a single wave. In practice the load on the revetment will be due to an irregular wave train containing a large number of waves of different height and frequencies of occurrence. The significant wave height, H_s, which

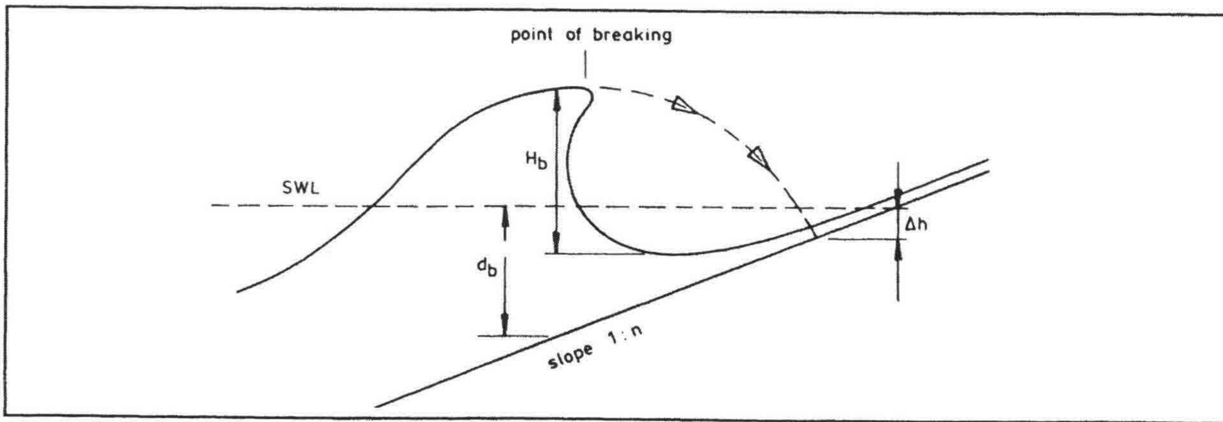


Figure 6.15: The breaking wave

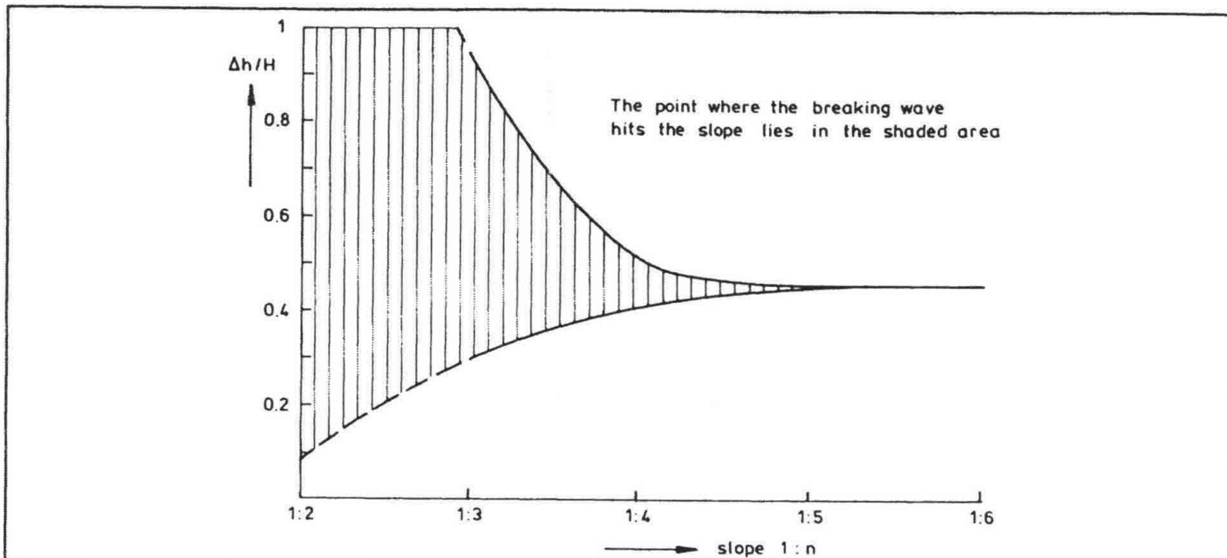


Figure 6.16: Attack level of the wave

characterises a particular wave field can be used for determining the wave impact. The number of times that this wave occurs is selected so that the same total 'fatigue load' is reached as that caused by the wave field as a whole. The calculation of wave impact should take into account the fact that only a small number of waves in the field will cause actually an impact on the revetment.

Construction schematization

It is important that the schematization of the construction approximates reality as much as possible. The most simple schematization is that of a plate lying on an elastic subsoil with a delayed response.

The method treated here is suitable for wave impacts and also for other types of loads which can act on the revetment. These include, for example, loads caused by colliding vessels, maintenance and recreations traffic, long-term loads and loads during the construction phase. The loads can be:

a. Static; that is loads which are always present. The way in which a construction reacts to static loads depends on the size of the load, the stiffness of the revetment and the subsoil, and the thickness of the revetment, amongst other things.
b. Dynamic; that is time-dependent loads. In addition to the factors mentioned above under static loads the speed, frequency and type of loading, the density of the revetment material and the damping and directly related mass of the subsoil are also important. When the revetment is frequently loaded the asphalt properties are altered: the strain at break reduces. This phenomenon is known as fatigue.

Here the static solution is applied to the selected schematization. The number of loading cycles is taken into account in the calculation of the breaking strength of the material. The duration of loading is incorporated into the stiffness modulus of the asphalt mix.
The loading is schematized as a line load. Since the wave impact is a distributed load in practice, corrections are applied.

This schematization is not so suitable for complicated constructions (varying layer thickness, multi-layer systems, joints etc.). In these situations a more extensive calculation involving, for example, finite element methods can offer the best solution.

Asphalt and subsoil properties
When designing it is essential to know the asphalt and subsoil properties. Preferably data should be obtained by carrying out specific tests for each design. If such tests are not possible, use can be made of the general values given in following tables. The materials used in the construction should then be carefully checked against these values. Mix compositions of the asphalt types are given in the table.

Table: Moduli of stiffness and related initial stresses and strains at break for different asphalt types.

		Initial strain at failure number of loading cycles					Initial stress at break number of loading cycles				
mix type	stiffness modulus NM^2	1	100	1000	10000	100000	1	100	1000	10000	100000
Asphaltic concrete	7 10^9	1.2 10^3	5.2 10^4	3.4 10^4	2.5 10^4	1.6 10^4	8.4 10^6	3.6 10^6	2.4 10^6	1.8 10^6	1.1 10^6
Asphaltic mastic	1 10^9	8.6 10^3	3.4 10^3	2.2 10^3	1.4 10^3	8.6 10^4	8.6 10^6	3.4 10^6	2.2 10^6	1.4 10^6	8.6 10^5
Dense stone asphalt	4.5 10^9	2.3 10^3	9.2 10^4	5.8 10^4	3.7 10^4	2.3 10^4	1.0 10^7	4.1 10^6	2.6 10^6	1.6 10^6	1.0 10^6
Open stone asphalt	7 10^8	3.4 10^3	1.3 10^3	7.9 10^4	4.8 10^4	3.0 10^4	2.4 10^6	9.1 10^5	5.5 10^5	3.4 10^5	2.1 10^5
Lean sand asphalt	1 10^9	1.1 10^3	4.2 10^4	2.6 10^4	1.7 10^4	1.1 10^6	1.0 10^6	4.2 10^5	2.6 10^5	1.7 10^5	1.0 10^5

Table: Mix compositions of the asphalt types

Mix	Composition (by mass%) crushed stone	sand	(very) weak filler	80/100 bitumen	Voids ratio (% v)
Asphaltic concrete	46.9	39.5	7.5	6.1	5
Mastic	--	64	17	19	0
Dense stone asphalt	60	25.6	6.8	7.6	5
Open stone asphalt	82.9	9.9	4.4	3.1	31
Lean sandasphalt	--	96	--	4	30

The asphalt properties (modulus of stiffness and stress at break) are temperature and loading duration dependent. Since the heavy storms, against which the revetment is mostly designed, occur mostly in the winter season a temperature criterion of 5°C can be accepted. The loading duration is wave height-dependent and can be deduced from previous text. Variations in this parameter, within practical limits, have little effect on asphalt properties.

Some properties and mix compositions of the most used asphalt types are given in the tables. Bitumen 80/100 is used by which an unfavourable loss in penetration is taken into account. The temperature is fixed at 5°C and the loading period 3 seconds.

The stresses at break for loading cycles of less than 10,000 are found by linear extrapolation on a log-scale. Although this method is not completely correct it gives presumably safe values.

Table: General values for the modulus of subgrade reaction of different soil types.

soil type	modulus of subgrade reaction c (N/m^3)
sand-- medium compacted (relative Proctor density 87-95)	1×10^7--1×10^8
-- well compacted (relative Proctor density 95-100)	1×10^8--3×10^8
sand + clay	3×10^7--8×10^7
sand + silt	2×10^7--5×10^7
clay-- low compressibility	2×10^7--5×10^7
-- high compressibility	$< 4 \times 10^7$
Peat	$< 5 \times 10^7$
Gravel	$> 7 \times 10^7$
Lean sand asphalt	$> 5 \times 10^8$

The parameter determining the subsoil is referred to as the modulus of subgrade reaction. General values of this parameter are given in above table.

Design

Design criteria are selected such that the stresses and strains developing in an asphalt plate of a certain thickness due to bending moments do not exceed the allowable values. The formula using a plate model on elastic subsoil reads:

$$h = 0.75 \sqrt[5]{\left(\frac{27}{16}\,\frac{1}{(1-\nu^2)}\,\left(\frac{P}{\sigma_b}\right)^4 \left(\frac{S}{c}\right)\right)}$$

in which:

h = thickness of revetment (m)
σ_b = asphalt stress at failure (N/m^2)
P = wave impact(N/m)
S = stiffness modulus of the asphalt (N/m^2)
ν = Poisson ratio for asphalt
c = Modulus of subgrade reaction (N/m^3)
0.75 = reduction factor

Usually the revetment is designed for a design parameter such as the significant wave height, H_s, which characterises the wave climate in a severe storm or at an extremely high design water level.

If the revetment is also subjected to loads of a normal daily wave climate then this should be taken into account in the layer thickness calculated with the design H_s. Distinction should be made between:

1. That part of the revetment on which only the design condition, H_s acts. No correction is necessary here.
2. That part on which the design conditions act together with the normal wave climate.
3. That part on which only the normal wave climate acts (and not the design wave).

That part of the revetment which, for example, lies above the (spring) tide zone is classed in Category 1. The revetment in the (spring) tide zone is acted on by the normal wave climate but, in general, not by the design wave, H_s, which occurs at higher water levels. This part is classed in Category 3. Dikes with a deep foreshore can be classed in Category 2.

Practical application of the wave impact formula
General values of layer thickness are given in the following figures for some standard mixes, for various significant design wave heights and subsoil parameters. These values are based on assumptions for application in Dutch conditions.

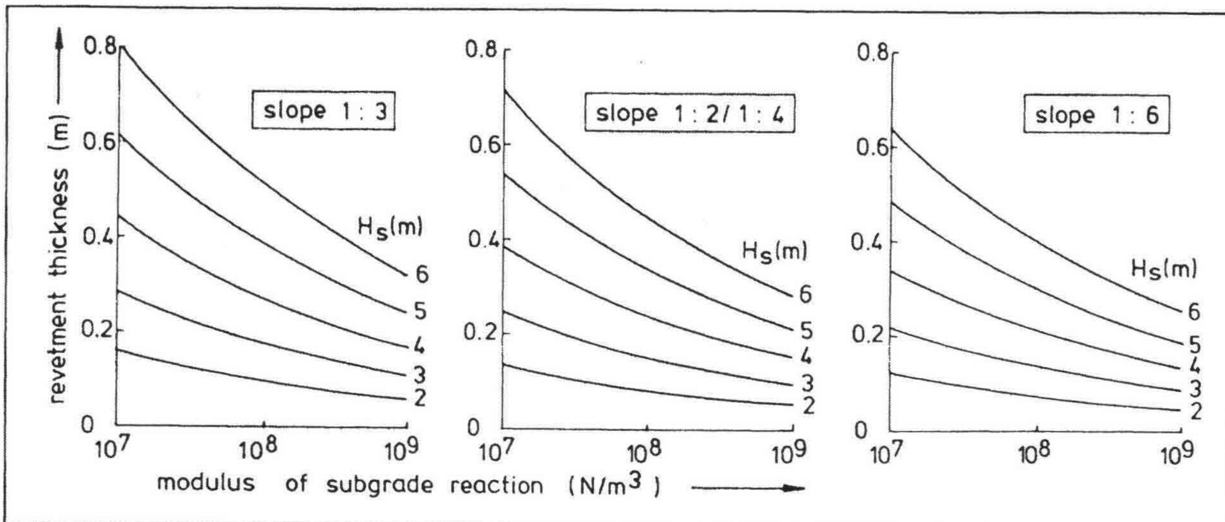


Figure 6.17: Revetment thickness/asphaltic concrete

The assumptions are:

-- the design storm has a duration of 36 hours (3 tidal cycles)
-- the number of wave impacts used for dimensioning is 10% of the total number of waves in the design storm
-- the waves in the storm are generated in a wave field which has a Rayleigh distribution
-- the relationship between the significant wave height H_s (m) which characterises the storm and the average wave period in the storm, T (sec) is $T=3.5 \times H_s^{0.5}$
-- the wave impact is determined as discussed earlier, $\rho_w=1000$ kg/m^3 and $g=9.81$ m/s^2
-- the asphalt properties are given in previous section. The number of loading cycles, n_s, related to the significant wave height, H_s, needed to determine the strength at break, is given in the table below. A value of 0.35 is taken for Poisson's ratio.

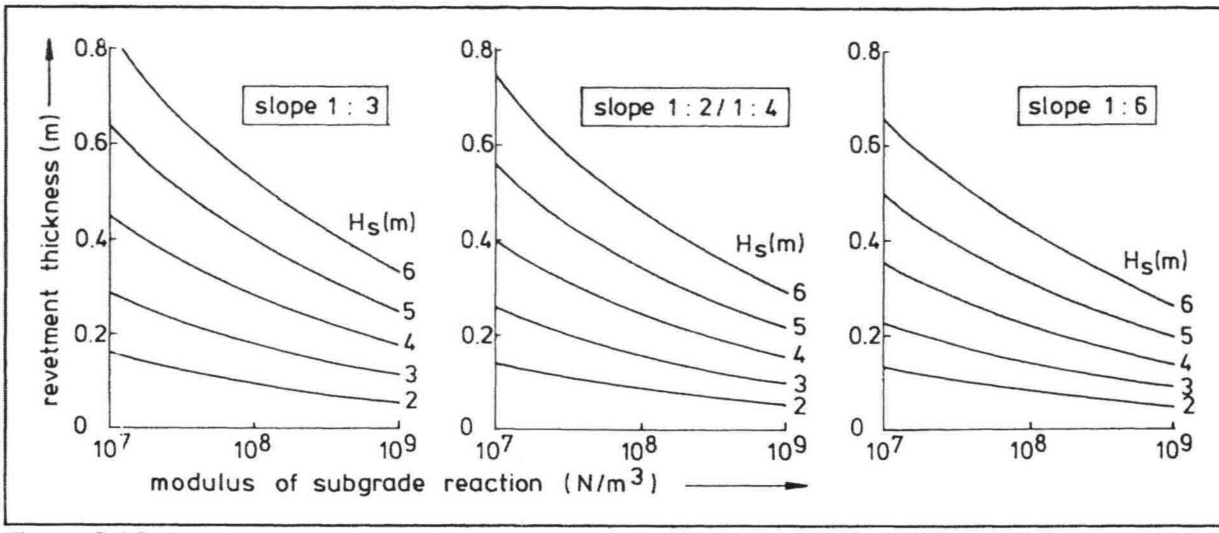


Figure 6.18: Revetment thickness/mastic

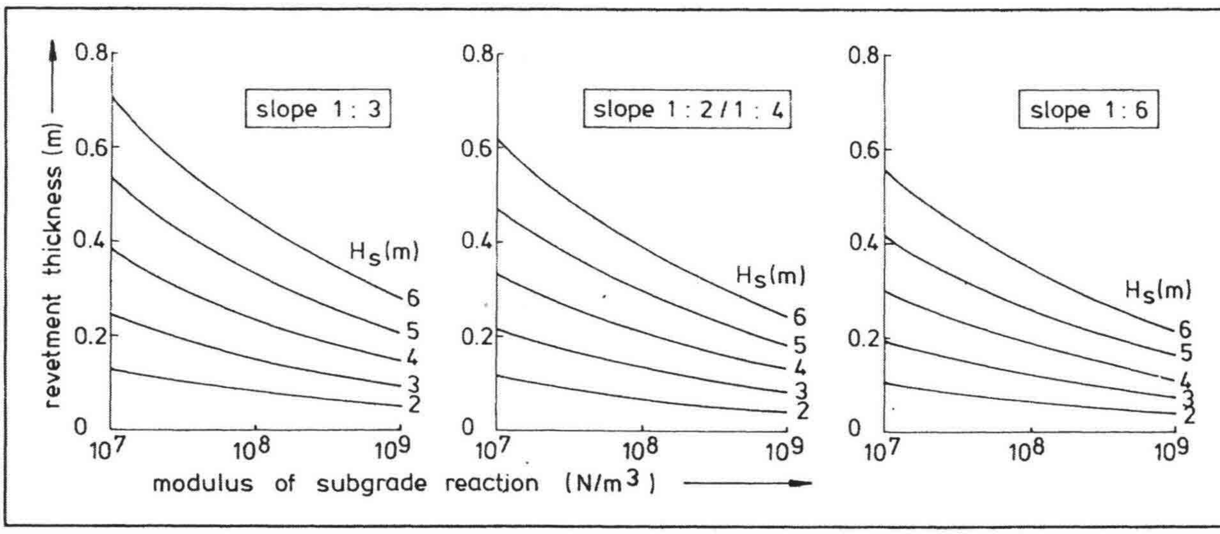


Figure 6.19: Revetment thickness/dense stone asphalt

In order to obtain an optimum design or if the design conditions differ greatly from these assumptions, then the calculation of layer thickness can deviate from the method used for determining the figures.

Adaptations can be made by:
- -- changing the mix composition;
- -- treating/compacting the subsoil;
- -- changing the dike geometry.

It is then essential to carry out extra checks on site and in the laboratory. This will also indicate the programme for the execution requirements. The following remarks should be noted for certain specific materials:

• Fully grouted stone

A fully grouted stone layer has, in general since it is composed of several layers of stone, considerable thickness. Because of this it is in most cases not necessary to design on the basis of wave impacts. Design is based on the normal wave impact formula using the material properties of the grouting mortar (mastic). In order to obtain the design layer thickness the value calculated must be multiplied by a factor lying between 1.4 and 1.75. These factors are valid if the shear stresses which develop between the mastic and the crushed stone can be transferred. It is not immediately obvious to what extent this condition is satisfied.

H_s (m)	n_s
2	9900
3	8000
4	6950
5	6200
6	5670
7	5250
8	4900
9	4630
10	4400

• Open stone asphalt

Because of the open character of open stone asphalt the wave

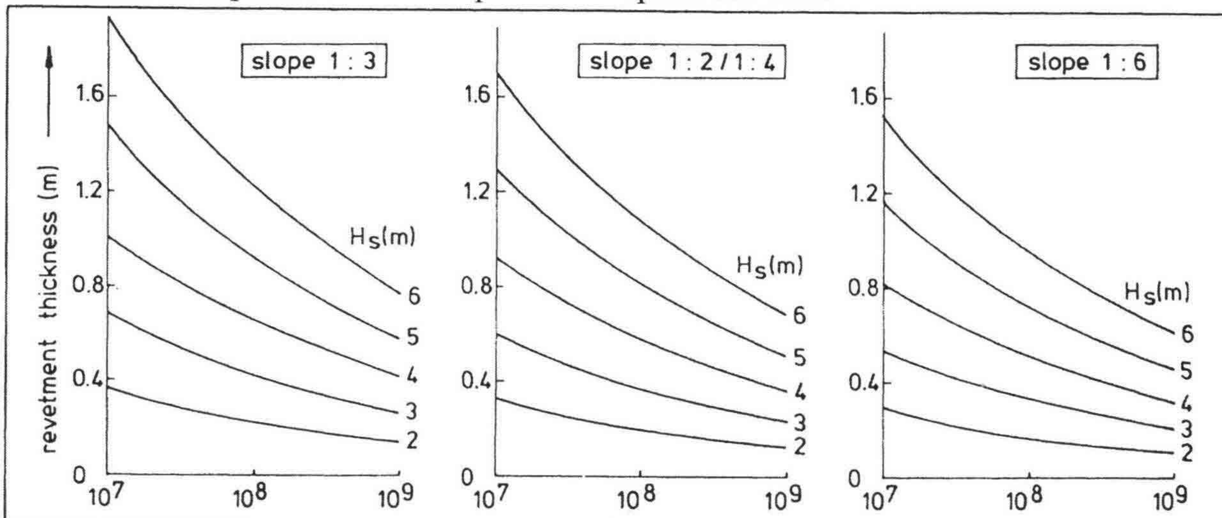


Figure 6.20: Revetment thickness/open stone asphalt

pressure can quickly propagate through the revetment, the result being that the load on the plate is less than that was indicated previously.

• Lean sand asphalt applied as a core material

The performance under wave impact of lean sand asphalt applied as a core material can be calculated using the Boussinesq approximation.

Design of underwater bed protection against hydraulic uplift pressures caused by currents and waves

Uplift pressures caused by currents

A mastic slab is often used to protect the bed against erosion caused by currents. In extreme situations fluctuations in the flow of water and groundwater can create pressure differences across a bed protection. If the pressure above a revetment plus its weight is less than the

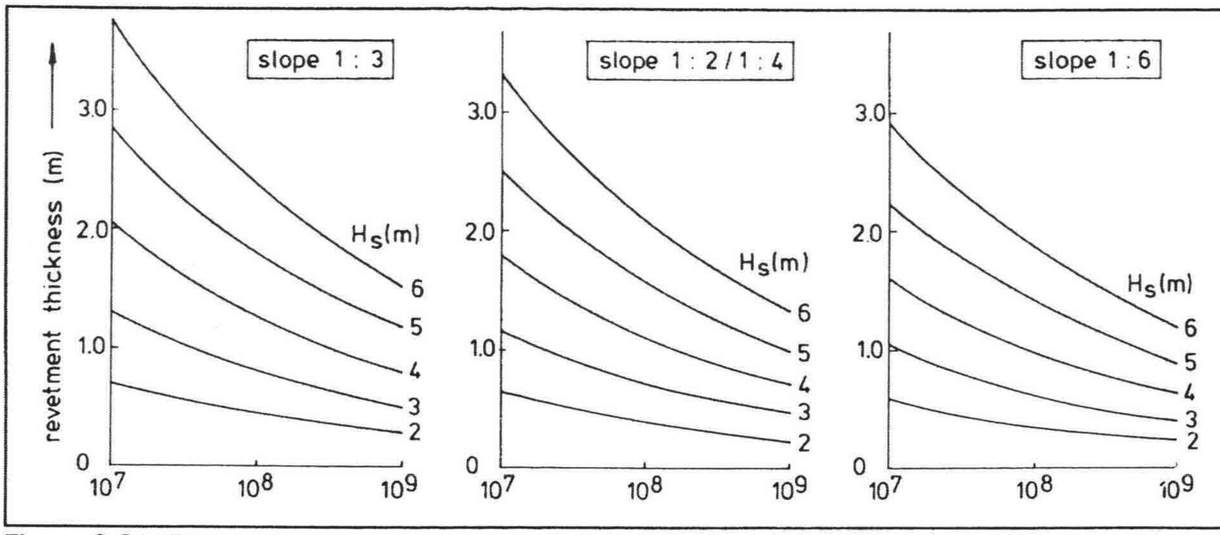


Figure 6.21: Revetment thickness/lean sand asphalt

pressure underneath, the revetment will tend to lift. This should be prevented. If a slab lifts a cavity develops underneath into which water flows, the extent depending on the duration of the pressure difference. Because of the viscous properties of the asphalt mix it will deform. Depending on the duration and the quantity of inflowing water, the deformation can be so large that the slab breaks. A simple solution to this problem cannot be given since the pressures which develop vary from situation to situation. For simple cases the water pressure can probably be estimated using simple formulas. For more complicated cases an extensive calculation programme or an electrical analogue can offer the solution. More important than the lifting of the revetment by excess water pressure is the stability of the edges of a bed protection or mattress.

Hydraulic uplift pressures due to wave action

An impermeable bed protection slab, lying on sand, can be lifted by differences in pressure above and below the slab caused by wave action. This must be prevented. Wave action causes groundwater movements under the bed protection which change the groundwater pressure. These changes, however, are not necessarily the same as those which occur above the slab and an upward pressure can result. Two situations can be identified:

a. The wavelength is longer than the bed protection.

The maximum pressure under the revetment, in this situation, can be estimated using the following formula. Obviously the weight of the revetment must be greater than the uplift pressure.

$$h \geq \frac{\rho_w}{\rho_a} \frac{H}{2} \quad (l < L)$$

in which:

h = thickness of bed protection (m)

ρ_w = density of water (kg/m:3)

ρ_s = bulk density of bed protection material (kg/m^3)

H = wave height (m)

l = length of the bed protection in the wave direction (m)
L = wavelength (m)

b. The wave length is much shorter than the length of the bed protection.
In order to prevent the slab from being raised its weight must be greater than the maximum uplift pressure underneath. An uplift pressure is caused by the pressure resulting from the wave action above the slab being, locally, less than the groundwater pressure below. The time-dependent pore water movements, which strongly determine the groundwater pressures, cause considerable damping to this effect.
Under normal conditions the phenomena of lifting by wave action is not of major importance. More important is the possibility of scouring at the edges of the bed protection as a result of erosion. If the sand at the edges is unstable it is recommended that the watertight bed protection is overlapped at the edges with an open sand-tight revetment.

Design of an asphalt revetment against currents

The performance of various asphalt types under such forces caused by currents is discussed in the present section. Under normal conditions an asphalt revetment is very resistant to flowing water. Considerable damage however can develop, for example, if the water carries hard objects such as stones. In addition currents can lift the edges of a plate or mattress and can turn them over. This can be prevented by, for example, increasing the weight of the edges or by burying them.

Asphaltic concrete

Asphaltic concrete is only attacked slightly by currents. If the water carries hard particles, however, impacts can occur which damage the revetment material.
In Los Angeles it was found that, in the prevailing temperatures with a minimum of about 10°C, an asphaltic concrete revetment can resist the erosive effect of debris in the water if the binder is softer than penetration 50. For Dutch conditions, with a minimum temperature around the freezing point, bitumen 80/100 is more suitable.

Mastic

Mastic is an overfilled mix with a relatively low stiffness. The stresses caused by impacts from debris etc. carried by the flow are generally small and can be withstood. It is important, however, that the edges of the plate do not "flap" by the current. This can be prevented by:

1. burying the edges of the plate so that the current can gain no purchase on it.
2. dumping crushed stone on the edges. These pieces will then penetrate into the mastic layer until they are in equilibrium. This process is dependent on the difference in density between the stone and the mastic, the shape of the stones and the viscosity of the mastic. (The penetration into a layer of stones takes longer than into loose stones.) Investigations indicate that a few centimetres of mastic slab originally 20 cm thick, when still remained under the stone, dumped underwater, placed five years ago.

A mastic slab can be built up from a number of separate layers laid like rooftiles over each other. If there is not good adhesion between these layers, because of the presence of sand pollution or inadequate heat transfer, flowing water can get hold of the unattached sections, lift them and break them off.

Grouting mortars
The following formula is often used for designing loose crushed stone against stationary or quasi-stationary flow:

$$D_{50} \geq b\frac{\overline{U}^2}{2\Delta g}\{\cos\alpha\,\sqrt{1 - tg^2\,\alpha/tg^2\phi}\}^{-1}$$

in which:
D_{50} = median diameter of the revetment material (m)
U = current velocity parallel to the axis of the channel (m/s)
α = slope angle

$$\Delta = (\rho_s - \rho_w)/\rho_w$$

ρ_s = density of the revetment material (kg/m3)
ρ_w = density of the water (kg/m3)
ϕ = angle of internal friction of the revetment
g = acceleration due to gravity
b = a stability parameter

The stability parameter is dependent on many factors. With a uniform, continuous flow, for conditions which occur in the Dutch waterways it averages, 1.4. For other situations reference should be made to literature.
With grouted crushed stone, the stability parameter b can be lowered. Preferably, the value of b should be established for each particular situation by model tests. The effect of currents on fully grouted stone is negligible.

Dense stone asphalt
Dense stone asphalt is an overfilled mix and, thus, very resistant to currents. The design should be such that the edges do not flap.

Open stone asphalt
Information about the resistance of open stone asphalt to currents is, as yet, not complete. Investigations have produced the following results:

- With stationary and quasi stationary flow very limited erosion was observed after 34 hours with current velocities of 6 m/s. The damage which did occur consisted mainly of the loosening of limestone particles.
- The material has also been tested with currents generated by ships screws. These tests were made on a bank revetment of 12 cm lean sand asphalt under a cover of 18 cm open stone asphalt in the Rhein-Main-Donau Canal. The currents were produced by the screws of a cargo ship with a draught of 2 m and an 800 HP motor at full strength for 5 minutes. No damage occurred.
- Investigations have been carried out into the resistance of open stone asphalt to wave attack which showed that under normal tide conditions and over a long period, the

material did not erode. The edges of an open stone asphalt mattress can flap when the critical steady flow velocity (2.5 to 3 m/s for a 15 cm thick mattress) is exceeded. This can be prevented by making the edges heavier. This phenomena of flapping can occur with all types of slabs and mattresses.

Lean sand asphalt

Indicative tests have shown that loosely dumped lean sand asphalt is resistant to currents up to 3 m/s. This resistance to currents can be increased by raising the bitumen content and by a certain amount of compaction. When lean sand asphalt is used as a core material there is generally only a need to take flow precautions during the construction phase. The loss of a certain amount of core material during construction can be accepted provided that on completion the core is sufficiently large. When lean sand asphalt is used as a filter layer it must only be exposed to direct currents during construction. This phase should be as short as possible. If lean sand asphalt is used as a permanent revetment there must be no loss of material. Therefore the waves and currents to which it is exposed should not be too large. Tests indicate that the maximum current velocity is about 3 m/s. More investigations into this aspect are desirable. Erosion can possibly be restricted by compacting or adapting the mix or providing it with a seal coat. These measures can affect the water permeability. Crushed stone in the weight range up to 60-300 kg can be laid directly on lean sand asphalt as a protection layer. By experience it appears that larger stones cause turbulence which can erode the lean sand asphalt. To prevent this an intermediate layer (of gravel for example) can be used.

Designing an asphalt revetment against irregular settlement and scouring

Irregular settlement

Depending on the speed of settling and the properties of the asphalt an asphalt revetment will not immediately adapt to irregular settlement. After some time however provided that there is no break the revetment will again lie on the subsoil. The bending of the asphalt plate due to irregular settlement and the time taken for it to adjust can be determined using simple formulas derived by mechanics. If the surface of the settlement is assumed to be circular, the time taken for the revetment to adjust, t, can be determined using the following formula. By repeating the calculation a number of times with related values of the stiffness modulus, t can be approximated:

$$\frac{t^3}{S(t,T)} = \frac{16\,v h^2}{3\rho_a g(1-\nu^2)\,u^4}$$

in which:

ρ_a = density of the asphalt (kg/m^3)
g = acceleration due to gravity (m/s^2)
ν = Poisson's ratio for asphalt
h = thickness of the revetment (m)
u = horizontal speed of settling (m/s)
v = vertical speed of settling (m/s)
t = time (s)
time at which settling begins: t = 0

S = stiffness modulus of asphalt (time and temperature dependent). The related time can for example be fixed at 0.5 t.m?

The speed of settling (v and u) is assumed to be constant.

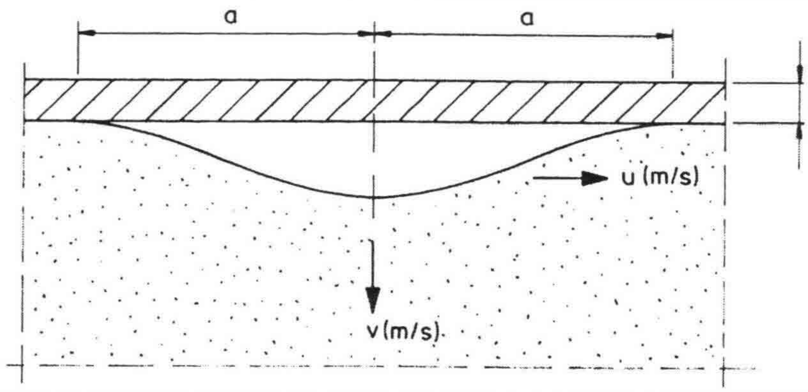


Figure 6.22: Settlement under an asphalt revetment

The largest bending stress will develop just before the plate touches the bed. The bending stress is then:

$$\sigma_b = \frac{3\rho_a g u^2 t_1^2}{4h}$$

t_1 = adjustment time

If the expansion of the settling has stopped before the revetment can adjust:

$$\sigma_b = \frac{3\rho_a g a^2}{4h}$$

a = See Figure 6.22

If the allowable stress at break σ_{bmax} is smaller than the stress σ_b which develops, the revetment will fail before the material can adapt to the subsoil surface. The bend in the plate can also be so big that after a long time the deformation capacity is exceeded and the plate breaks. The presence of a cavity under the revetment can cause extra stress in conjunction with other loads for example a wave impact. This can lead to failure. In addition large settlements can lead to a reduction in the layer thickness (viscous flow) which reduces the strength of the revetment. If the asphalt revetment is to follow irregular settlement without cracking this will depend on the speed of settling and the asphalt properties.

-- Asphalt mastic is a very suitable material for adjusting to irregular settlement because it is reasonably viscous. In the above mentioned formulas the stiffness modulus S can be replaced by $3\eta/t$ in which η is the value of the viscosity of the mastic.
-- Grouted crushed stone must be able to adjust to irregular settlement without losing its cohesion. It is more able to do this when the voids are completely filled.
-- It is important that open stone mattresses remain in contact with the subsoil. If the mattress is anchored, stresses can develop in certain sections which in combination with wave induced forces can lead to failure.

-- Because of its limited ability to adjust and in view of its function, great care is needed when using lean sand asphalt as a filter layer or revetment. Failure will not arise initially because of large deformation but because of too great a deformation speed. Because of its large bulk when lean sand asphalt is used as a core material its ability to adapt to settlement is of much less significance than that of a sand asphalt filter layer. The core should be designed to act monolithically under differential settlement. The tension level and the deformation speed should however he checked.

-- An asphaltic membrane which adjusts to settlements will stretch, see Figure 6.23. If the extension is ΔL and the original length L then the strain is $\Delta L/L$ (that is $\Delta L = \sqrt{\Delta h^2 + \Delta x^2 - \Delta x)}$. This strain must not exceed the maximum allowable value. Reinforcing in the membrane enables the strain which otherwise would concentrate in one place and could lead to excessive extension.

Scouring

Scouring usually takes place so quickly that the asphalt revetment cannot adjust. The dike should therefore be designed in such a way that the revetment cannot be undermined for example by providing a good toe and bed protection. Because of its good viscosity asphalt mastic can adjust to undermining to a large extent and is therefore often applied as bed protection. If scouring occurs at the end of a mastic apron it will bend. The time for the end of the plate to reach the bottom of the hole can be determined from the following:

$$t = \sqrt[5]{\frac{2h^2 z\eta}{\rho_a g (1 - \nu^2) v^4}}$$

in which:

h = thickness of the revetment (m)
η = viscosity of the mastic (Pa s)
ρ_a = density of the mastic (kg/m^3)
g = acceleration due to gravity (m/s^2)
ν = Poisson's ratio for asphalt
v = horizontal scouring speed (m/s)
z = depth of scour (m)
t = time when bending begins: (s)

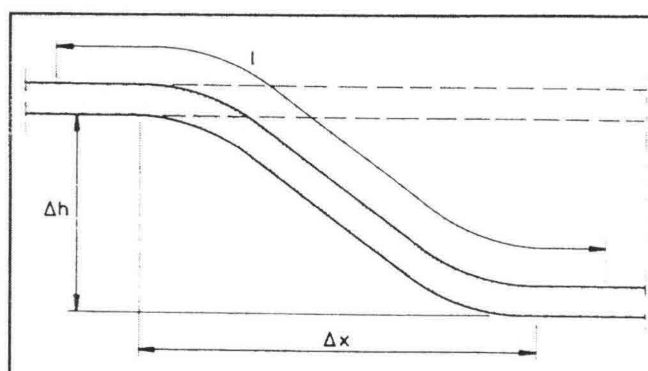


Figure 6.23: Extension of a membrane

The speed of scouring is assumed to be constant.

The largest bending stresses will develop at a distance l(t) from the end of the slab just before the end of the slab settles onto the bed of the scour hole. If these stresses exceed the limiting value the revetment will break.

$$\sigma_b = \frac{3\ \rho_a\ g\ v^2 t^2}{h}$$

in which

σ_b = bending stress (N/m^2)

t = adjustment time (s)

In principle, as soon as the end of the slab touches the bed, the scouring process stops. The mastic slab however does not lie in contact with the bed everywhere and in between the slab and the bed there may still be some cavities.

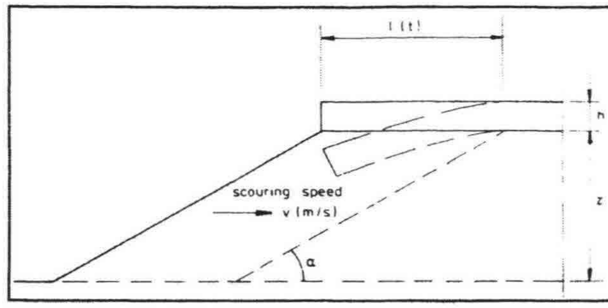


Figure 6.24: Scouring of a mastic slab

The length over which the slab is not supported is increased since the bed underneath adjusts to its own natural flatter slope. The slab will then bend further to complete its adjustment. New stresses develop in the slab which in combination with the reduced slab thickness can lead to failure. In practice a crack can develop at A (figure 6.25).

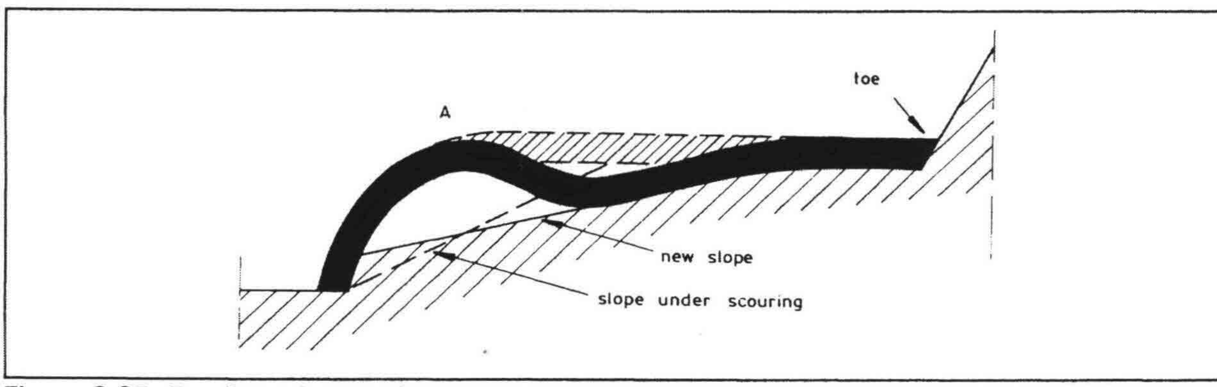


Figure 6.25: Bending of a mastic slab at a scour hole

Failure can also occur in a mastic slab lying on a sand bed which overlies a clay layer. In this situation wave pressures cannot propagate through the clay layer and will build up to cause large uplift pressures under the mastic slab. Sand under the slab can be lost through cracks and as a result undermining can proceed. Because of the stiffness properties of the material cracks are more likely to develop in winter than in summer. In warmer periods it is possible that the cracks will flow together and re-seal. The reduction of layer thickness due to viscous flow should not be overlooked. The length of a mastic slab in front of a dike must be so large that the toe cannot be undermined and the slope stability is endangered.

Determination of the maximum slope
In order to prevent the revetment sliding off the dike body the slope angle must be less than the angle of internal friction. For a relatively impermeable revetment the slope angle in places where water is likely to occur behind the revetment should be no larger than:

$$\tan \alpha \leq \tan \phi \left[1 - \frac{\rho_w}{\rho_n}\right]$$

in which:
α = slope angle (degrees)

ϕ = angle of internal friction of the subsoil (degrees)
ρ_w = density of water (kg/m^3)
ρ_s = density of wet soil (kg/m^3)

For a slope of cohesionless material where groundwater can flow out freely, in order to prevent slope instability, the angle should be:

under water:

$$\tan\phi > \frac{\tan\alpha}{\left[1-\left(\frac{\rho_w}{\rho_n-\rho_w}\right)\frac{i}{\cos\alpha}\right]}$$

above water:

$$\tan\phi > \frac{\tan\alpha}{\left[1-\frac{\rho_w}{\rho_n}(1+\tan^2\alpha)\right]}$$

in which:
i = potential gradient at the surface

The allowable slope is also determined by the internal stability of the mix itself in both the construction and the completed phase. The method of construction can also be a determining factor in the choice of slope.

Asphaltic concrete
Depending on the mix composition asphaltic concrete is stable on a slope of about 1: 1.7 even when hot. After completion the mix is stable at the same angle. The construction method is generally the limiting factor, particularly the applicability of the construction equipment.

Mastic
Mastic is preferably used on a horizontal subsoil. For a lean mastic the maximum allowable slope is about 10° and for a fat mix 5° depending on the duration and temperature regime.

Grouting mortars, surface grouting
The method of applying the mastic must be such that in the whole surface the stone remains in place.

Pattern-grouting
The grouted areas of stone must be stable under the prevailing conditions. The areas must satisfactorily lock up the interlying stones. It is thus important that the grouted areas remain as lumps. The mix composition and the method of execution must be such that the grout does not stay in the upper layers or sag completely through the layer.

Fully grouting
Because of its internal skeleton this type of revetment has high stability.

Dense stone asphalt
Dense stone asphalt is preferably used on a flat subsoil.

Open stone asphalt
Open stone asphalt is stable on slopes up to about 1: 1.5 when hot. The stability of open stone asphalt mattresses should be such that they do not slide as a whole down the slope. The maximum allowable slope is about 1: 2.5. It is possible to anchor mattresses to the slope. In this case it is essential that they remain in contact with the subsoil.

Lean sand asphalt
Because of its viscosity lean sand asphalt has the tendency to creep down a slope just like other asphalt mixes.
1. In constructions where deformation plays an important role, such as foundations and abutments, creep has to be limited.
2. For constructions in which deformation is not important the only limitation is safety. The resistance of failure of granular materials is characterized by the angle of internal friction ϕ and the cohesion c.

The failure mechanism can be studied using a slip circle analysis or the Prandtl wedge method for example.

For present purposes the equilibrium method is used to calculate the stability of the material. Here it is assumed that deformations remain sufficiently small if the relationship between the shear stress which develops and the maximum allowable shear stress is equal to or less than 0.75. For bituminous products the stability problem is more complicated because of the viscosity. A time-dependent component must be taken into account.

Investigations have been carried out on lean sand asphalt made from sand from the Eastern Scheldt and 3 to 4% bitumen 80/100. The objective was:

1. To establish the minimum angle of internal friction ϕ and thus the stress situation for which the deformation speeds are small. Under such shear stresses ϕ was in the range 20° to 24°
2. To establish the maximum angle of internal friction ϕ applicable when deviator-stresses produce deformation speeds which do not decrease but remain constant or increase. Under such shear stresses ϕ was in the range 30° to 34°.
3. To establish the relationships between the stress conditions and deformations and the deformation speed between the maximum and minimum.

There are several ways in which the failure criteria can be established:
1. Bishop's slip circle analysis. In this method the total shear strength which develops at the critical slip surface is determined. The Bishop method is the most traditional and readily applicable method.
2. A finite element method.

3. The plate method.
   This is a modification of the Bishop method. The underlying principle is that the sliding of adjacent plates of asphalt can be calculated with the existing method and that from this the relationship between deformation speed and shear stress can simply be found. In the method it is assumed that stiff plates can slide over each other by means of imaginary viscous interlayers. The summation of the separate deformations indicates the extent of the total deformation.

Large deformations developing in a short time indicate unallowable loads and thus failure.

Asphaltic membranes

Asphaltic membranes are generally covered. In situ prepared membranes are often covered with earth; prefabricated membranes with earth rubble concrete asphalt etc. The following failure mechanisms can develop:

1. The membrane causes the protection layer to slide off.
   To prevent this the friction component along the membrane should be larger than the weight component of the protection down the slope that is: f > tau α in which: f = the friction component between the membrane and the subsoil or protection layer and α = the slope angle of the membrane.
2. The shear stress transfer over the membrane can cause such large deformations that it can result in changing geometry or damage of the protection. The deformations which develop as a result of shear forces acting on the membrane should not exceed certain limits.

example of an asphalt structure

The Brouwersdam

As an example of a dike with a closed covering construction the Brouwersdam is discussed.

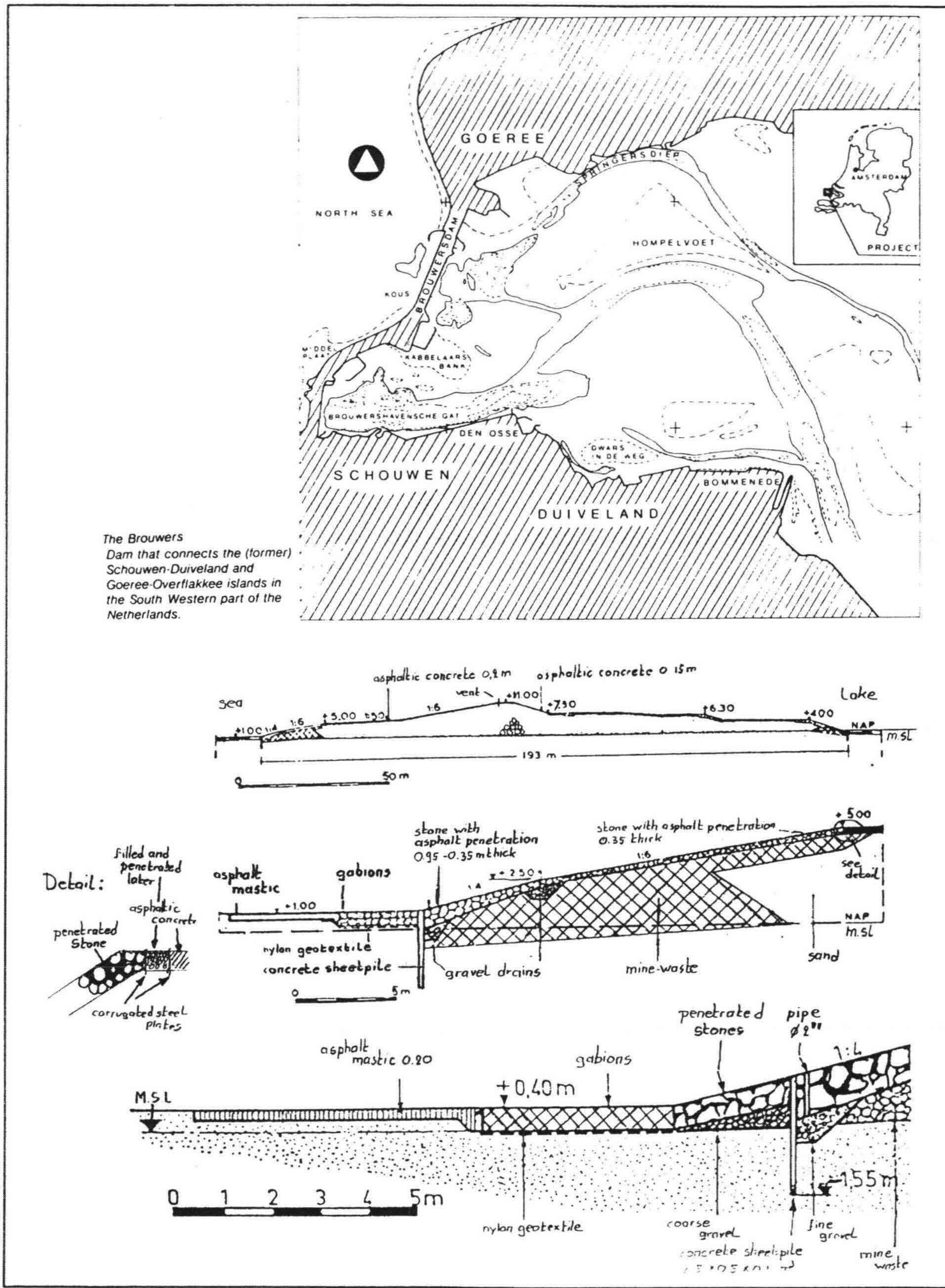
Figure 6.26 shows the locality and cross-section of the dam. The dimensions of the cross-section had to be selected in such a way as to satisfy the functional requirements of the dam, while optimizing the required dimensions with a view to cost. Thus a number of possible combinations of crest-height and outer-slope gradient were compared with each other, viz.:

a. crest height 16 m above MSL, outer slope 1:4, no berm;
b. crest height 13 m above MSL, outer slope 1:6, no berm;
c. crest height 11.5 m above MSL, outer slope 1:8, no berm;
d. crest height 11 m above MSL, outer slope 1:6, berm at storm-tide level.

In calculating the crest height in case d. it was assumed that the wave run-up with such a cross-section would be equal to three-quarters of the wave run-up of an interrupted slope, provided that the width of the berm was at least a quarter of the maximum anticipated wave length.

Wave run-up against a smooth slope may be calculated with the "Old Dutch Formula", z= 8 H tan α, where:

- z = the wave run-up in meters exceeded by 2 % of the waves;
- H = the significant wave height at the foot of the slope in metres. In this case the wave height in front of the dam is 5 m;



The Brouwers Dam that connects the (former) Schouwen-Duiveland and Goeree-Overflakkee islands in the South Western part of the Netherlands.

Figure 2.26: The Brouwersdam

α = the gradient of the slope;
z = 3/4 (8 * 5 * 1/6) = 5 m (in which is 3/4 the previous mentioned berm reduction factor).

For Zeeland the design condition is 1/4000 per year. The 1/4000 waterlevel in that location is MSL + 5.35 m. The calculated wave run-up is 5 m. This gives a crest height of MSL + 10.35 m. Because the subsoil is very good (no settlement is expected), and for the next 100 years a sealevel rise was expected of only 20 cm, the crest height was rounded off to 11 m above MSL.

In order to guarantee the applied reduction coefficient for run-up, it is essential that the berm width is at least 0.25 times the wave length.
The period of the design wave is 12 seconds. Because of the gentle foreshore and the gentle slope of the dam, shallow water conditions may be used. That means that the wave length can be computed with:

$$L = c * T = \sqrt{(gh)} * T = \sqrt{(9.81 * 4.35)} * 12 = 78.4$$

On quarter of the wavelength is approx. 20 meters. Because there is enough space available, this has been rounded off to 25 m.

The core of the dam consists of a dam with dumped concrete blocks (dumped from a cable car above the closure gap). A small section was closed using concrete caissons. After the closure, the dam was built out using sand, brought in place using hydraulic fill.

On the outer side a flexible sheet of asphaltic mastic was placed. This sheet will follow the (possible) erosion holes in front of the dam and protect the real toe construction.
The toe construction consists of a layer of two gabions (1 * 1 * 0.5 m^3), filled with <u>round</u> gravel 80 - 300 mm, placed in two layers in the baskets, and compacted very well. The gabions were made from galvanized steel with a plastic coating to prevent corrosion in the aggressive sea-water. The gravel has to be round, otherwise it would damage the plastic coating. It was compacted well, in order to prevent movement of the stones inside the gabions.
This row of gabions is required to prevent a high overpressure under the asphalt cover of the dike, especially during falling water.
If the water-pressure inside the dike is higher than outside the dike, the asphalt will be lifted somewhat from the sand body. Then there will be no friction between the asphalt and the sand and the asphalt will slide down along the slope.
The sheetpiling is necessary to increase the seepage-length of the groundwater going in and out. It also makes construction somewhat easier. The gravel drain just landward of the sheetpiles is only required during execution. Because the sand body was placed by hydraulic fill, there was a big outflow of water. It is not desirable to have all this water flowing around the sheetpiling. Therefore a gravel drain was made, with small outlet-pipes of 2 ". These pipes were sealed of after construction.
The natural stone (10/60 kg) was penetrated with 50 % bitumen. This resulted in a strong, watertight construction.
In the crest of the dike there is a small vent every 5 m to release air-pressure under the asphalt during high water.

The connection between the penetrated stone and the asphaltic concrete at the berm had to be made very careful. No leakage is allowed over there. Because both constructions were made with heavy equipment this was a big problem. Therefore both constructions were ended with a sheet of corrugated steel plates (the have a long surface) and the gap in-between was filled later on with small stones and penetrated very careful with smaller equipment.

At the inner side of the dike a road was constructed, At the inner toe there is also a drainage structure. Drains are under the road, in order to prevent that during storms the drainage after flows over the road.

6.2.4 grass slopes

One of the options for reinforcement is a slope protection of grass on a bed of clay, rather than stone, concrete or asphaltic protection. This option is only feasible in cases where vast mud-flats (high foreshore) and grasslands stretch away on the seaside of the existing dikes and are inundated only during storm surges. Also, the wave action has to be much reduced (maximum up to 2.5 m on "deep" water).
Ideally, the vegetation lining should take the form of a uniform sward and have maximum percentage cover so that the unvegetated area exposed to the erosive flow is minimal. Species with clumpy growth habits should be avoided because of the danger of flow concentrations which can give rise to high local flow drag and localized erosion. Care must also be taken to avoid invasive species which become difficult to eradicate and shade out other species, including even those of engineering value. One example in Japanese knotweed (*Reynoutria japonica*) which has invaded many river banks and distributed land sites in the UK over the last decade and which spreads very rapidly by flooding or by earth-moving operations. On no account shout this or similar species be chosen as part of an ecological succession. For mor details on the ecological aspects of vegetation on slopes is referred to COPPIN & STILES, 1995.
A full scale model study has been done, which is an absolute requirement as grass cannot be scaled down. The investigations were done in the Delta Flume where a five metre wide section of the grass dike was reproduced on full scale. The model consisted of a sand core covered with a clay layer on a slope 1 on 8. Sods of grass with the depth of the roots of approximately 40 cm were laid on top of the clay layer (the grass was taken from an existing dike that was reinforced ten years ago). During the tests, the wave heights and periods and water levels (tidal cycles) were varied continuously according to predetermined boundary conditions during the design storm surge. The maximum H_s was equal to 1.85 m with T_p = 5.6 sec. (plunging breaker falling on a water cushion). The measured maximum velocity on the slope (1:8) was about 2 m/s. After 30 hours of continuous random wave attack the condition of the grass dike was still exceptional well. The surface erosion speed of clay protected by grass was not more than 1 mm per hour. In a number of additional tests, the durability of the grass and the enlargement of holes, previously dug in the grass, were studied. Although wave action considerably enlarged some of these holes, the residual strength of the dike was such that its collapse was far from imminence.
The second investigation was carried out in a large (site) flume on slope 1 on 4. Special equipment was used to simulate the run-up and run-down velocities on this slope. Two qualitatively different grassmats on clay were used. The grass-mats were tested with the average velocity of 2 m/s (average over 40 hours of test) and the thickness of a water layer

of about 0.6 m. The maximum velocity was about 4 m/s. Erosion speed of the clay surface was 1 to 2 mm per hour up to 20 hours depending on quality of grass-mat. After 20 hours of loading the erosion speed started to grow much progressively for a bad quality grass-mat. Similar process took place for a good quality grass-mat but after 40 hours of loading.

Some additional information on resistance of unprotected clay-surface (slope 1 on 3.5) were obtained during the investigation carried out for the Eastern Scheldt dikes. Also in this case two qualitatively different clays were used (fat and lean clay). The surging-breaker conditions were applied to eliminate the effect of wave impact (H_s = 1.05 m, T_p = 12 s, max. velocity 3 m/s). The erosion on the upper part of slope was for both clay-types the same and equal to about 2 - 3 cm after about 5 hours of loading. After the same time, the erosion below S.W.L. was about 7 cm for a good clay, while for a lean clay a cavity of about 0.4 m depth was created. This latest probably because of the local non-homogeneity of clay. Also during this investigation a number of additional tests on the erosion of different sublayers (incl. clay) at locally damaged top layers (some protective units were removed) were performed.

All the tests mentioned above indicated that the strength of the grass slopes is strongly affected by the quality of clay and the condition of grass, especially its rooting. The general design rules cannot be defined yet. It proved that the inhomogeneity of clay is the most important design parameter.

7 Filters

7.1 Filter types, locations and functions

Three types of filters can be distinguished in hydraulic and geotechnical engineering:
A Fine soil protection filters (Figure 7.1)
B Pressure relief filters (Figure 7.2)
C Discharge filters (Figure 7.3).

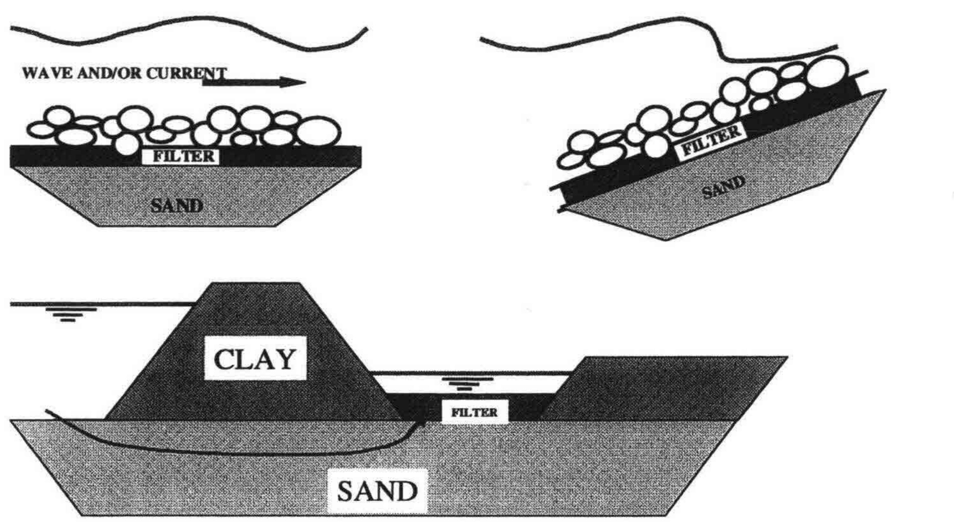


Fig. 7.1 Three examples of 'fine soil protection filters'

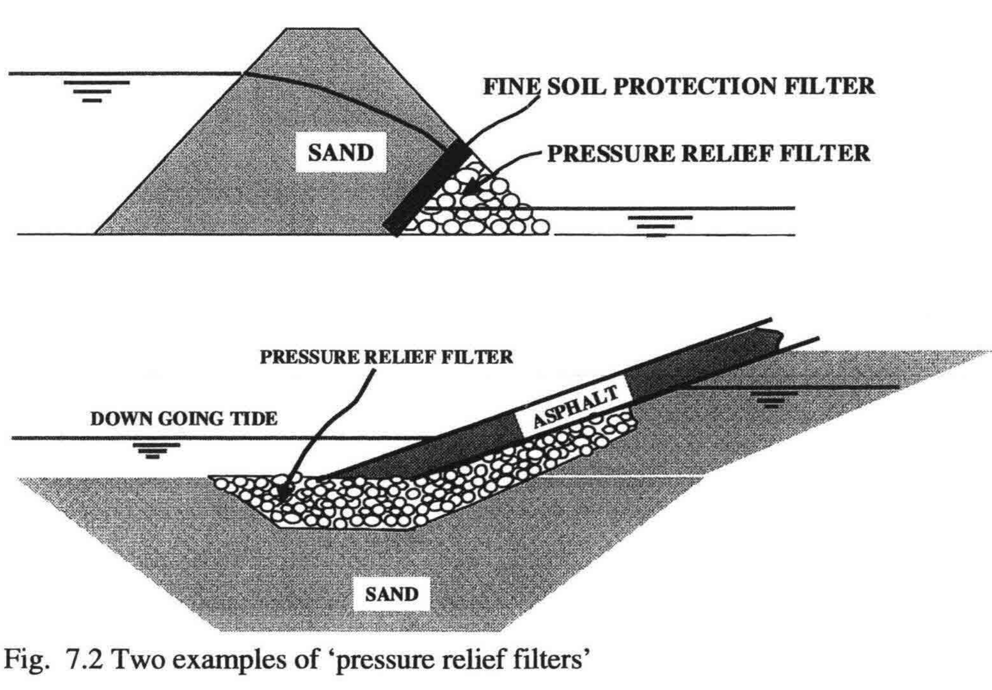


Fig. 7.2 Two examples of 'pressure relief filters'

Only the types A and B are relevant for dykes. Type A is especially needed at locations in and around a dike where erosion of sand or other fine grained material (failure mode 5 'internal erosion') may occur due to large gradients on the interface of soil and water. Type B is useful at locations where large pore pressures endanger the sliding or uplift stability of the soil (failure modes 3, 'micro-instability' or 5 'macro-instability').

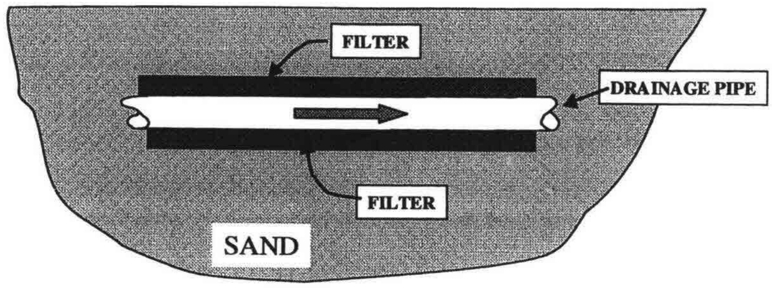


Fig. 7.3 Example of 'discharge filter'

Each filter, whatever the type is, must prevent the transport through the filter of one material (grains or particles), but allow for the transport of another material (water). So each filter has two functions (Figure 7.4):

I 'Filter stability', i.e. the prevention of the transport of fine grains.
II 'Sufficient permeability'.

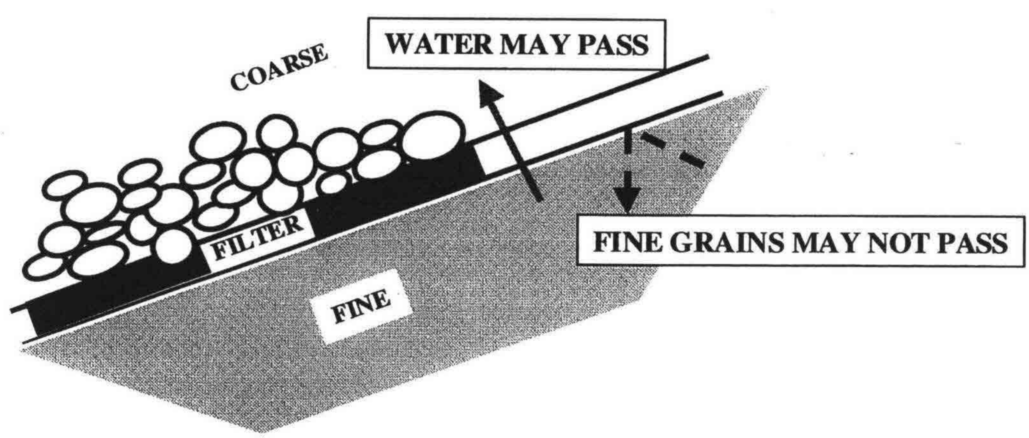


Fig. 7.4 The two basic functions of any filter

Remark: in several publications other functions like 'segregation prevention', 'separation' are mentioned. These functions, however, are basically not different from function I. The filter layer/geotextile may have other functions, but these are no filter functions. Filter requirements, directly derived from these functions are discussed in the subsequent sections. Less attention will be paid to the requirements that can indirectly be derived from the functions: requirements with respect to construction (can the filter structure be brought in place such that it full fills its function?) and to durability (can the filter structure remain functioning long enough?).

7.2 Granular filters

7.2.1 Description and orientation

Granular filters are designed in successively coarser layers proceeding outward from the finer soil. According to the first filter function, the first layer should hold the grains of the sub soil or core material of the dike, while the second layer should hold the grains of the first one, the third should hold the grains of the second one etc.. Relevant for the functioning are the 'interfaces' between sub soil or core material and first layer, between first layer and second layer etc. The finer of the two materials on both sides of the interface is always called the 'base', the coarser the 'filter'. Thus, the first layer is called 'filter' when considering the interface with sub soil or core material, but 'base' when considering the interface with the second layer.

The coarsest outer layer has often another function, e.g. the function of cover layer of a slope protection, which should be stable under the prevailing open boundary conditions (hydraulic loads by waves and current). Then, it is not always called a 'filter layer' and , if the 'first layer' is also the outer one, even no layer is called 'filter layer'. Even in that case, however, there is an interface and the structure should meet the requirements of a filter. See Figure 7.5.

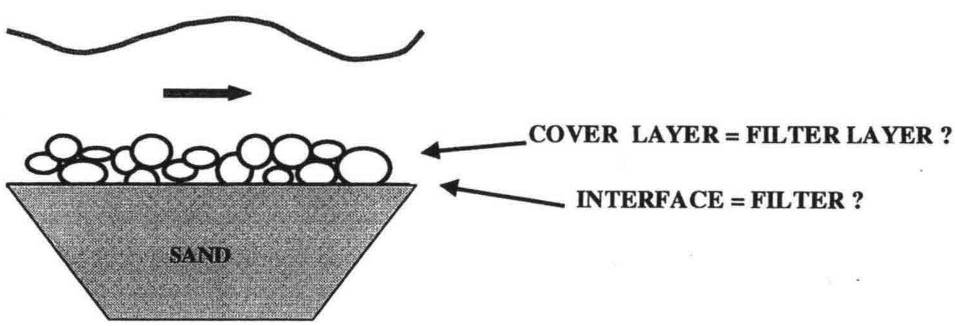


Fig. 7.5 No commonly accepted definition for number of layers if one filter layer is also cover layer

Relevant for the dimensioning is the orientation of the interface and the question whether the underlying material is the finer one (as in most cases) or the coarser one (sometimes called 'up side down' filter). See Figure 7.6.

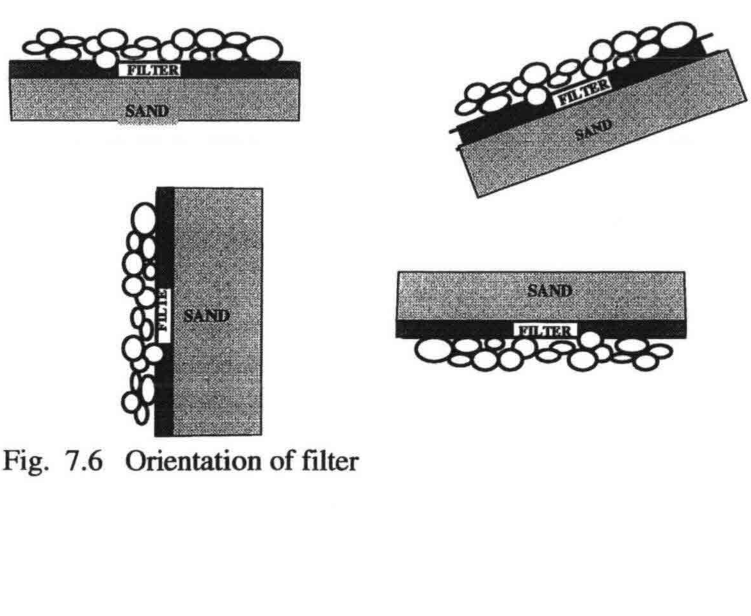


Fig. 7.6 Orientation of filter

The most traditional filter consists of several layers of uniformly graded material (Figure 7.9). Often such a construction is costly if it is to be constructed under water. Especially the layers of finer material are difficult to bring in place and keep in place (section 7.2.5). Several alternatives are applied:

- Replace the 2 or 3 layers of fine graded uniform material with one layer of well-graded material. Advantages and disadvantages will be discussed in section 7.2.5.
- Replace the 2 or 3 layers of fine graded uniform material with one layer of geotextile. See [Pilarczyk 1998], Ch. 10, figure 11 and section 7.3.
- Design a geometrically open filter. See section 7.2.2.

7.2.2 Requirement Ia 'interface stability'

The first filter function, the prevention of the transport of fine grains or particles, is relevant in the first place for the fine grains of the 'base material'. 'Interface stability' means that these fine grains should be protected against being transported through the interface and through the pores of the relatively coarse filter material (Figure 7.4). In [Pilarczyk 1998], Chapter 4, Figure 4c, this transport is called 'suffosion'.

The traditional design criterion can be characterised as '**geometrically tight**' or 'geometrically closed', which implies pore (grains) sizes too small to allow the fine grains to pass through. Such filters are relatively simple to design and all that is required is knowledge of the grain size distributions and the pore or opening size distributions of the filter.

The following geometrically tight criterion can be applied (Figure 7.7):

$D_{15f}/D_{85b} < 5$

where the indices 'b' and 'f' are used for the base and the filter respectively, thus the finer and coarser materials respectively and the numbers refer to the grain size distribution curve.

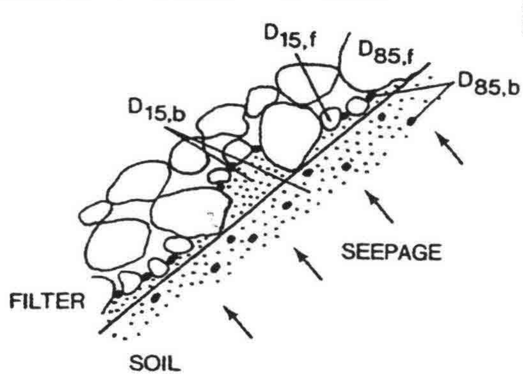


Fig. 7.7 Interface stability of granular filter

Application of above geometrical tight criteria is still the best in cases of heavy hydraulic loads and in cases where the interface is nearly vertical of the 'base' rests on top of the filter. There are many situations, however, in which '**geometrically open**' filters are most suitable to prevent an uneconomically large number of filter layers, which would be required according to the geomerically tight criterium. The application of the criteria for 'geometrically open' filters is based on the principle that the hydraulic load must be too small to initiate erosion of the base (fine) material. These criteria, however, require more knowledge about the hydraulic loads on the filters, caused by the water movement along and inside the structure.

Extensive series of laboratory tests have been carried out on interface stability, with hydraulic gradients both parallel to the interface and perpendicular and both stationary as well as cyclic. A theoretical foundation of the results has been developed by Klein Breteler and an extension of the experiments to situations with a slope has been made. See [Pilarczyk 1998], chapter 10, section 2.3 and/or chapter 12, Fig. 18.

7.2.3 Requirement Ib 'internal stability'

The first filter function, the prevention of the transport of fine grains, can also be relevant for the fine grains within the core material of the dike or within the granular material used for a filter layer. The risk of this type of transport is only present if the granular material in question is very well-graded, i.e. if the coarsest grains within the material are much coarser than the finest grains within the same material. The most simple requirement reads, accordingly:

$C_u = D_{60}/D_{10} \leq 10$

However, if $C_u > 10$, the risk of internal instability is only present if the grain size distribution is "gap-graded", i.e. if the material consists of very fine grains and very coarse grains, whereas the middle sized grains are practically absent. No gap is present in the well-graded material if it meets the following **geometrically tight** requirement, formulated by Kenney and Lau:

$$\left[\frac{F_{4D} - F_D}{F_D}\right]_{min} > 1.3$$

where F_{4D} and F_D are two (dependent) characteristics (cumulative mass %) of the grain size distribution curve. Moving along the curve, values of $(F_{4D}- F_D)/F_D$ will vary and the minimum value of $(F_{4D}- F_D)/F_D$ is found at the flatter part of the grain size distribution curve, as illustrated in Figure 7.8. The requirement can also be simplified to:

$D_{10} < 3\ D_5$
$D_{20} < 3\ D_{10}$
$D_{30} < 3\ D_{15}$
$D_{40} < 3\ D_{20}$

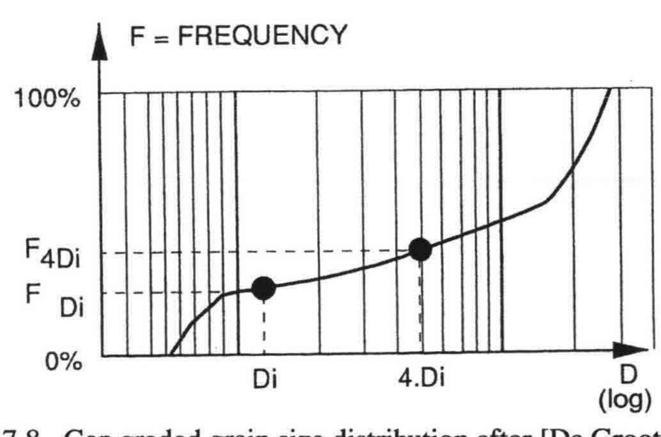


Fig. 7.8 Gap graded grain size distribution after [De Groot et al 1993]

In cases of strong hydraulic loadings the geometrically tight criterion is still the best. If however, the hydraulic gradient is smaller than unity ($i < 1$), the following **geometrically open** criterion (Den Adel et al, 1988) can be applied, which defines a critical gradient (the 'strength', i_{cr}), to be compared to the actual gradient (the 'loading', i):

$$i_{cr} = 0.5 \cdot \left[\frac{F_{4D} - F_D}{F_D} \right]_{min}$$

Now stability is guaranteed as long as holds that $i < i_{cr}$ (loading < strength). The actual gradients (i) can be determined as explained in section 8.2.

7.2.4 Requirement II 'sufficient permeability'

GENERAL

Water conveyance or drainage is the other function of a filter. How this requirement should be quantified, largely depends on the function and location of the filter in the whole structure.

The best way to quantify the requirement is to make pore flow calculations, predict the consequent pore pressures and analyse the consequences in terms of stability. For filters in slope protections the permeability must be sufficient to prevent excess pore pressures to contribute to instability of the structure mentioned. This requires the prediction of any excess pore pressures and the prediction of the resulting stability (Fig. 7.9).

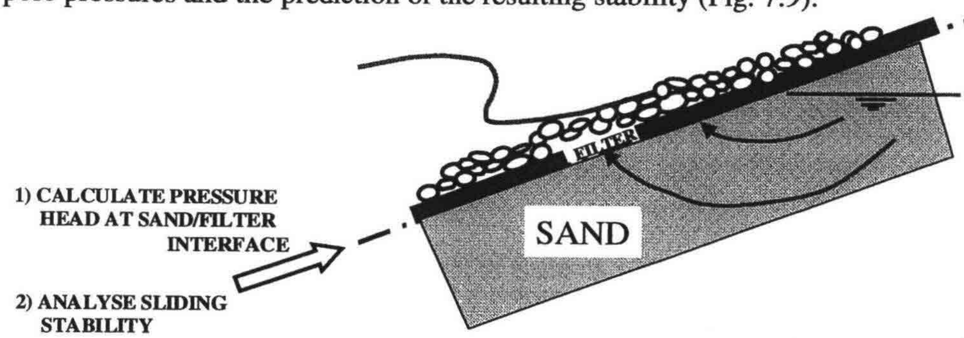


Fig. 7.9 Best design approach for requirement 'sufficient permeability'

Here, however, simplified design rules will be presented for two common cases.

SLOPE OR BED PROTECTION WITH GRAVEL AND STONE

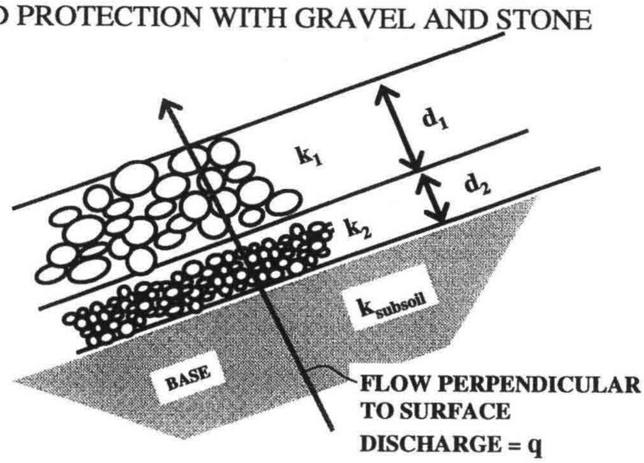


Fig. 7.10A Granular filter with several layers, each with its own permeability

Consider the upward flow with discharge q perpendicular to the surface. The slope protection must be designed such that this flow will not cause uplift of the slope or bed protection from its base and will not cause sliding of the slope protection along the interface with the subsoil. This is more or less guaranteed if the upward force by the flow on the slope protection is smaller than the upward force on a part of the subsoil with the same thickness. The upward force on a layer with thickness **d** is proportional to the hydraulic resistance. The hydraulic resistance is equal to the ratio of head difference **ΔH** and discharge (per unit area) **q**, but also to the product of head gradient **i** and thickness **d**: hydr.resist = **ΔH/q = i.d**.

This yields the following 'SIMPLIFIED DESIGN RULE':

$$\frac{\Delta H_1 + \Delta H_2}{(d_1 + d_2)\cdot q} < \frac{(d_1 + d_2)\cdot i_{base}}{(d_1 + d_2)\cdot q} \Rightarrow \frac{d_1/k_1 + d_2/k_2}{(d_1 + d_2)} < \frac{1}{k_{base}}$$

This rule can be simplified even more by replacing it by the 'LAZY DESIGNERS RULE':

$$k_1 > k_2 > k_b \quad \text{OR} \quad k_f > k_b$$

where k_f is the permeability of the filter layer (in case of just one layer) and k_b that of the base.

The rule $k_f > k_b$ is practically equivalent to $D_{15f}/D_{15b} > 1$. A granular filter structure like sketched in Figure 7.10B results with the above requirement.

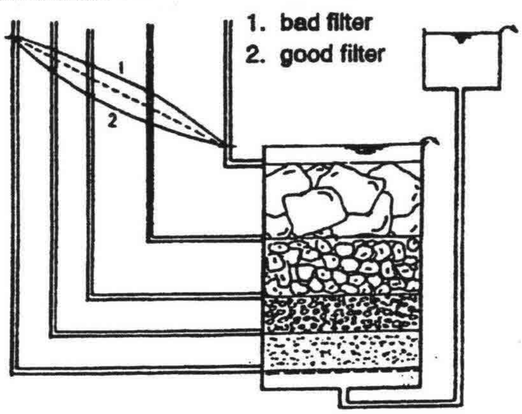


Fig. 7.10B Granular filter with several layers and possible pore pressure distributions depending on permeabilities of different layers

Remark: in [Pilarczyk 1998], Chapter 4, section 8 and Chapter 10, the factor '1' is replaced with '4 to 5' or '5'. This requirement is taken over from other publications. There is, however, no justification for this requirement. It can better not be applied.

The 'LAZY DESIGNERS RULE' is a very well-known rule. It is easy to apply, but may be too conservative on case of 'pressure relief filters' and, especially, 'fine soil protection filters'[De Groot et al 1993]. The rule is, for example, too strict if d_2 is small, whereas k_1 (and d_1) are large. Then a value $k_2 < k_{subsoil}$ is still also O.K. according to the 'SIMPLIFIED DESIGN RULE'.

BLOCK REVETMENT

Special types of slope protections are revetments where concrete (or natural stone) blocks are placed on a filter layer. The blocks make up the cover layer. For this structures, the ratio between the hydraulic resistance of the cover layer perpendicular the the surface and the hydraulic resistance of the filter layer ***parallel*** to the surface should be limited. This ratio is expressed as the 'leakage length'. It will not be discussed here.

7.2.5 Construction and execution

A filter layer of which the grain sizes meet the above requirements, function well if its thickness is at least ca 2 times D_{50}. The execution should be such that this thickness is guaranteed in the whole area of the filter layer. This is not as easy in case of construction of a filter under water. The chance of a missing part is relatively large. See Figure 7.11. Depending on the equipment used, the accuracy of the positioning system, the wave and current conditions, the experience of the contractor and the quality control, a minimum layer thickness of 0.3 m tot 2.0 m is recommended for under water construction of the fine grained filter layers.

Another problem is erosion of the fine grained soil during construction if the coarses material is not brought in place quick enough (Figure 7.12).

Construction of well-graded material is difficult for two reasons:
- De-mixing may occur during transport and dumping in the barges (coarse material here; fine material there)
- The fines tend to arrive on top of the coarse material due to their relatively small fall velocity (Figure 7.13).

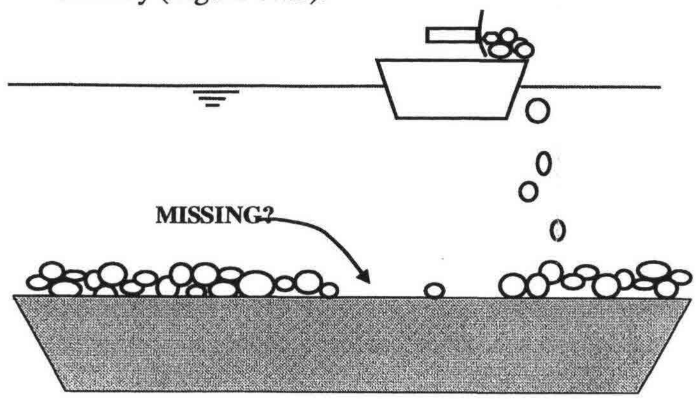


Fig. 7.11 Limited accuracy with construction under water

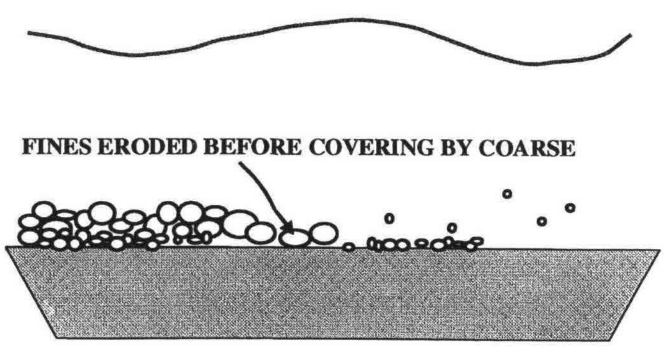


Fig. 7.12 Execution problem: erosion of fines before protection of coarse is ready

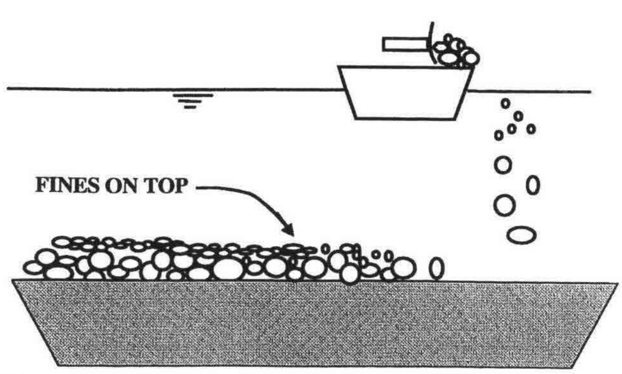


Fig. 7.13 Execution problem: fines arrive on top of coarse due to difference in fall velocity

7.3 Geotextile filters

7.3.1 Material aspects

Geotextiles are permeable textiles made from natural or artificial fibres. This sub-section will only deal with artificial fibres.

Geotextile manufacture

Geotextiles are manufactured from a variety of artificial polymers:

1. Polyamide (PA);
2. Polyester (PETP);
3. Polyethylene (low-density LDPE and high-density HDPE);
4. Polypropylene (PP);
5. Polyvinylchloride (PVC);
6. Chlorinated polyethylene (CPE).

The first four are the most widely used although many variations are possible. Additives are also employed in geotextile manufacture to minimise ageing, to introduce colour and as anti-oxidants and UV stabilisers.

Comparisons of properties of the four main polymer families are shown in the figure 7.6. These are very broad, because there are many variants within each group. Some properties (such as strength) are also greatly influenced by the different processes of manufacture. A classification of geotextiles based on the type of production and the form of the basic elements is given in figure 7.7 below.

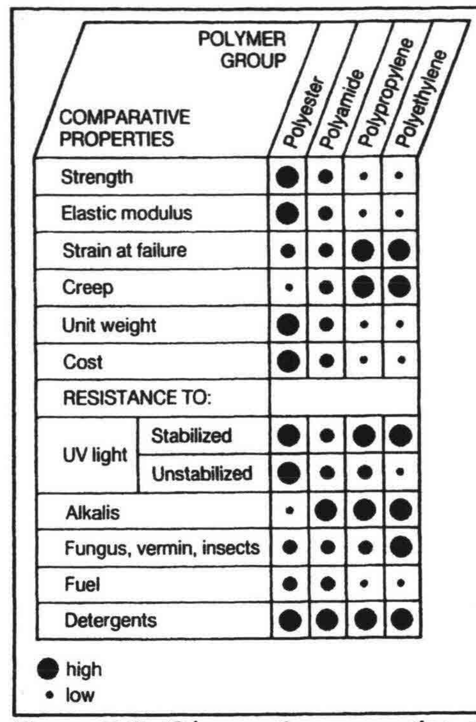


Figure 7.6: Comparative properties of general polymer families

The basic elements used in geotextiles are monofilaments, multifilaments, tapes, weaving film and stable fibres. Monofilaments are single, thick, generally circular cross-sectioned threads with a diameter ranging from 0.1 mm up to a few millimetres. Multifilaments (yarns) are composed of a bundle of very thin threads. Yarns are also obtained from strips and from wide films. Tapes are flat, very long plastic strips between 1 and 15 mm wide with a thickness of 20-80 μm. A weaving film is sometimes used for the warp 'threads' in a fabric.
The basic fibre is a fibre of length and fineness suitable for conversion into yarns or non-woven geotextiles. For non-woven fabrics the length is usually about 60 mm.

Woven geotextiles have a sieve-like appearance of two-dimensional nature and relatively uniform size of the openings. Nonwoven geotextiles are manufactured by linking randomly deposited fibres, resulting in a more complex textile structure, more three dimensional and with greater thickness than wovens. Since the introduction of geotextile filters a debate has been ongoing regarding the advantages and disadvantages of woven versus nonwoven geotextiles. The answer is often to be found in the design and application requirements.

It has been experienced that either woven or nonwoven geotextiles perform adequately when filtering stable granular soils, as long as the geotextile meets the filter design criteria. Nonwovens may provide more resistance to extrusion of fine graded soils, while woven geotextiles may provide more resistance to intrusion due to higher tensile modulus. When

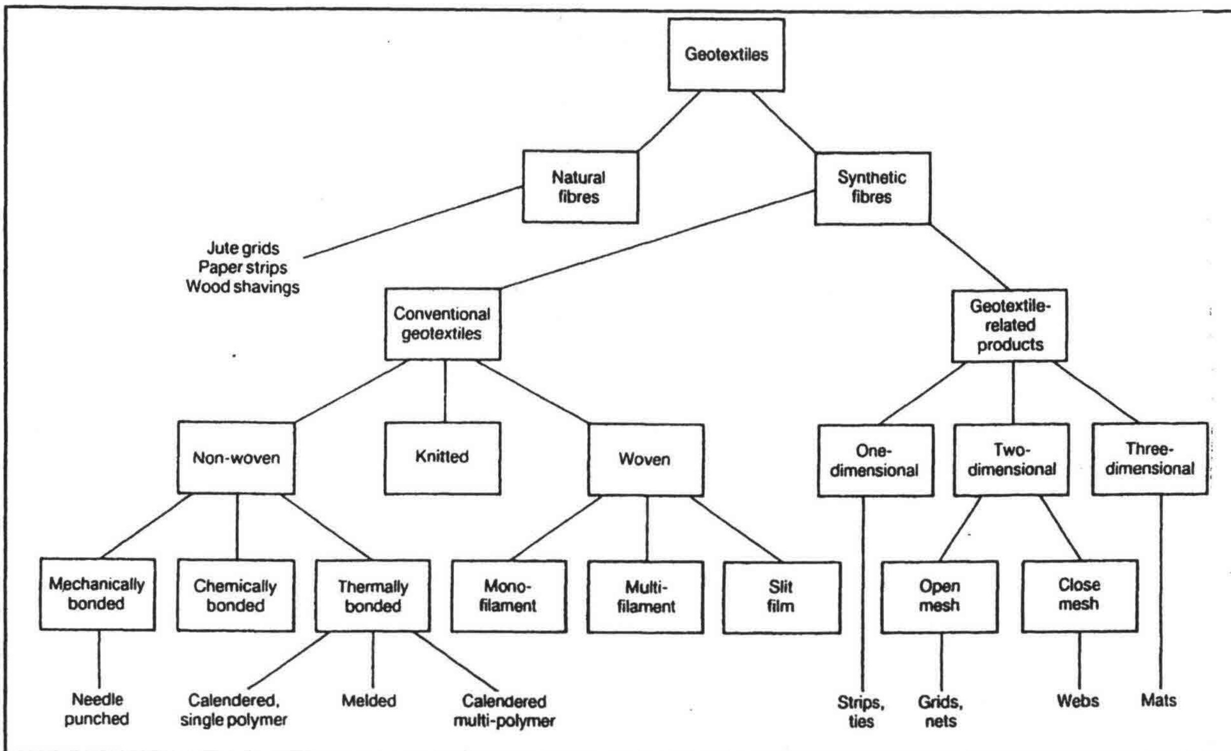


Figure 7.7: Geotextile classification group

unstable soils are encountered, it is experienced in general that thick nonwoven geotextiles (needle-punched,'fluffy' texture) are better suited for applications which require stringent retention performance from the filter. Alternatively, thin (sieve like) woven geotextile filters are better suited to allow fines to escape easily through the geotextile for applications that require a high permeability filter zone.

Woven geotextiles
A woven fabric is a flat structure of at least two sets of threads. The sets are woven together, one referred to as the warp, running in a lengthwise direction, and the other, the weft, running across. Woven geotextiles can be categorized by the type of thread from which the fabric is manufactured.

Monofilament Fabrics are used for gazes or meshes which offer relatively small resistance to through-flow. The mesh size must obviously be adapted to the grain size of the material to be retained. Monofilament fabrics are made principally from HDPE and PP.

Tape fabrics are made from very long strips of usually stretched HDPE or PP film, which are laid untwisted and flat in the fabric. They are laid closely together, and as a result there are only limited openings in the fabric.

Split-film fabrics are made from mostly fibrillated yarns of PP or HDPE. The size of the openings in the fabric depends on the thickness and form of the cross-section of the yarns and on the fabric construction. These split-film fabrics are generally heavy. Tape and split-film fabrics are often called slit-films.

Multifilament fabrics are often described as cloth, because they tend to have a textile appearance and are twisted or untwisted multifilament yarns. The fabrics are usually made from PA 6, PA 6.6 or from PETP.

Besides the above-mentioned monofilament fabrics, special *mesh-type constructions* are produced such as those with a monofilament warp and a multifilament weft which have outstanding water-permeability and sand-retention properties. Other examples include open meshes in which the woven or unwoven warp and weft threads are attached at crossing points by chemical or thermal bonding and other meshes constructed by using knitting techniques.

Non-woven geotextiles
A non-woven geotextile is a textile structure produced by bonding or interlocking of staple fibres, either monofilaments or multifilaments arranged at random, accomplished by mechanical, chemical, thermal or solvent means. Non-woven gazes are structures with large meshes which are formed by placing threads or tapes at predetermined distances on top of one another and bonding them at the intersections by a chemical, thermal or mechanical process.

Geotextile-related products
These products are distinguished in one-dimensional (strips, ties), two-dimensional (grids, nets, webs) and three-dimensional (mats) products. Grids are lattices made from perforated and then stretched polymer sheets. Three-dimensional mats are produced by extruding monofilaments into a rotating profile roller, followed by coating so that the threads adhere to each other at crossings which are spatially arranged. The matting material itself occupies less then 10% of the mat volume. The mats are 5-25mm thick and about 1-6m wide.

Characteristics and properties
The geotextile properties stem primarily from their functional requirements. Since the geotextile can have a variety of functions, requirements are diverse. For reinforcement the emphasis is on mechanical properties such as E-modulus and strength, for filters it is on properties such as water permeability and soil tightness. The durability required will depend on the specific application and lifetime required. Geotextiles must also fulfil secondary functional requirements related to the execution of the work (e.g. a certain amount of UV resistance is needed) or it must have resistance to mechanical wear and tear if construction equipment is to be driven over the fabric. The suitability of a geotextile should be checked against these functional requirements during the design phase of the project.

Although specification requirement tests need only be carried out once, the following quality control tests may be required during production: tearing strength, grab test strength, tensile strength, strain at breaking load, moduli and mass distribution. These tests should be made in both the length and width directions. The thickness, the mass per unit area and the bursting strength may also need to be checked, and in some applications water permeability and sand-retaining properties will be important. A large number of national and a few international standard test methods are available covering these requirements.

Dimensions
The maximum standard width available for both woven and non-woven fabrics is 5-5.5m. The length is limited by the available transport facilities and ease of handling on-site. Depending on the mass per unit area, the length generally lies in the range 5-200 m.

Jointing is necessary to obtain greater dimensions. In practice, large areas can be covered by overlapping sheets. Where physical continuity is required without overlap then heat welding (some non-wovens) or stitching may be used. The seam forms the weakest link in the geotextile construction and should therefore be checked thoroughly against the specifications. The thickness of most geotextiles lies between 0.2 and 10 mm when unloaded, although this may sometimes reduce under pressure.

In general, the mass of non-woven geotextiles lies in the range 100-1000 g/m², 100-300g/m² being the most commonly used. Woven fabrics can be heavier and masses between 100 and 2000g/m² are possible. The greater demand is for the lighter grades in the range 100 200g/m². Generally, the lighter types of geotextiles are used as separators, the heavier woven fabrics for reinforcement and the heavier non-wovens for fluid transmission.

Mechanical properties
The mechanical properties of geotextiles depend on a number of factors: temperature, atmospheric conditions, the stress-strain history, the mechanical properties of the material and fibre structure, the structures of the yarn and of the geotextile, the direction of anisotropy, and the rate of loading and ageing. Most fabrics exhibit cross-contraction under loading. However, light tape fabrics and fabrics with so-called 'straight' warp construction do not exhibit cross-contraction and construction strain (strain due to fibre straightening). Test methods can be categorised into those that do not prevent cross-contraction (uniaxial) and those that do (biaxial). Methods commonly employed for tensile testing are strip tensile, grab tensile, cuff tensile, plain-strain tensile, wide width tensile and biaxial.

A great variation in both strength and stiffness exists. The strength varies generally between 10 and 250kN/m. Non-woven and woven PP fabrics are not ideal in situations where high strength is combined with low strain because of the large elasticity of these geotextiles.

All melt-spun synthetic polymers, as used in geotextiles, have visco-elastic behaviour, which means that the mechanical behaviour is time-dependent. This becomes manifest in creep and relaxation phenomena. Creep data for polymer materials can be presented in several ways. Often log ϵ (strain) is plotted against log t (time) for various levels of the ultimate short-term load U, i.e. 50% U, 25% U, etc. The sensitivity to creep of polymers increases considerably in the sequence PETP, PA, PP and PE. For geotextiles that are loaded for prolonged periods of time (10 100 years) the permissible load for polyester is the order of 50% of the tensile strength, for polyamide 40%, and for polypropylene and polyethylene below 25%.

Burst and puncture strength of geotextiles is important in coastal and shoreline rock structures. In these tests a circular piece of geotextile is clamped between two rings and loaded directly by gas or water pressure or by a physical object, Puncture tests can be used for investigating the resistance of a geotextile to puncturing by, for instance, falling stones.

The other tests available include the California Bearing Ratio (CBR) plunger tests, the cone drop test and the hammer test (BAW). Test methods are also available for strength parameters such as tear strength, abrasion resistance and friction coefficient.

Chemical properties
One of the characteristic features of synthetic polymers is their relative insensitivity to the action of a great number of chemicals and environmental effects. Nonetheless, each plastic has a number of weaknesses which must be taken into account in the design and application. Specifically, the life of geotextiles can be affected by oxidation and by some types of soil/water/air pollution. Many synthetic polymers are sensitive to oxidation. The end result of oxidation is that mechanical properties such as strength, elasticity and strain absorption capacity deteriorate and the geotextile eventually becomes brittle and cracks.

Investigations have shown that, provided the geotextile is not loaded above a certain percentage of the instantaneous breaking strength, the thermo-oxidative resistance will determine the theoretical life of the material. The allowable load for PETP is, at most, 50%, for polyamides it is somewhat lower, and for polypropylene and polyethylene it is about 10-30%. (This guidance only applies where the geotextile functions as a filter and is not withstanding mechanical loads.)

Specific additives have been developed to counteract these processes. These can be grouped according to their protection function as either anti-oxidants or UV-stabilisers. In fact, the thermo-oxidative resistance of a geotextile is determined by a number of factors: the thermo-oxidative resistance of the polymer itself, the composition of the anti-oxidant packet, the effect of the thermo-oxidative catalytic compounds in the environment, the effects of processing on the long-term thermo-oxidative resistance, the resistance of the anti-oxidant additives to leaching by water and the practical site conditions.

7.3.2 Requirement I 'filter stability'

The filter stability has only to do with 'interface stability'. The most strict **geometrically tight** filter rule is that the smallest particles cannot pass the largest opening in the geotextile. The soil tightness of a geotextile on a particular type of soil can be determined by reconstructing the situation in a model laboratory apparatus and then carrying out measurements using the hydraulic boundary conditions.

Additionally the soil tightness can be characterised by a requirement with respect to the ratio of geotextile opening size and grain diameter. Correspondingly, the most strict geometrically tight filter rule reads:

$$O_{98} < D_{15,b}$$

where O_{98} is the effective opening size of the geotextile which corresponds to the average sand diameter of a sand fraction 98% of which remains on the geotextile during sieving. There are various methods of sieving, varying from country to country. The main differences being that some use dry sieving and others wet sieving, with either one-directional flow or alternate flow.

The design rule $O_{98} < D_{15,b}$ should be applied if there is a strong cyclic loading through the filter (varying in direction) and if no loss of material is accepted. In many situations, however some loss of fine material is not detrimental to the functioning of the filter, since a small layer under the geotextile can act as part of the total filter system. The finer parts are washed through the textile, whereas the coarser particles remain behind and act as a filter for the remaining soil, provided the sub soil is internally stable (section 7.2.3). In that case, a practical stability rule for partially closed geotextiles reads:

$$O_{98} < 2D_{85,b} \qquad \text{OR} \qquad O_{90} < (1 \text{ to } 2)D_{90,b}$$

Design rules for **geometrically open** geotextile filters have also been developed. See [Bakker et al 1990].

7.3.3 Requirement II 'sufficient permeability'

BASIC PROPERTIES

The water permeability of a geotextile incorporated into a structure depends on the geotextile itself, the subsoil, the load imposed on the face of the geotextile, the hydraulic load, the blocking of the geotextile, the clogging of the geotextile, the water temperature and the composition of the water. The pore size and the number of pores per unit area of a geotextile primarily determine the permeability. The subsoil and imposed load determine how much water has to be discharged and the compression of the geotextile.

Flow through the fabric is normally laminar when the geotextile is embedded in the soil, but becomes turbulent when subjected to wave action (for example, under a coverlayer of rip-rap or blocks). Blocking of flow occurs when soil particles partly wedge into the pores. This normally only arises in situations of unidirectional flow rather than the oscillating flows, which frequently occur under wave action. Clogging of flow occurs when fine particles (for example, iron particles) settle on or in the geotextile or at the interface between the geotextile and the subsoil. Reductional factors in permeability of geotextiles due to clogging of the order of 10 have been found.

Permeability is usually measured in the laboratory using values of hydraulic gradient low enough for laminar flow. For thin, more permeable fabrics, the permeability test may be performed on several separated layers to increase the measurable water head and still maintain laminar flow.

The hydraulic behaviour of a geotextile is not as much determined only by the permeability of the geotextile, $\mathbf{k_g}$, but rather by the 'permittivity' ψ with:

$$\psi = \frac{k_g}{t_g} = \frac{q}{\Delta H}$$

where:

ψ = permittivity of geotextile [1/s]
k_g = permeability of geotextile [m/s]
t_g = thickness of geotextile [m]
q = discharge (per unit area) perpendicular to geotextile [m/s]
ΔH = head loss over geotextile [m]

The permittivity of geotextiles varies between 0.01/s and 10/s. Many publications focus on the permeability instead of the permittivity, although the permittivity is much more relevant in hydraulic engineering and should be preferred.

DESIGN RULES
Like with granular filters, the best way to quantify the requirement is to make pore flow calculations, predict the consequent pore pressures and analyse the consequences in terms of stability.

In case of bed or slope protections the same 'SIMPLIFIED DESIGN RULE' can be applied as presented in 7.2.4:

$$\frac{d_1/k_1 + d_2/k_2}{(d_1+d_2)} < \frac{1}{k_{base}} \Rightarrow \frac{d_1/k_1 + 1/\psi}{(d_1+t_g)} < \frac{1}{k_{base}}$$

In case of other structures, the rule could be applied that the permittivity should be larger than that of 0.01 m or 0.1 m of the base soil:

$\psi > k_s/(0{,}1 \text{ m or } 0{,}01\text{m})$ which is equivalent to $k_g > k_s \cdot t_g/(0{,}1 \text{ m or } 0{,}01\text{m})$

In many cases of 'fine soil protection filters' even this rule is too conservative.

Unfortunately, many authors advocate the 'lazy designers rule': "The geotextile should be more permeable than the base soil". E.g. [Giroud 1988]:

$k_g > i_g \cdot k_s$ where $i_g > 1.0$
$k_g > k_s$ where $i_g < 1.0$

where k_g is the geotextile hydraulic conductivity or permeability, k_s is the soil permeability and i_g is the hydraulic gradient in the soil adjacent to the geotextiel filter.

Much attention must be paid to the determination of the value of ψ (or k_g) in view of two factors which may cause a serious reduction of the permittivity: 'blocking' and 'clogging'.

Blocking is the phenomenon that large grains or particles seal the openings of the geotextile. In that case, the permittivity can decrease dramatically. In case many particles have nearly the same size as the O_{90}, the head difference over the geotextile can increase with a factor 10 to 20 compared to the situation without grains and with the same flow discharge through the geotextile (Figure 7.14).

Clogging is the trapping of (very) fine particles in the openings of the geotextile, leading also to a decrease of permeability. Such decrease can also be caused by chemical processes if the water is contaminated with chemicals, e.g. iron. In contrast to blocking, clogging is a time dependent process. The idea is that it stabilizes at a certain level after a certain time, but not much experimental evidence is available to support this thesis.

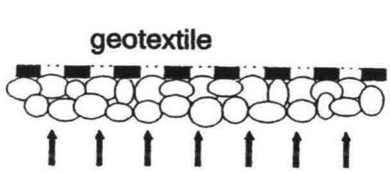


Fig. 7.14 Blocking of geotextile after [Veldhuijzen van Zanten 1986]

Blocking is especially relevant for woven geotextiles, whereas clogging is especially relevant for non-wovens. If a high value of the permittivity is critical for the design, laboratory tests should be performed where geotextile and base soil are tested together. If the water near the filter may be contaminated, tests should be done with this type of water and the test period should be rather long. Otherwise, it is recommended to take into account a reduction of the permittivity with a factor 10 to 30:

$$\psi_{design} \approx 0.03 \text{ to } 0.1\ \psi_{specification} \qquad \text{AND} \qquad k_{g,design} \approx 0.03 \text{ to } 0.1\ k_{g,specification}$$

where 'specification' refers to the values presented by the producer of the geotextile, usually based on tests without soil. In case of non-wovens, a reduction factor smaller than 10 may be acceptable if the porosity, n > 30%. The porosity can be calculated using the following formula:

$$n = (1 - \mu/(\rho \cdot t_g) \cdot 100\%$$

where μ is the geotextile mass per unit areas and ρ is the density of the geotextile material.

7.3.4 Durability

Selection of a geotextile should include provisions to ensure that the geotextile is durable enough to perform its function for the design life of the project, especially if the geotextile is located at a place where replacement is hardly possible. Durability considerations typically address the geotextile's resistance to:

- ultraviolet light
- adverse chemical environments.

7.3.5 Placement of the geotextile and construction of the dike

Selection of the geotextile should include provisions to ensure that the geotextile survives construction, its own placement as well as placement of materials on top of it. Tensile strength, tear resistance and puncture resistance, are mainly determined by the execution method, especially for under water filters. The type of geotextile and the way of placement should be such that all geotextile parts are placed at the right place and effective overlaps are guaranteed.

Placement (partly) under water can be done by connecting pieces of geotextile and other components together to make a mattress. Some types of mattresses can be brought in place by floating them in position and subsequently sinking them down to the sea bed or slope. See the figures 10.5, 10. 6 and 10.7 of this lecture notes. See also figure 7.15. An alternative is winding the mattress around a large floating drum, floating the drum in position and unrolling the mattress off the drum.

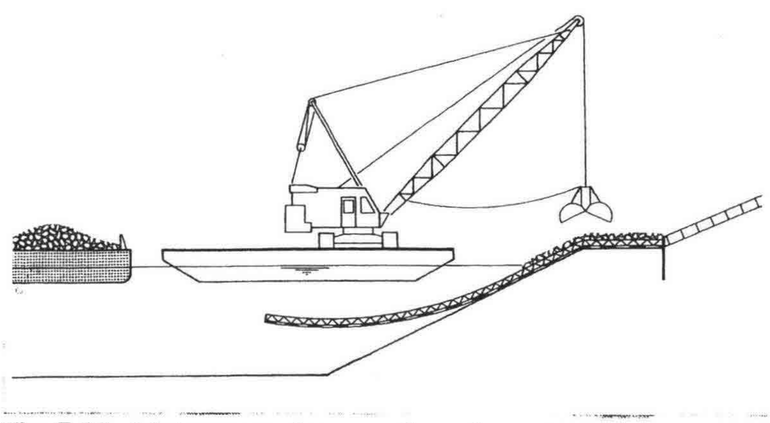


Fig. 7.15 Mattress on slope, partly under water.
Placement at high water; stone dumping partly at low water as in sketch

7.3.6 design methodology

The following design methodology represents a compilation of years of research and experience in geotextile filtration design. The text of this section is provided by Nicolon Industrial Textiles. The approach is a logical progression through nine steps, as follows:

- Step 1. Define the Application Filter Requirements
- Step 2. Define Boundary Conditions
- Step 3. Determine Soil Retention Requirements
- Step 4. Determine Geotextile Permeability Requirements
- Step 5. Determine Anti-clogging Requirements
- Step 6. Determine Survivability Requirements
- Step 7. Determine Durability Requirements
- Step 8. Miscellaneous Design Considerations
- Step 9. Select a Geotextile Filter

Detailed discussion of these steps is provided, followed by design examples for several applications.

STEP 1. DEFINE THE APPLICATION FILTER REQUIREMENTS

Geotextile filters are used between the soil and the drainage medium. Typical drainage media include natural materials such as gravel, and geosynthetic materials such as geonets, cuspated drainage cores, and other pre-fabricated drainage materials.

1.1 <u>Identify the Drainage Material</u>

The drainage medium adjacent to the geotextile must be identified for the following reasons:

- drainage media with a large amount of voids or pore volume could influence the selection of the retention criteria; and
- drainage media with sharp contact points (such as highly-angular gravel) will influence the survivability requirements.

1.2 <u>Define Retention versus Permeability Trade-Off</u>

The drainage medium adjacent to the geotextile often affects the selection of the retention criteria. Due to the conflicting nature of the retention and permeability filter requirements, it is necessary to evaluate whether retention or permeability is the favoured characteristic of the filter. For example, a drainage material that has relatively little void volume (e.g., a geonet or a strip drain) will require a high degree of retention from the filter. Alternatively, where the drainage material void volume is large (e.g., a gravel trench), the permeability and anti-clogging criteria are favoured.

STEP 2. DEFINE BOUNDARY CONDITIONS

2.1 Evaluate Confining Stress

The confining pressure is important for the following reasons:

- For coarse-grained soils, high confining pressures tend to increase the relative density (I_D) Of the soil, hence increasing the soil's resistance to particle movement. This affects the selection of retention criteria.
- For fine-grained soils, high confining pressures decrease the hydraulic conductivity of the soil (k_s), and increase the potential for the soil to extrude through the geotextile filter.
- For all soil conditions, high confining pressures increase the potential for the geotextile and soil mass intruding into the flow paths of the drainage material. This could reduce the flow- capacity of the drainage media, especially with thin geosynthetic drainage cores.

2.2 Define Flow Conditions

Flow conditions can be either steady-state or dynamic. It is important to define the flow conditions because the retention criteria for steady-state flow are different than for dynamic flow conditions. Chart 1 provides the numerical retention criteria for steady-state flow conditions; Chart 2 is for dynamic flow conditions. These charts will be explained in subsequent sections.

Standard dewatering drains, wall drains and leachate collection drains are examples of applications with steady-state flow conditions. Shoreline and coastal embankment protection layers are typical examples of applications where waves and water currents may cause dynamic flow conditions.

STEP 3. DETERMINE SOIL RETENTION REQUIREMENTS

3.1 Define Soil Particle-Size Distribution

The particle-size distribution of the soil to be filtered should be determined in accordance with test method ASTM D 422. The particle-size distribution curve is used to determine parameters that are necessary for selection of numerical retention criteria. Charts 1 and 2 indicate the use of particle-size parameters for this purpose. These charts show that the amount of gravel, sand, silt, and/or clay affects the first quantitative step in selection of the retention criteria. For predominantly coarse-grained soils, the grain-size distribution curve is used to calculate specific parameters, such as C_c, C_u, and C'_u, that govern the retention criteria

3.2 Define Soil Atterberg Limits

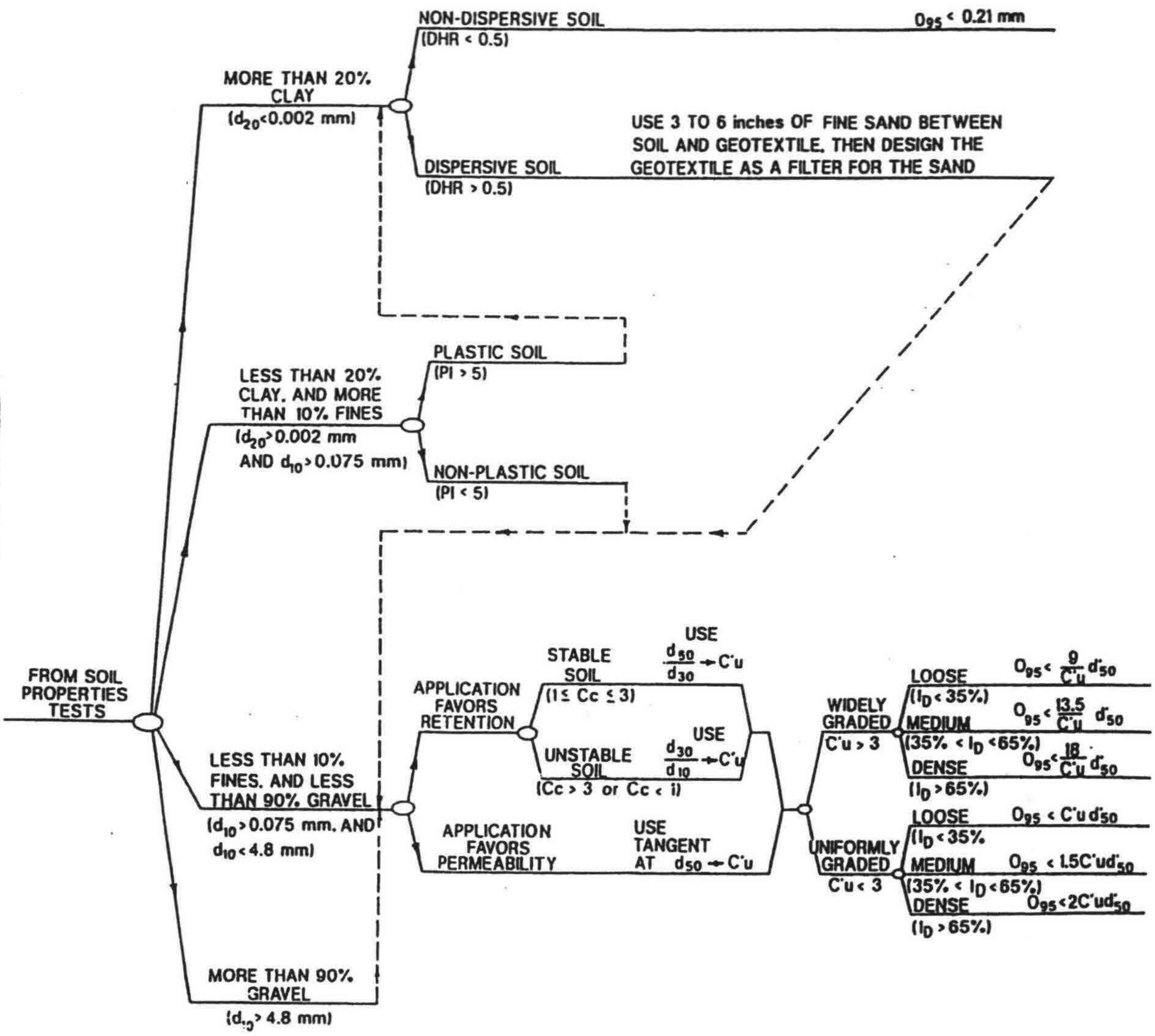
If the soil contains a considerable amount of fine particles, the plasticity index (PI) of the soil should be determined from the Atterberg Limits test method ASTM D 4318. Charts 1 and 2 show how to use the PI value for the purpose of selecting appropriate numerical retention criteria for fine-grained soils.

3.3 Define Soil Dispersion Potential

If the soil is predominantly fine-grained and somewhat plastic, the dispersion potential of the soil should be evaluated using the Double Hydrometer test method ASTM D 4221. Charts 1 and 2 show how to use the double hydrometer ratio (DHR) in selecting appropriate numerical retention criteria.

3.4 Define Soil Density Conditions

If the soil is predominantly granular, and steady-state flow conditions prevail, then the relative density of the soil should be determined in accordance with ASTM test method D 4254. For non-critical applications, the guidelines provided in the following table may be used to estimate the relative density of soil for the given application.



NOTES:

d_x is the particle size of which x percent is smaller

$C'u = \sqrt{\frac{d'_{100}}{d'_0}}$ where: d'_{100} and d'_0 are the extremities of a straight line drawn through the particle-size distribution, as directed above; and d'_{50} is the midpoint of this line.

$Cc = \frac{(d_{30})^2}{d_{60} \times d_{10}}$

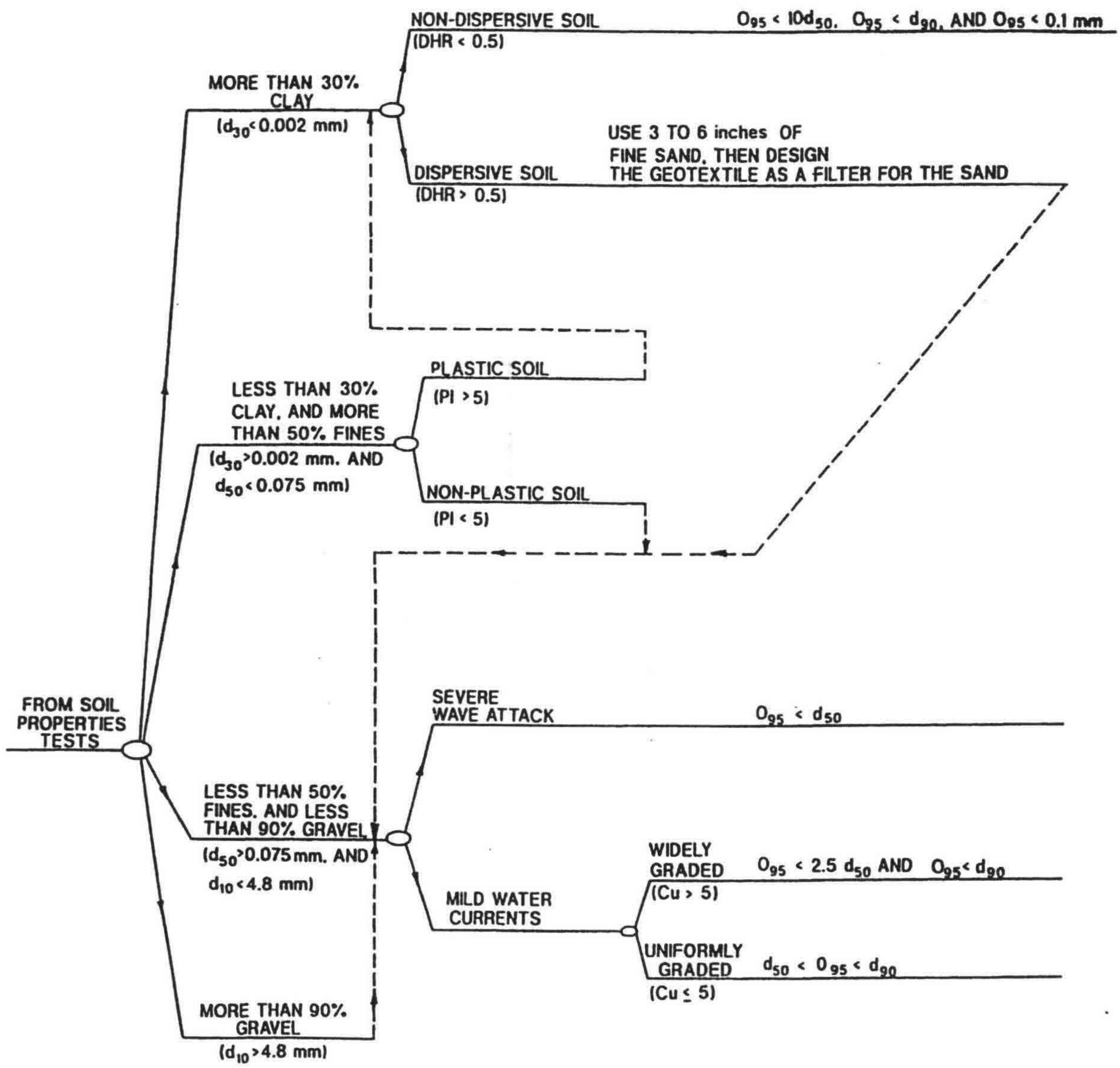
I_D is the relative denisty of the soil

PI is the plasticity index of the soil

DHR is the double-hydrometer ratio of the soil

Portions of this flow chart modified from Giroud (1988)

Figure 7.8: Chart 1, Soil retention criteria for steady-state flow conditions



NOTES:

$$Cu = \frac{d_{60}}{d_{10}}$$

d_x is the soil particle size of which x percent is smaller
PI is the plasticity index of the soil
DHR is the double-hydrometer ratio of the soil
O_{95} is the geotextile opening size
Portions of this flow chart modified from Heerten (1985)

Figure 7.9: Chart 2, soil retention criteria for dynamic flow conditions

Soil condition	Low confining pressures (typ ≤ 50 kPa)	High confining pressures (typ > 50 kPa)
Unconsolidated sedimentary deposits or uncompacted hydraulic fill	$I_D \leq 35$ %	35 % $< I_D <$ 65 %
Consolidated residual deposits or compacted fill	35 % $< I_D <$ 65 %	$I_D >$ 65 %

3.5 Determine the Maximum Allowable Geotextile Open Size (O_{95})

Charts 1 and 2 show that the final step in determining soil retention requirements is to evaluate the maximum allowable opening size (O_{95}) of the geotextile which will provide adequate retention of the soil. The O_{95} value can be determined from Apparent Opening Size test method ASTM D 4751; this value can often be obtained from the geotextile manufacturer's literature as well.

STEP 4. DETERMINE GEOTEXTILE PERMEABILITY REQUIREMENTS

4.1 Define the Soil Hydraulic Conductivity (k_s)

The soil hydraulic conductivity (permeability) should be determined by one of the following methods:

- For critical applications, such as earth dams, the soil permeability should be measured in the laboratory using representative field conditions in accordance with test method ASTM D 5084.
- For many non-critical applications the soil hydraulic conductivity can be estimated from the following figure, using the characteristic particle size, D_{15}, of the soil.

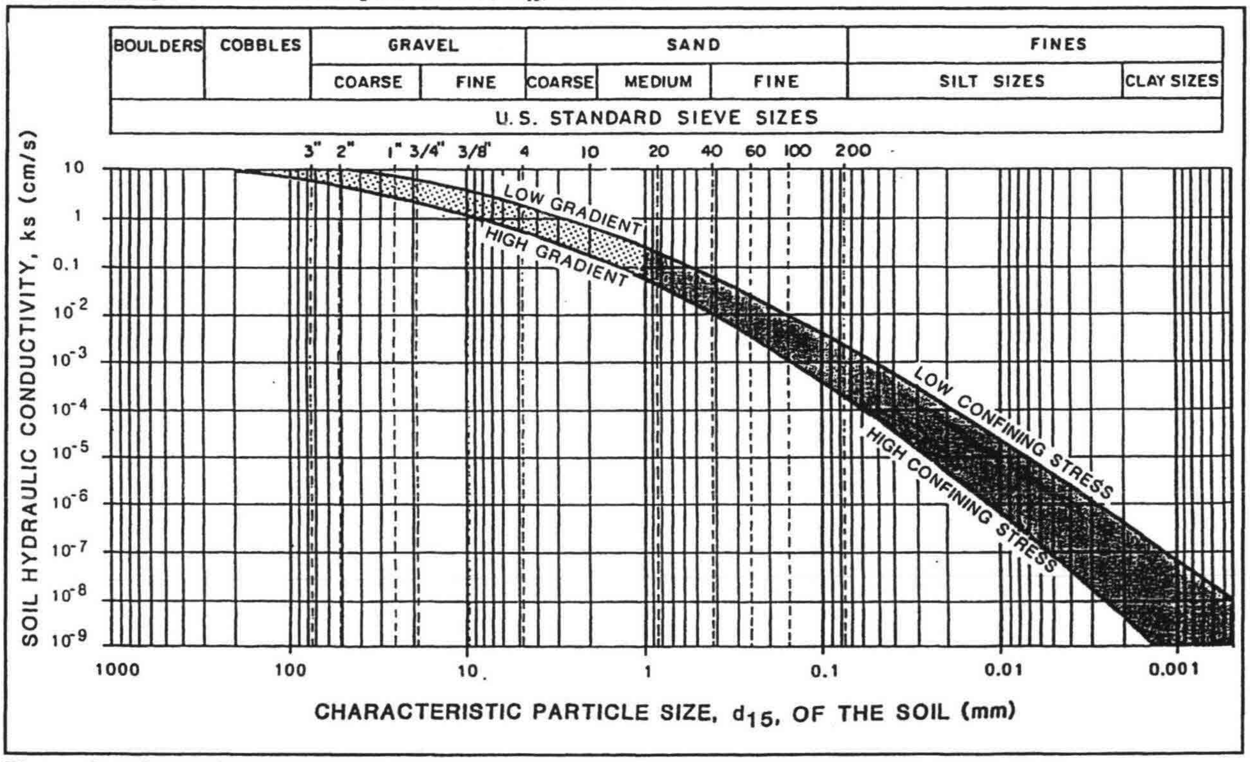


Figure 7.10: Typical hydraulic conductivity values

4.2 Define the Hydraulic Gradient for the Application (i_s)
The hydraulic gradient will vary depending on the application of the filter. Anticipated hydraulic gradients for various applications may be estimated using the following table (Giroud, 1988).

Drainage application	Typical Hydraulic Gradient	
Standard dewatering trench	1.0	
Vertical wall drain	1.5	
Pavement edge drain	1	*
Landfill LCDRS	1.5	
Landfill LCRS	1.5	
Landfill SWCRS	1.5	
Dams	10	*
Inland channel protection	1	*
Shoreline protection	10	*
Liquid impoundments	10	*

* Critical applications may require designing with higher hydraulic gradients than those given.

4.3 Determine the Minimum Allowable Geotextile Permeability (k_g)
After determining the soil hydraulic conductivity and the hydraulic gradient, the following equation can be used to determine the minimum allowable geotextile permeability [Giroud, 1988]:

$$k_g > i_s\, k_s$$

The hydraulic conductivity (permeability) of the geotextile can be calculated from the permittivity test method ASTM D 4491; this value can often be obtained from the manufacturer's literature as well. The geotextile permeability is defined as the product of the permittivity, ψ, and the geotextile thickness, t_g:

$$k_g > \phi\, t_g$$

STEP 5. DETERMINE ANTI-CLOGGING REQUIREMENTS
To minimize the risk of clogging, the following criteria should be met:
- Use the largest opening size (O_{95}) that satisfies the retention criteria.
- For nonwoven geotextiles, use the largest porosity available, but not less than 30 percent.
- For woven geotextiles, use the largest percent open area available, but not less than 4 percent.

STEP 6. DETERMINE SURVIVABILITY REQUIREMENTS
Experience has shown that the type of drainage material placed adjacent to geotextile and the construction technique used for placing these materials can affect the damage to the geotextile. The most common technique for assuring the construction survivability of the geotextile is to specify minimum index strength properties that vary with the severity of the installation. It is noted that some engineering judgement must be used in defining the severity of the installation.

The following figure provides guidance in selecting required geotextile strength properties to ensure survivability for various degrees of installation conditions.

STEP 7. DETERMINE DURABILITY REQUIREMENTS
If the installation or application of the geotextile filter will result in extended exposure to sunlight, anti-oxidants (such as carbon black or titanium dioxide) are recommended for added resistance to degradation due to ultraviolet light.

If the geotextile application will result in exposure to adverse chemicals (such as in waste-containment landfill applications), the chemical compatibility of the geotextile should be evaluated.

Installation conditions	Contact stresses	GRAB STRENGTH (lbs)	ELONGATION (%)	SEWN SEAM STRENGTH (lbs)	PUNCTURE STRENGTH (lbs)	BURST STRENGTH (psi)	TRAPEZOID TEAR (lbs)
MODERATE INSTALLATION CONDITIONS (TYPICAL DRAINAGE APPLICATIONS)	HIGH CONTACT STRESSES (ANGULAR DRAINAGE MEDIA) (HEAVY COMPACTATION) OR (HEAVY CONFINING STRESS)	180	N/A	160	80	290	50
	LOW CONTACT STRESSES (ROUNDED DRAINAGE MEDIA) (LIGHT COMPACTION) AND (LIGHT CONFINING STRESS)	80	N/A	70	25	130	25
SEVERE INSTALLATION CONDITIONS (SHORELINE PROTECTION AND ARMOURED SYSTEMS APPLICATIONS)	HIGH CONTACT STRESSES (DIRECT STONE PLACEMENT) (DROP HEIGHT > 3 ft)	200	15	180	80	320	50
	LOW CONTACT STRESSES (SAND OR GEOTEXTILE CUSHION) (DROP HEIGHT < 3 ft)	90	15	80	40	140	30

Figure 7.11: Survivability strength requirements (after FHWAm, 1985)

STEP 8. MISCELLANEOUS DESIGN CONSIDERATIONS

Other considerations which should be addressed in design of geotextile filters are:

- the geotextile structure;
- intrusion of the geotextile into the drainage layer;
- extrusion of fine-grained soil through the geotextile when subjected to high confining pressures;
- abrasion of the geotextile due to dynamic action;
- intimate contact of the soil and geotextile;
- biological and bio-chemical clogging factors; and
- safety factors.

STEP 9. SELECT A GEOTEXTILE FILTER
The design considerations presented in Steps 3 through 8 provide a logical methodology for obtaining the required properties of the geotextile filter. To summarize, these properties are as follows:

- Step 3 gives the maximum allowable Apparent Opening Sieve (O_{95}) of the geotextile;
- Step 4 gives the minimum allowable permeability, k_g or permittivity (of the geotextile;
- Step 5 gives the minimum allowable porosity (n), or percent open area (POA) of the geotextile;
- Step 6 gives the minimum allowable physical strength requirements of the geotextile;
- Step 7 provides guidance to help ensure adequate durability of the geotextile; and
- Step 8 involves miscellaneous considerations which are specific to certain applications and should qualitatively be integrated into the decision process.

The final step is to select a geotextile filter using the properties obtained from these steps. The properties should be compared to the geotextile properties provided in geotextile manufacturer's product literature.

TESTING FOR CRITICAL APPLICATIONS
The design methodology presented thus far is intended to guide the designer through a series of logical steps for the selection of geotextile filters for non-critical applications. These guidelines were developed from a combination of theory, empirical data, and experience pertaining to geotextile filters.

Critical applications, where loss of life or significant loss of property may result from failure of the filter, may require laboratory and/or field tests. Results of performance tests provide data regarding the behaviour of the candidate geotextile filter when subjected to the actual (or closely simulated) boundary and soil conditions anticipated from the design application.

Performance testing may take many forms, depending on the application and the consequences of failure of the filter. The following list references commonly performed test methods which may be used to provide additional information regarding filter behaviour.

7.4 Lessons about filters

Filt 1 Strongly graded granular filter is attractive AND dangerous:

- Variation in composition
- Internal instability if gap-graded
- De-composition with placing under water

Filt 2 Filter with fines on top of coarse need to be geometrically tight

Filt 3 Transitions often give problems (section 9.2)

7.5 Additional references (Chapter 7)

Bakker, K.J., Klein Breteler, M. & Den Adel, H. 1990.
New criteria for granular filters and geotextile filters under revetments.
22 nd Int. Conf. Coastal Eng. Delft, The Netherlands, Ch. 114, 14 pp

De Groot, M.B., Bakker, K.J. & Verheij, H.J. 1993.
Design of geometrically open filters in hydraulic structures
Filters in Geotechnical and Hydraulic Engineering, Brauns, Heibaum & Schuler (eds).
Balkema (Swets & Zeitlinger), Lisse, the Netherlands. pp 143 – 154.

Pilarczyk, K.W. (ed) 1998.
Dikes and Revetments. Design, Maintenance and Safety Assessment.
Balkema (Swets & Zeitlinger), Lisse, the Netherlands. ca 580 pp

Veldhuijzen van Zanten, R. (ed) 1986
Geotextiles and geomembranes in civil engineering.
Balkema (Swets & Zeitlinger), Lisse, the Netherlands. ca 670 pp

8 Geotechnical aspects

8.1 General

8.1.1 Character of dikes

The character of dikes is described in chapter 3. The following additional remarks are relevant for the geotechnical aspects:

- Dikes are often located in flat areas with much soft soils (sand, clay peat). For economical reasons most of the material used for the construction of dikes consists of locally available material, thus soft material.
- The soft material of the sub soil and the soft material of the dike tend to reduce in volume during construction and during the years after construction. The resulting crest settlement may continue over a very long period (centuries), although the rate of settlement reduces continuously if no further load is placed on top. Settlement and sea level rise may force the dike owner to raise the crest level from time to time and widen its base to guarantee stability. In the Netherlands this is done roughly every 50 years. As many dikes are more than 500 years old, they may have been reconstructed already more than 10 times. After any crest level rising, the rate of settlement increases again.

8.1.2 Character of soil

Soil is used for dikes in two different ways:

- Soil as foundation material: the soil that will not be removed. The dike will be constructed on top of this soil
- Soil as construction material: (soft?) soil that will be removed from a place (in the neighbourhood), treated (if necessary), transported and brought in place.

Soil has a strange character. Depending on the way it is used the following features are important:

- The properties of the sub soil may vary from place to place. Soil investigations are an essential start of any design. Geological information will be very helpful in the interpretation. Geological knowledge will also be helpful to decide about the intensity of the soil investigations. Do we need 1 boring or cone penetretation test every 50 m or every 200 m? For example: most of the sub soil may consist of nearly impermeable clay. At some locations, however, permeable sand may be available over a limited distance, due to the geological history. This may have serious consequences for the design. Geologists may be able to predict whether sand may be present and over which distance.
- Soil properties change by (un)loading. Loading and unloading may have occurred in the past (sedimentation and erosion or ice caps) and may have changed the structure of the soil (density, orientation of the grains, horizontal stress): a nearly irreversible change. Increased loading does not only change the structure of the soil, it also changes the reversible properties of the soil: shear strength, stiffness etc are all functions of the stress level. Thus, loading of the sub-soil by the construction of the dike will influence the properties of the sub-soil material. Digging out and handling of soil for construction of the dike causes changes in the properties of the soil in question (other density, different orientation of the grains, breaking of the cementation bonds if any were present). The type of handling, in particular any densification after dumping may have a large influence on the properties of the applied soil. Several geotechnical tests can be used to predict and quantify these influences.
- Soil properties may change in time due to consolidation, creep and/or cementation. Most gravels and sands are not very sensitive to such changes, unless much calcium is present

in the water which may cause cementation. Clay and peat, however, are sensitive to consolidation and creep. Fast loading of clay and peat can be dangerous!

8.1.3 Numerical models

Many numerical models are used in geotechnical engineering. There exists not one best model. Soil behaviour is so complicated and the loading situations are so variable, that a universal model is practically impossible to make. The more complicated the models are, the less clear is how the model works ('black box') and how reliable the answer is.

The choice of the model depends in the first place on the problem considered:
- Stability?
- Settlement?
- Other deformation?
- Pore flow?
- Permanent or temporary load?
- Slow or fast loading?

More about mathematical models for stability and (shear-)deformation in section 8.5.7.

8.1.4 Relevant geotechnical aspects

The potentially relevant failure modes are discussed in section 4.2.4. The following of these modes can be considered to be geotechnical failure modes and will be discussed here (see also [Pilarczyk 1998] chapter 4, section 5):

3 micro-instability of inner slope and sliding-instability of surface layers (section 8.3)
4 sliding or macro-instability of inner or outer slope (section 8.5)
5 internal erosion, in particular piping (section 8.4; internal erosion by filter instability is discussed in chapter 7)
6 flow slide of sub-soil or fore-shore and other types of liquefaction (section 8.7)
10 settlement, followed by failure modes overflow/overtopping, erosion of crest, erosion of inner slope and/or sliding of inner surface layer or slope protection (section 8.6).

Many of these failure modes are strongly influenced by pore pressures and pore flow. The modes 3, 4 an 6 depend on the shear strength of the soil and the shear strength can be strongly reduced by any excess pore pressures. Therefore the pore flow and pore pressures underneath and in the dike will be discussed first.

8.2 Pore flow, phreatic surface and pore pressures

8.2.1 Basic examples of stationary flow

The subsoil underneath a dike is never completely impermeable. Neither is the dike, as it is made of soil as well. Although dikes aim to keep the water away, always a small quantity of water will flow through the dike and its subsoil. This small quantity has a large influence on many failure modes. Two basic examples are shown here.

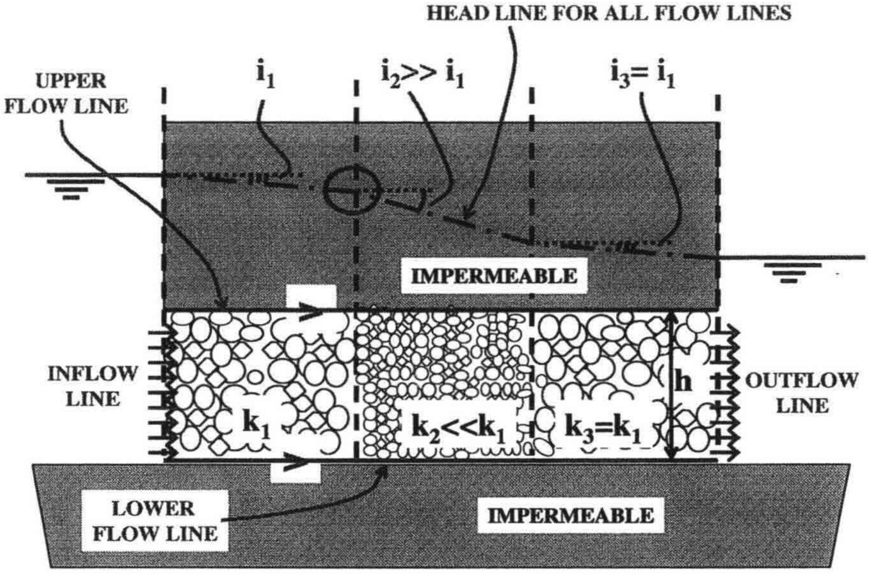


Fig. 8.1A Flow through a tunnel with different types of soil

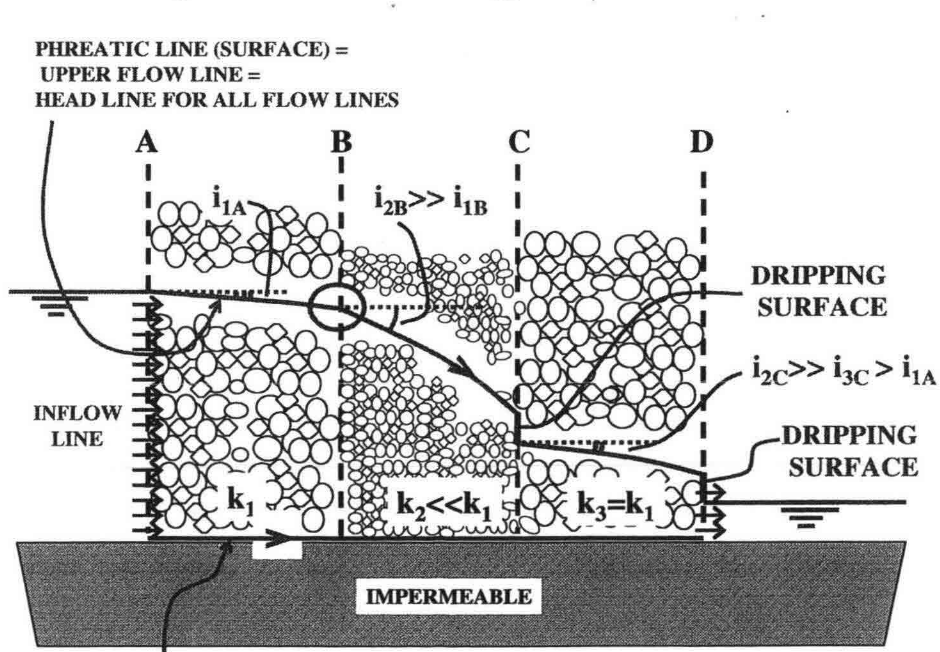


Fig. 8.1B Flow through different types of soil with phreatic surface.

In case of a subsoil consisting of different types of permeable soil and a clayey dike, the flow through the subsoil is often similar to the flow through a tunnel filled with different soil types. See Figure 8.1A. The flow can be characterised by the inflow line, the outflow line, the upper flow line, the lower flow line and the headline. In fact, all lines in the sketch represent surfaces. 'Inflow ***surface***' would be a better expression than 'Inflow ***line***'. However, 'line' is more commonly used and will be used here as well.

The discharge **Q** (in m^2/s) of water through the tunnel is the same for each section. The relationship with gradients $\mathbf{i_1}$, $\mathbf{i_2}$ & $\mathbf{i_3}$ and the permeabilities $\mathbf{k_1}$ $\mathbf{k_2}$ a& $\mathbf{k_3}$ follows from Darcy and the given tunnel height **h**:

$$Q = h \cdot i_1 \cdot k_1 = h \cdot i_2 \cdot k_2 = h \cdot i_3 \cdot k_3$$

The gradient is constant in each of the three sections. The values of the gradients are inversily proportional to the permeabilities.

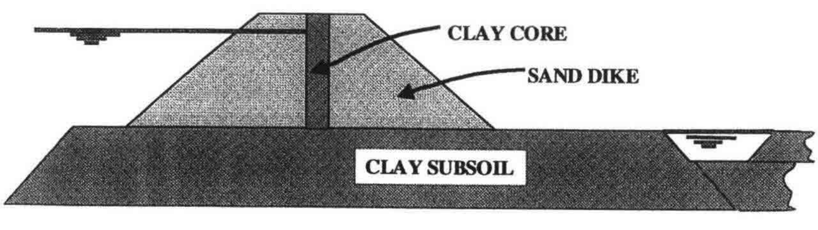
In case of a clayey subsoil and a dike with different types of permeable soil, the flow through the dike is often similar to the flow through a flume filled with different soil types. See Figure 8.1B. This flow is more complicated, because a phreatic surface (or 'line')is present:

- The upper flow line and the head line coincide both with the phreatic line
- This line is curved in each section because the flow height h reduces in the flow direction
- The line not only shows a change in direction at each boundary between the different soil types, but also a jump where the water flows from a low-permeability soil into a soil with higher permeability or into the space without soil. The jump can be called a 'dripping surface' as drips of water appear out of the low-permeability soil and fall down on the lower phreatic surface in the more permeable soil. This (vertical) part of the phreatic line is no flow line, because the water flows through it.

8.2.2 Permeable and 'impermeable' elements

The sub-soil usually consists of several types of soil. The permeability may vary significantly. Most important is the flow through the soil layers that are most permeable. The other, less permeable, layers can often be considered to be 'impermeable' for this flow. The permeability of the soil used for dike construction is often different from that of the most permeable sub-soil layer. The permeability may also vary within the dike. This especially true if special 'impermeable' elements are applied in the dike.

In addition to the dike elements sketched in Figures 3.1 and 3.2, and discussed in section 3.1, attention is needed for any impermeable or nearly impermeable elements, such as a vertical impermeable core through the crest or an impermeable revetment connected to and impermeable row of sheet piles, which may be connected to an 'impermeable' horizontal (clay) layer. See Figure 8.1C.



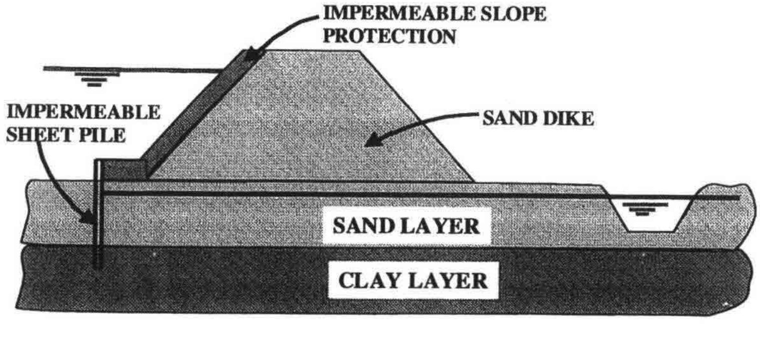


Fig. 8.1C Dike elements relevant for pore flow and pore pressures

8.2.3 Estimate of permeability

The sub-soil usually consists of several types of soil. The permeability may vary significantly. Typical permeabilities are:

gravel	$k \approx 10^{-3}$ m/s
medium sand	$k \approx 10^{-4}$ m/s
fine sand	$k \approx 10^{-5}$ m/s
sandy silt	$k \approx 10^{-6}$ m/s
clay	$k \approx 10^{-8}$ m/s

The permeability of gravel, sand and clay can be estimated from the following formula. The accuracy is such that the calculated value may not differ more than a factor 3 from the real value:

$k \approx \frac{g}{\nu} \cdot \kappa$ where κ is the 'intrinsic permeability' and ν is the kinematic viscosity

$\nu \approx 1.2 \cdot 10^{-6}$ m²/s and $\kappa = [c_\kappa - 0.22 \ln U] \cdot 10^{-3} \cdot D_{10}^2 \approx 0{,}001 \cdot D_{10}^2$

where U is the uniformity index defined by $U = D_{60}/D_{10}$ and $c_\kappa \approx 1.2$ for densely packed sand and $c_\kappa \approx 1.8$ for loosely packed sand. For $D_{10} = 100$ µm, this results in $\kappa \approx 1.0 \cdot 10^{-11}$ m² and $k \approx 10^{-4}$ m/s.

The permeability of clay above the water table increases significantly (factor 1000 or 10 000) if it not becoming wet very regularly due to the development of cracks (fissures). The upper part of a clay layer in the slope protection of a dike cannot function as an impermeable layer, where it lies above the daily high water level.

8.2.4 Prediction of stationary flow

In many cases the flow through the dike may be considered to be stationary. The following procedure can be followed:

1. Determine the hydraulic boundary conditions. Most relevant is usually the situation with extreme high water table outside and a low water table inside (that may be the water level of the nearest polder ditch).
2. Determine the permeabilities of the different soil layers and dike elements.
3. Calculate flow velocities and pore pressures. This requires often a repetition of calculations, starting with a first estimate of the location of the phreatic line (surface) and correcting this in the subsequent calculations.
4. Determine the excess pore pressures along the potential rupture surfaces and other parameters relevant for the analysis of the different failure modes.

This procedure can be followed for a prediction with a sophisticated pore flow calculation model. However, it can also be applied for a first estimate. It is recommended to make such an estimate first, in order to recognise the problems and potential pitt falls. The procedure is worked out for a first estimate below. See Figures 8.2, 8.3 and 8.4. The estimate is valid for a vertical cross-section of dike and sub-soil. 'Lines' in this cross-section are surfaces in 3 dimensions:

1. Sketch the water tables at both sides of the dike.
2. Estimate the permeabilities and determine the part of the soil through which most of the water will flow, indicated here as ***'flow layer'***. If the most permeable soil material connects the water area's at both sides of the dike, then the flow layer will consists of the most permeable material (see Figures 8.2, 8.3 and 8.4A). In other cases the water has to flow subsequently through different soil materials. See, for example Fig. 8.4B,

where the water flows first through gravel, then through sand and finally through gravel.

3A Connect the intersection of the water tables with the soil slopes as a ('provisional') first estimate of the phreatic line.

3B Indicate the inflow line (surface in 3-D) for the flow layer (the part of the water/soil interface where the water flows into the soil) and the outflow line (the part of the water/soil interface where the water flows out of the soil)

3C Indicate the upper and the lower flow lines of the flow layer. A flow line gives the path of a water particle that enters the soil at any point of the inflow surface. The upper flow line coincides with the upper boundary of the flow layer or with the phreatic surface, whatever is the lowest. The lower flow line coincides with the lower boundary of the flow layer. If the flow layer is very thick and extends to a great depth, the following flow line may be taken as the lower flow line: the line starting at the inflow surface in a point at a distance equal to half the dike width upstream from the seaward dike toe, running in a gentle curved line to the outflow surface at a corresponding distance from the inner dike toe and reaching its greatest depth underneath the middle of the dike at a depth equal to the dike width (or half the dike width) below the dike base.

3D Correct the provisional phreatic surface:

- If the water flows subsequently through different soil materials (e.g. Fig. 8.4A), the phreatic line should be made (nearly) horizontal in the most permeable material
- If the phreatic line runs through the flow layer and if the height of the flow layer underneath the phreatic line varies, the slope (= gradient) of the phreatic line should be made roughly inversily proportional to the height of the flow layer underneath the phreatic line. Thus, this slope is steeper if this height is smaller. Example: the curved phreatic line in the Figures 8.3 and 8.4.
- If the provisional phreatic line crosses the inner slope, such that it runs through the air: it should be lowered such that it coincides with the inner slope. This part of the slope becomes also part of the outflow line.
- If the water flows first through a relatively impermeable soil material and then through a more permeable material and if the provisional phreatic line crosses the intersection between these two materials, then the phreatic line may partly follow this intersection, even if the intersection is vertical or if the line comes upside down. See Figure 8.4A. Water flows (drips) through this part of the intersection.

3E Devide the flow layer in two sub-layers, if the flow layer is relatively thick and/or if it has a very irregular form. The devision should be done with a 'middle' flow line in such a way that the discharge through the upper sub-layer is roughly equal to the discharge through the lower sub-layer. This can be realised by deviding the height of the flow layer everywhere in h_{upper} and h_{lower} such that

$$h_{upper}/h_{lower} = (l_{middle} + l_{lower})/\ (l_{lower} + l_{middle})$$

where l_{lower} is the length of the lower flow line, l_{middle} is the length of the middle flow line and l_{upper} is the length of the upper flow line. If usefule, the sub-layers can each be devided in two sub-sub-layers.

3F Sketch the pressurehead line along each flow line. The pressure head line for the upper flow line coincides with the phreatic line, of present. For other flow lines the following can be stated. The pressure head at the inflow point is equal to the high water level; the pressure head at the outflow point equal to the level of the free water table at the inner side of the dike. A straight line between these points can be taken as a first estimate of the pressure head line. Corrections are needed where the distance between two flow lines perpendicular to the flow direction, varies. The slope of the pressure head line should be inversily proportional to this distance.

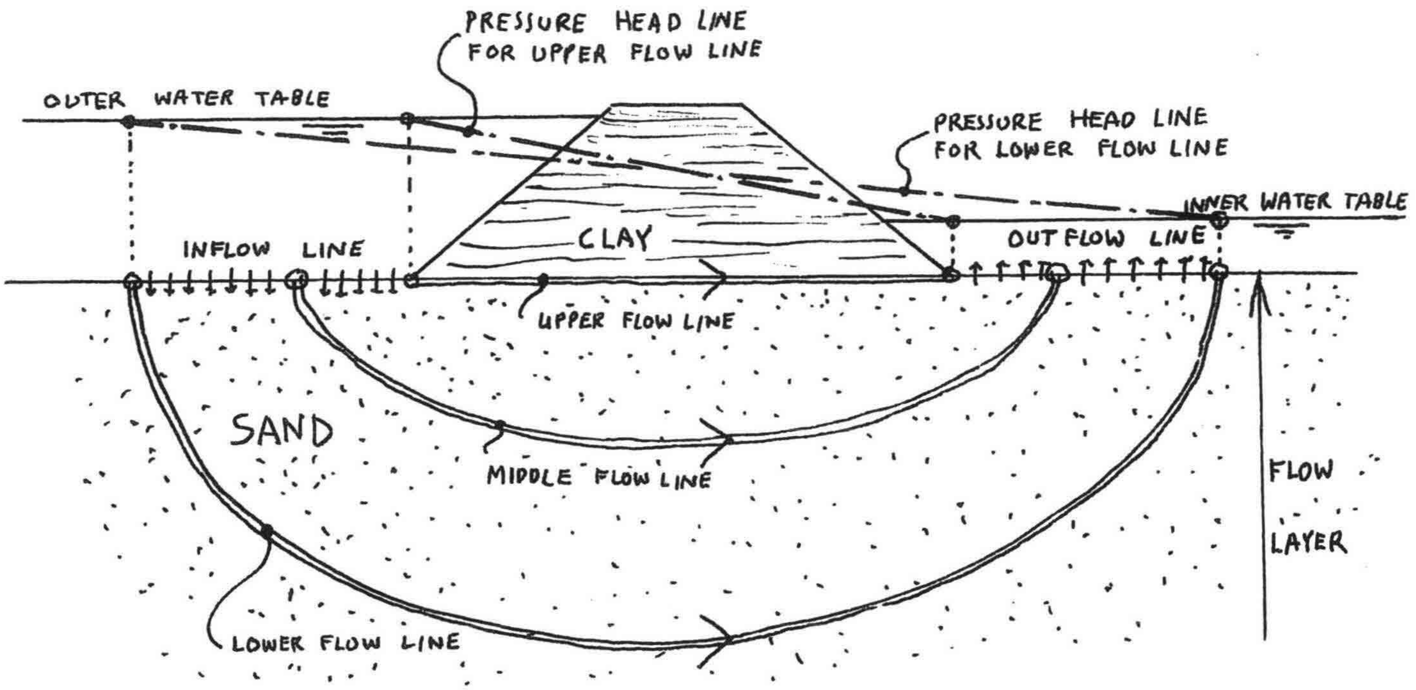


Fig. 8.2 Illustration of procedure for first estimate of pore pressures for clay-dike on top of homogeneous sandy sub-soil

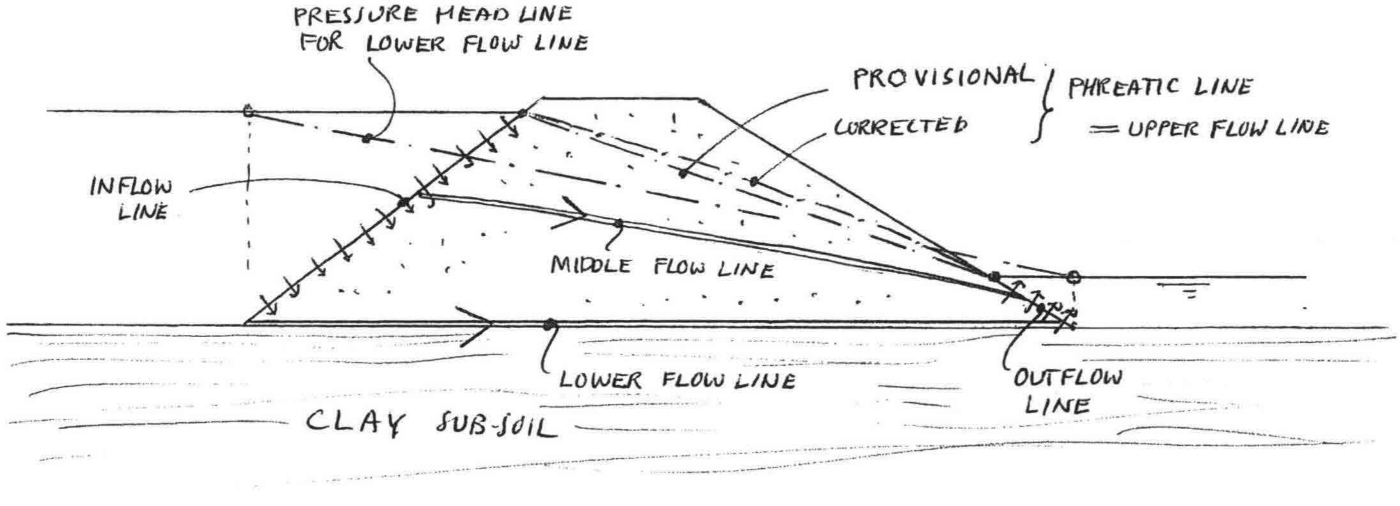


Fig. 8.3 Illustration of procedure for first estimate of pore pressures for sand-dike on top of clay sub-soil

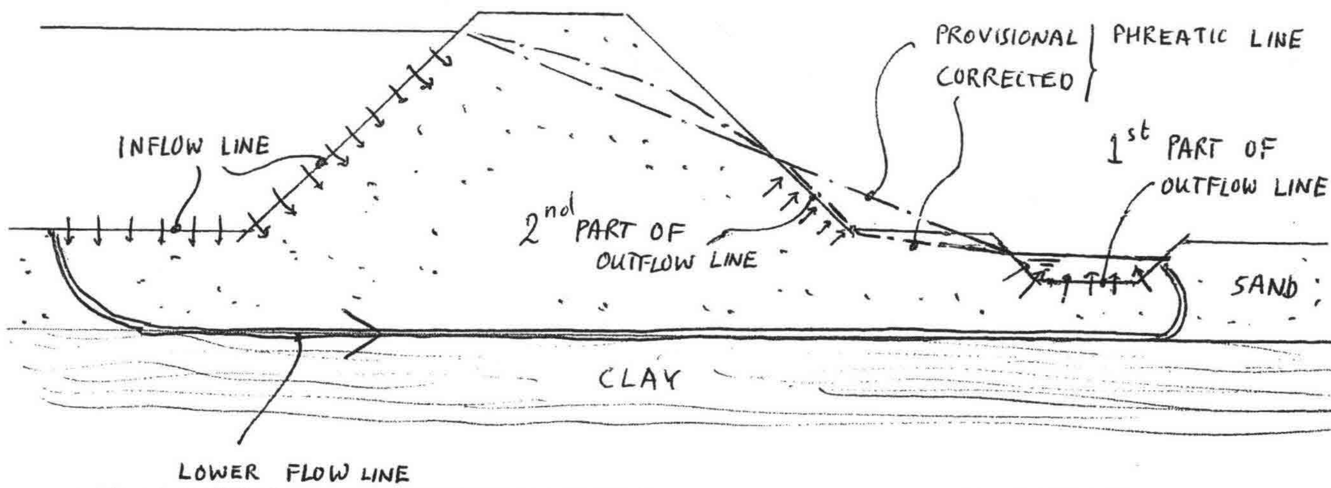


Fig. 8.4A Correction of phreatic line 'in the air'. Where corrected line runs along the slope it coincides with 2nd part of outflow line.

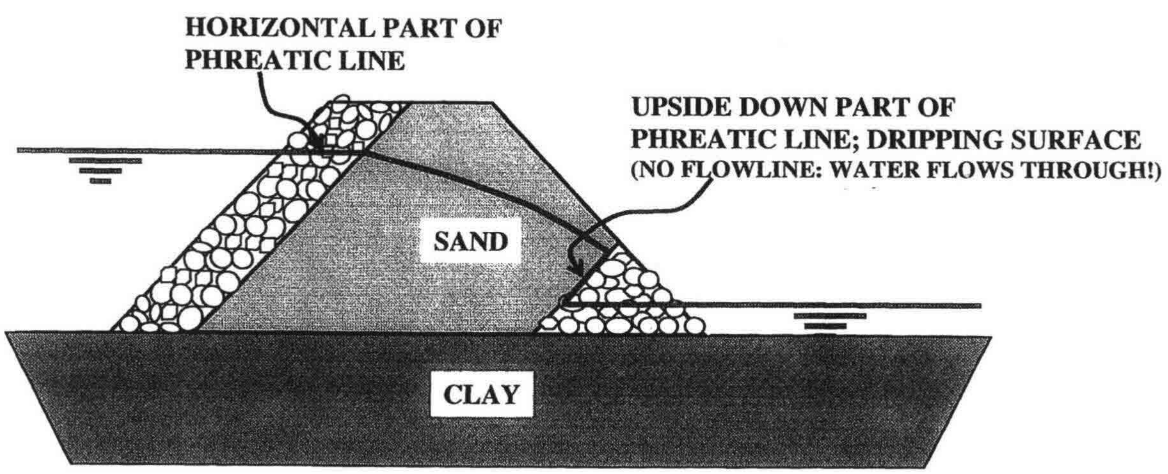


Fig. 8.4B Correction of phreatic line if water flows subsequently through different soil materials

8.3 Micro-instability and sliding-instability of surface layers

8.3.1 General

The term 'Micro-instability' in general is used for the instability of individual soil particles. This instability is usually caused by erosion (scour). The following types may be relevant for dikes:

- erosion of fine soil particles at the interface with coarse particles or a geotextile: 'filter interface instability' (sections 7.2.2 and 7.3.2)
- erosion of fine soil particles out of a skeleton of coarse particles: 'filter internal instability' (section 7.2.3)
- erosion of particles out of a body of sand or silt just underneath a relatively stiff layer or structure such that small canals appear: 'piping' (section 8.4)
- erosion of particles on the inner slope of the dike under the influence of outflowing pore water: 'micro-instability' in a narrower sence.

Last type of micro-instability is discussed here. The analysis is just equal to the analysis of the sliding instability of thin surface layers at the iner slope of a dike. 'Thin' means here 'thin with respect to its length', where the length should be taken parallel to the slope and such that the conditions (load by the out-flowing water and soil properties) are constant. Then, the approximation of an infinite long, straight, homogeneous slope is justified, where each small part of the thin layer must be in equilibrium at itself.

The equilibrium can be found with a Bishop analysis, where the circle has an infinitely large radius and intersects the surface over a very small depth. It is more simple, however, to apply an equation or graph.

Two situations deserve further examination. See Fig. 8.5A and 8.5B.

8.3.2 Groundwater flowing out of an under water slope

This situation is sketched in Figures 8.3 and 8.5A. Most dangerous is the situation if the soil along the slope has no cohesion (gravel, sand or silt). Then the following criterium for stability (without safety factor) can be applied:

$$\tan\varphi \geq \frac{(\rho_s - \rho_w)\sin\alpha}{(\rho_s - \rho_w)\cos\alpha - \rho_w \cdot i}$$

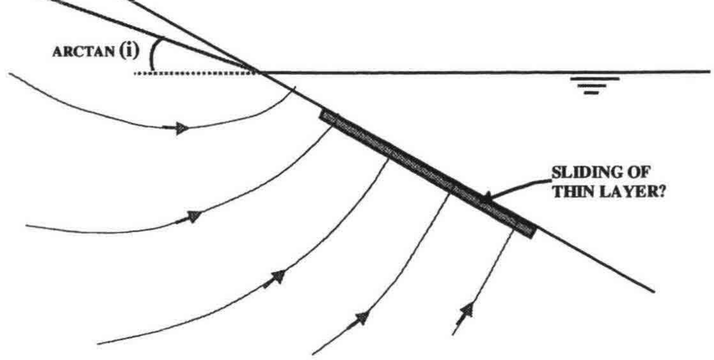


Fig.8.5A Micro-instability at inner slope under water

See [Pilarczyk 1998] chapter 5, section 3.3 and realise that the pressurehead along the slope is constant, thus the water flows out perpendicular to the slope and $\theta = \alpha - 90°$. The pressurehead gradient is also perpendicular to the slope and is indicated with 'i'.

8.3.3 Groundwater flowing out of an above water slope

This situation is sketched in Figures 8.4 and 8.5B. This occurs mainly on the inner slopes. Again most dangerous is the situation with cohesionless soil. Then the following criterium for stability (without safety factor) can be applied:

$$\tan\varphi \geq \frac{\rho_s \sin\alpha}{(\rho_s - \rho_w)\cos\alpha - \rho_w \sin\alpha \cdot \tan(\alpha - \theta)}$$

See [Pilarczyk 1998] chapter 5, section 3.3 and realise that the phreatic surface runs along the slope, yielding: $i = \sin\alpha/\cos(\alpha-\theta)$. In most cases $0 \leq \theta \leq \alpha$. Then, the most dangerous is the case where the groundwater flows out horizontally: $\tan\theta = 0$ and $i = \tan\alpha$, yielding for sand with $\rho_s \approx 1800$ kg/m^3 and $\varphi \approx 30°$ a critical slope angle of 1 : 4.

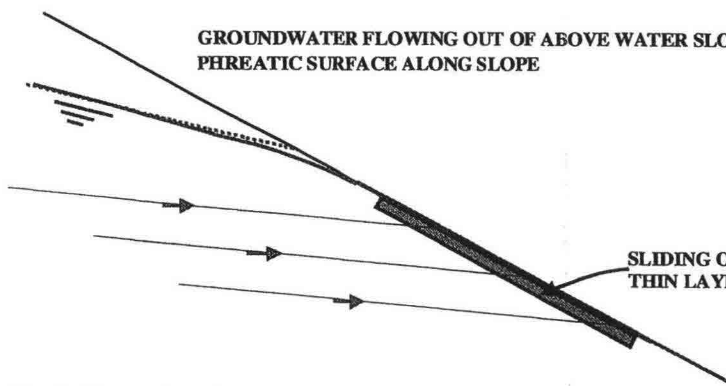


Fig.8.5B Micro-instability at inner slope above water

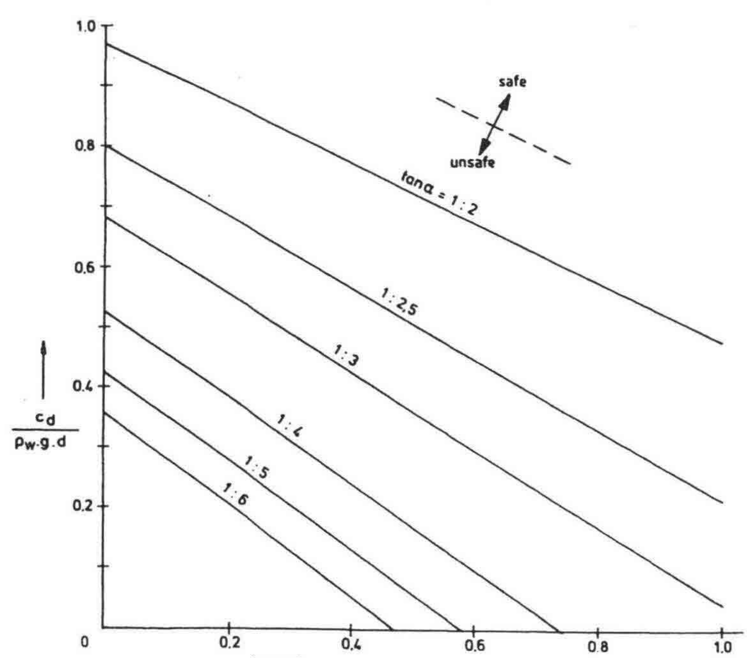


Fig. 8.6 Equilibrium conditions for groundwater flowing horizontally out of an above water slope. Safety factor 1.2 applied. $\rho_s/\rho_w = 1.8$

In many cases, however, inner slopes of dikes are covered with a cohesive soil layer. Then, the equilibrium depends on the thickness d of the potentially sliding layer and the graph of Figure 8.6 can be applied. The thicker the layer, the lower the safety. Realise, however, that the assumption of 'thin' (or 'infinite' slope length) may not be valid any more. The situation is more favourable, if the layer is not 'thin', and can better be analysed as a case of macro-instability (section 8.5), e.g. with a Bishop analysis.

8.4 Piping and sand carrying wells or 'sand boils'

8.4.1 General

Piping under dikes occurs as a result of the entrainment of soil particles by the erosive action of seepage flow. The piping phenomenon is preceded by the formation of boils discharging water in which sand is carried along. Such boils, which manifest themselves especially at periods of high water level retained by the dike, are frequently observed, not only along river dikes, but also along sea-dikes.

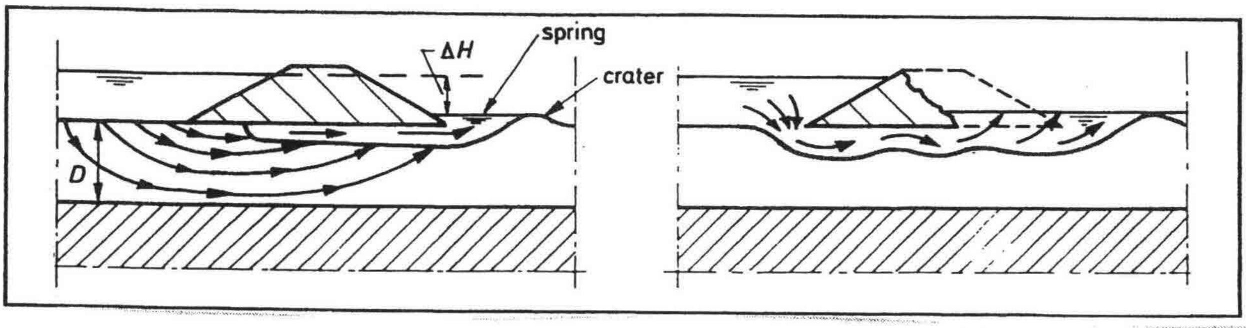


Fig.8.7 Piping in sand layer underneath clay-dike and sand-carrying boils

8.4.2 Conditions for piping

Three conditions must be fulfilled to get dangerous piping:

(A) Condition for a sand boil at the outflow point: the local flow gradient large enough for uplift of soil, thus: $i_{upward} \geq \dfrac{\rho_s - \rho_w}{\rho_w}$ See Figure 8.8A

(B) Condition for pipe in sand: there must be a 'roof' of clay, concrete or otherwise. See Figure 8.8B.

(C) The groundwater flow discharge, q, must be strong enough for continuing erosion: $q \geq q_{\text{erosion for all pipe lengths}}$. See Figure 8.8C.

The values of q and $q_{\text{erosion for all pipe lengths}}$ are mainly influenced as follows:

q larger if:

- Water level difference over dike, ΔH, larger
- value of $(L - l_{pipe})$ smaller
- permeability of sand, k, larger (i.e. D_{10} larger)
- sand layer thickness, D, larger

$q_{\text{erosion for all pipe lengths}}$ smaller if:

- Largest 30% of sand grains, D_{70}, smaller
- value of l_{pipe} smaller

Consequently, condition (C) can be rewritten as:

$$\frac{\Delta H}{L} > \left[\frac{\Delta H}{L}\right]_{critical} \quad \text{where} \left[\frac{\Delta H}{L}\right]_{critical} = \frac{1}{c} \quad \text{and} \quad c = c(D, D_{10}, D_{70})$$

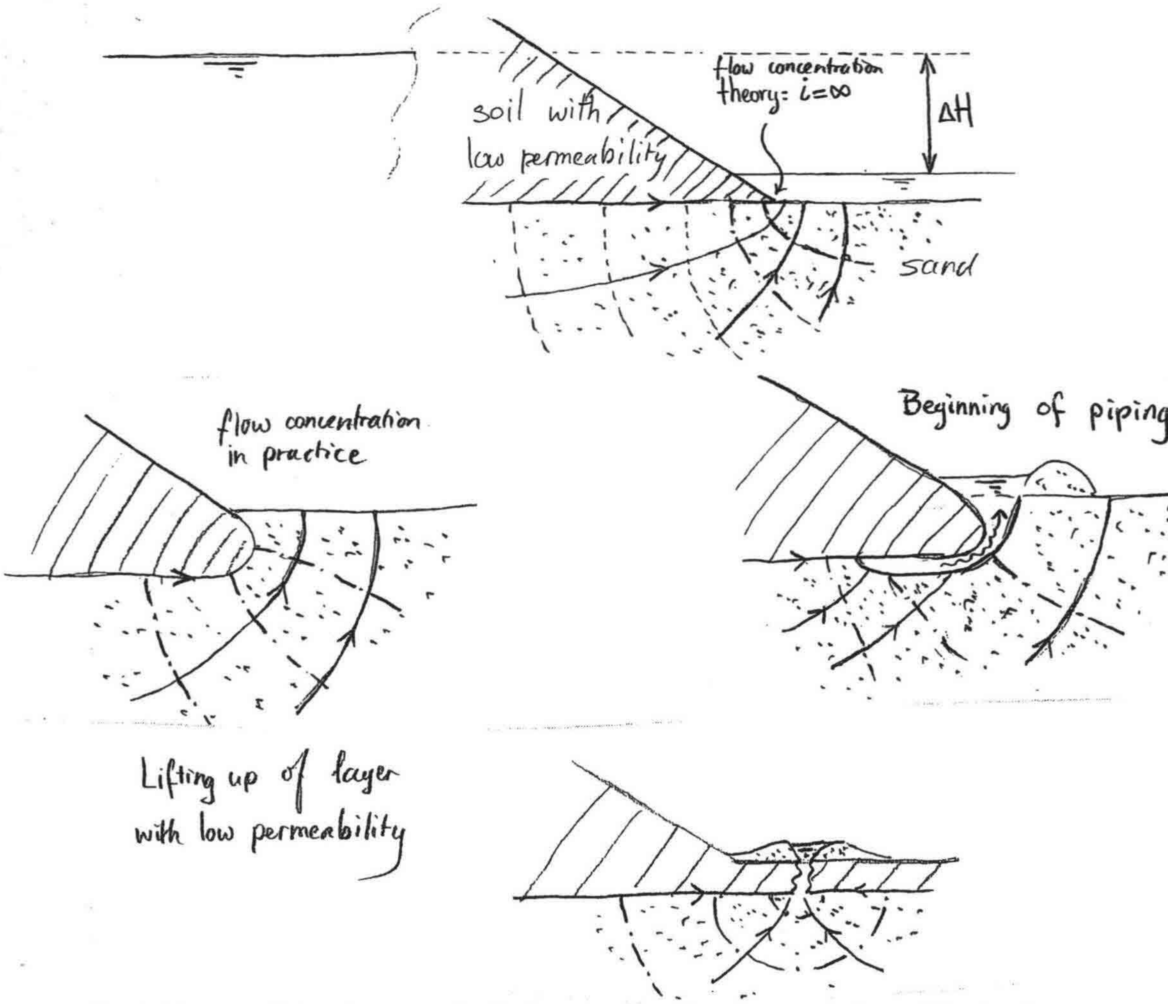


Fig.8.8A Condition for a sandboil: flow gradient large enough for uplift of soil

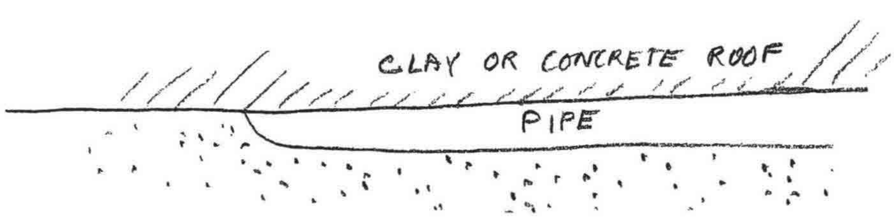


Fig.8.8B Condition of a 'roof' present above pipe in sand

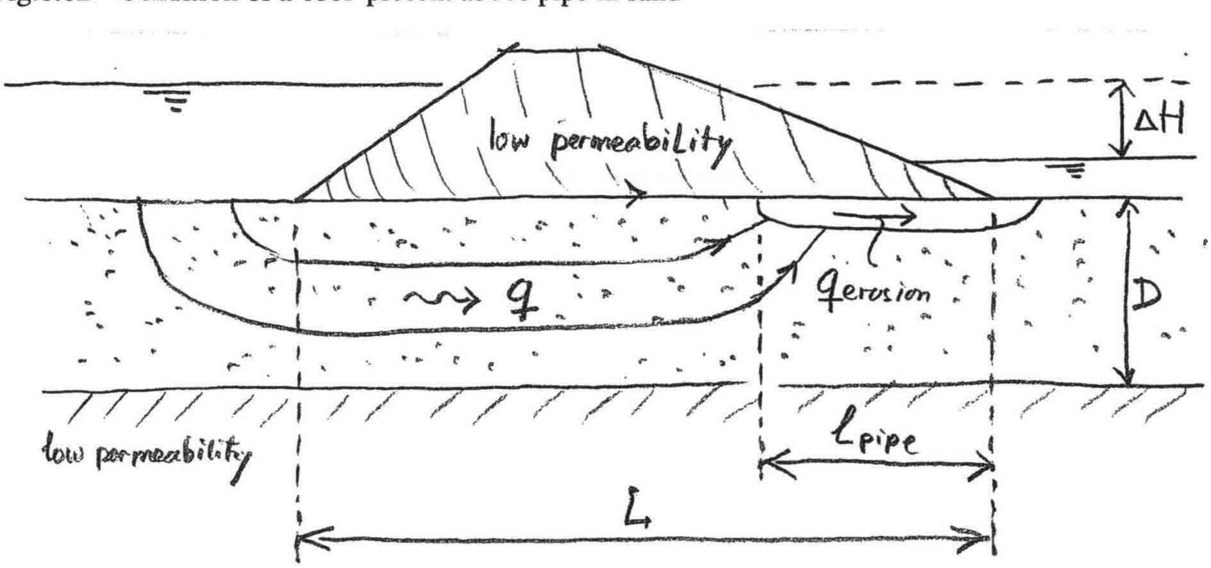


Fig.8.8C Condition for critical flow discharge

8.4.3 Simple analysis of condition (C) for piping

The magnitude of the seepage flow in a water-bearing sand stratum under a dike will depend on the difference between the water level retained by the dike (intake side) and the water level in the polder (outflow side) and the geometry and permeability of the sand stratum (including seepage path length and thickness of the stratum). For piping the intensity of the seepage flow (hydraulic gradients) at the outflow zone is more particularly of importance.
Whether a particular outflow gradient will indeed cause piping is moreover determined by the properties of the sand stratum, of which the mean grain diameter D_{50} and the coefficient of uniformity $U = D_{60}/D_{10}$ are considered to be the principal parameters.
The existing criteria, such as those applied in the Netherlands more particularly to the assessment of river levees and aiming to prevent incipient piping, relate to water level differences of long duration. For this purpose the seepage flow can be conceived as steady flow.
For river levees this is an obvious approximation because the relevant flood waves are usually of several weeks duration. On the other hand, in the case of sea-dikes such long periods of high water generally do not occur. For this reason the danger of piping in sea-dikes is less.

The standard design rules were developed by Bligh and Lane, with regard to piping and underflow associated with rock-fill dams on the basis of a statistical analysis on such structures which had failed and which had not failed. In connection with this a minimum necessary seepage length L_k under the structure was determined:

$$L_k = c * \Delta H,$$

where ΔH is the total head loss (overall difference in water level) across the structure and c is a coefficient depending on the type of soil, see table.
These rules, though originally intended for structures such as dams, have -in the Netherlands and elsewhere- been applied also to river dikes for determining a minimum necessary seepage path length in the sand subsoil of an impermeable clay dike and of any poorly permeable top layers of soil on the foreland (more particularly the winterbed of the river).
For Dutch conditions this comes down approximately to a seepage path length of $L_k \geq 15 * \Delta H$ (for example ranging from $12 * \Delta H$ for course sand to $18 * \Delta H$ for fine sand, acc. to Bligh).

The entry point of the groundwater flow can be taken at the sea/river side toe of the dike or, if there are nearly impermeable layers on the foreland, at a distance L_1 as indicated in figure 8.9.

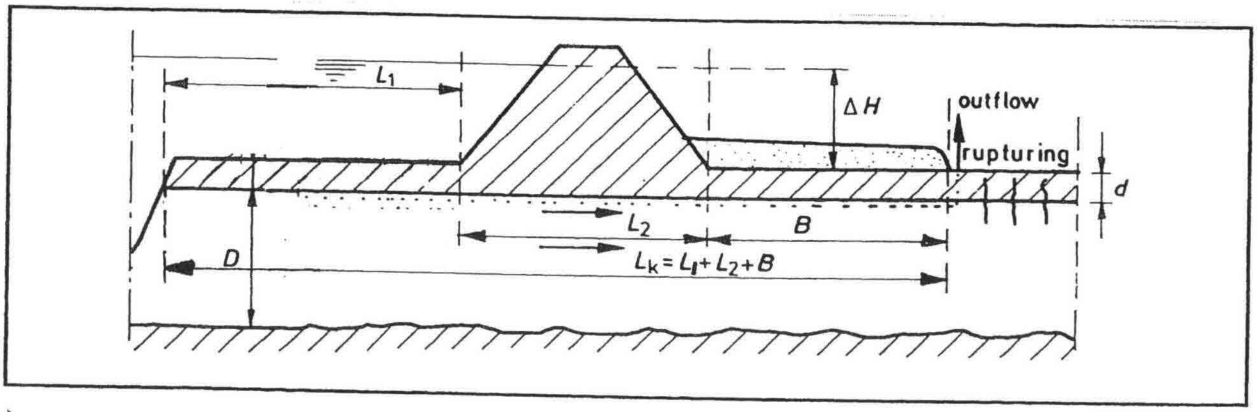


Fig. 8.9 Determination of minimum seepage path length

Piping will, according to Lane, not occur for the following condition:

$$\frac{\Delta H}{L*} < \frac{1}{m \cdot c_L}$$

where

$L* = (L_1 + L_2 + B)/3 + d$

m - modelfactor taking into account the scatter in empirical observations and also the poorly representative character of those observations with regard to flood defences. $m \approx 1{,}67$ (expected value 1,67; standard deviation 0,33)

c_L - See table.

Piping will, according to Bligh, not occur for the following condition:

$$\frac{\Delta H}{L} < \frac{1}{m \cdot c_B}$$

where

$L = L_1 + L_2 + B + d$

m - modelfactor as with Lane

c_B - See table.

TYPE OF SOIL	c_L (Lane)	c_B (Bligh)
very fine sand or silt	8,5	18
fine sand	7,0	15
medium-grained sand	6,0	-
coarse sand	5,0	12

8.4.4 Sophisticated analysis

The fundamental objection to the methods of Bligh and Lane is that they do not take account of the potential pattern of the seepage flow in a clear-cut manner. It is known from research into ground-water flow that seepage into (and therefore the hydraulic gradients at) the ditch at the inner toe of the dike depends not only on the seepage path length, but also on the thickness of the water-bearing sand stratum (aquifer) and the width of the ditch. For relatively low thicknesses of the sand stratum (<approx. 6 m) the gradients are considerably smaller than for large thicknesses, so that in such cases the danger of piping is much less.

In the framework of the raising of the dikes in the Netherlands, much research has been done on piping. The generally accepted analysis is by Sellmeijer. He designed a mathematical model describing the phenomena of piping. Not only the threshold of piping as given in the above formula, was taken into account, but also the transportation of grains through an already formed channel. His model consists of a numerical solution of the relevant differential equations. By curve fitting the results of this model, the following formula was obtained [WEIJERS AND SELLMEIJER, 1993 or TONNEIJK, 1992].

$$H_c = \alpha c \frac{\rho_s-\rho_w}{\rho_w} \tan\theta (0.68-0.10\ln c) L$$

with $\alpha = (D/L)^{\frac{0.28}{(D/L)^{2.8}-1}}$

and $c = \eta D_{70} [\frac{1}{\kappa L}]^{1/3}$

For the L in this formula one may use the L_k as defined in figure 8.10. The parameters ρ_s and ρ_w are the density of sediment and water respectively.
The parameters θ and η are the angle of repose and a resistance-coefficient for the material to be washed away respectively (White's coefficient), for which values of 41° and 0.25 are recommended (for all sands where piping can play a role).
κ is the intrinsic permeability of the soil. The intrinsic permeability is expressed in m², and is related to the hydraulic permeability and to the temperature. See section 8.2.1.

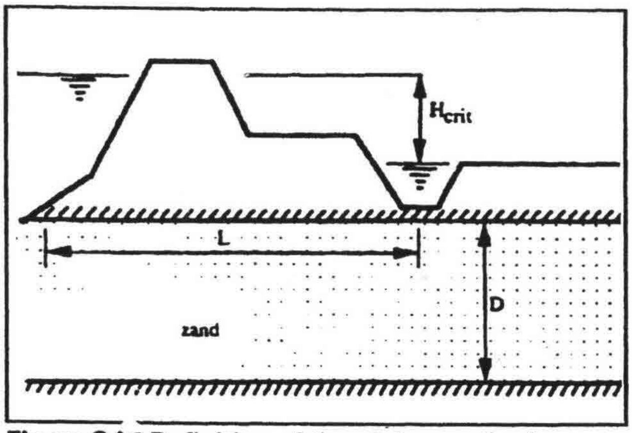


Figure 8.10 Definition of the piping method acc. to Sellmeyer

In the above equations the coefficient D/L plays an important role. See figure 8.10.When the thickness of the permeable layer D becomes small, the necessary L can be reduced. Based on the new equation, new rules for safety against piping in dike stability in the Netherlands have been established. For situation where D is relatively small (say less than 10 m), a reduction of about 20 - 30% of L is possible. For a dike to be raised, with many houses in the vicinity, this can mean the difference between demolition of the houses or not.

For the computation of piping a routine in CRESS is available.

Example:
Given is a situation as depicted in figure 8.9 The angle of repose of the sand is 41°, White's drag coefficient is 0.25, groundwater has a temperature of 5 °C and the sediment has a D_{10}, a D_{60} and a D_{90} of resp. 220, 287 and 300 μm. We assume normally packed sand, a gravel content of 15 % and a silt content of 0.5 %. (these are data from a real dike near Kampen in the Netherlands)

*The result of the computation is plotted in figure 8.11. Suppose the dike has to cope with a H_{crit} of 5 m. Assume both the inner and outer slope are 1:3 and the crest is 2 m wide. Then the total width of the dike is 2*3*5+2 = 32 m.*
From this figure it becomes clear that if the thickness of the sand package below the dike is less than 5 m, the dike is stable against piping. However, if the sand layer is 12 m thick, a minimum seepage length of 40 m is required. This means that a "piping-berm" of 8 m (40-32) is needed.

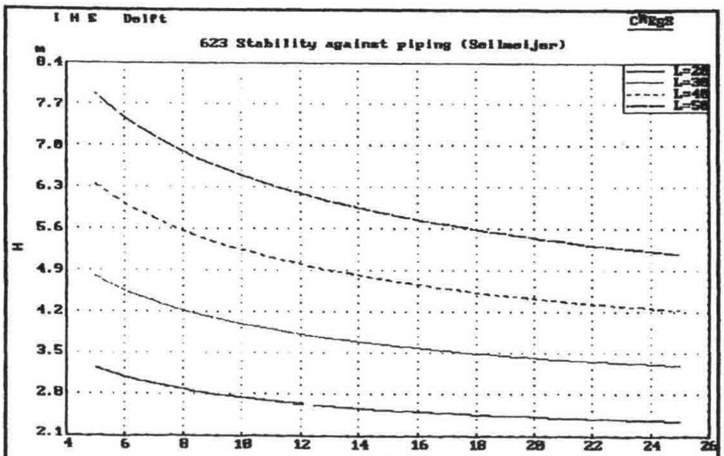


Figure 8.11 Critical head difference as function of L and D (Sellmeyer method)

8.4.5 Measures against piping

Three measures against piping are illustrated in Figure 12.

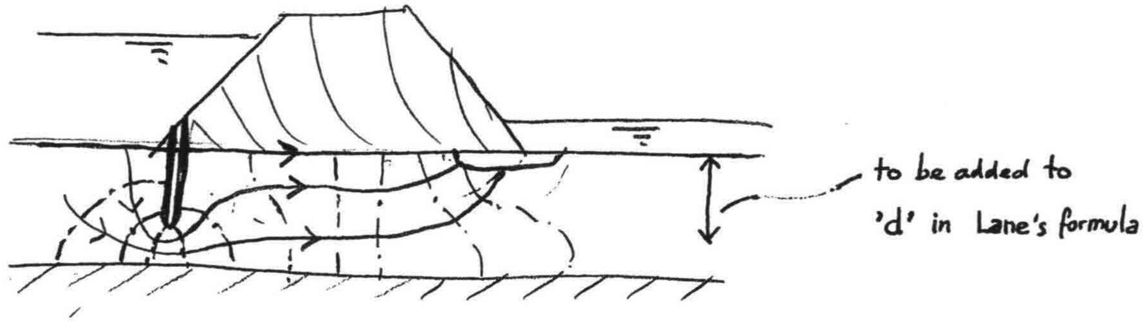


Fig.8.12A Increase flow route by impermeable screen

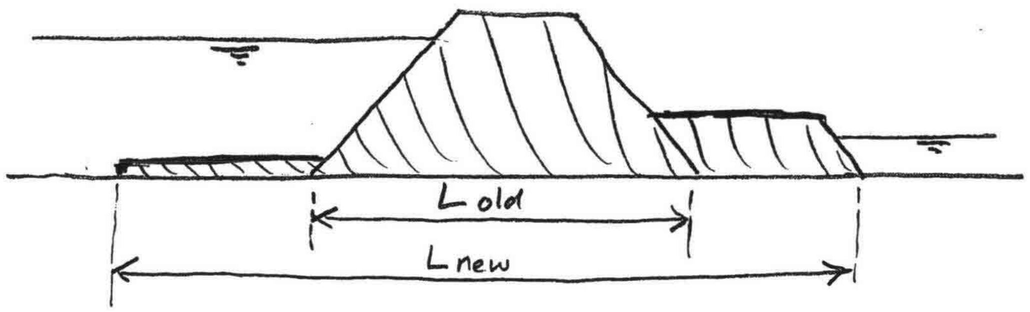


Fig.8.12B Increase flow route by (thick), impermeable berm at inner side and/or impermeable layer at outer side.

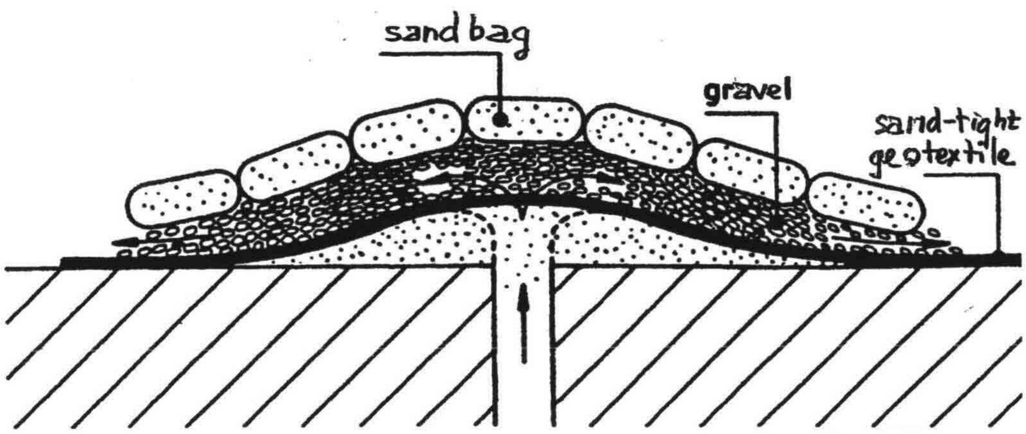


Fig.8.12C Filter on top of pipe

8.5 Macro sliding-stability of inner and outer slope

8.5.1 General approach

In general, the stability is a matter of equilibrium between the weight of the soil, including pore water (the load) and the shear stresses. The shear strength is given by:

$$\tau = c + \sigma \tan\phi$$

The effective grain stresses (σ) are influenced by the pore water pressures, so pore water influences both the loading and the strength. See figure 8.13.

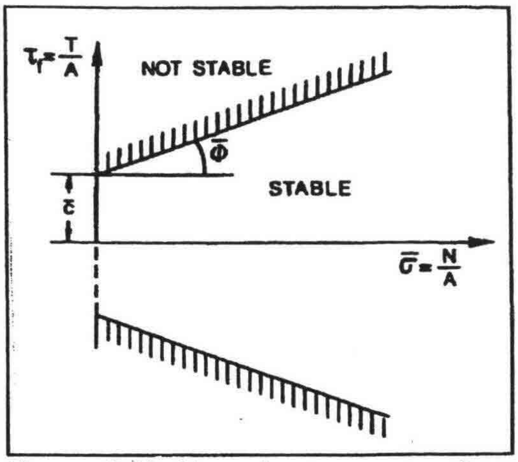


Figure 8.13 Normal stress vs. shear stress

Figures 8.14 and 8.15 give pictures of the slope instability of a dike. The shear stress along a possible slip-circle should withstand the weight of the soil-mass above the circle, expressed as the equilibrium of moments around the centre of the slip-circle. A critical situation occurs after high sea levels when the waterlevel suddenly drops and the slope is filled with water and heavy, increasing the load while the outflow of water decreases the strength.
There are methods that calculate the stability of the slip-circle as a whole, assuming a constant τ along the circle. Since τ is directly related to σ, it will be clear that the shear stress along the slope is not constant.
In a homogeneous soil mass, the assumption of a slip-circle is reasonable, but when there is a considerable non-homogeneity in the soil, the sliding will take place along the weakest layers, where c and/or ϕ are low, hence the shear strength is low, see figure 8.16 The same can occur when σ is low due to high pore pressures.

There are many calculation models for simple and complex situations, that latter mostly based on finite element methods. A relatively simple method is the Bishop-method, as discussed in detail in the lectures on soil mechanics. For the calculation of the stability of dikes in the Netherlands, very often the computer program Plaxis is used. This program is able to also compute the plastic deformation of the soil, as well as situations where the sliding plane is not a circle, but contains a straight section.

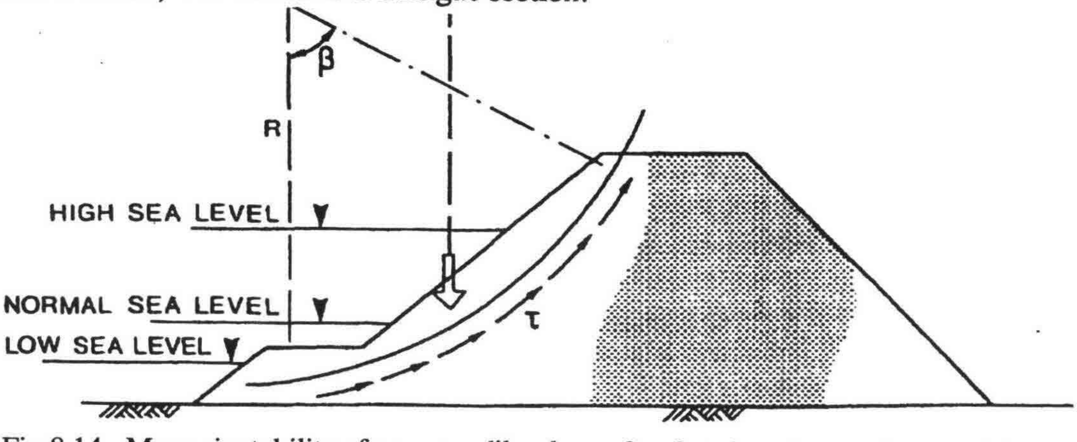


Fig.8.14 Macro-instability of an outer dike slope after fast drop down of water table

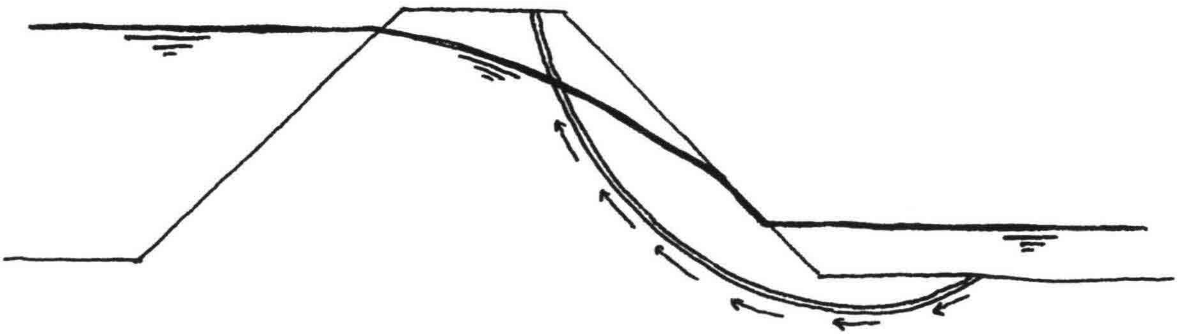


Fig.8.15 Macro-instability of inner slope of river dike at the end of extreme high water period

In some very special cases with a soft clay layer underneath a relatively strong clay or peat layer, the soft clay may be squeezed out.

Three factors have a large influence on the macro-stability:

- slope angle
- excess pore pressures
- shear strength of the soil

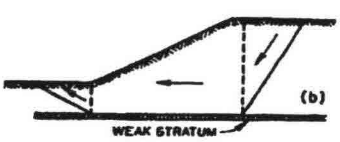


Fig.8.16 Circular rupture surface or straight rupture surface

8.5.2 Analysis macro-stability according to Bishop

The most applied method to analyse the macro-stability in two dimensions is the method worked out by Bishop. See [Pilarczyk 1998], chapter 19. The method is based on the assumption of circular rupture surfaces. Stresses along the surface are found from the vertical equilibrium of slices. The moment equilibrium yields a safety factor, SF. The horizontal equilibrium is not taken into account. See Figures 8.17 and 8.18. Numerical codes are available to analyse all possible circular rupture surfaces and determine the most critical one, i.e. the one with the lowest safety factor.

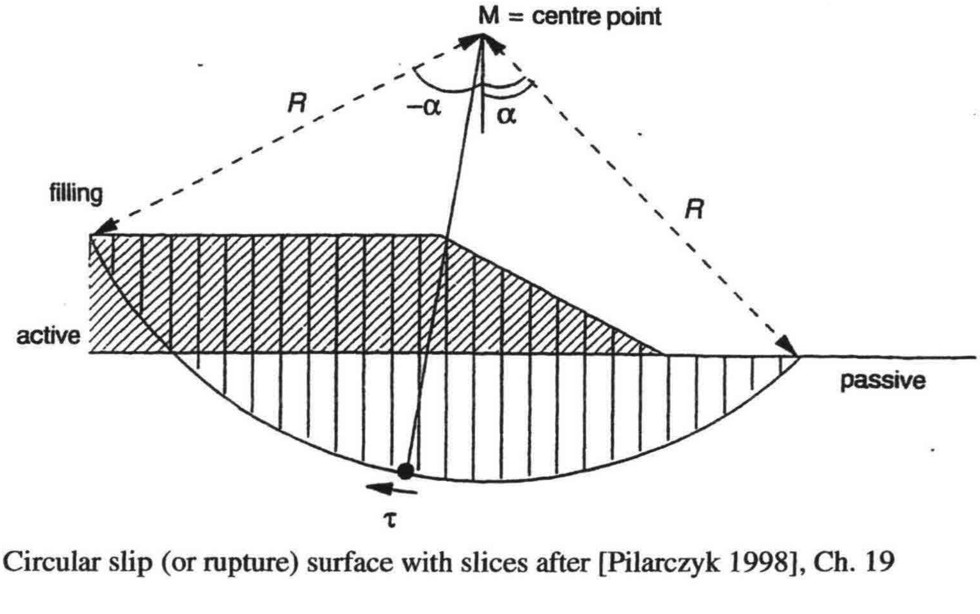


Fig.8.17 Circular slip (or rupture) surface with slices after [Pilarczyk 1998], Ch. 19

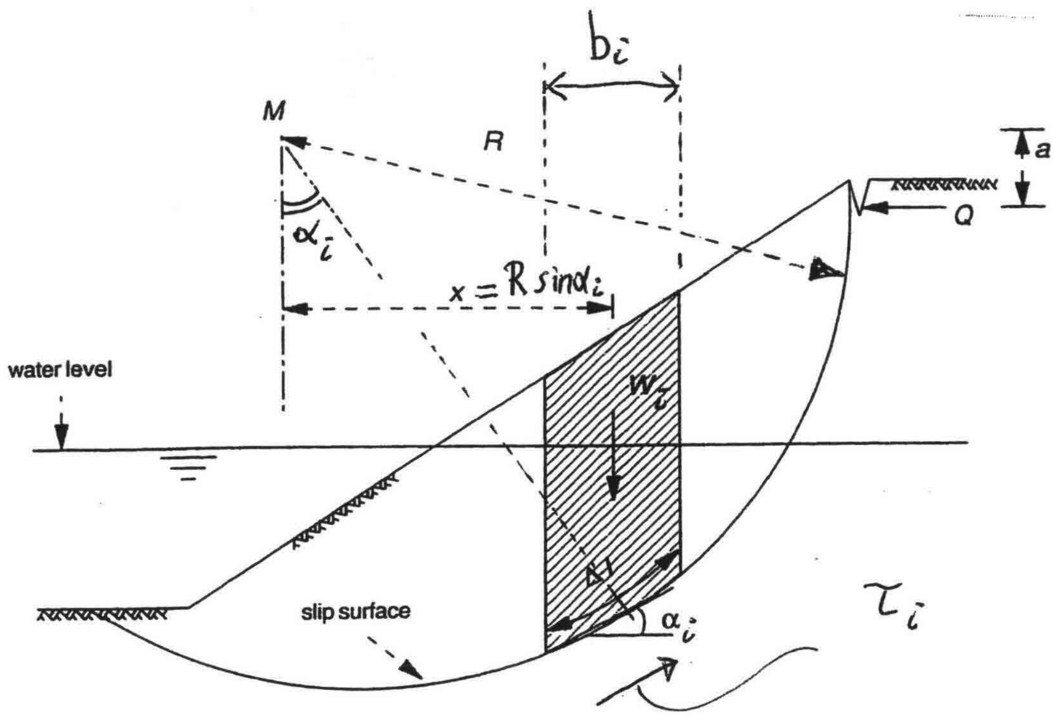


Figure 8.18 Principle for calculation of circular slip surfaces [Pilarczyk 98] Ch.19

$$SF = \frac{\text{total resisting moment}}{\text{total driving moment}} = \frac{\sum_{i=1}^{n} \tau_i \cdot \frac{b_i}{\cos\alpha_i} \cdot R}{\sum_{i=1}^{n} W_i \cdot R \sin\alpha_i} = \frac{\sum_{i=1}^{n} \left[\frac{(c_i' + \sigma'_{ni}.\tan\phi_i).b_i.R}{\cos\alpha_i} \right.}{\sum_{i=1}^{n} [\gamma_i.h_i.b_i.R.\sin\alpha_i]}$$

where SF = stability factor, i = number of slices concerned, c_i' = cohesion at the sl surface of slice i (kPa), σ'_{ni} = effective stress perpendicular to the slip surface of slice i (kPa ϕ_i = angle of internal friction at the slip surface of slice i, b_i = width of slice (m), R Radius (m), α_i = angle of slip surface of slice i with the horizontal, γ_i = unit weight of tl soil in slice i (kN/m), h_i = height of slice (m). The radius R above and below the dividi line in formula can be omitted as can the width b_i, provided that the widths are the same f all slices.

8.5.3 Analysis macro-stability with 'wedge' method

A simple method which can give a first impression of the safety factor by means of a calculation by hand, is the 'wedge' method [Pilarczyk 1998], Chapter 19. The rupture surface is approximated to a surface with 3 straight parts, the central part of which is horizontal. See Figure 8.19. The vertical and horizontal equilibrium of the central soil part (shaded in the figure) are considered. The moment equilibrium is not taken into account. The equilibrium of the lower triangular soil part to the left yields the passive horizontal effective force, F_p, along the vertical plane between the left part and the central part. The equilibrium of the higher triangular soil part to the right yields the active horizontal effective force, F_a, along the vertical plane between the central part and the right part. These horizontal forces can be derived from the horizontal effective stresses as indicated in Figure 8.20 and the formulas, if the ground surfaces of both parts is horizontal. P_p and P_a are the horizontal forces due to pore pressures.

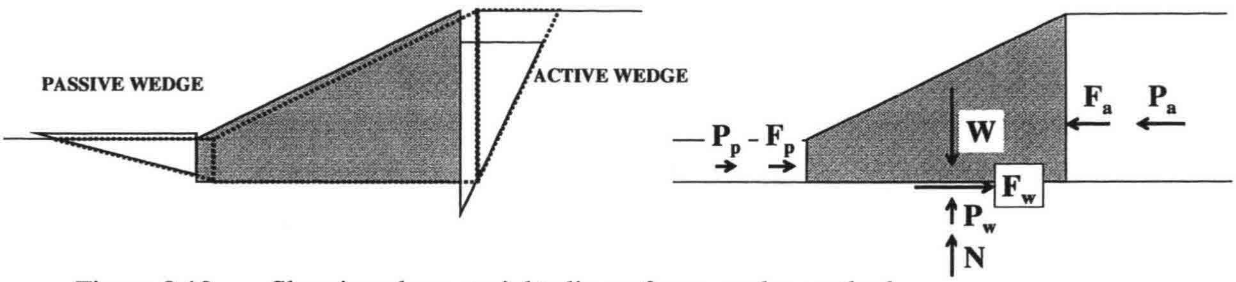


Figure 8.19 Shearing along straight slip surfaces: wedge method

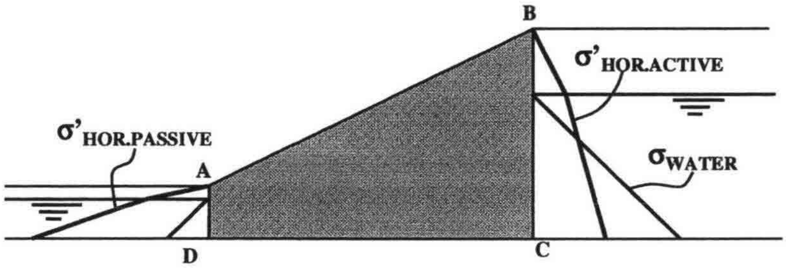


Figure 8.20 Passive and active horizontal effective stresses

$$F_p = \int_D^A \sigma'_{\text{HORIZONTAL}}\, dz = \int_D^A K_p \sigma'_{\text{VERTICAL}}\, dz$$

$$F_a = \int_C^B \sigma'_{\text{HORIZONTAL}}\, dz = \int_C^B K_a \sigma'_{\text{VERTICAL}}\, dz$$

where $K_p(c, \phi) \approx 3.0$ and $K_a(c, \phi) \approx 0.3$

The vertical effective stresses can be derived from the vertical equilibrium of the column of soil and the pore pressure σ_{WATER}.

$$F_w = \int_D^C \tau_{\text{strength}}\, dx = \int_D^C (c + \sigma'_{\text{VERTICAL}} \cdot \tan \varphi')dx$$

where φ' is the effective (or "real") friction angle

The vertical equilibrium of the central part: $N = W - P_w$

The horizontal equilibrium of the central part: $F_w > F_a - F_p + P_a - P_p$

REMARK: The safety factor, SF, can be defined in at least two ways:

$$SF = \frac{F_w + F_p + P_p}{F_a + P_a} \quad \text{or} \quad SF = \frac{F_w}{F_a - F_p + P_a - P_p}$$

Both are correct. In some situations they give quite different results, although always holds:
- in case there is just equilibrium, they both yield SF = 1.
- in cases of equilibrium they both yield SF ≥ 1
- in cases of instability, they both give SF ≤ 1.

Different possible safety factor definitions are quite comon in soil mechanics.

8.5.4 Analysis macro-stability during earthquakes

The analysis of the macro-stability during earthquakes requires 4 major steps:

1 Prediction of 1/100 years or 1/1000 years magnitude at rock base (right part of Figure 8.21). Extrapolation from past registrations. Problems:
   - Prediction of expected horizontal distance to center of earthquake
   - Measure of "magnitude": max. horizontal acceleration?

2 Prediction of the maximum horizontal acceleration at earth surface (left part of Figure 8.21). This acceleration will be larger than the one at rock base, if excitation dominates and smaller if damping dominates. Which of the two dominates depends on the dynamic properties of the soft soil layers.

3 Stability of the dike without liquefaction. This can be approximated by making a quasi-static calculation with the horizontal inertia force as an external load (Figure 8.22). More sophisticated is a 2-dimensional dynamic analysis with a finite element model yielding the deformation at the end of the earthquake.

4 A liquefaction analysis if loose or medium dense sand or silt is present below the phreatic surface. See section 8.7.4.

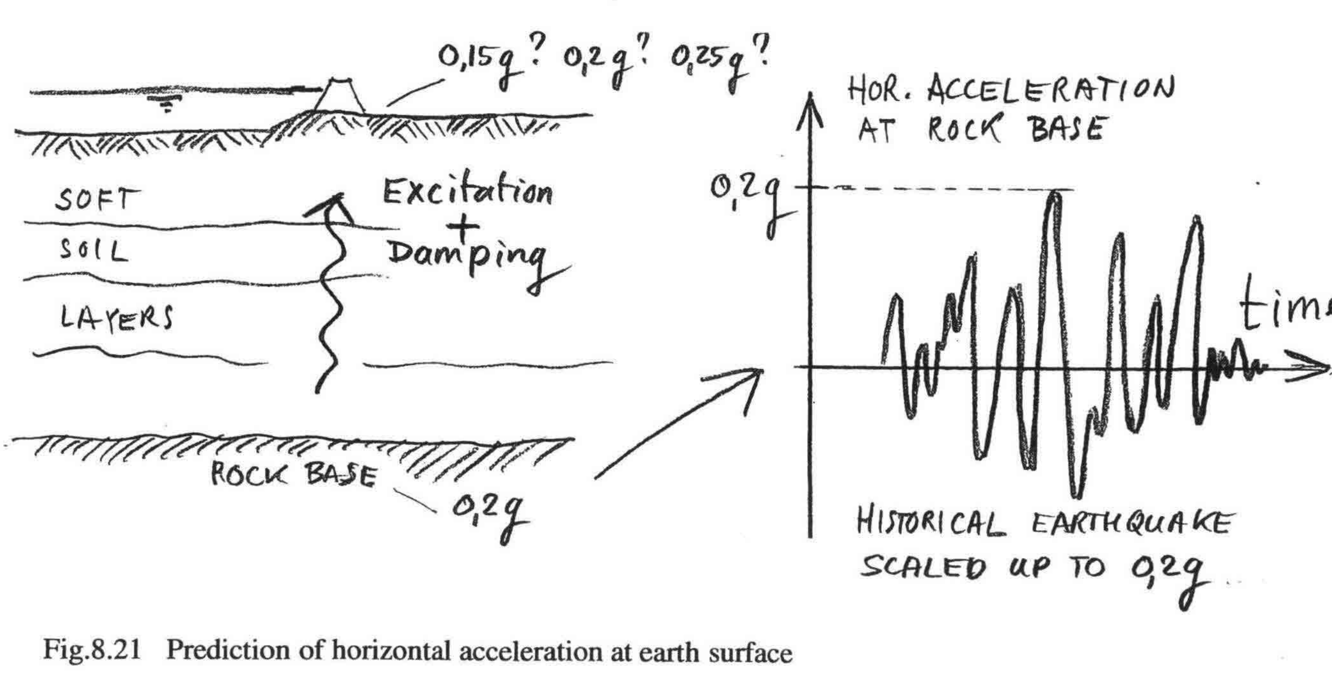


Fig.8.21 Prediction of horizontal acceleration at earth surface

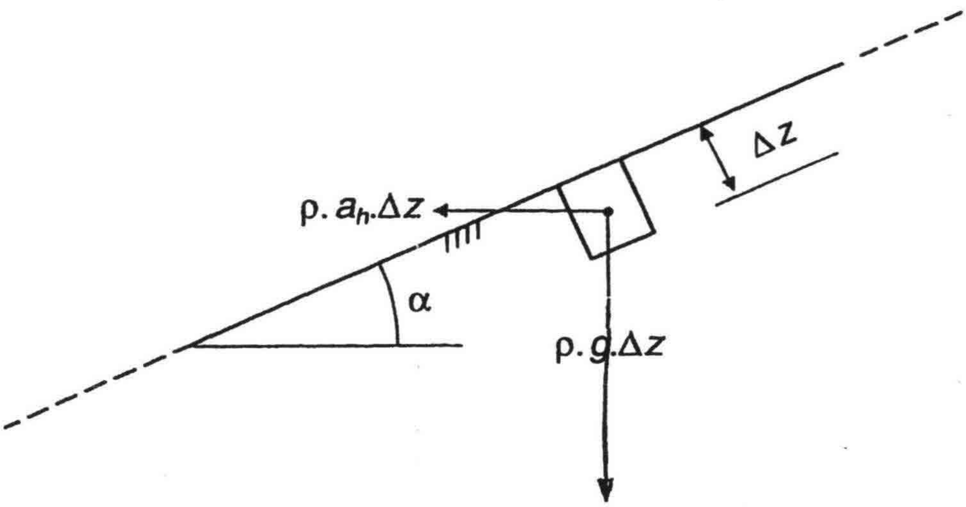


Fig.8.22 Stability of infinite slope under earthquake loading after [Pilarczyk 1998] Ch.19

8.5.5 Analysis macro-stability during construction

The macro-stability will be smaller during construction than in the final stage, If clay or peat are present in the sub soil or if the dike is made with this soil material, because consolidation of these materials needs to take place before they reach their final strength.

The consolidation process can be studied with the help of oedometer tests. See [Lubking 2000], Chapter CS (Compaction and Settlement). One must realise, however, that these tests only simulate the 1-dimensional behaviour. Oedometer tests yield 2 results:
- Final compression (= settlement) as a function of (vertical) load. [Lubking 2000] CS, section 1.5 and 2.1 yields the final settlement, s as a function of σ_0', σ_c', σ_v', C_1, C_2 and h.
- Variation in time: a) consolidation (primary compression)
  b) creep (secondary compression). [Lubking 2000] CS, section 3.3 yields the hydrodynamic period Δt_e as a function of s, h, k and (σ_v' - σ_0') and U as a function of time and Δt_e.

The translation of these results into a prediction of the strength is explained in [Lubking 2000], Chapter IS (Instability and Strength). It is summarised below:

DRAINED loading (e.g. by Consolidated Drained tri-axial test) yields the real strength properties of the skeleton:
- Effective (or "real") cohesion c'
- Effective (or "real") friction angle φ'

$\tau_{strength} = c' + \sigma \cdot \tan\varphi' = c' + \sigma' \cdot \tan\varphi'$

where σ is the mean total stress and σ' is the mean effective stress, defined as $\sigma' = (\sigma'_x + \sigma'_x + \sigma'_v)/3$, with x and y indicating the two horizontal direction and v the vertical direction. In normally consolidated soil $\sigma' \approx 0.7\sigma'_v$. Thus:

$\tau_{strength} \approx c' + 0.7\sigma'_v \cdot \tan\varphi'$

UNDRAINED loading (e.g. by Unconsolidated Undrained tri-axial test) yields the apparent strength, relevant for quick loading, or the "undrained shear strength, c_u":
$\tau = c' + \sigma' \cdot \tan\varphi'$ is also valid here. However, $\sigma' = \sigma - u = \sigma'_0 = \sigma_0$ (u is the excess pore pressure, σ_0 the initial value of the mean total stress and σ'_0 the initial value of the mean effective stress) is constant, even if σ increases, because u increases with the same amount, at least in the brief period considered. Thus:
$c_u = \tau_{strength} = c' + \sigma'_0 \cdot \tan\varphi' \approx c' + 0.7\sigma'_{v0} \cdot \tan\varphi'$

DURING CONSOLIDATION under constant load σ, u decreases and σ' increases with the same amount. Define the initial vertical stress is σ_{v0} and the vertical stress after loading is σ_{v1}. With the approximation: $\sigma' \approx 0.7\ \sigma'_v$, the following shear strength is found as a function of the degree of consolidation U:

$$\tau_{strength}(U) = c' + \sigma'(U) \cdot \tan\varphi' \approx c' + 0{,}7 \cdot \sigma'_v(U) \cdot \tan\varphi' = c' + 0{,}7 \cdot \{\sigma_{v0} + U \cdot (\sigma_{v1} - \sigma_{v0})\} \cdot \tan\varphi'$$

8.5.6 Vertical drainage to increase speed of consolidation

Consolidation may take years, yielding insufficient macro-stability if the dike is immediately constructed to its full height. Thus, it may take years to construct the dike. To reduce this period vertical drainage can be applied to increase the speed of consolidation. See Figure 8.23. The vertical drains are usually made of geosynthetics and brought in place by large cranes.

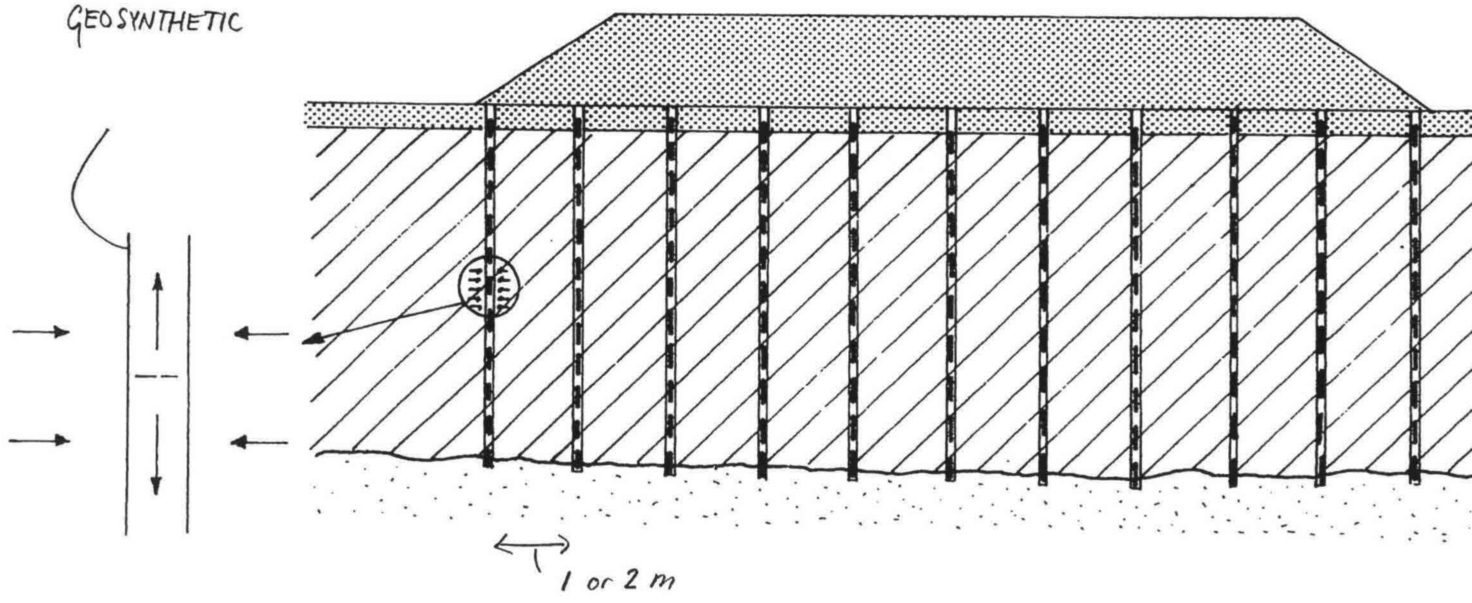


Fig.8.23 Vertical drainage to increase speed of consolidation

8.5.7 Mathematical models for macro-stability

The 'Bishop analysis' and the 'wedge method', discussed in section 8.5.3, are models that just care for stability. The models can be characterised by:

- A 'rigid-plastic' soil model; the shear-stress/shear-strain model is simplified as indicated in Figure 8.24A.
- Output: safety factor
- A limited number of rupture surfaces is considered; the limitation is very strong for the 'Wedge' method where just one (straight, horizontal) plane is analysed for each calculation; the limitation is much less with 'Bishop', where all possible circular surfaces are considered; non-circular surfaces, however, are not considered.
- Usually not all 3 equilibrium equations are taken into account: the 'Wedge' method does not analyse the moment-equilibrium; 'Bishop' does not analyse the horizontal equilibrium.

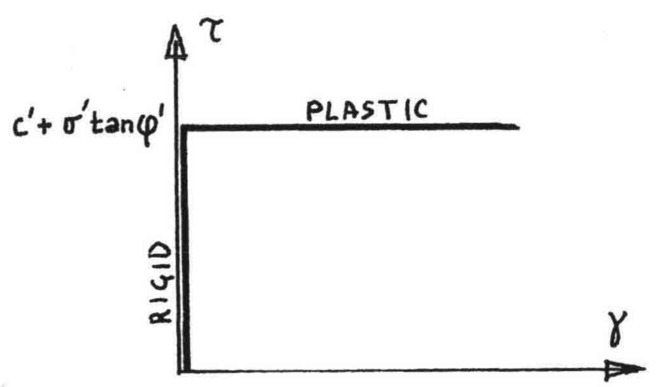


Fig. 8.24A Soil model for Bishop analysis

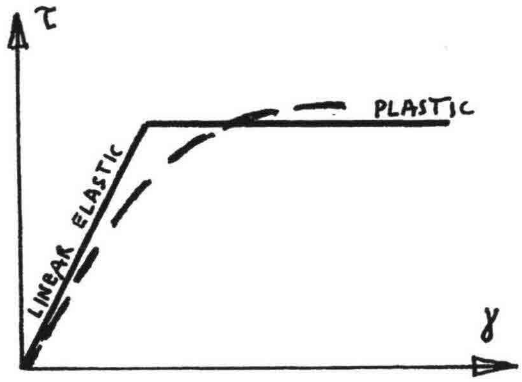


Fig. 8.24B Soil models for FE-models

Most Finite-Element models, like PLAXIS, are made to predict (shear) deformation. Macro-instability can be considered as the result of an extreme shear deformation. Therefore, these models can also be applied to analyse macro-instability. The models can be characterised by:

- A 'linear-plastic' soil model or a more complicated one (Figure 8.24B)
- Output: displacements of points (Figure 8.24C); most FE-models cannot continue to complete failure (numerical instability if small load-increase yields very large deformation); a safety factor is defined differently and may be different from the one found by, e.g. Bishop, even if all assumptions are the same.
- All potential rupture surfaces are automatically considered, at least for as far boundary conditions, element sizes and element properties admit.
- All equilibrium equations are automatically taken into account.

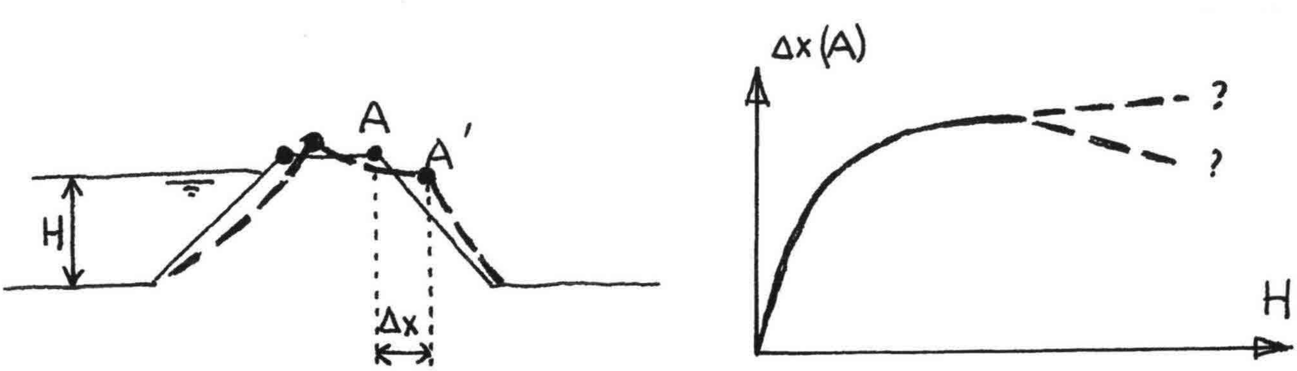


Fig. 8.24C Result of FE-calculation: displacements of points

When selecting the soil model, bear in mind:

- Soil is not elastic in primary ('virgin') loading; it is near elastic in unloading and reloading
- The near elasticity observed in unloading and reloading, is not linear. The meaning of the bulk modulus E, the shear modulus G, the contraction ν is limited.
- The stiffness of the soil as found in the 'oedometer' test is sometimes presented as a kind of bulk modulus 'E_{oed}'. This is NOT a modulus of elasticity, but a modulus for plastic deformation.

Nevertheless, in many cases it is acceptable to simplify, in a numerical, the soil as a linear elastic material (with or without a plastic upper limit expressing the shear strength), provided the user of the model realises the implications of the simplification.

8.6 Too much crest settlement

The failure mode 'too much crest settlement' can be analysed with the final compression found in oedometer tests. See section 8.5.5 and [Lubking 2000], section CS (Compression and Settlement). Apart from the primary compression, ending after complete consolidation, also the secondary compression, due to creep, should be taken into account.

In principle, the compression of all the soil in the dike body and all soil layers underneath the dike should be considered. However, the layers at great depth below the dike are hardly compressed due to:

- the lateral spreading of the dike weight loading (see [Lubking 2000], section SD, figure 5a+b
- the large initial vertical stress, which makes any increase relative small.

8.7 Liquefaction of sand or silt and flow slides

8.7.1 Liquefiability of loose, saturated sand and silt

General information about liquefaction of sand and silt can be found in [De Groot et al 2006].

The basic difference between loose and dense sand is illustrated in Figure 8.25, copied from [Lubking 2000], section SD.

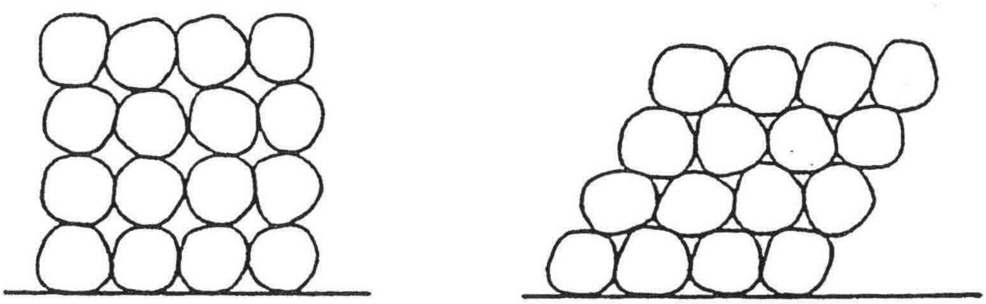


Figure 8.25 Diffence between loose and dense sand, silt or gravel with uniform grains

This difference causes a difference in behaviour when dry sand is loaded with shear: loose sand tends to decrease in volume, 'contraction'; dense sand tends to increase in volume, 'dilation' or 'dilatation'. See Figure 8.26.

When the sand is completely saturated and the pore water can not drain out of the soil skeleton, as in 'undrained shear tests', the volume of the sample cannot change during shearing ('incompressible' pore water). Then, these tendencies will cause the following behaviour:
- increase in pore pressure, yielding decrease in effective stress and decrease in shear resistance during shear of loose sand
- decrease in pore pressure ('suction'), yielding increase in effective stress and increase in shear resistance during shear of dense sand.

See Figure 8.27.

The tendency to contraction is stronger when the shear load is repeated. Thus, cyclic (shear) loading of loose and even medium dense dry sand or silt leads to densification of the soil. Compare the way sand underneath roads and in other foundations is often densified: by vibrating loads. Undrained cyclic loading of loose and medium dense sand or silt causes an increase in pore pressure and a reduction in shear stiffness of the soil. See Figure 8.28. With enough cycles liquefaction occurs. The number of cycles to liquefaction, N_l, is a function of the relative shear load amplitude (shear stress amplitude divided by the original vertical effective stress) and of the density index.

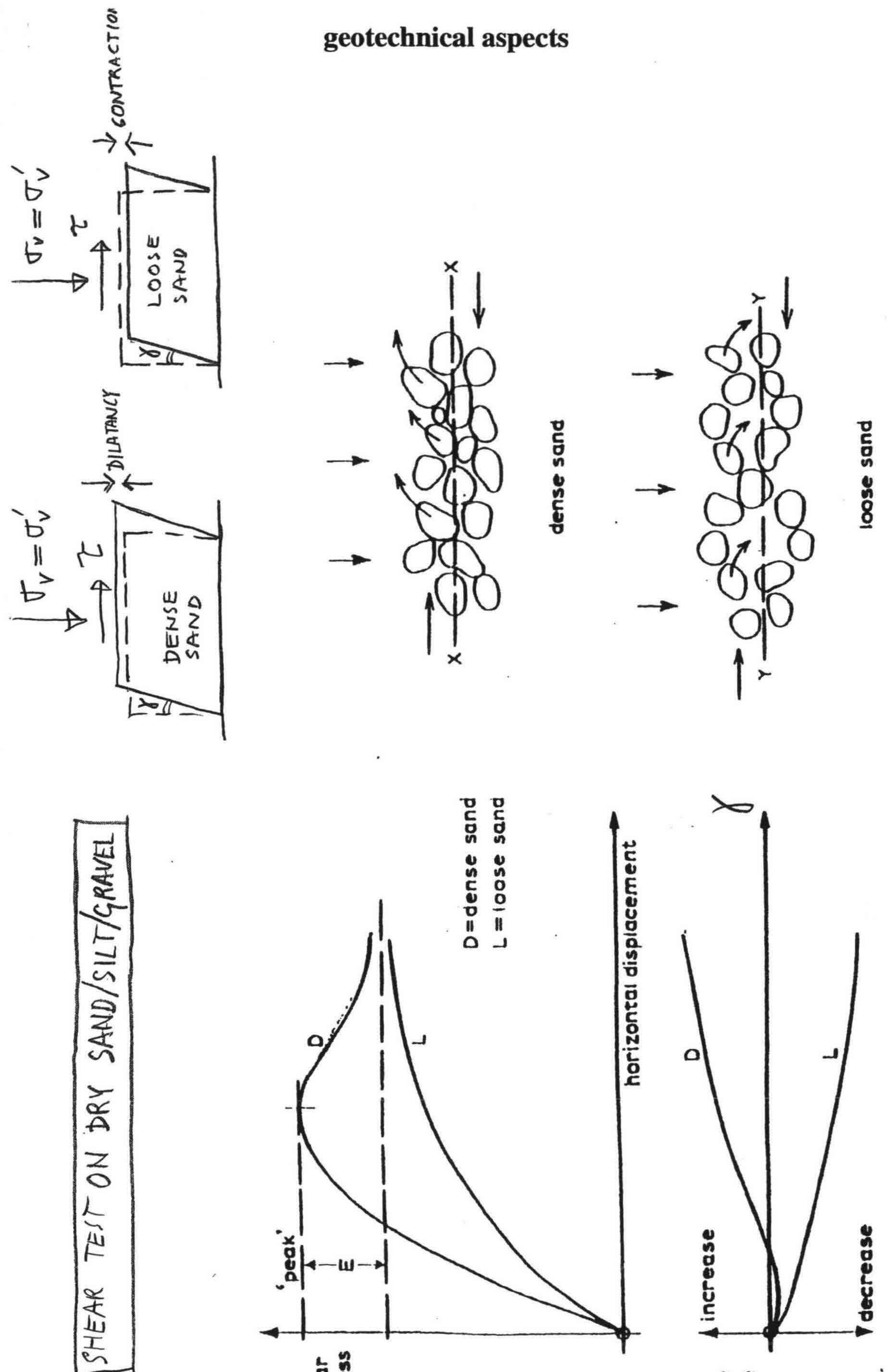


Figure 8.26 Contraction in loose sand (silt) and dilation in dense sand (silt) during shear

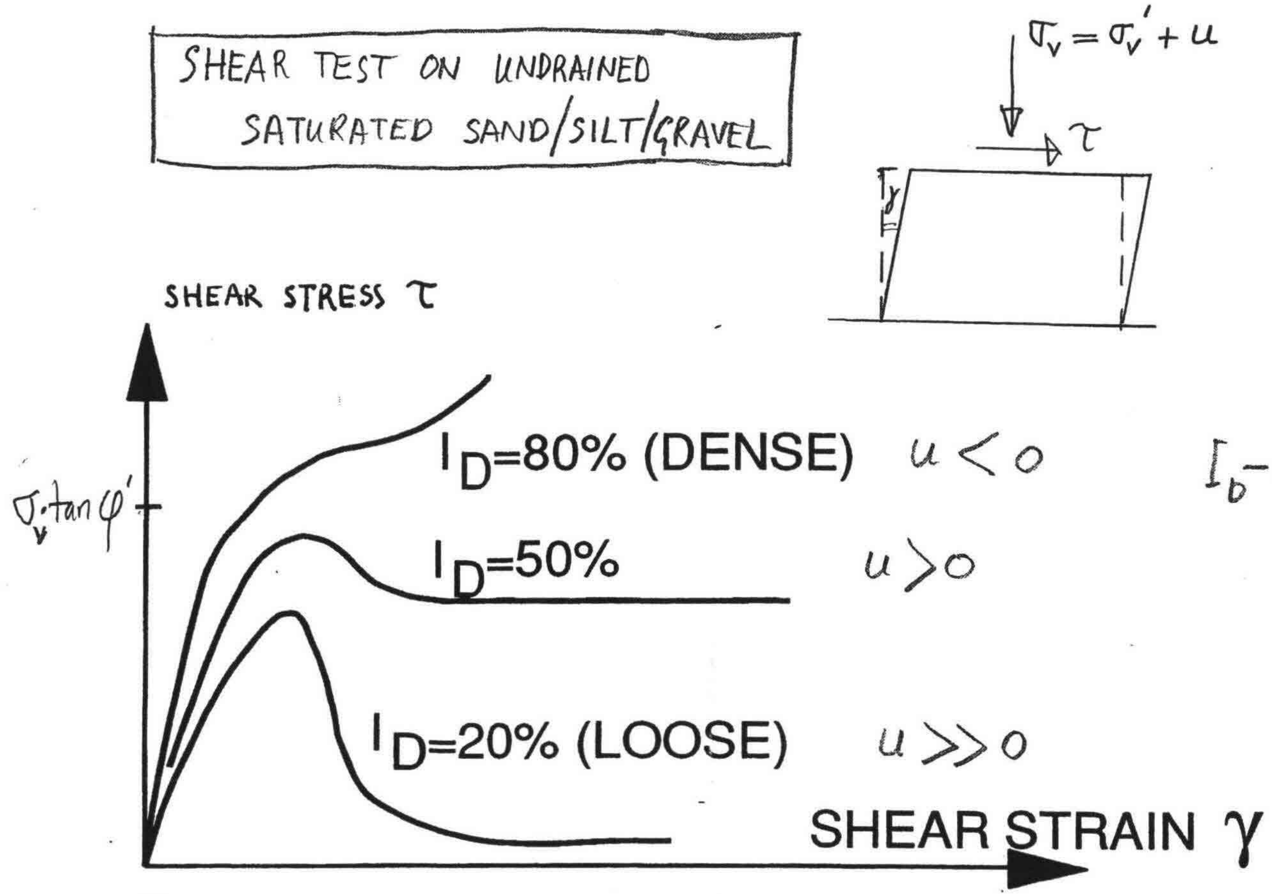


Figure 8.27 Behaviour of sand, silt or gravel in undrained static shear loading

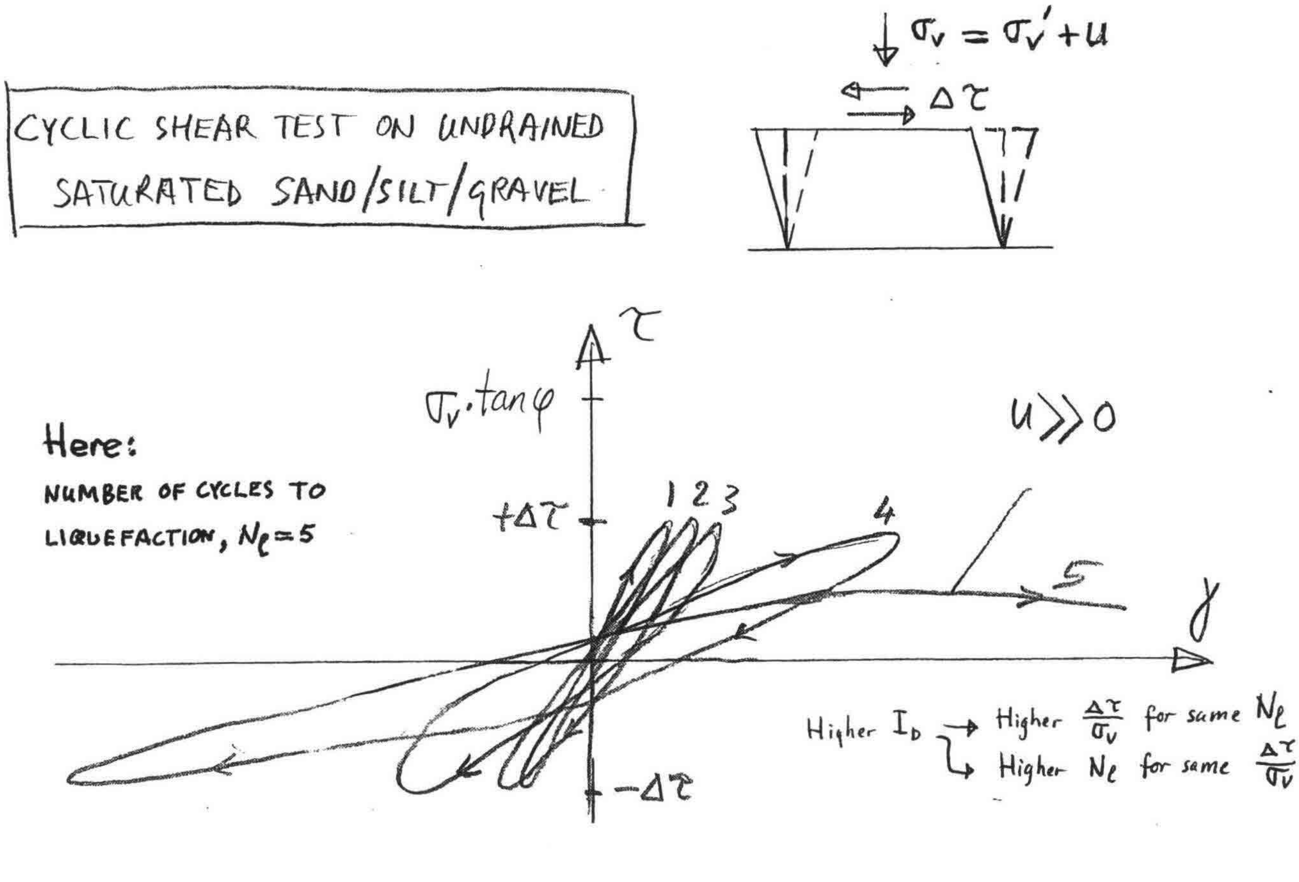


Figure 8.28 Behaviour of sand, silt or gravel in undrained cyclic shear loading

8.7.2 Three types of sand liquefaction relevant for dikes

In many seas, estuaries and rivers, sand has settled at a low density. Such sand is sensitive to liquefaction. Three types of liquefaction may be dangerous for dikes:

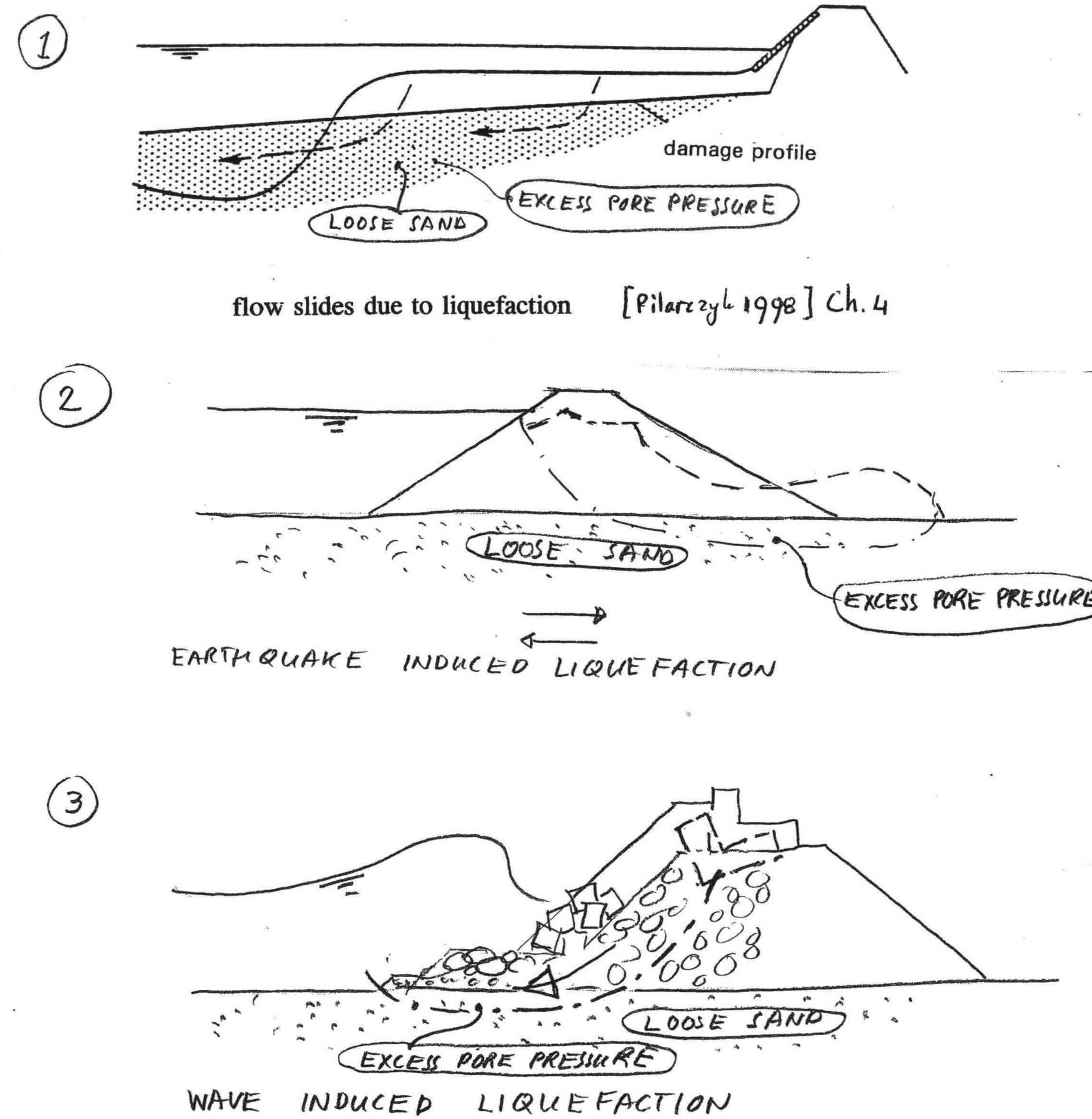


Figure 8.29 Three types of sand liquefaction potentially relevant for dikes

8.7.3 Liquefaction flow slides

More than 1000 flow slides occurred during the last 150 years along the shores of the estuaria in Zeeland. Figure 8.30 shows two profiles of the foreshore of a dike along a tidal channel in Zeeland, each at two different moments: one before and one after the flow slide that occurred at a calm day in March 1966. Please note the scales, and imagine the quantities of soil moved by this process. In this case, no body saw it happen. It must have happened in one night or one hour. In the morning the dike inspector observed that ca 100 m of his 5 m high dike was missing.

Because flow slides had surprised people quite often in history, several safety measures have been developed:
- One of the first measures was that in liquefaction-prone areas, a second dike was constructed behind the primary dike. In case of disappearance of the primary dike, the spare dike could take over its function.
- Later, a systematic survey helped to find the conditions for flow slides to occur. An empirical relationship for the critical steepness of the fore shore slope and the critical depth of the channel was found. Soundings of all fore shores were made every 50 m to 100 m twice a year. Whenever a slope steeper than the critcal one was discovered, an under water slope protection was provided. To date vast section of the fore shores of the Oosterschelde and the Westerschelde have been covered with mattresses.

Figure 8.31 shows a part of the Southern shore of the Westerschelde. The two dikes are indicated. The thick curved lines at the toe of the outer dike indicate the upper boundaries of historical flow slides. The mattresses covering the foreshore are indicated with the year of placement.

More fundamental research has been performed in the 1980's and 1990's. See [Silvis & de Groot 1995] and [Stoutjesdijk et al 1998]. The following conditions for a flow slide to occur have been formulated (figure 8.32):

1. Presence of a relatively thick layer of loose, relatively fine saturated sand or silt (layer thickness D > 10 m with coarse sand; D > 1 m with silty sand; D > 0.3 m with silt)
2. Cotangens of slope angle cotα > 3 to 7
3. Waterdepth h > 5 m to 20 m
4. Slide must be introduced by:
   - vibration (light earthquake; pile driving; etc)
   - local scour
   - quick water level descent
   - dredging
   - other small load change.

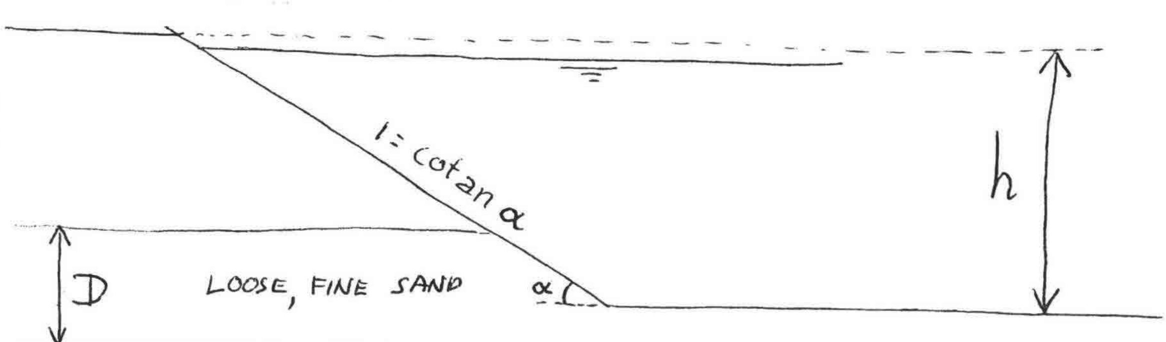


Figure 8.32 Conditions for flow slide

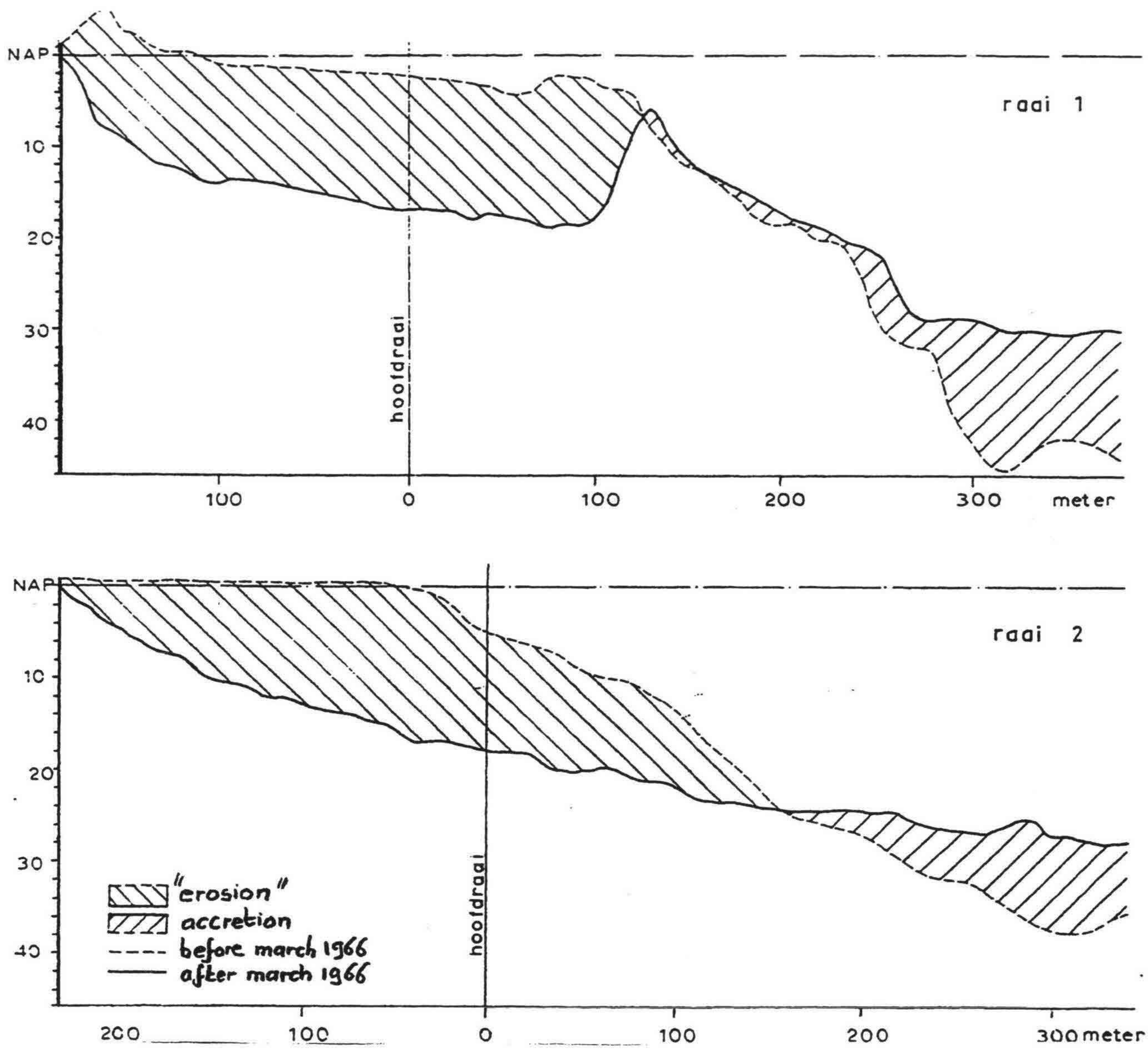


Figure 8.30 Profiles before and after flow slide

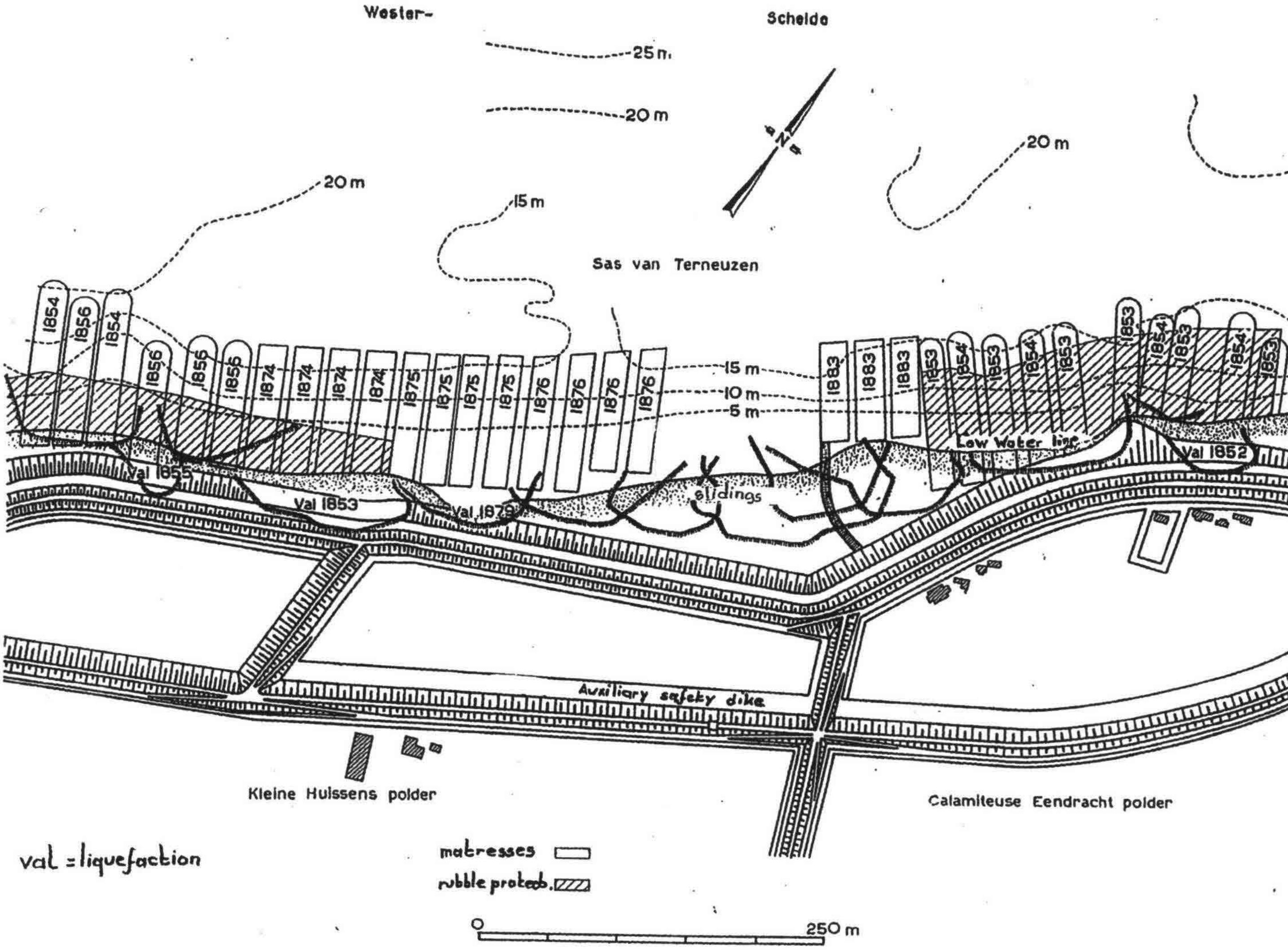


Figure 8.31 Historical flow slides and measures against flow slides

The prediction of the flow slide risk includes:

- site investigations, preferably many cpt's and some borings; if necessary electrical density measurements to get a more accurate prediction of the density
- laboratory investigations: classification (grain sizes, minimum and maximum densities, shape of the grains) and, sometimes, drained tri-axial tests to determine the sensitivity to liquefaction as a function of density and stress level
- soundings to determine the geometry
- verification of the seismic activity in the region.

Potential measures against flow slides:

- Fix gentle slope with slope protection
- Compact sand. This must be done before the slope is steep. Otherwise the compaction would induce a flow slide! Compaction of loose, saturated sand is doen by cyclic loading (vibrators, falling weight or whatever). This cyclic loading causes liquefaction and subsequent drainage of some excess pore water, at least if the sand is not too fine (silt or silty sand can virtually not be compacted when saturated), after which the sand is more dense.

8.7.4 Earthquake induced liquefaction

In seismic active areas, earthquake induced slides are a serious danger if part of the sub soil consist of loose or even mdium dense sand or silt. Many engineering tools are available to quantify this risk, e.g. [Committee on Earthquake Engineering 1985] or [Youd et al 2001]. If the dike body is constructed under water with sand, the density index will be low and compaction of the sand is needed.

8.7.5 Wave induced liquefaction

Wave induced liquefaction is mainly a problem in cases of very heavy wave attack, such as with breakwaters, if loose sand or silt is present in the subsoil or the dike body. An analysis can be made according to [De Groot et al 1991].

8.8 Construction sand body under water

The construction of a sand body under water, if no special measures are taken, results in:
- very gentle slopes
- very loose sand.

This is also the case for the under-water part of a sand body that is partly made above the water.

The slope depends, if no coarse material bunds are used, on the dumping method (dumping from hopper barge; dumping from split barge; hydraulic fill with pipe-end at the water surface or with the pipe-end near the sea/river bed, with or without diffusor; progressing sand body partly above the water, made by hydraulic fill with pipe-end on the above-the-water part of the sand body; etc). See [Van 't Hoff 2001]. The steepest slope is 1:20 to 1:10 for medium to fine sand.

After the dumping process, the slopes can be made steeper by dredging part of the slope. Given the loose packing, however, the chance of a liquefaction flow slide is very large. If a sand body is made partly under water and partly above the water, then the slopes around the water table become even more gentle by the action of waves and currents: a typical beach profile results with slopes of 1:50 to 1:30 above the level 'Mean Low Water – H_s' and below the level 'Mean Low Water + H_s'. These values are for fine to medium sand. With coarse sand steeper slopes result and with gravel slopes of 1:6 to 1:3 may result.

The under water slopes of a soil body can be made much steeper (ca 1:3) by application of bunds made of gravel, quarry-run, clay or geosystems. First two rows of such bunds are placed, after which sand is dumped in between these bunds. Subsequently two new rows of bunds are placed on top of the earlier constructed soil body, sand is dumped in between these new bunds at a higher level. See Figure 8.32.

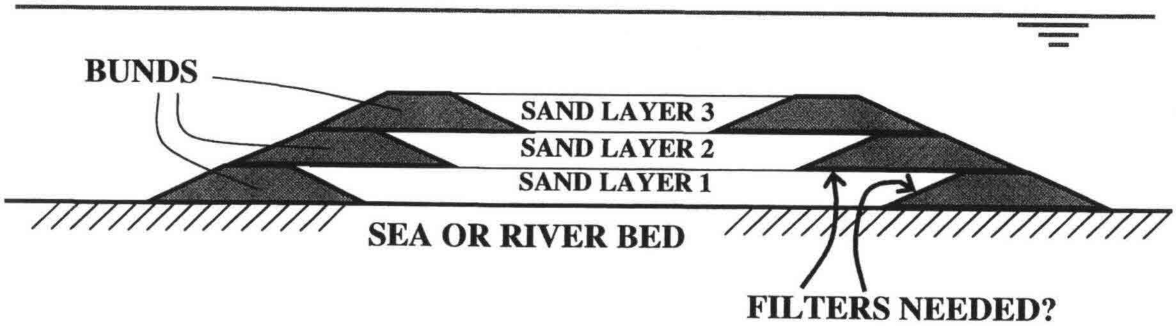


Fig. 8.32 Construction of a soil body under water with bunds and sand

8.9 Lessons about geotechnical aspects

8.9.1 Four lessons about the strange character of soil

Soil 1 Soil varies from place to place

- Sufficient soil investigations essential
- Geology helpful

Soil 2 Soil properties change by (un)loading

- What happened in the past (geology!)?
- What influence by handling?
- What influence by construction?
- What is the stress level and what will it be?

Soil 3 Soil properties change in time

- Consolidation
- Creep
- Cementation

Soil 4 Fast loading of clay and peat can be dangerous:

- Loading yields first excess pore pressures
- Loading yields later increase of strength
-

8.9.2 Three lessons about calculating with soil

Calc 1 Soil model and relevant properties depend on question:

- Stability?
- Settlement?
- Other deformation?
- Pore flow?
- Permanent or temporary load?
- Slow or fast loading?

Calc 2 Soil is seldomly elastic; if it is, then usually non-linear

- Meaning of elasticity modulus E and contraction ν limited

Calc 3 There exists not one universal calculation model

8.9.3 Three lessons about liquefaction of sand

Liqui 1 Liquefaction danger where sand is loose and under water

Liqui 2 Sand placed under water is nearly always (very) loose

Liqui 3 Repetitive loading of loose sand under water first yields liquefaction, then compaction (and more resistance to liquefaction)

8.10 Additional references (Chapter 8)

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9 Various aspects and special revetment types

9.1 limits of the protection

The upper and lower limits of the protection are determined by the loads. As can be seen from the following figure, sometimes the upper limit of the slope may cause some problems. The cover layer as such seems alright and also the filter design has been given attention. But the run-up of the waves is such that they attack the area on top where there is no cover and where the filter layers cannot withstand the wave forces. Even the original unprotected soil is eroded by the waves and a total collapse is the result. The protection should either be made higher or strong enough behind the top to withstand the loads.

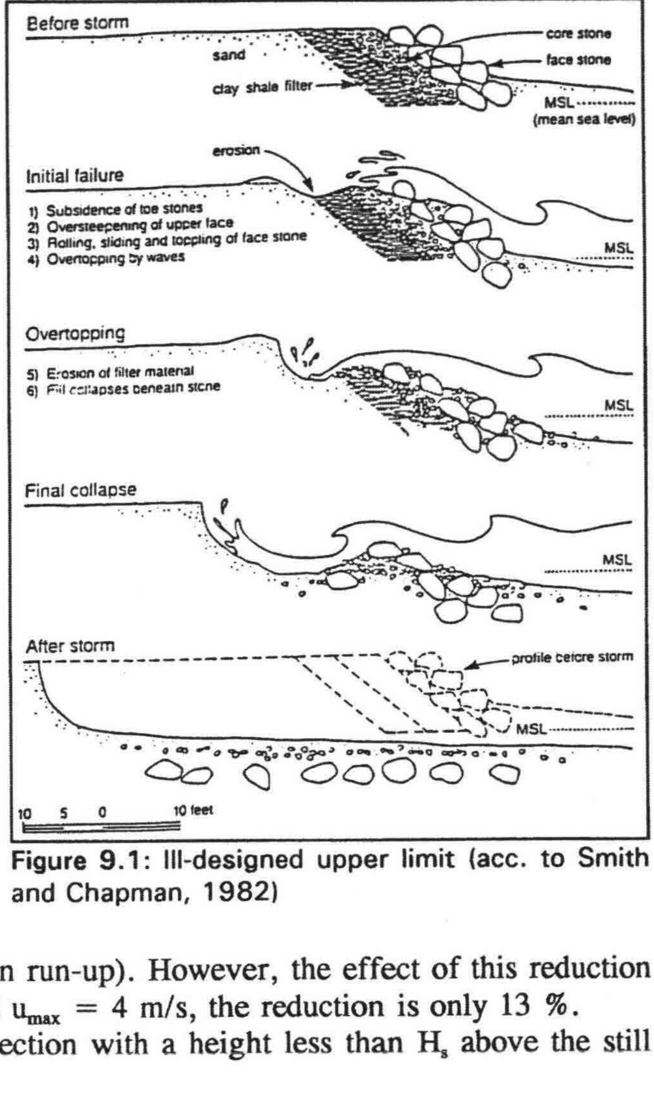

Figure 9.1: Ill-designed upper limit (acc. to Smith and Chapman, 1982)

Usually the lower limit is the value Rd, as defined before in the chapter on wave run-up and run-down. The upper limit can be the maximum Run-up (Ru), however, some reduction might be possible. In case the slope above the revetment can withstand a maximum velocity u_{acc}, on can determine where on the slope this velocity is not exceeded. This point is defined with:

$$z = R_u * \cos[\arcsin(\frac{u_{acc}}{u_{max}})]$$

where u_{max} is the maximum velocity on the slope and R_u the 2% run-up (see chapter on run-up). However, the effect of this reduction is not very much. In case u_{acc} = 2 m/s and u_{max} = 4 m/s, the reduction is only 13 %.
In any case one should never make a protection with a height less than H_s above the still water level.

9.2 transitions

Special attention should be given to transitions because they often form the Achilles heel of the protection; most damage starts at transitions. Transitions can have both a reduced strength and an increased load. Due to the strip, the pressure gradient in the filter can increase while due to a difference in settlement between strip and blocks there can be a hole. Therefore they should be avoided as much as possible, at least in the area of maximum attack.
It is however impossible to avoid them completely, because it is very expensive to use the same protection designed for the maximum attack) along the whole slope. Also in the case

where an existing protection has to be extended, a transition can not be avoided. Sometimes, a transition is used because it can limit damage to certain areas. This means that in case of an overload, the damage will be restricted to a small area. Because the damage cannot spread out, the danger of a failure of the whole dike becomes less.

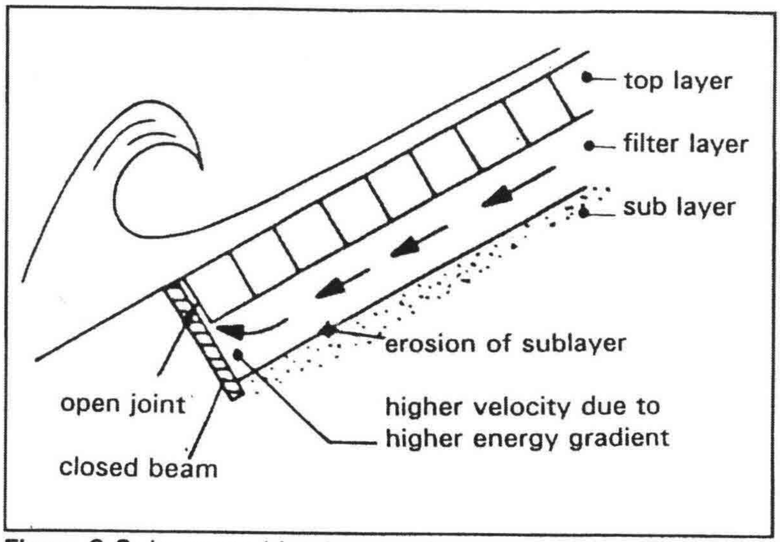

Figure 9.2: increased load and reduced strength

The ideal transition is equally strong, flexible and sustainable as the adjoining layers. Since the ideal is never reached completely, at least the following should be given attention:

1. *relation permeability - overpressure.* If it is impossible to maintain the same permeability and the transition induces extra pressure, the transition should be designed to withstand it.
2. *sandtightness a.* Never allow a seam that goes through the whole protection, always make some overlap, avoiding loss of material trough seams.
3. *sandtightness b.* Transitions are not only important in the toplayer, but in the filter as well. Figure 9.3 gives an example of an incorrect transition where clay can penetrate in the filter layer under the blocks.
4. *care.* Probably the most important. Give extra attention during construction, inspection and maintenance to transitions.

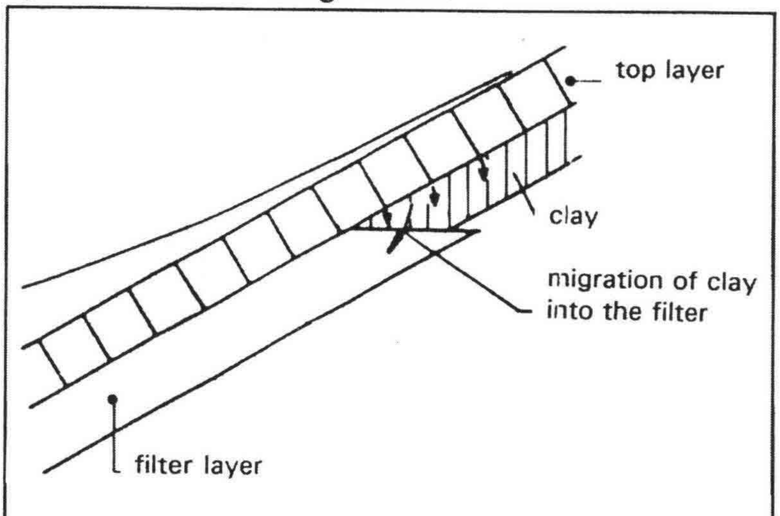

Figure 9.3: migration of clay into the filter

It is impossible to give rules for all situations; much depends on the local situation with regard to loads, available materials, experience, etc. Four general types of transitions will be discussed in the following section.

<u>open-open</u>
As said before, it is important to prevent a seam through the whole protection by continuing the filter layer, see figure. This however, can cause a new problem. When there is a

transitions

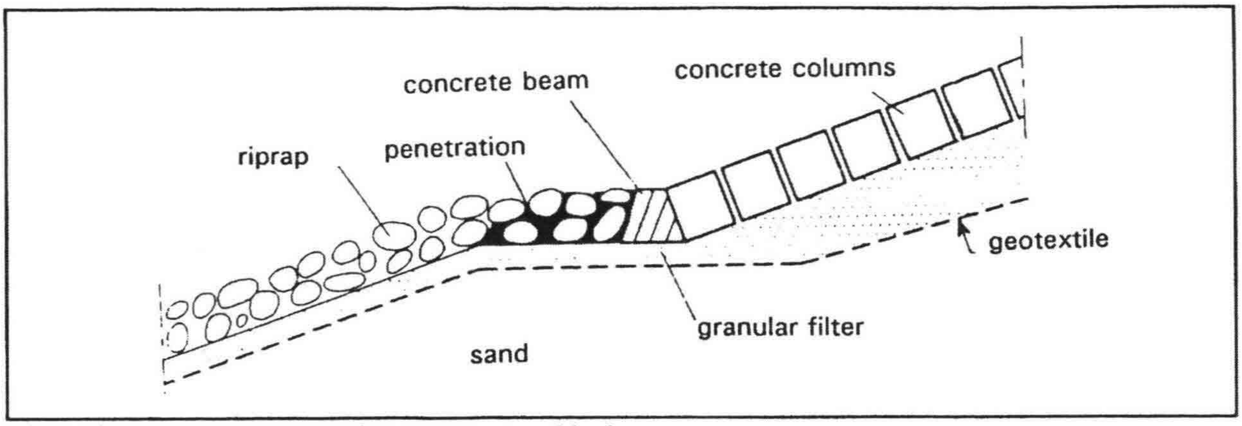

Figure 9.4: transition from riprap to square blocks

considerable differences in permeability (or leakage length), the pressure under the least permeable revetments can lead to uplifting. In the figure this is the case for the lower placed blocks, which get much more pressure via the rip-rap layer. The solution can be the penetration of the upper meter of the rip-rap, hindering the wave penetration. When the transition is at the point of a changing slope angle, e.g. at a berm, provisions should be made to assure a proper placing of rectangular blocks. This can be done by placing a concrete beam with an adapted shape, see figure 9.4.

<u>open-closed</u>
When the filter layer is continued under the closed layer in order to avoid a seam, the closed layer has to be made thicker to withstand the extra (wave) pressure, penetrating through the filter layer under the closed layer.

<u>closed-closed</u>
A filter is usually not necessary here. The most important is to avoid a leak at the transition. With overpressure this would lead to loss of soil material. Figure 9.5 gives an example of a transition from asphalt to clay.

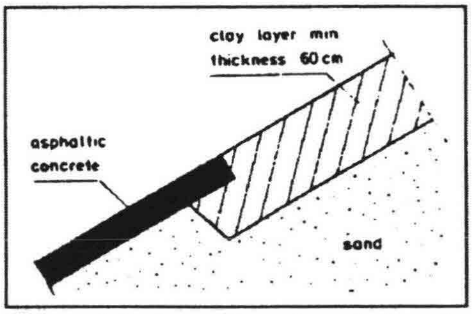

Figure 9.5: transition closed-closed

<u>protection-vegetation</u>
This transition is often necessary for the upper limit of a protection. An abrupt transition can give a seam, aggravated by grazing cattle and the fact that the roughness is different, giving an extra load in wave run-up. Therefore a gradual transition is preferable, possibly with open blocks through which grass can grow.

9.3 toes

Toes are a special kind of transition, i.e. from a slope to a horizontal part. The primary function is the <u>support</u> of the revetment. When the friction along the slope becomes to little, either due to a drop in the waterlevel or to waves, the toe has to deliver the resisting force. So, it should be able to withstand a horizontal force. In rock bottom this is very easy and

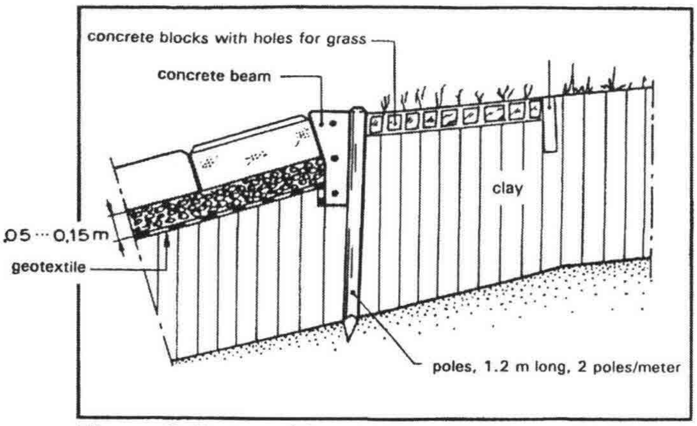

Figure 9.6: transition to vegetation

very much like the connection of two wooden roof beams.

In soft(er) bottoms, toes are often made of piles, sheet piles, strips or combinations. It should be remembered that those can only give a reaction force by means of passive soil pressure from which some displacement is necessary, leading to deformation of the revetment, leading to seams, etc. Therefore when considerable sliding forces can be expected, a layer of stone in front of the toe might help.

When the toe is threatened by scour, some bed protection is needed, like a mattress covered with stones, which can also be useful for the above mentioned horizontal forces.

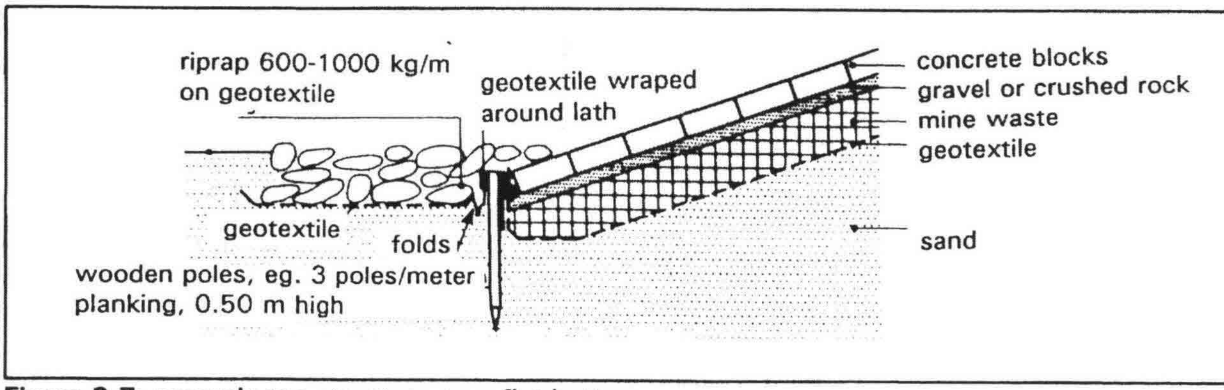

Figure 9.7: example toe structure on a flat bottom

When the toe consists of a closed layer or is constructed with sheet piles, the pore pressure under it can form a threat to the soil directly besides the toe, see figure 9.8. The solution can be a combination of piles, a strip and some granular filter with enough space between the piles to let the pressure escape.

9.4 vertical walls

Vertical walls sometimes are used as water retaining structures or as beach-walls. Very often a vertical wall, made as coffer-dam, is used as a temporary structure. These cofferdams will not be discussed in these lectures. Cofferdam-like structures as permanent water-retaining structures are in principal not advisable, because of the maintenance problems. They should only be used in those cases where other structures are not possible. In dike improvement plans, when a lot of houses or other buildings are in the neighbourhood of the existing dike, often there is no space for improving the existing dike. In such cases a cofferdam may be a solution. The main problem is indeed how to maintain such a structure. This will be discussed in more detail in chapter 13.

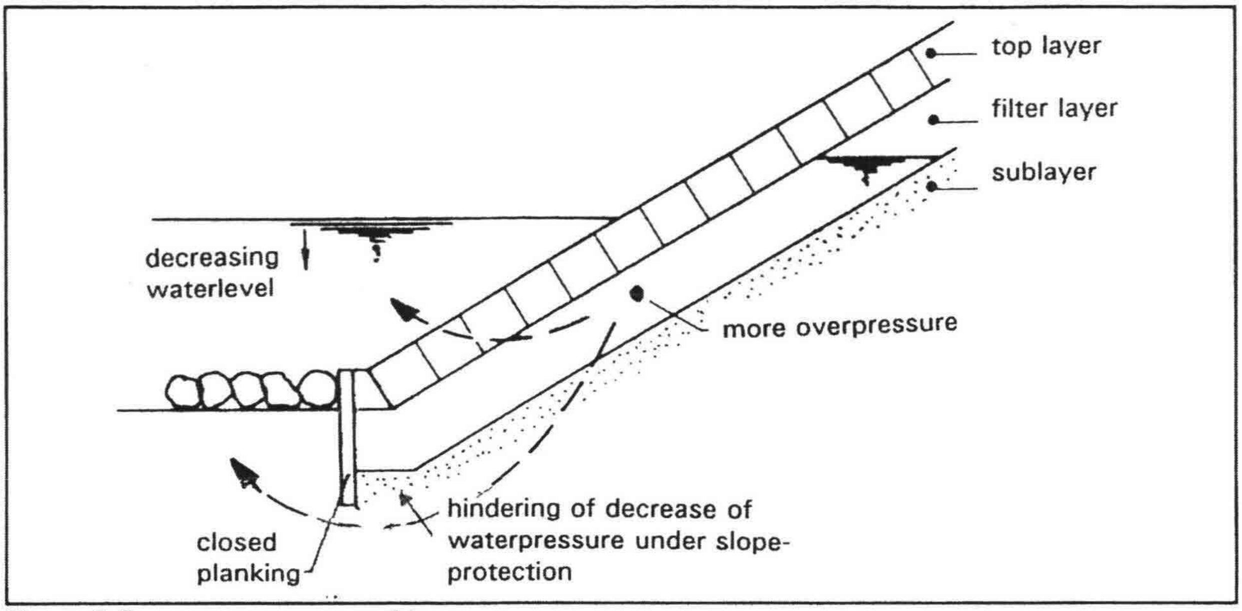

Figure 9.8: water pressure problems

Vertical walls are used along beaches as "beach walls". Usually they were constructed with the idea that they were able to stop the coastal erosion. This is clearly not the case. They only prevent loss of land at the cost of extra erosion in front of the beach wall. So usually the beach in front disappears. Heavy wave attack follows, causing constructional problems. it is therefore not very wise to build a beach wall as coastal protection.

However, in some cases their might be a reason for a vertical wall. Examples of vertical walls are:

- Construction of a nice promenade along the beach. For the development of recreation it is often necessary to make an easy opportunity to stroll along the seafront. A nice promenade with a vertical wall may provide this; however, one should realise that this is only possible along very stable coastlines. For eroding and for unstable coastlines the construction of a beachwall implies always artificial beach nourishment for maintaining a beach in front of the wall.
  An example of such a wall is the retaining wall in Vlissingen. This is an old brick wall not enough strength. Behind the brick wall a steel sheetpiling has been placed to guarantee the structural strength of the wall. The "beach" in front of the wall is fully protected with an asphalt cover, to prevent scouring. Characteristics of this wall are:
  - storm surge level 5.5 m above m.s.l.
  - design wave 4.5 m
  - a deep channel in front of the wall (m.s.l. -30 m)
  - a steep foreshore (slope 1:3)
  - many (historical) buildings on top of the dike (at m.s.l + 9.5 m)
  - width of the dike+ houses approx. 50 m
  - level of the polderland behind the dike (with the centre of the town of Vlissingen) at approx. m.s.l.

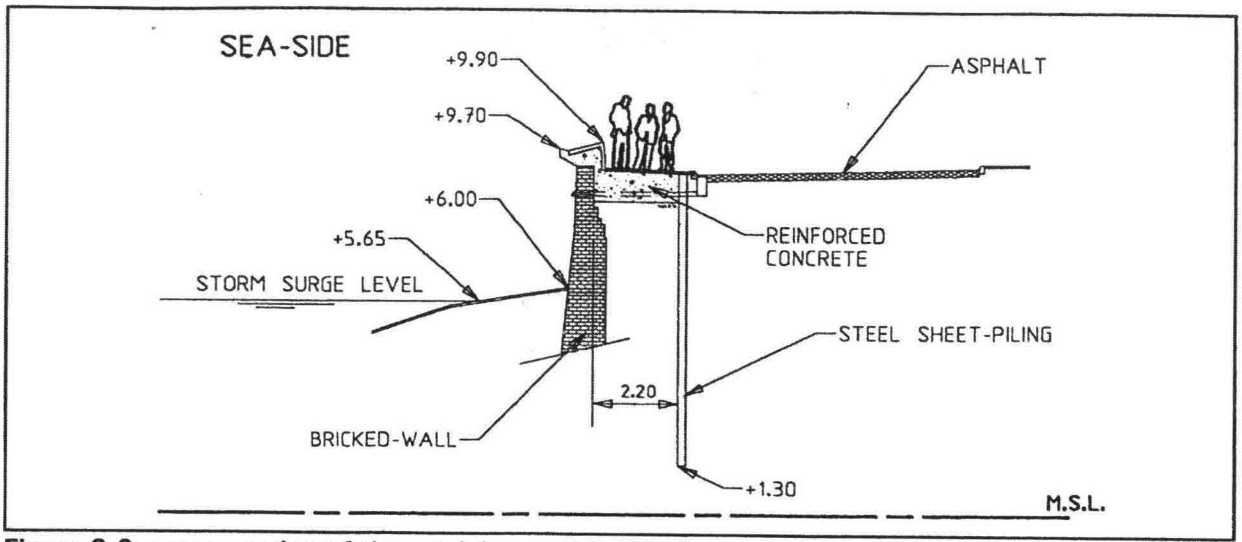

Figure 9.9: cross-section of the retaining wall in Vlissingen

- Creation of a mooring facility for ships. A quay-wall is usually placed in very calm water, without any current or wave action. The design parameters for a quay-wall are fully determined by the forces caused by the ship (bollard pull, etc.) and the handling equipment (crane forces). This item will not be discussed in this lecture.
- Lining along a canal. Sometimes not enough space is available for the construction of a sloping bank (for example when there are houses, industrial area or other types of vary valuable land). In these cases a vertical wall can be considered. Disadvantages are the considerable wave reflection, which hinders shipping, especially when both banks are vertical. In that case, with intensive traffic, the waterway is a dashing, turbulent pool. From an environmental point of view this type of canal-lining is not recommended because of the hindered interaction between land and water.

Vertical walls can be build as gravity structures (mainly in case of seawalls), as steel, concrete or wooden sheetpiles. Sometimes it is necessary to anchor construction (anchor-wall or grout-anchors). For the design of sheetpiles is referred to the lectures on sheetpiles (geotechnics).
Along minor canals, a hardwood-construction can be very effective and economic. Several types of shore protection structures are possible, like a wooden sheetpile, a palisade, piles with horizontal planking or piles with braided wooden strips. Instead of wood, bamboo can be a good replacement in some cases. Pine-wood cannot be used without conservation (conservation is not advisable from an environmental point of view).
In the Netherlands there is at this moment a strong movement against the use to tropical hardwood, because such use is said to stimulate the uncontrolled disforestation of the tropical rain-forest.
On the other hand, provided the exploitation of a forest is done in a sustainable way, the advantage of wood is that it is a renewable resource.
The discussed wooden structures can be used for waves up to approx. 50 cm (either caused by wind or by ships)

vertical walls

In the design of vertical walls, attacked by heavy waves, the determination of the wave impact is important. This can be done using the Goda-method (see also the lectures on vertical wall breakwaters). A routine to determine the forces is available in CRESS.
In case of a beach wall, one has also to realize that during a severe storm, an erosion hole is formed, just in front of the wall. This hole may reach to a considerable depth (several meters). The depth of this hole can be determined using a time-dependent dune-erosion program (e.g. Durosta, developed by Delft Hydraulics for the Dutch Technical Advisory-board for Waterdefences, [STEETZEL, 1993]). The foundation of the wall has to be deeper than the maximum depth of the scouring hole. See figure 9.10.

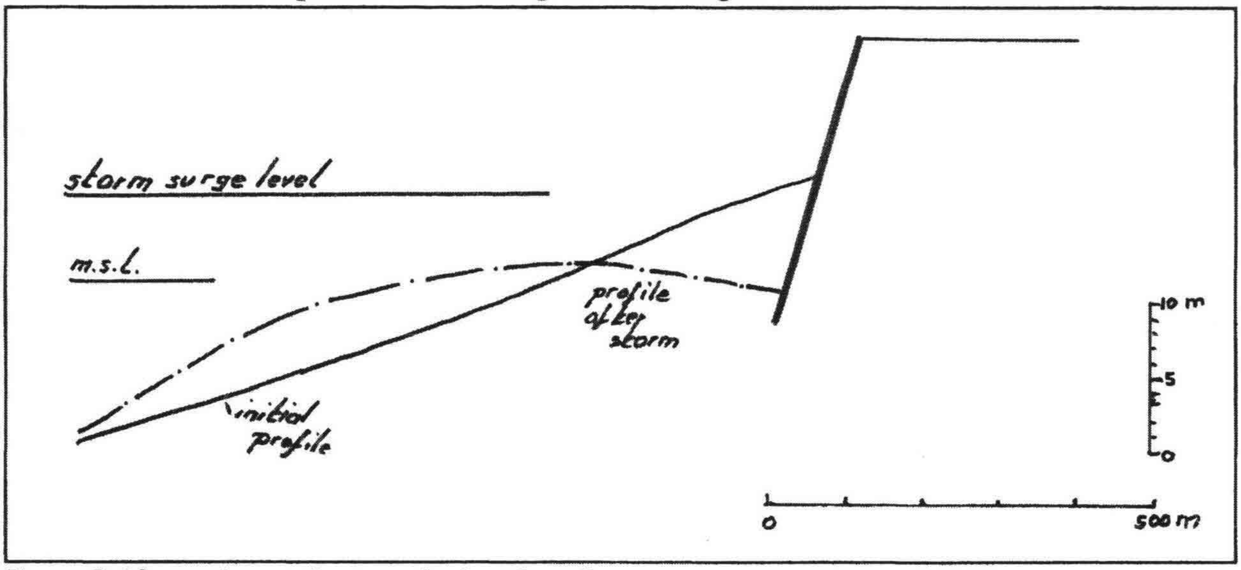

Figure 9.10: erosion at the toe of a beach wall

9.5 gabions

A gabion is a box or mattress-shaped container made out of hexagonal (or sometimes square) steel wire mesh strengthened by a selvage of heavier wire, and in some cases by mesh diaphragms which divide it into compartments, see figure. Assembled gabions are wired together in position and filled with quarried stone or coarse shingle to form a retaining or anti-erosion structure. The wire diameter varies but is typically 2-3 mm. The wire is usually galvanised or PVC-coated. PVC-coated wire has to be used for marine applications and for polluted conditions.
The durability of gabions depends on the chemical quality of the water and the presence of waterborne attrition agents. The influence of the pH on the loss of the galvanic zinc protection is small for pH values in the range 6-12 and there are examples of gabions with negligible loss over 15 years. Grouting of the stonefilled gabions or mattresses can give some protection to the wire mesh against abrasion and corrosion, but this depends on the type of grout and the amount used.

The dimensions of gabions vary, but typically range in length from 2 to 4 m (mattress, 6 m), with widths about 1 m (mattress, 2 m), and height 0.3-1.0 m (mattress, max. 0.3 m). The

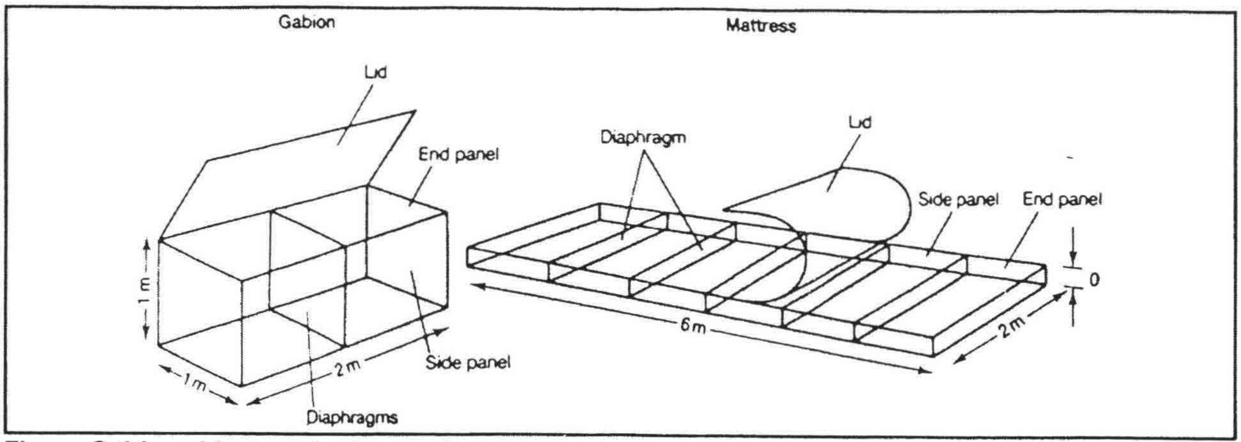

Figure 9.11: gabions and wire mesh mattresses

mesh size is typically 50-100 mm.

Gabions are traditionally used in earth retaining structures and in river projects. Recently they are also used in marine works.
The units are flexible and conform to changes in the ground surface due to settlement. Prefabricated gabions can be placed under water. Gabions can thus be used in a wide variety of marine works: groynes, dune and cliff protection, protection of pipelines and cables, and as toe protection. Mattress-shaped gabions are flexible and are therefore able to follow bed profiles both initially and after any scouring which may take place. Gabions can also be piled up to the form retaining walls or revetments. In order to prevent migration of solid through the structure they may be used in conjunction with geotextile filter layers.
An important aspect of a gabion structure is that vegetation can grow in the structure, which is highly advantageous for an environmentally friendly shoreline protection (see also next section).

Table: Stability limits of gabions related to flow velocity (acc. to AGOSTINI, 1988)

Type	Thickness (m)	Rock fill thickness: Size (mm)	Rock fill thickness: d_{50} (m)	Critical velocity (m/sec)	Limit velocity (m/sec)
Reno mattress	0,15 - 0,17	70 - 100	0,085	3,5	4,2
		70 - 150	0,110	4,2	4,5
	0,23 - 0,25	70 - 100	0,085	3,6	5,5
		70 - 150	0,120	4,5	6,1
	0,30	70 - 120	0,100	4,2	5,5
		100 - 150	0,125	5,0	6,4
Gabions	0,50	100 - 200	0,150	5,8	7,6
		120 - 250	0,190	6,4	8,0

In certain applications the gabion structure needs impermeability or weight to counter uplift. To give these characteristics, the stone is grouted with mastic or a cement-bound grouting. The weight of the structure can also be influenced by the choice of the density of the stone blocks with which the gabion or gabion-mattress is filled. However in cases where gabions have to be grouted one should question if one really needs the gabion as such. In case of grouting it is often also possible to place riprap without gabions in penetrate the riprap with asphalt or cement.

For the calculation of gabions in flow conditions one can use the design manuals of the manufacturers [AGOSTINI, 1985; AGOSTINI 1988]. In these design manuals also some formulas are given for stability under wave action. These formulas underestimate the required layer thickness. In the same manuals also a table for stability limits is given. The values in the table (see the following table) are much more reliable than the formulas given in these manuals. Therefore for the calculation of the stability of gabions in wave-action is referred to the table or to the computational method described in chapter 6 of these lecture notes. For both wave and current conditions a routine is available in the CRESS package.

Table: Stability limits of gabions subject to wave action (acc. to AGOSTINI, 1988)

Dimensions			Permeable soil						Inpermeable soil					
Reno mattresses Gabions	Rock material		1:1,5		1:2		1:3		1:1,5		1:2		1:3	
(m)	D (mm)	d_{50} (mm)	H_C (m)	H_L (m)	H_C (m)	H_L (m)	H_C (m)	H_L (m)	H_C (m)	H_L (m)	H_C (m)	H_L (m)	H_C (m)	H_L (m)
0,15-0,17	70-100	85	0,60	0,90	1,00	1,30	1,50	1,80	0,40	0,60	0,75	0,90	1,20	1,40
0,23-0,25	90-150	120	0,90	1,20	1,30	1,60	1,80	2,10	0,60	0,70	0,90	1,20	1,40	1,60
0,30	120-180	150	1,20	1,40	1,60	1,90	2,10	2,40	0,70	0,80	1,20	1,30	1,60	1,80
0,50	180-320	250	1,70	2,00	2,20	2,50	2,70	3,00	0,90	1,10	1,40	1,60	2,00	2,20

D = dimensions of the rock
d_{50} = average size of the rock
H_c = critical height (initial deformation or sliding in excess of 0.05 m)
H_L = maximum height (initial settlement or sliding in excess of 0.25 m)

9.6 detached shore protection

It is not always necessary to place the protection directly at the shoreline itself. In some cases a detached shoreline protection may be advantageous. This is the case in deeper water when some mooring facilities have to be created, but also in shallow water this can have significant benefits. Between the shoreline and the protection a shallow pool is created. This pool may have high natural values (see the following section of environmentally friendly shore protection) or can be useful for recreation (swimming, etc).
In other cases the quality of the subsoil can be the reason of constructing a detached protection.

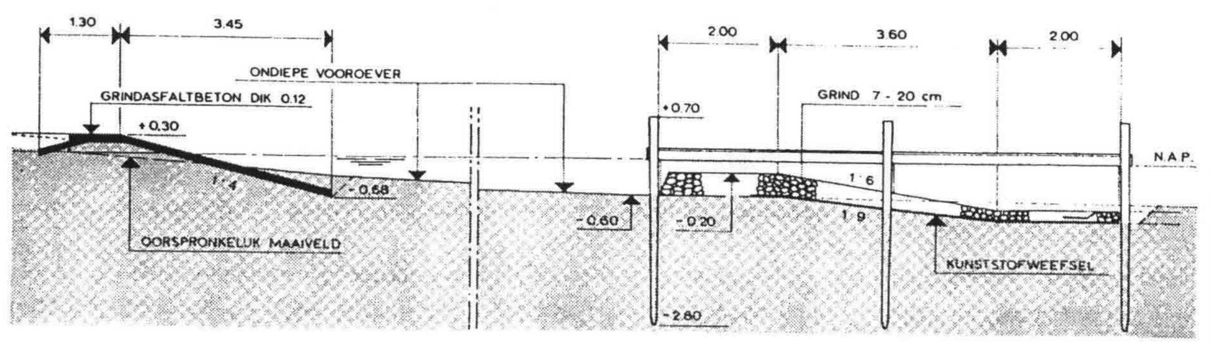

Figure 9.12: detached shoreline-protection along lake Grevelingen

In figure 9.12 an example is given of such a detached shoreline protection.
The design of a detached shoreline protection can be identical to a normal protection. In most cases such a protection will consist of riprap. In those cases the Van der Meer formula is very well applicable. The allowable damage can be quite high, because usually some changes in the shape of the profile of the protection is no problem at all. Also overtopping by storm waves is no problem. Therefore these constructions can be quite low.
In fact these protections very often can be designed using the design rules for berm-breakwaters.

9.7 environmentally friendly shore protection

In the Netherlands the policy on the design and management of banks bas been revised recently. The new approach is set forth in several policy documents. The management of a watercourse should be related to the management of connected waters and the functions of the adjacent land. What is required is integrated management of a water basin. This approach demands that a thorough analysis be made of the functions, management and general situation in the whole area concerned. Such analysis is of prime importance in the case of the design and management of banks. Banks form the boundary between water and land and therefore themselves consist of both elements. A functional analysis based solely on criteria deriving from hydraulic engineering is no longer adequate; other requirements and related criteria must also be taken into account.

In functional terms a watercourse bank may be variously described as:

- at once a dividing-line and a transition between water and land, each having its own functions;

- a means of protecting the land against the water: it affords protection against high water levels, waves and currents;
- a means of protecting the water against the land: it affords protection against cave-ins and the formation of shallows;
- a habitat for specific aquatic plants and animals, in this sense banks play an ecological role related to the general ecological function of the aquatic environment;
- an environment through which specific bank related plants may spread, and animals move, across the land, thus forming part of the ecological infrastructure;
- a scenic element affecting the overall recreational enjoyment of the landscape;
- an element in water-related activities, such as shipping, fishing and recreation.

There are many types of banks, which differ in respect of functions, morphology, soil composition, hydrological conditions and vegetation. In actual fact, no two banks are alike. But even on one long bank considerable variations can occur, or even be introduced, greatly enhancing the diversity of the scenery and its wildlife.
It is therefore ill-advised to seek one uniform type of environmentally friendly bank. It would be wrong to try to eradicate the diversity which has arisen, which is in may cases of great value. The term "environmentally friendly" was opted for in the new policy to emphasise the contrast with the usual civil engineering approach to bank protection. The qualification "multipurpose" would, perhaps, have been more to the point.

In general the embankments have to be designed in such a way that plants and animals can grow and reproduce, whereas the protection itself blends nicely into the surrounding environment. Thus, they do not only provide a habitat for plants and animals, but they are also attractive for recreational purposes. The environment-friendly structure should resist the occurring hydraulic forces induced by currents, win-waves and ship-induced water motions. In some situations the vegetation will have a protective function. In that case the stems and leaves are expected to improve the stability of the subsoil against erosion (passive role of the vegetation). However, in other situations the vegetation will only play an aesthetical role.

Several types of protections can be distinguished [BOETERS *et al.*, 1991]. The difference between the various types is determined by the function of the vegetation in relation to the hydraulic loads (aesthetical or protective role). In the figure the following types are shown schematically:

a) Embankments without any protection, resulting in the most natural environment for animals and plants. This type can only be applied in the case of low hydraulic loads and/or a very large available construction width.
b) Partially protected embankments, i.e. the lower part of the embankment is protected against (ship-induced) currents, the upper part is protected by the vegetation. This type is suitable for smaller canals and rivers in which relatively low hydraulic loads occur. The vegetation then has to be temporarily protected against incoming ship or wind waves during the first stage after construction.
c) Structures with a shallow pool behind a fixed protection. In the pool plants and animals find their habitat. The water in the pool is refreshed through gaps in the fixed protection. The fixed protection absorbs all the hydraulic loads. This type may be applied in the case of medium to heavy hydraulic loads and sufficient construction

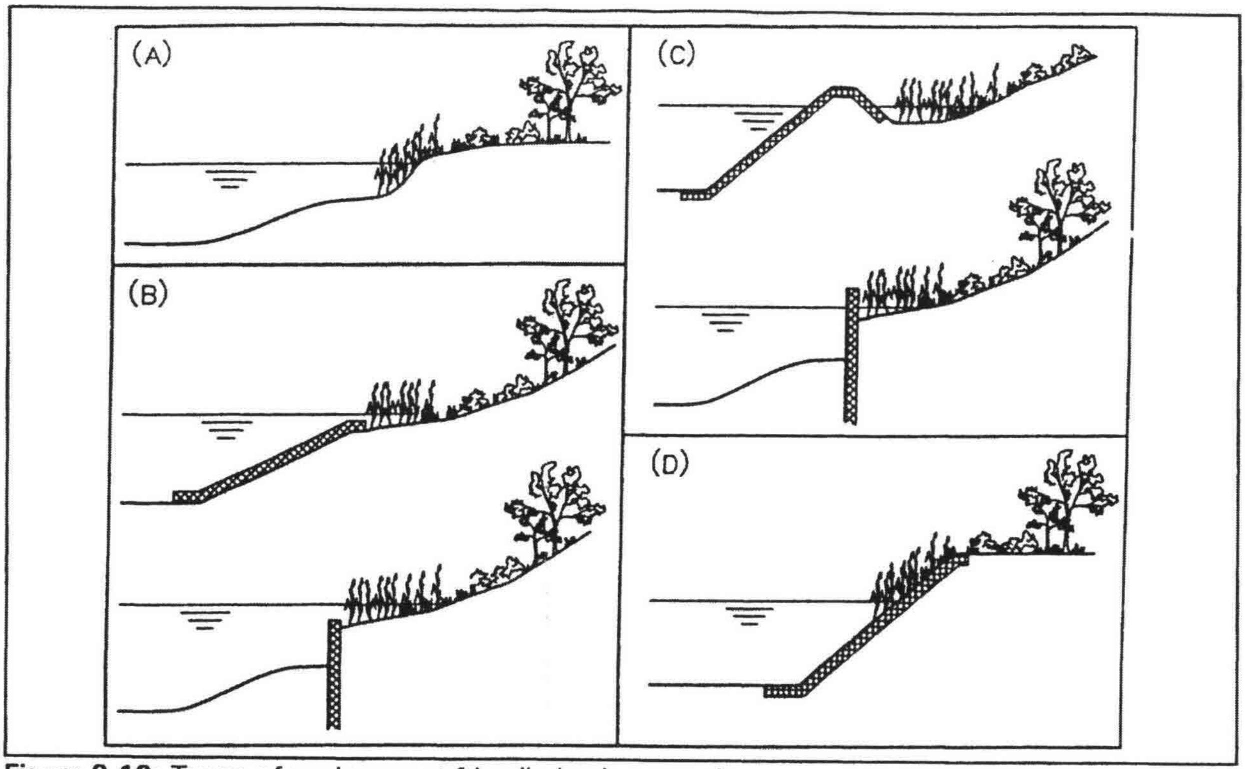

Figure 9.13: Types of environment-friendly bank protection

width.

d) Protections through which plants can grow and develop. For these structures block mattresses can be used, but also gabions and specially designed protections consisting of open riprap layers. This type of structure is suitable in the case of little available width and medium hydraulic loads.

The above examples are mainly for shorelines along canals. For shorelines along lakes and estuaries, identical types can be distinguished, but sometimes with different dimensions.

After damming a number of estuaries in the Netherlands, the embankments were no longer under influence of the forces of the tides, and started to erode. Of course, this erosion was very undesirable. because the shorelines had an important recreational and ecological function. On some places direct protection (type B and D) was placed, but from an ecological point of view this was not very successful. Therefore on many of these shorelines a detached shore protection was constructed. In fact a type c protection. For an example see the previous section on detached shore protection. In this environment there was hardly any growth of vegetation on the gravel dams. However, they are very important as resting place and nesting place for waders (birds), who feed in the shallow pond behind the gravel dams.

With several special technical means one may improve the ecological quality of an embankment. Several examples will be given in the following sections:

cellular blocks
A promising structure with respect to providing a natural habitat for plants and animals living along the water is the application of cellular concrete revetment blocks. The holes (cells) in the blocks are filled with clay, and allow vegetation to grow. The blocks are permeable enough to prevent the occurrence of high uplift pressures, while, on the other hand the individual blocks support each other against severe wave attack.The erosion of the cell fillings is decisive for the stability of the slope protection and the growth of the vegetation in the first stage. As soon as vegetation has established itself, erosion will be prevented by the roots and leaves of the plants.

blocks with special top
Several types of algae grow on rather porous stones. These algae do not grow on concrete or asphalt. Therefore ecologists oppose sometimes when a revetment of limestone is replaced by a revetment of concrete blocks, like Basalton. To overcome this problem at this moment blocks are available where the top layer consist of crushed limestone or with porous volcanic rock (Basalton Ecoblock). These blocks provide a good substratum for various types of algae.

the use of plants as wave damping device
Reed planted on a shallow berm is able to damp the wave action considerably. Both laboratory and prototype tests have been performed.
Stems of the plants and trees will reduce the exerted shear stresses on the bed, while the roots will increase the strength. About the last aspect little is known. Recent research with respect to the wave damping in reed stands has resulted in the following formula:

$$\frac{H_B}{H_i} = \exp\left(\frac{-cB}{\cos\beta}\right)$$

with:

- c = 0.05 for about 125 stems/m^2 and c= 0.12 for about 400 stems/m^2
- H_i = the incoming wave height,
- H_B = the wave height in the vegetation at B meters from the water,
- B = width of the vegetation
- β = angle of wave incidence.

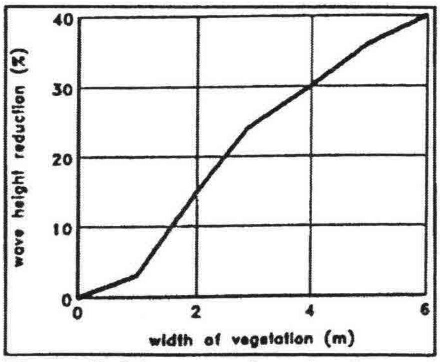

Figure 9.14: wave damping in reed (from Boeters *et al.*,1991)

The formula is valid for waterdepth between the plants up to 1.0 m and the plants emergine above the water. For relatively low waves the damping effect is indicated in the figure.
If a forest of willows or mangroves is present it is assumed that the same formula can be applied. However, the formula is not verified yet. This assumption seems correct, because the hydraulic roughness, viz. the Chezy-coefficient, has in both situations (reed or willows) nearly the same value (C_{reed} about 5 to 10 m$^{0.5}$/s and C_{trees} about 10 - 15 m$^{0.5}$/s) in spite of the quite different number of stems per unit area and stem thickness. [BOETERS *ET AL*, 1993]. For large Mangrove forests SCHIERECK & BOOIJ [1995] suggested a $C_{mangrove}$ of 20 m$^{0.5}$/s.

Because reed needs some time to grow, and also because the vegetation is not always present, sometimes a construction with a geotextile is used. Reed is planted, and the reed should grow trough the geotextile. Outdoor experiments in the Netherlands [REITSMA, 1995] indicated that the penetration through wovens is mainly determined by the geotextile weight (kgm^{-2}) and the penetration through non-wovens by the pore size of the geotextile. Vitality of a reed belt can be characterized by shoot length and shoot diameter. For both wovens and non-wovens these parameters are mainly determined by the tensile strength of the geotextiles. The following criteria were formulated to select geotextiles to be used in ecologically sound bank protection works:

wovens	weight $\leq$ 194 g/m^2
	$\ln(F_{2.5}) \leq 2.76$
non-wovens	pore size (O_{90}) $\geq$ 95 μm

(The value $F_{2.5}$ is the penetration force (N), ie. the force needed to press a metal cone with a diameter of 2.5 cm through the geotextile).

The same is also valid for mangroves. Mangroves have a considerable effect on the wave action along the coastline. But until now there are no clear design rules for mangroves.
An other problem with the application of mangroves as coastal protection is that no general techniques exist for the planting of mangroves. It is quite questionable whether it is possible to create a mangrove forest in front of an (eroding) coastline. Research in this field is still continuing.
In any case it is clear that cutting mangroves will have a negative effect on the action of the waves on the coastline. So one should always try to avoid cutting mangrove forests. It is not only bad for the ecology of the area, but also bad for the coastal protection.

the regulation of various recreational users

In some cases it is also possible to regulate the recreational use of the shorelines, by selection the appropriate shoreline-protection material. Along a natural reserve it was not desired that boaters landed there. Therefore a detached protection was used with relatively big, sharp-edged rock (the weight of the rock was many times the required weight from an hydraulic point of view). Because of this boaters did not approach this shoreline with their vulnerable polyester boats.
Using coarse gravel and fine rock may prevent swimmers to cross areas. And if a connection for swimmers over a detached dam is necessary, a simple wooden bridge can be provided.
At those places where windsurfing should be concentrated, one should provide easy access with for cars an a either a small sloping structure or a low, vertical wall.
In this way, one can regulate the use of the area by adapting the type of shoreline protection. Anticipation on human behaviour works usually better than regulations and placing signs.

10 Bottom protection

10.1 erosion problems

Because of current and wave action in the neighbourhood of a dike and/or revetment, scouring may occur. In general the stability of loose material under the influence of current is discussed in the lectures on sediment transport. The stability under influence of waves is discussed in the lectures on breakwaters. The stability of the bottom and the bottom protection behind a sill will be discussed in detail in the lectures on the closure on the closing of tidal inlets. In most cases scouring is not allowed, because it endangers the stability of the dike structure. Scouring is prevented by protection of the bottom with a bottom protection.

10.2 design of bottom protection

See also lecture notes on design of closures

Prefabricated mats for hydraulic constructions have been used extensively as a medium for combating bottom and embankment erosion. The range of mat types have increased considerably during the last decade. Formerly, use was made of handmade mats of natural materials such as reed, twigs/branches (willow), rope, quarry stone and rubble. Today, geotextiles, bituminous and cement-bonded materials, steel cables and wire gauze are used as well as the more traditional materials.

Functions and principles

A distinction must be made between mattresses used for the protection of continually submerged beds and foreshore and mats employed for slope or front face protection of embankments. Bottom protection mats are concerned with:

- The mitigation of groundwater-flow in the underlying seabed to prevent horizontal transport and erosion;
- Prevention of piping of soil particles;
- Assisting in the overall geotechnical stability of a larger structure;
- Offering resistance to loads imposed by anchors, trawler-boards, etc. and thereby protecting pipes and cable-lines;
- Providing a protection system with the flexibility to allow for anticipated settlements.

Mattresses for embankment retention have similar functions, i.e:

- Prevention of soil transport;
- Resistance to wave action;
- Provision of flexibility to allow for settlement.

Additionally, they may be required to:

- Not trap refuse; and
- Support vegetative growth.

Mats are manufactured from the following elements:

1. *The carrier:* Fabric, gauze, cables and bundlings by which the necessary strength is obtained for its manufacture, transport and installation.

2. *The Filter:* A geotextile which satisfies the conditions with respect to permeability, soil density, strength, etc. In some embankment-mat constructions the geotextile is applied separately.
3. *The Ballast:* Concrete blocks, packaged sand, gravel, stone or asphalt necessary for the stability of the protection. Quarry stone can be used to serve as a material to sink the mat.
4. *The Connections:* Pegs, wires, etc. by which the ballast material is fixed to the filter and the carrier materials.

In addition to the mat proper, a further deposit of gravel, aggregates, quarry stone, slag, etc. may be necessary when the ballast material stability in itself is insufficient or when extra protection is demanded.

The carrier and the filter can often be combined in the form of a single geotextile with the necessary mechanical and hydraulic properties. Sometimes the filter is combined with the ballast, which is then designed as a granular filter.

In the past mattresses were build only using natural materials (in Europe with brushwood an reed, in the Tropics with bamboo and reed). Because the construction of these natural mattresses is quite labour intensive, in Europe a more capital intensive alternative has been looked for. Using a geotextile the mattress can be made with less manpower, and is therefore in the industrialised countries cheaper than the natural mattress.

10.3 mattresses with only brushwood or bamboo

Fascine mats are typically up to 100 m long and 16-20 m wide. Historically, these mats were manufactured from osiers which were clamped between the so-called 'raster' of bundles of twigs/branches bound together to a circumference of 0.3 m. These bundles are called "wiepen".
'Plugs' were driven into the crossing-points of the bundles to serve as anchor points for towing and sinking the mats (for bottom protection) or collar-pieces (for foreshore protection). Because of the destruction of the osiers by pile-worms attack, a reed layer was incorporated into the mats.
These fascine mats are usually made on embankments in tidal areas or on special slipways. The construction on of the mattress must takes place in the dry, so that the mattress must have some way of being floated.

In tidal regions, the mattress is, as a rule, made on a piece of ground which remains dry for a sufficiently long time. The mattress is built up on this in one tide, after which the mattress floats at high water. If it is not possible to finish in one tide, as in the case of big mattresses, then, in the available time, a part of a mattress is completed to the stage of fixing the upper grillage.
The wiepen are left projecting 2-3 m out of this, so that in the following low-water period of work they may be well built in and the mattress may be completed. If the work is on the upper reaches of rivers or on an inland waterway and no use can be made of ebb and flood, then the mattress is made on a primitive slipway of wooden planks from which, when it is completed, it is pulled into the water (for instance by tugboats).

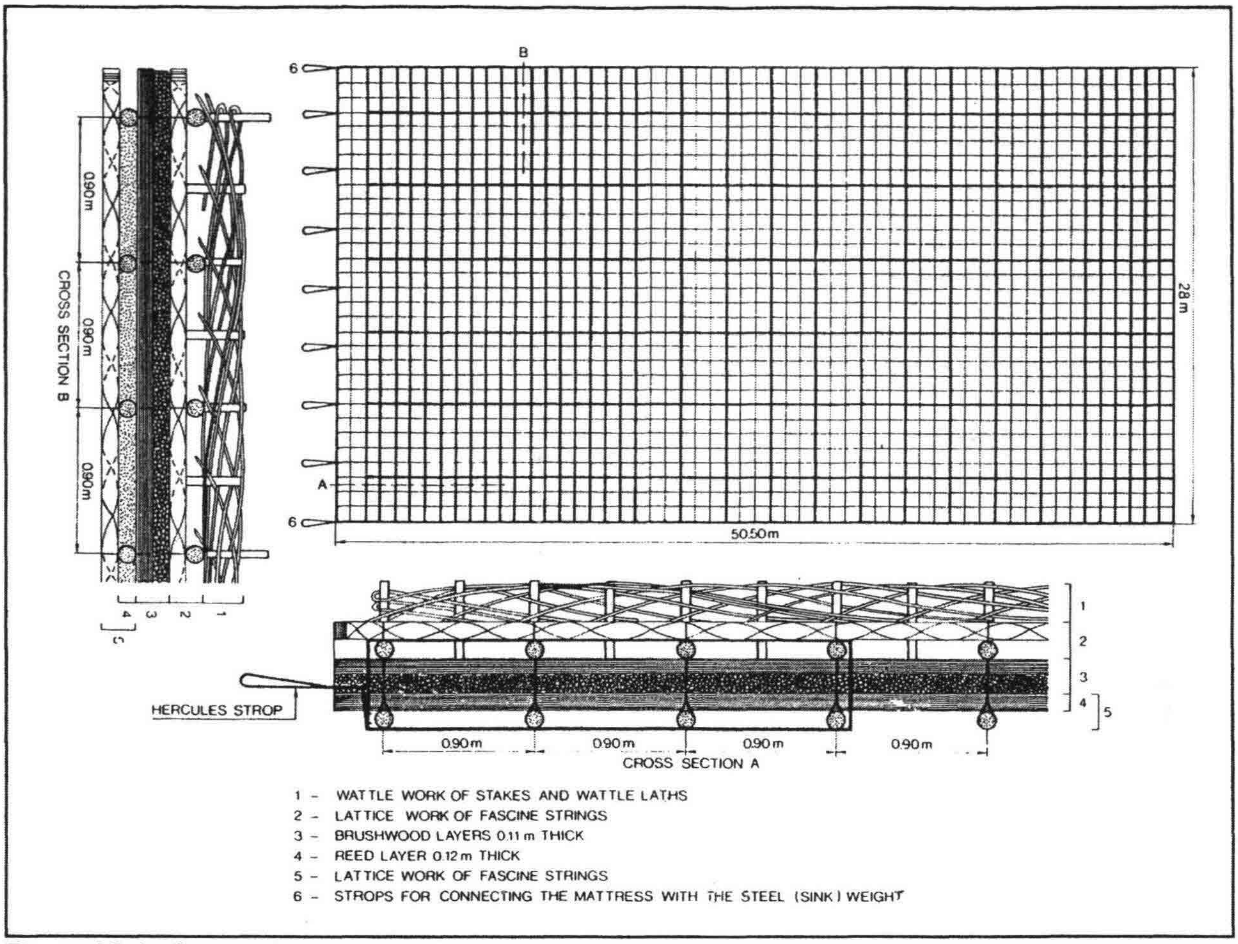

Figure 10.1: Conventional mattress

If the mattress constructions are extensive, somewhat more money may be spent on the "building place" for the mattresses. For the making of a big part of the mattresses for the Enclosure Dam of the Zuiderzee a sort of small "dry dock" was made inside a soil dam with closable opening. The closed dock is pumped dry, the mattress is constructed, the water allowed to rise, the opening is made free again and the mattress is floated out.

Normally, a sunken mattress consists of a filling of 3 layers of brushwood shut up between a lower and upper grillage of fascine poles (wiepen). To make this construction, first the lowest wiepen of the lower grillage are laid out on the site on 0.90 m centres. Perpendicular to these the upper wiepen of the lower grillage are laid such that square compartments are formed. At each intersection a lashing rope is bound round each of the wiepen crossing each other, which is long enough to be bound round the wiepen crossing each other in the upper grillage.

To prevent losing the rope in the brushwood filling between the two grillages a stake is driven into each crossing of the lower grillage, to the upper end of which the end of the lashing rope is made fast. This stake also gives the advantage that it indicates the position of the intersection of the lower grillage, which is necessary in making the upper grillage as these intersection are no longer visible because of the brushwood lying on top of them.

When the brushwood filling has been placed and the upper grillage laid, and the rope bound round the intersection of the upper grillage, the stake has then done its job and is removed again. The lowest layer of the brushwood is laid perpendicularly on the lower wiepen of the lower grillage (thus in between the upper wiepen of the lower grillage). In each compartment 3 bundles brush wood are placed.

On the lowest layer the intermediate layer is placed. Next perpendicular to the intermediate layer, the third layer is laid. In the third layer all faggots on the perimeter are laid with the trunk end outwards to strength the edge of the mattress. The lower wiepen of the upper grillage are then laid perpendicularly on the third layer. The above mentioned lashing ropes are bound round the intersections of the wiepen of the upper grillage and the brushwood filling is simultaneously pressed together as firmly as possible. Wattle hurdling is set up on the completed upper grillage; on the two outermost wiepen and then on alternate wiepen. In this, the hurdles are first woven on the lower wiepen and then, over these, those on the upper wiepen.

The space between the hurdles on the two outermost wiepen is called the gangway. The inner hurdle of the gangway is woven over the top of all the other hurdles. To prevent the mattress being fixed to the site by the point of the hurdling stakes, they are only driven in during transport of the mattress.

Naturally, care must be taken that the stakes still do not protrude through the mattress, because the mattress would not then lie close along the bottom and water would be able to flow underneath it which could result in scour. In connection with the transport of the mattress from the building yard to the point of sinking, which usually takes place by means of a tugboat, towing bollards should be made on the mattress which come up about 0.50 m above the mattress and round which a towrope can be laid, These bollards consist of a heavy stake which is driven in vertically, with six stakes round it in an oblique position.

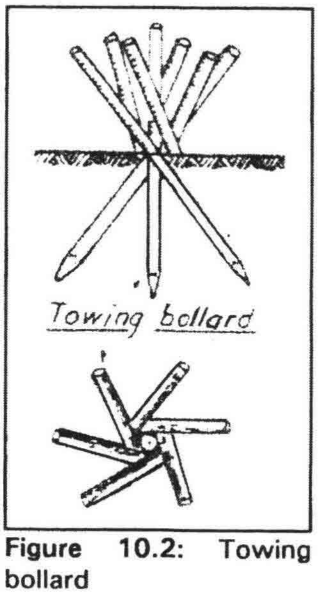

Figure 10.2: Towing bollard

The oblique stakes stand wide apart at their upper end to prevent the towrope from slipping off and to form a well-fitting whole round the centre stake, bollards may not be placed too near the outer perimeter of the mattress because of the danger of them tearing out. Usually 4.50 m is taken as the minimum distance for this. The spacing of the bollards is 30 - 40 m and depends on the strength of the current. In order to be able to move freely round the bollards, the hurdles are omitted just there, that is on three successive wiepen in the length and on five in the breadth. At the intersection of the wiepen where the bollard is placed, the wiepen of lower and upper grillage are strengthened by firmly binding to them four (in less important cases, two) special strong faggots.

mattresses from natural material

In order to be able to support the mattress from the ballast vessels during loading, anchor straps are attached. These are 1.10 m long, have a perimeter of 7 cm and are fastened by a loop to the intersection of the second wiep of the lower grillage so that their position corresponds with the position of the bollards on the ballast vessels i.e. about every 10 m. At the upper end of every anchor strap, a loop is made with a steel eye through which the sinking rope is threaded, by which the mattress is supported from the ballast vessels before loading.

The shape of the mattresses must be simple (rectangle or trapezium). In fixing the dimensions of a mattress as component of the total construction, it is necessary to take into account the fact that it is not possible to lay the mattresses precisely alongside one another The width of the remaining gaps may be estimated at 0.5 m to 1/10 of the depth. Care is taken to get as few gaps as possible, parallel to the bank because the current also running parallel to the bank would be able to scour the bottom in the region of such a gap, with consequent damage to the mattresses and bank. In practice, it appears that a mattress with a width of 20 m is easiest to manage. The length may be increased almost without limit. Mattresses of 200 m length have been sunk without difficulty.

As far as the composition of the mattress is concerned, the above description is sometimes departed from. In mattress constructions of less importance, two instead of three layers of brushwood are applied between the grillages. In seawater, 1 or 2 layers of brushwood are replaced by reed layers in order to diminish attack by marine borer. These mattresses require a heavier loading because they have a greater buoyancy. This greater buoyancy can also prove an advantage if sinking must be in the sea in places where work can be carried out only in calm weather, so that sometimes the finished mattress must lie in wait before linking may proceed.

A mattress completely of brushwood would then be too heavy which is inconvenient for transport and also for sinking because one can then no longer move freely over the mattress for the handling of tow ropes and the placing of stone.

If several mattresses have to be laid on top of one another to form a dam, for example in the closing of a dike, then no hurdles should be placed on those parts which are to be covered by the subsequent one and a smaller quantity of stone (approx. 0.4 tons/m). In this case the mattresses can be made thicker.

For the construction of a normal mattress with 3 layers of brushwood. each 100 m is needed:

- 2100 ordinary brushwood faggots
- 17 bundles of wiep tiers
- 25 bundles stakes
- 25 bundles poles
- 18 kg lashing rope.

For taking the anchors out and placing of ballast 4 - 6 men per 100 m^2 mattress are necessary. The average rate of work of a fascine worker amounts to 40 - 45 m^2 per week, including the unloading of the brushwood and the sinking.

Instead of brushwood, bamboo can be used without any problem.

For transporting, a barge loaded with ballast stone is set crosswise along the short side of the mattress. The towropes from the towboat are fixed to the bollards on one side of this barge, while the bollards on the other side of the barge are connected to the bollards on the mattress. If possible, transport is in the direction of the current. In towing big mattresses, a second tug is employed to push the mattresses from behind. When the mattress has arrived at the place indicated by marks, anchors are 100-300 m long. In order to be able to pull up the anchors later, buoys are fastened to them.

The finished mat or collar-piece is then towed to its destination and sunk using quarry stones which were placed in position by hand, with the uppermost raster-work of bundles serving to maintain the stone in place during sinking. Nowadays very often mechanical sone dumping vessels are used. After sinking, more and heavier stones are discharged onto the mat if this was found to be necessary.

10.4 mattresses with geotextile

Modern mats are made in accordance with the same principles. The twigs/ branches and reed are often replaced by a geotextile into which loops are woven and to which the raster-work of bundles are affixed. A reed mat may be affixed onto the geotextile in order to prevent damage to the cloth during the discharge of stones onto the mat. The geotextile extends out beyond the raster-work with the aid of lath outriggers so that, on sinking, the mats overlap. Sometimes a double raster-work is applied to give the mats a greater rigidity and more edge support to the quarry stone load. The mat construction can serve as a protection for the bottom, a foreshore and even as an embankment protector. In the last case the mat is hauled up against the embankment itself.

Modern alternatives to fascine mats include mats where the quarried stone is replaced by open-stone asphalt reinforced with steel mesh, or by 'sausages' of cloth or gauze filled with a ballast material such as sand, gravel, bituminous or cement-bound mixtures.

10.5 placing of bottom protection

In the past conventional mattresses were sunk along traditional lines. This entailed transporting the mattress well before the turn of the tide to the place where it was to come to rest, where it was then anchored. The actual sinking to the bottom was effected by mooring a number of barges loaded wit 10/60 kg stone around the mattress, the stones then being transferred to the mattress by hand. This method of application was used for mattresses on steep slopes whose longitudinal axis was at right angles to the direction of the flow.

The anchor cables are hauled in until the mattress lies a little upstream of its final position. How much depends on the local circumstances and is a question of experience and practical insight. When the mattress lies "on line", barges loaded with ballast stone are brought to lie around it. These barges are likewise anchored and fastened to one another with ropes over the top of the mattress. Through the anchor straps of the mattress, the sinking ropes are

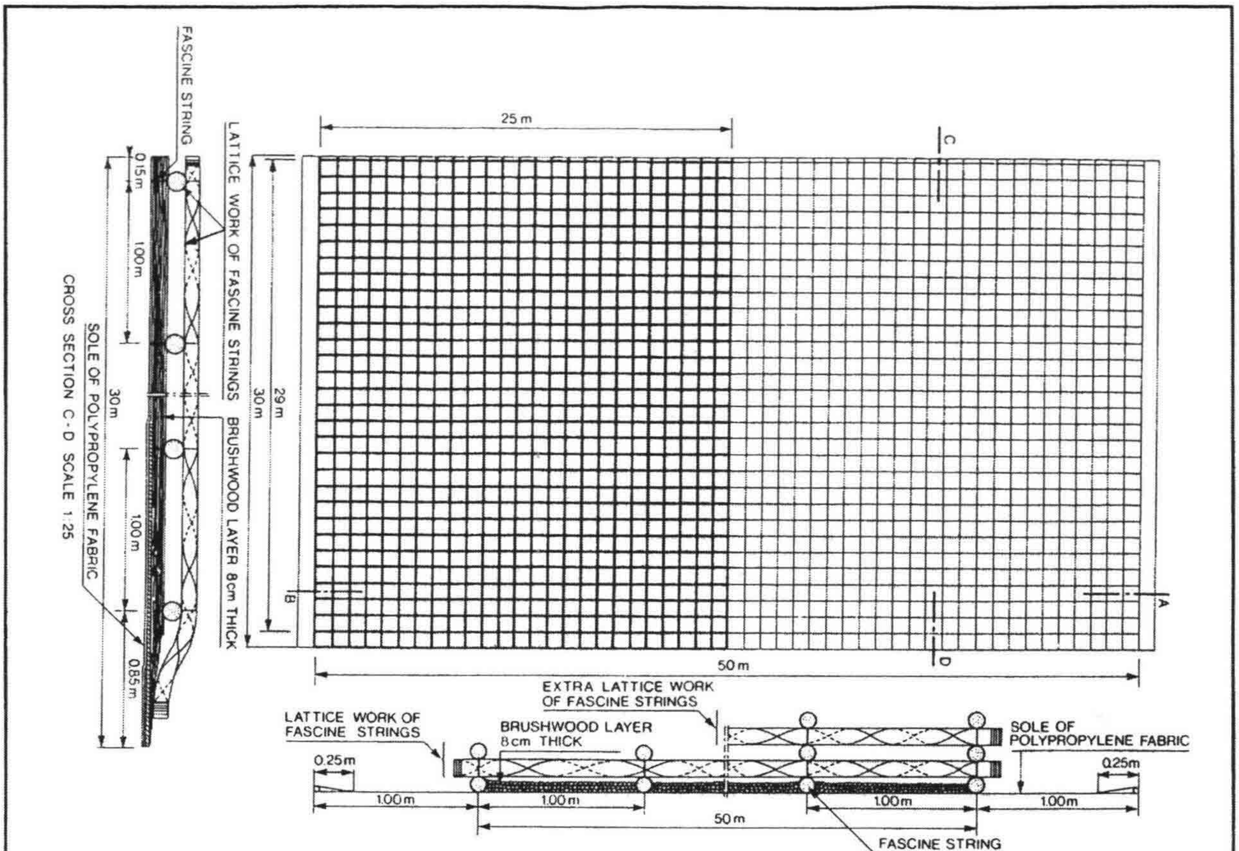

Figure 10.3: modern polypropylene mattress

placed and these are fixed to the bollards of the barges. Finally, buoys are fastened to the corners of the mattress. Loading can then be started. This takes place from the middle outwards. The workers form rows on planks laid out on the mattress and pass the stone from the barges from one to the next. The last man of each row throws the stone onto the mattress so that the ballast is uniformly distributed.

The mattress sinks slowly and begins to pull more and more heavily on the sinking ropes. The time must be so allocated that, in the case of tides, loading is finished by the time of slack water. Loading is now carried on until only the gangways project above water. The mattress then hangs like a bag between the ballast barges. When the time of slack water is approaching, the master of the works gives the command "In the gangways", and the gangways are rapidly filled up with stone.

The workers arrange themselves along the sinking ropes and at a command, let all of these of simultaneously. In order to prevent any calamity by the sticking of one or more sinking rope, every worker has a sharp knife with him to cut through any sticking sinking rope. If the operation runs successfully, the middle of the mattress touches the bed first and thereafter the sides. In this way the mattress will fit the bed well at every point. If the work is in flowing water, such as occurs in the Dutch upper rivers, the mattress rotates around the upstream anchorage during sinking. In some cases, even in the tidal district the mattress will be sunk preferably, in a (weak) current in a fixed direction. The mattress is then loaded beginning at the upstream end and the current ensures that the mattress does not capsize.

A completely different method of sinking was applied in the outlet sluices in the Afsluitdijk at den Oever, where the depth was very great (up to 30 m) and where no current was present. The mattress, which had very big dimensions, was brought slightly loaded with 40 kg/m^2 , above the place where it had to lie on the bed. As the brushwood gradually became saturated with water the mattress sank down slowly after some time along guides placed for this purpose. After the mattress had been sunk, the barges where drawn together above the mattress by means of the mooring ropes and the stone tipped until the load on the mattress reached 300 kg per m^2 .

After sinking and loading, the position of the mattress is observed at still water by pulling the buoy ropes tight which were fastened to the mattress and measuring their position with respect to the sights set out on the shore. The amount of stone necessary to make the mattress sink depends on the time that the mattress has laid in the water because the buoyancy gradually decreases.

The necessary amount of stone varies therefore, between 100 and 250 kg/m^2 while the size of the second loading depends on the circumstances and varies between 250 and 1000 kg per m^2 mattress. The loading stone which must be passed from hand to hand has a composition of 10 - 40 kg. For the second loading heavier stones are used, such as: 10 - 80 kg (light stone) and 80 - 200 kg (heavy stone). The stone sizes are chosen according to the amount of wave and current attack which is to be expected. The tipped stone consists of basalt (high specific gravity) or types of limestone from Belgium.

After 1967 a new placing method was employed. With this method the stones were no longer transferred to the mattress by hand but by means of special stone-dumping vessels known as stone dumpers, which released their load onto the mattress mechanically. In order to enable the stone dumper to transfer its load to a mattress, the latter had first of all to pressed down to the bottom in some manner. This is done by attaching a weight to one of the narrow ends of the mattress (the head). See figure 10.4. This is a hollow steel pipe which can be filled with water, so that the edge of the mattress is carried down to the bottom. This operation is carried out at the turn of the tide, with the mattress ending up more or less vertical in the water. In order to create sufficient space between the mattress and the moored pontoon rom which the steel pipe was lowered down, the angle of the mattress had first to be reduced. This was initially done

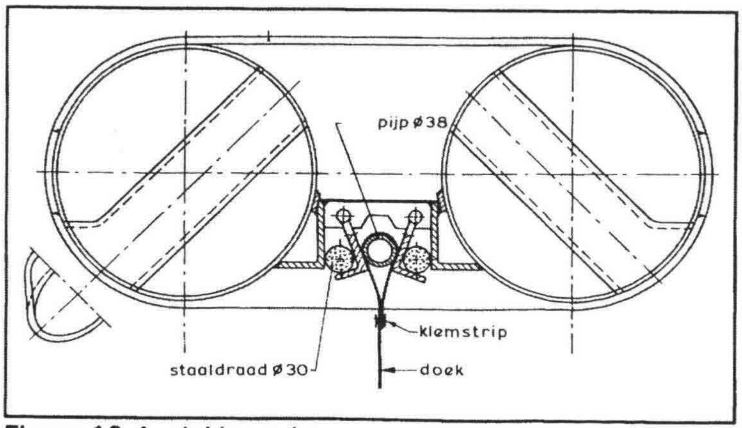

Figure 10.4: sinking tube

by making use of the current pressure. Later, tensile force was also exerted on the tail of the mattress ("stretched" sinking). The advantage of this method is that it cuts the time required for the sinking manoeuvre and that it also enabled mattresses to be sunk along their longitudinal axis which was not in the direction of flow, e.g. for sinking along the natural slopes.
After finishing the sinking operation, the pipe ø 38 mm is pulled away, which loosens the pipe from the bottom protection. Then the pipe is filled with compressed air, so it regains it floating power and returns to the surface of the water.
After this operation the sinking tube can be re-used for the next sinking procedure.
After sinking the mattress it usually is covered with a secondary layer of somewhat heavier stone to give is more stability under design conditions. In figure 10.5 the whole sinking procedure is given in four steps.
These procedures can also be carried out at sea with moderate waves, when the appropriate equipment is available.

In case of a shortage of stones, also sandbags can be used. This was done in a bottom protection work in Bangladesh. There the mattress was made from bamboo with a spacing of 0.75 x 0.90 m, three layers of palm-leaves laid crosswise and a top grating of bamboo. The total thickness of the palm-leave layers was 5 cm, this being sufficient to ensure an adequate bearing capacity of the ballast and also a correct filter function. The palm-leaves are called locally goalpata, being half-split leaves of a special dwarf palm growing in the mangrove forest and used as roofing material in the South of Bangladesh.
The cross-connections of the grating were made with jute rope, the rope used for the bottom grating being also used for the top grating to connect both gratings together. The tensile strength of the rope was about 100 kg. About 50 labourers were required to execute this work.

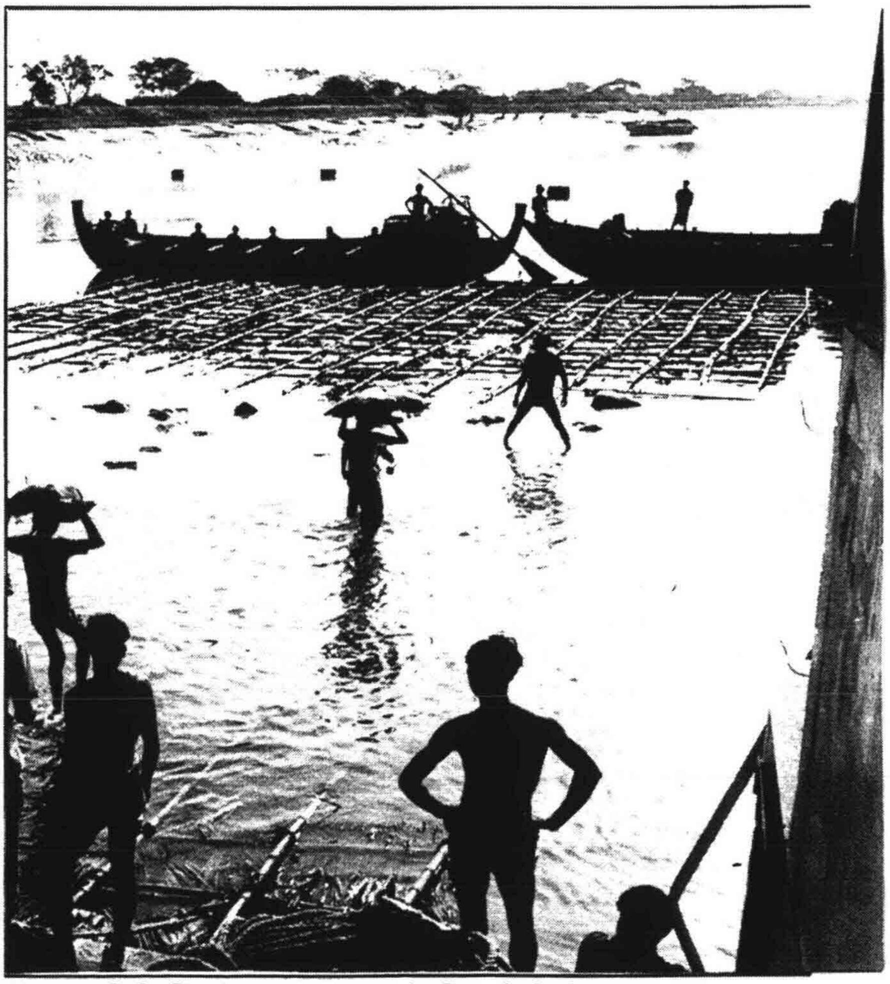

Figure 10.6: Sinking a mattress in Bangladesh

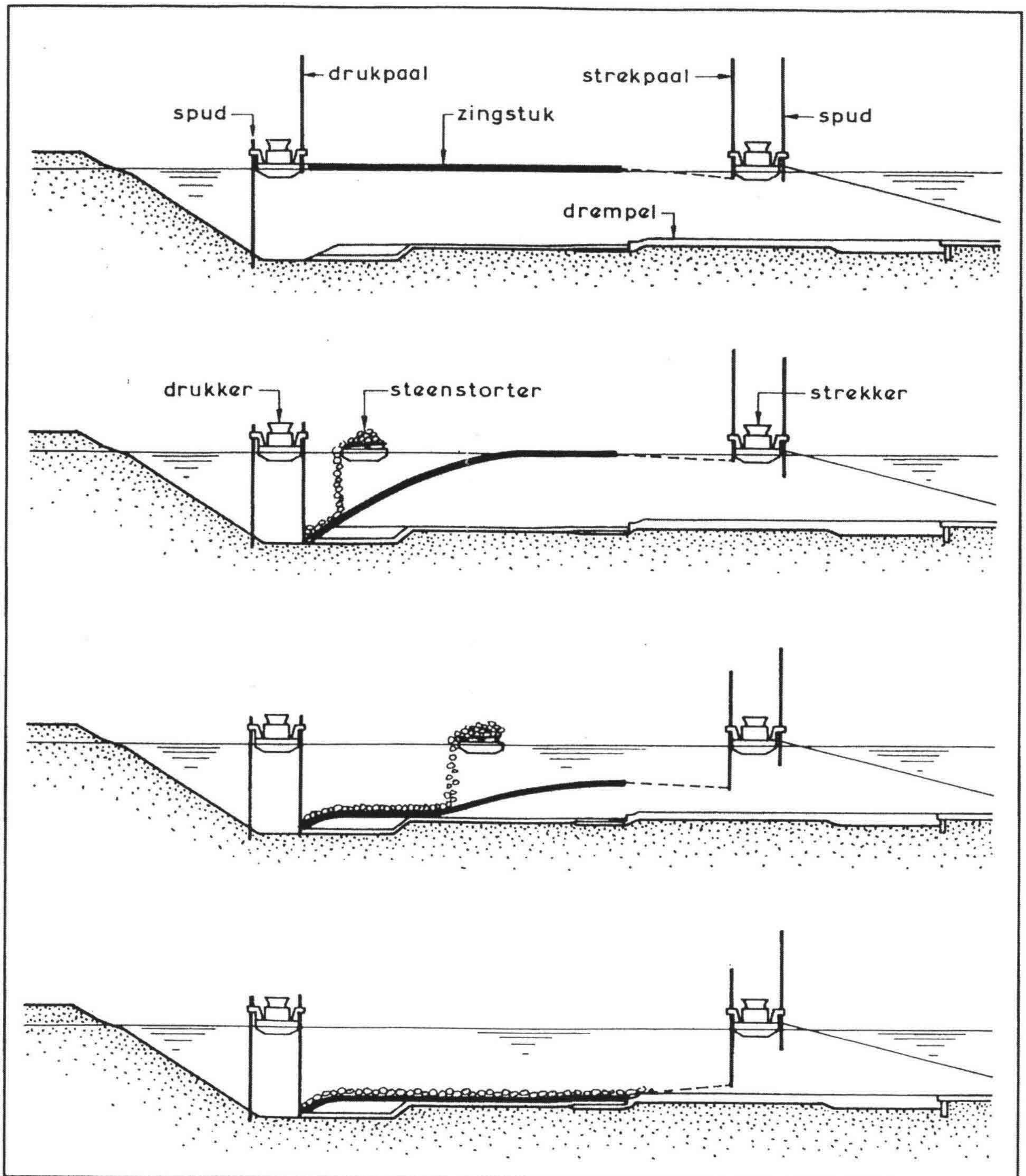

Figure 10.5: The sinking procedure

A practical problem with this type of mattress is that the floating capacity of the mattress is quite limited. Therefore transportation can only be done during calm weather and with extra floating capacity. The grid spacing (i.e. using more bamboo) was not possible, because then the mattress would become to stiff for sinking down.

placing of mattresses

The sinking of a mattress was achieved by ballasting with 4500 jute bags of 50 kg, filled with clay. Clay is the only material available in the south of Bangladesh. River boulders are found in the North and North-East, but the transport cost to the south are too high.

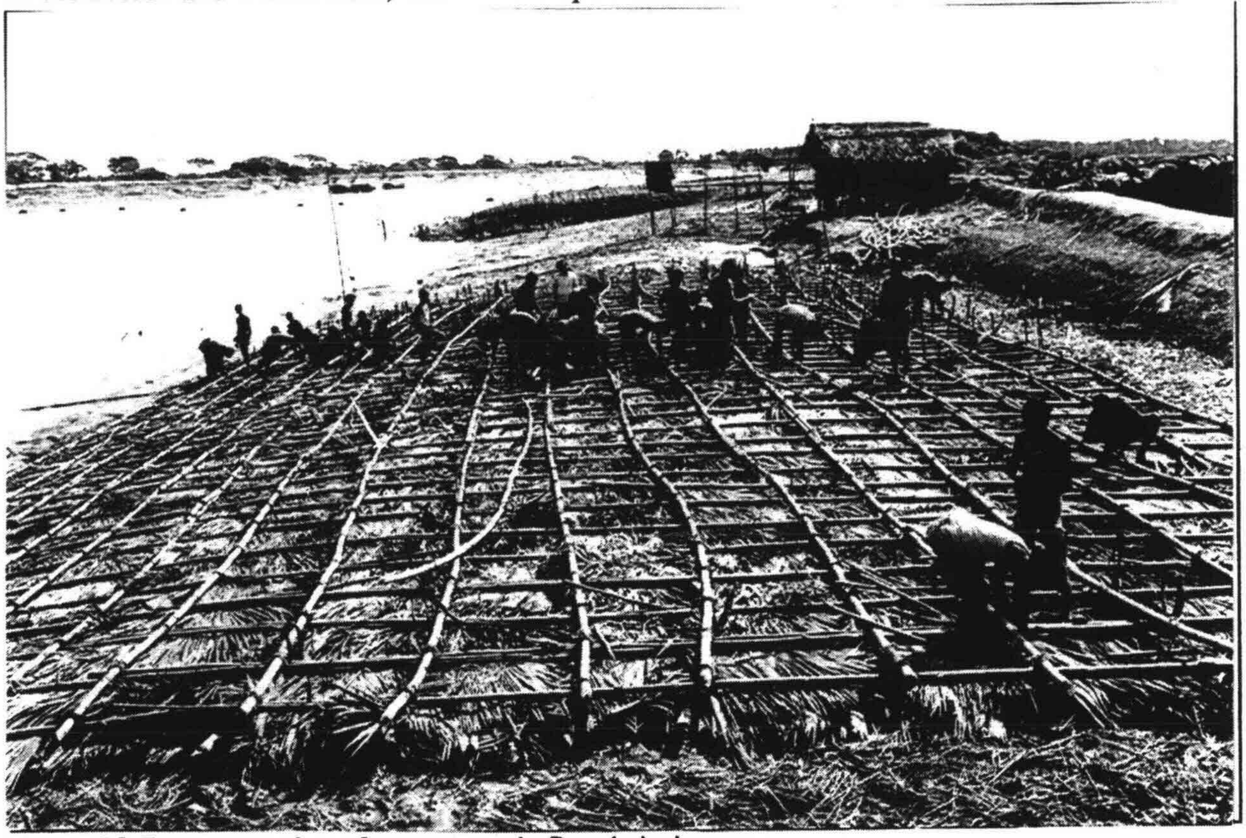

Figure 10.7: construction of a mattress in Bangladesh

An area of 7200 m^2 of channel bed was protected against scouring by 16 mattresses of 15 x 30 m. A mattress was attached to a flat top barge loaded with the jute bags and floated by the current to the sinking position. The mattress and the barge were then moored in the correct position using locally made jute anchor ropes with a diameter of 38 mm. The mattress withstood currents up to 3.5 m/sec without damage.
Instead of palm-leaves and full bamboo, also a protection made of split bamboo can be used. The appropriate choice of the materials depends very much on the availability of local materials. Importing geotextiles is usually quite expensive.
Especially when the protection is always under water, the life-time of the structure, when using natural materials, can be very long. In the Netherlands brushwood bottom protections, placed 1.5 century ago, are still in a very good condition, as long as they stay under water.

11 Special aspects of dikes

11.1 Gaps

In some cases it is necessary to make a gap in a dike, for example to allow traffic crossing the dike at locations where it is not possible to build a road over the dike, because of shortage of space (this happens often in urban areas).

Figure 11.1: Gap in a streetcar crossing in the river-levees of New Orleans

Figure 11.1 shows an example of such a gap in New Orleans.
Technically it is not really a problem to design such gap structures. The main problems are in the field of maintenance and de probability of failure.
Because the doors in such a structure are seldom used, the moving mechanisms tend to corrode, plants are growing in the grooves, etc. No-one is really worried about this, because the door is "never" used. But when the high water comes, also the problems start. Because of plants, etc., it will be quite impossible to close the doors of fig.11.1.
The weakest point is usually the man who has to close the gap.
In a recent analysis in the Netherlands it was decided that a reasonable assumption for human risk were as follows:

- probability that the operator is not present at the correct moment 0.1 %.
- probability that inhabitants warn the authorities that the operator is not there and that the situation becomes critical: 99 %.
- probability that the doors fail: 0.1 %
- probability that the emergency closure (with wooden beams in grooves) fails, given a failure of the doors: 1 %.

In the case investigated the threshold of the gap was 3.37 m above mean sea level. The probability that the waterlevel in the river becomes higher that 3.37, but lower than 5.00 m (= crest of adjacent dikes) is 3.10^{-2} per year.

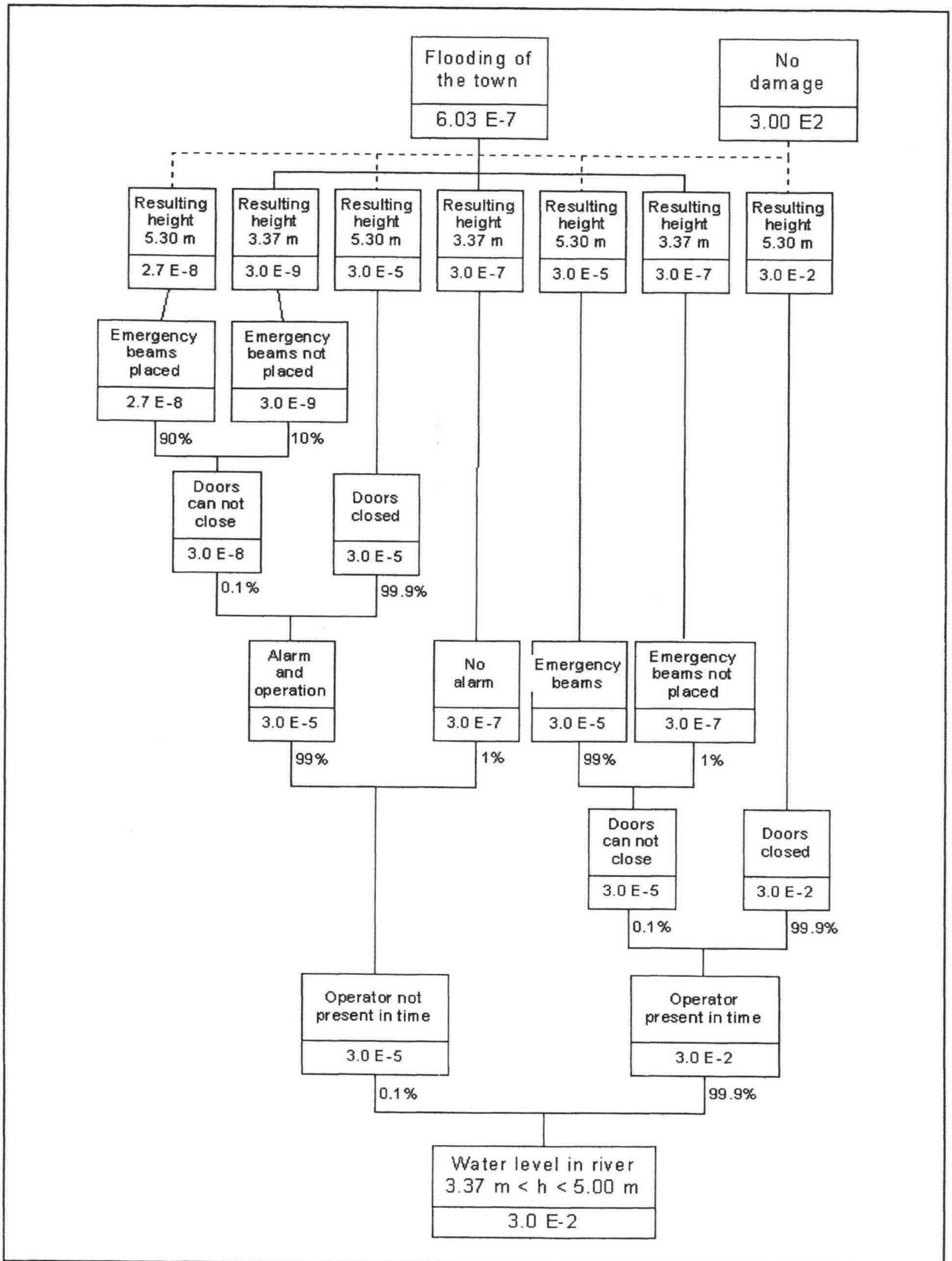

Figure 11.2: Events tree for failure of a Gap (Schoonhoven, from TNO report B-90-334, bt Vrouwenvelder and Van der Meer)

This leads to a probability of flooding of the town, because of failure of the gap closing structure of 3.10^{-7}. See the tree in figure 11.2.
You see that one such a gap does not contribute much to the probability of flooding of the dike circle. However, when the area is quite big, (e.g. a total dike length of 50 km), and every km there is one gap, then the contribution to the failure is $50 * 3.\ 10^{-7} = 1.5 * 10^{-5}$. When the total allowed probability of failure is in the order of $10 * 10^{-5}$, then the contribution of the gaps is already 15 %.

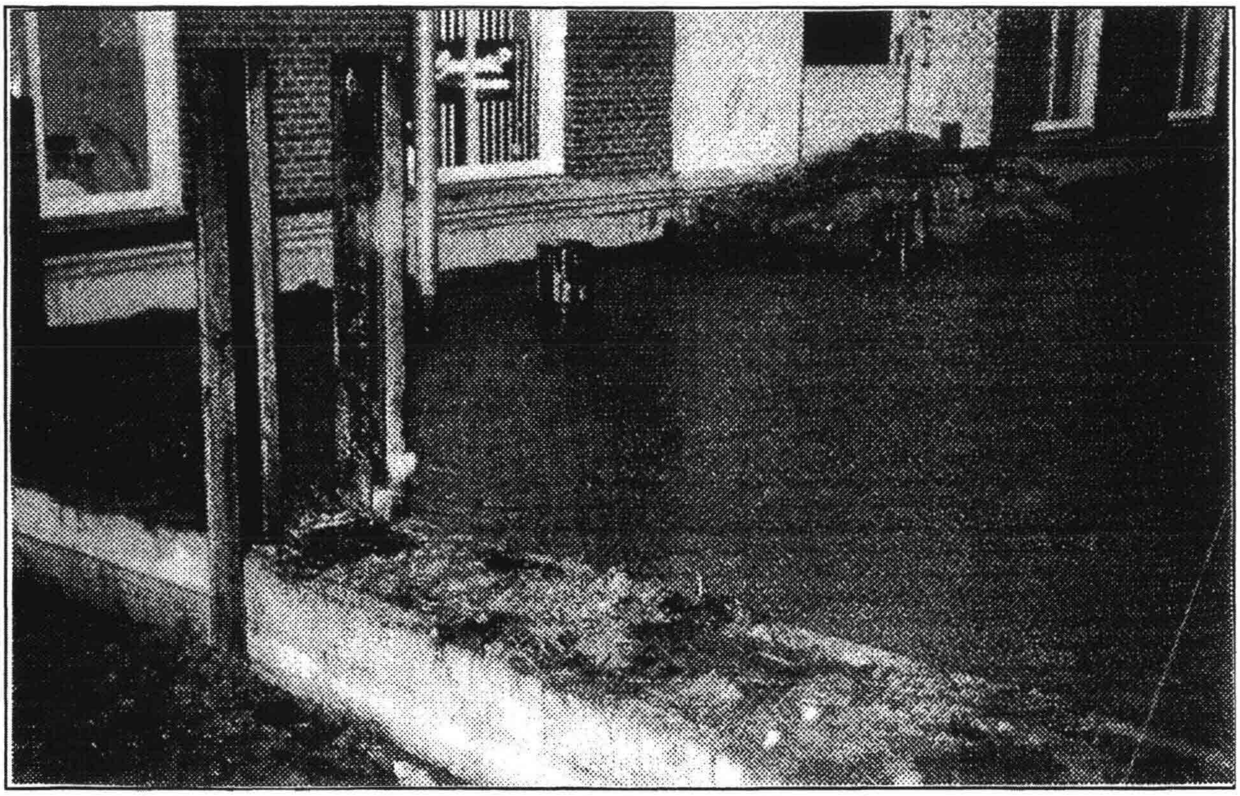

Figure 11.3: Gap temporary closed with wooden beams and horse dung

Gaps can be closed by doors, like in the case of the streetcar-crossing in New Orleans, but mostly closure is effectuated by placing two sets of heavy wooden beams. The space between the beams is filled up with a filling material with low permeability. Tradiationally in the Netherlands for this purpuse horse-dung is used.

11.2 special profiles (cofferdams, etc)

In some locations hardly any space is available. In such cases a solution might be the construction of a cofferdam. In general such a cofferdam (mostly two connected steel sheetpile walls are suggested) is not a good solution. Especially when the soil conditions are not too good, it is extremely difficult to design the structure (the forces on the connecting rods are extremely difficult to determine). But the most significant disadvantage of cofferdams is the maintenance of the structure. The life-time of a dike is very long, order of a century. Therefore

one should apply materials which hardly decay, ore can be replaced easily, like an asphalt revetment. Replacing of a steel sheetpiling, or replacing of the connection rods is a very costly operation, and will probably be postponed as long as possible. Experience has shown that this will be postponed (mainly by politicians who have to supply the money) until after the next disaster.
Therefore it is recommended to try not to use cofferdams, etc.

11.3 Emergency measures

In case of an emergency (e.g. the water is higher than expected, a start in a failure during high water is observed) immediate action has to be taken. In order to be able to take such action, one has to be prepared. This means that an emergency plan has to exist, dikes have to be monitored during extreme situations and material for emergency situations has to be available. For the monitoring ("guarding the dike") is referred to chapter 16.2 of this lecture note.
In setting up an emergency plan (disaster mitigation plan) first an inventory is made of all potential problems. For example, what to do if there is a failure of the revetment, what to do if some sliding occurs, what to do if piping is observed, what to do if water starts to flow over the dike.

Figure 11.4: Emergency wall of sandbags around a well

In order to prevent flooding (a water level, higher than the dike-height is expected), a temporary dike is needed. The standard way of doing this, is using a dike consisting of sandbags. These sandbags can be made of jute or nylon. "Equipment" needed are: many people, hand-shovels and a needle to close the bags. For filling sand has to be available. The bags should not be made too heavy, otherwise one runs into trouble while handling them (30 kg maximum). For a 1 m high dike one needs approx. 5000 bags and 150 tons of sand.

Although standard sandbags give maximum flexibility, their use is quite labour intensive, and also the time required to build up a temporary dike is quite long. Therefore in western Europe and the United States often bigger units are used. An example of such a big unit is the so-called "Big-Bag". These are bags with a size of approx. one cubic meter, normally used for the transport of rough semi-manufactured articles from one factory to an other. Of course one cannot handle these bags by hand, but handling them with a mechanical shovel is quite easy. They are also filled with mechanical shovels, so the production rate is quite high. In the Netherlands, during floods mainly second-hand bags were used. The price of these (second hand) bags is quite low, approx. 10 guilders per bag. For 100 m of dike, one needs 100 big bags and approx. 170 tons of sand.

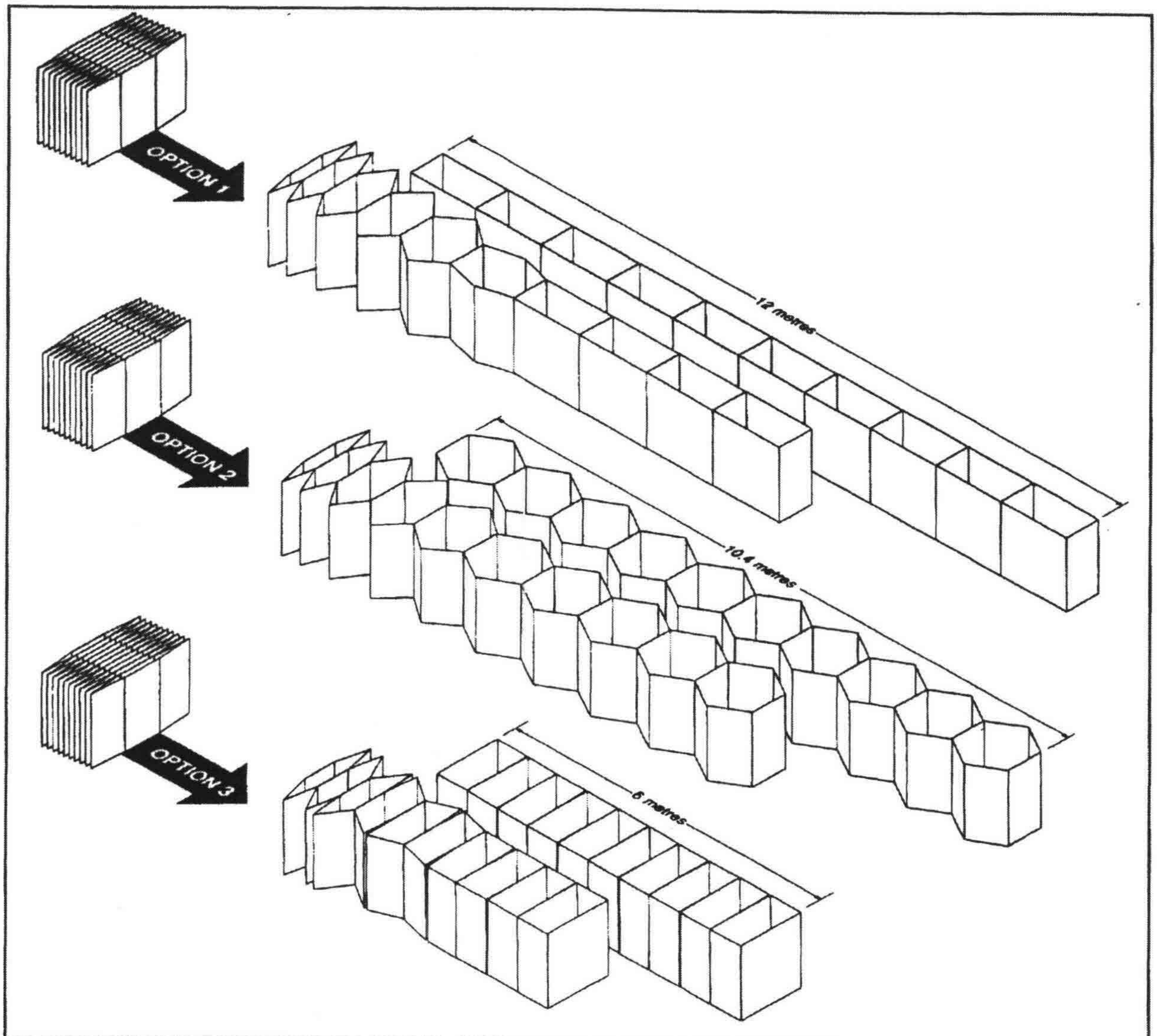

Figure 11.5: The Hesco/Bastion Concertainer

An other method is the use of prefabricated elements, especially developed for this purpose like the Concertainer or the Aqua Barrier. The concertainer is basically a hinged galvanised wire panels, which can easily be joined to create a honey-rate structure. The boxes can be filled with sand, and so the form a vertical, temporary water retaining structure. After use the concertainer can be recovered by lifting it, so the sand fall out of the compartments. The disadvantage is that one has to keep these elements in stock, and that they are relatively expensive. Concertainers are available in heights of 50 cm and 100 cm. The element usually applied as temporary retaining structure is the 10-segments element, which is (unfolded) 10 m long, and requires 15 tonnes of fill material. So, for 100 meter one needs 10 elements and 150 tonnes of sand. For informations, contact Hesco, Unit 37, Knowsthorpe Gate, Cross Green Industrial Estate, Leeds LS9 0NP, UK, fax +44.532.483501
The Aqua Barrier is based on standard EUR-pallets and can hold water up to 1 m deep. The pallets are supported by triangular, steel elements, which can be folded together to minimise

storage space. The pallets are loosely covered with an impermeable geotextile (LDPE or HDPE, 1 kg/m^2).
This type is well fitted for temporary water retaining structures which are in use quite often (e.g. once every year) for the protection of isolated placed, like a single house. Especially in those areas where it is very easy to rent the standard pallets commercially this type is attractive. One steel support is needed per pallet (8.3 kg). The maximum retaining height is 0.95 m, there are 125 pallets and supports needed per 100 m of temporary dike (and of course two 50 m rolls of LDPE).
(for info: Geodesign AB, Teknikringen 1, 58330 Linköping, Sweden, fax +46.13.211958)

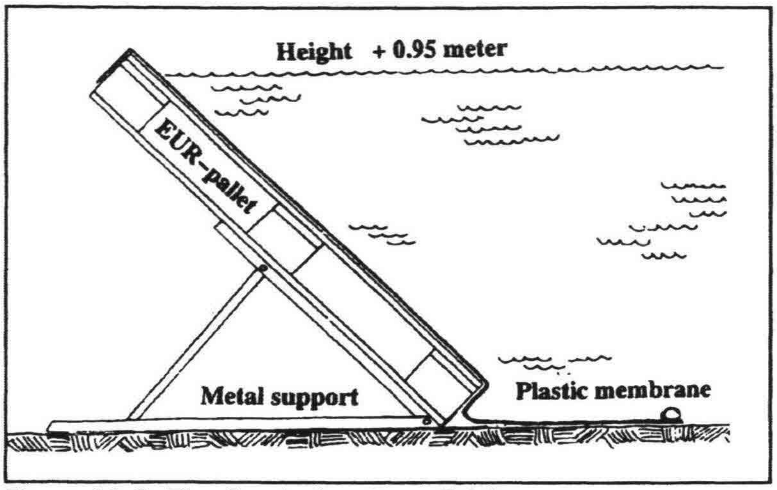

Figure 11.6: The Geodesign Aqua Barrier


Figure 11.7: Geotextile used to protect a grass slope

In case of damage to the outer slope of a dike (damage to revetment and/or grass slopes) one can prevent further damage by covering the damaged spot with a geotextile. Usually this is a quit effective method, however it is rather difficult to place the geotextile. During the last flood in the Netherlands, specially trained Navy divers were employed to connect the geotextile under water to the bottom. Especially when there are strong currents and/or high waves, such operation is quite risky.

A part of good dike management is to have material for emergency in stock. Special storage sheds have to be constructed on regular intervals along the dikes. These storage sheds can also be used as local office during an emergency. Therefore it is vital to have a telephone connection to these sheds.

14 Dike improvement works

In many cases a dike already exists, but needs to be improved. In the past this was mainly done by raising the crest-level of the dike. But this caused many problems; the dikes became too steep and too narrow.

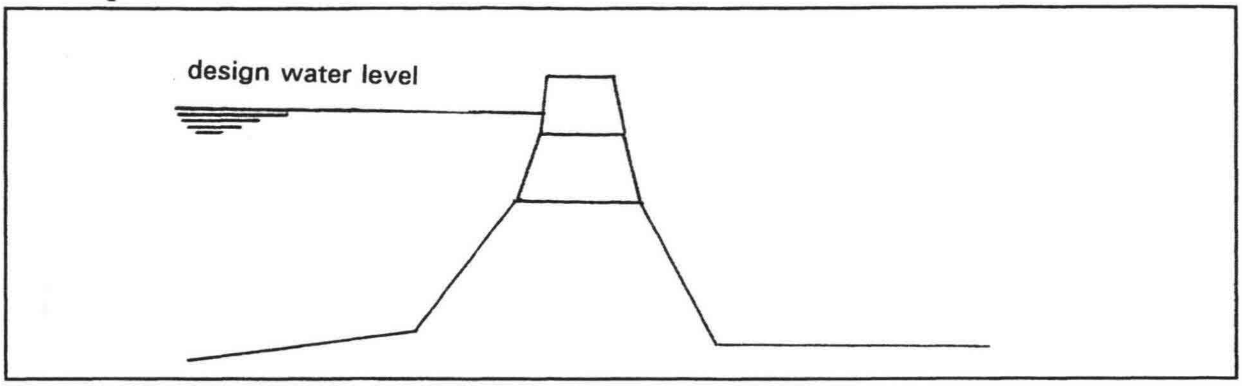

Figure 14.1: raising the dike height in former years

Nowadays dike improvement is therefore very often more widening the dike, than increasing the height. When a good revetment exist, one tries to make the dike improvement at the landward side of the dike. The advantage is that it is not necessary to make a (very costly) new revetment. Only in those cases were no space is available at the inner side, the dike is improved at the outer side. In those cases a new revetment has to be made. Both situations are illustrated in the figure below.

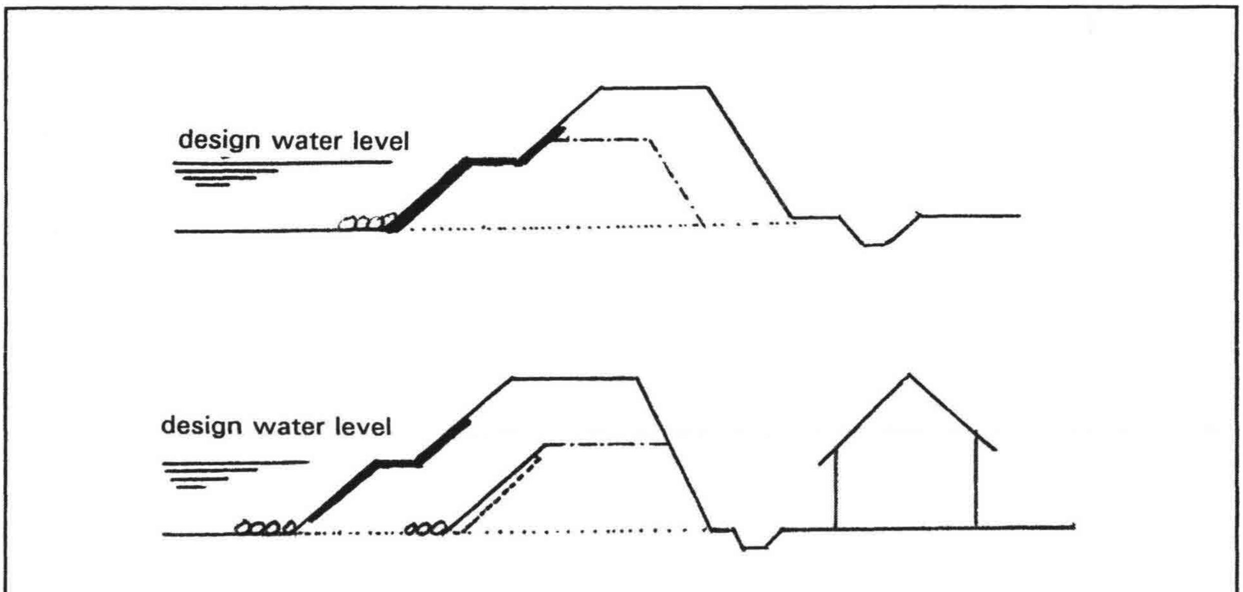

Figure 14.2: Improvement of an existing dike

Making an improvement at one side of an existing dike one has to take into consideration the uneven settlement. The soil under the existing dike was already under a load, and will

therefore be compressed significantly less. The improvement outside the existing dike will settle much more. In total this will lead to an uneven settlement.
Very often in the Netherlands mine-waste is used as material under a new revetment. This material acts somewhat like clay, but is much cheaper. It is very doubtful that mine waste in other countries has the same properties. There are also a lot of environmental problems with mine waste. The material itself is very clean, but the mounds of waste near the mines are very often contaminated with toxic material. Therefore application of mine-waste always requires a lot of care.
The execution of works at dikes in the Netherlands is only allowed outside the stormy season. So no significant works at a dike is allowed between 1 october and 1 april.

Practical examples of improved cross-sections can be seen in the following figures. The dike along the Dortsche Kil (a tidal river) has been made in such a way that the complete revetment at the outer slope could remain in place. The clay at the inner side of the dike had been removed and stored temporarily. Then the sand body was placed at the inner side of the dike, making it much wider. At the same moment also a road was constructed behind the dike. Finally the inner slope was covered again with clay.

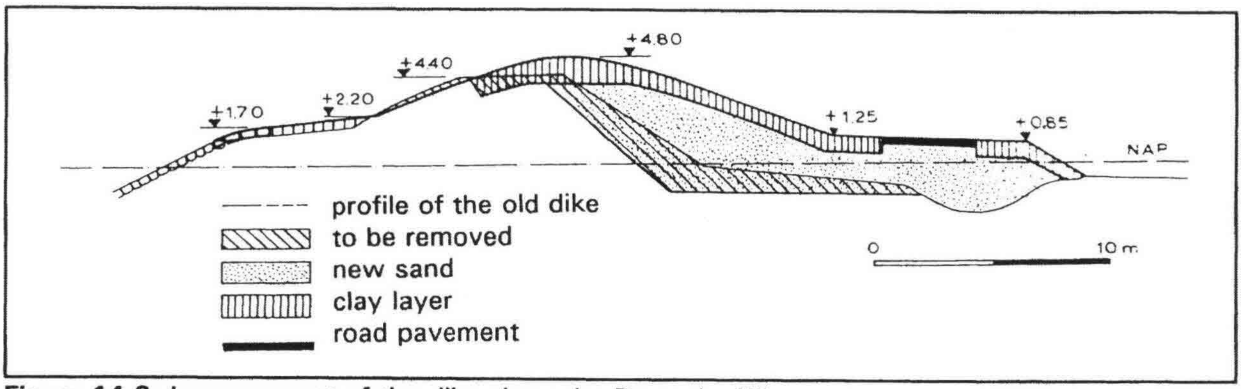

Figure 14.3: Improvement of the dike along the Dortsche Kil

The dike near Ossenisse (at the end of an estuary) had to be improved considerably. So a complete new dike body has been build behind the existing dike. In this case the main purpose of the improvement was an increase of height. Both the design waterlevel and the design wave height were much higher than the original ones from a century ago (caused both by a rise in waterlevel and a subsidence of the subsoil). To reduce the height a berm was designed at Design Water Level, which was approx. at the height of the original dike crest.

Improvement of the river dike along the river IJssel near Schoonhoven was not possible on the landward side, because of the existing build-up area. Therefore an improvement was made on the outer side. This resulted in a complete new (and costly) revetment. In order to reduce the required width of the dike the inner slope of the improvement was made extremely steep. Also in this case 2 % of the wave might overtop the dike. To prevent erosion of this extreme steep section by the overtopping waves, it has been covered with a slope protection of masonry.

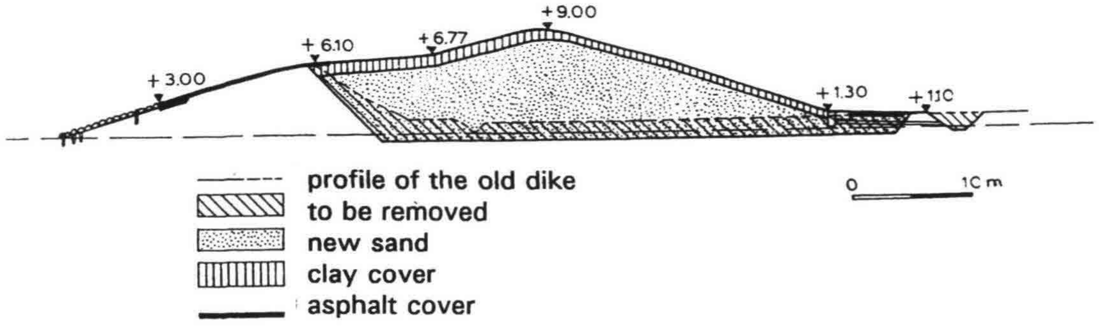

Figure 14.4: Improved dike near Ossenisse

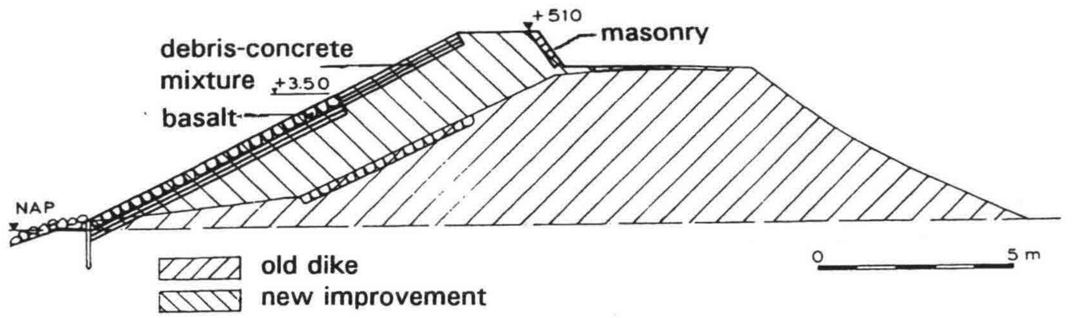

Figure 14.5: Improvement of a river dike near Schoonhoven

In very extreme cases, in build-up areas, no simple solution is possible. In case of the village of Sliedrecht a special analysis has been made for finding alternatives. When there are houses on both sides of the dike, it is nearly impossible not to remove the houses at one side. Because of social and cultural reasons, rebuilding of houses on the improved dike is necessary. However, from an hydraulic point of view, one should not build in a dike body. Therefore is was decided in some sections to transform the existing sloping dike in a cofferdam. In this example the houses on the landward side of the dike are demolished. The two anchored sheetpiles are in the dike, making a cofferdam. This cofferdam is dimensioned in such a way that it is stable in itself (so from a hydraulic point of view the slopes on both sides are allowed to erode away). A new bund is placed at the landward side, on which new houses can be build. In this way there is from a sea-defence point of view a full separation between the dike and the adjacent houses (the houses are not longer on the dike). From a architectonic point of view, the dike and the houses are still one unity.
Such solutions should not be used to often. Control of the strength of the hidden sheetpiles over many decades is very difficult. Constructions like this cause a lot of management problems in the long run. Therefore a dike-manager always has to try to avoid situations like this.

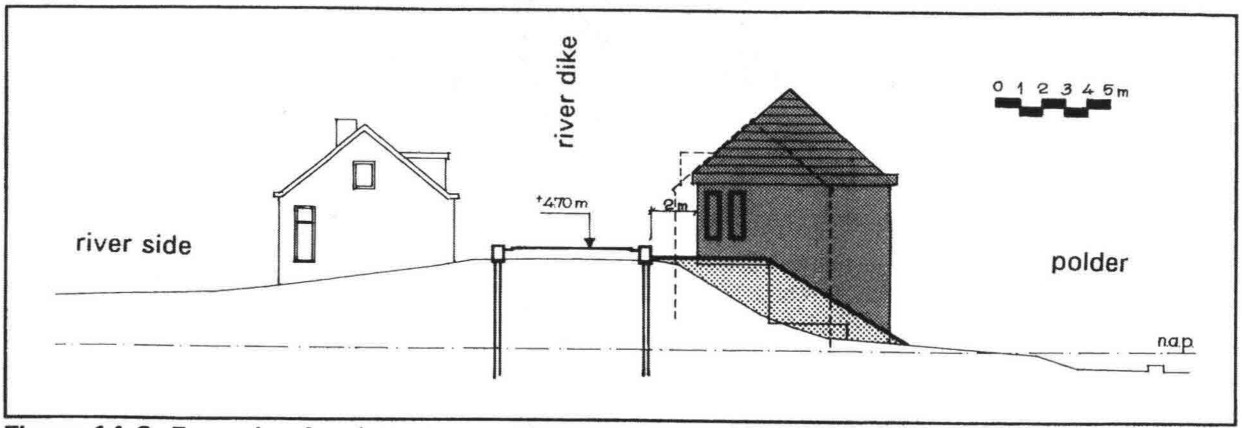

Figure 14.6: Example of an improvement in build-up areas (Sliedrecht)

In the Netherlands we were confronted in recent years with a lot of problems with our dike improvement projects. People living near the dike want to maintain the landscape. Of course this is often in contradiction with the requirements of easy dike-design. This resulted in a long and sometimes even endless procedure in order to find alternatives acceptable to everyone. In most cases these alternatives became rather expensive, and not attractive from a management point of view, like the sheetpiles in the above example.
In order to make procedures easier, the Technical Advisory Committee on Waterdefences has prepared a number of guidelines to incorporate cultural and landscape values in dike design. These guidelines were drafted in close cooperation with the various social and environmental groups, and are therefore acceptable for everyone.

16 Management and maintenance

Sea defence works have to be designed, constructed and maintained in such a way that for a stated period of time (reference period) they will perform all agreed functions with satisfactory reliability and without excessive maintenance. The reference period may vary between 1 and 50 to 100 years and has to be regarded as a help parameter for the design decisions. The true lifetime of the structure may be larger than the reference period.
During the lifetime of the structure especially the maintenance is a continuously returning task.
The responsible maintenance engineer must devise a maintenance program and this will generally comprise:

* establishing standards and criteria
* an accurate data base
* maintenance planning
* execution

The implementation of a quality assurance programme is beneficial. For quality assurance to be effective it is necessary to define the purpose of the scheme, the conditions of use, the expected lifetime and the serviceability. A proper monitoring is essential.

16.1 the history of dike management in the Netherlands

The unique form of self-government-- the "waterschap" - which may be regarded as having fostered the independent spirit of the western Netherlands, began to emerge in the first half of the twelfth century. Best translated today as "Water Board", the "waterschap" grew from the need to "manage" water levels in the western lowlands. The relatively isolated village settlements there tended to look after their own affairs, with little interference from their feudal lord.
The "buurschap" emerged, in which a district's inhabitants took care of their communal interests. Among these interests was building and maintaining embankments, dikes and sluices. The few records which survive would seem to indicate that a "waterschap" was usually the result of co-operation between a number of "buurschappen".
Up to the twelfth century only those communities directly bordering a sea arm or river actively built and maintained dikes. Further inland the ground was still far enough above sea level - except in very exceptional circumstances - to be troubled by excessive water. However, the marshy inland region of what are now the provinces of Utrecht and North and South Holland was largely uninhabited and the feudal lords later actively encouraged settlement by giving settlers the status of freemen. The oldest known "waterschap" was founded in this region.

During the 13th century a second type of water authority emerged as a result of active endiking of land along the coasts and rivers and better drainage of areas inside existing dikes, namely the polder and its administration by a "polderschap". The tasks of water and polder boards were, of course, practically identical, being the management and control of water levels in their districts. By 1300, amalgamation of local polder and water boards into bigger units, the "Hoogheemraadschappen", was in progress, encouraged by the Counts of Holland

and the Bishops of Utrecht. We know that such co-ordinating bodies already existed by then for, among other, Rijnland, Schieland, and Amstelland.
The upkeep of the great Lekdike was split between two, the "Hoogheemraadschappen" for the Lekdike Bovendams and Benedendams, upriver and downriver of the dam at Vreeswijk. The responsibility for maintaining the embankments, ditches and tracks in these early centuries fell upon those whose land directly abutted on to them and was divided up in proportion to the size of their holding. The "aldermen" of the "buurschap", the community, conducted a "schouw", a "viewing", at which they toured their area to see whether the farmers had met their obligations. An official of the community, the "schout", or "bailiff", saw to it that the verdicts of the local court were executed when farmers were found lacking.

In the polders, too, the viewing board of aldermen acquired the legal authority to draw up and execute the necessary by-laws. Costs were met from a levy exacted on property, assessed according the area of land held. The Board was elected by and answerable to a general meeting attended by all the levy-paying landholders. The Water Boards have continued to function in much the same form to this day. The French, during the Napoleonic period of occupation, attempted to sweep them away but failed. Their 1810 Dike Act was introduced to impose far-reaching amalgamation and a national inspectorate and control of finances. The Kingdom of the Netherlands, which succeeded the French regime, withdrew the Dike Act in 1835 and recognised anew the authority of the Water Boards.
However, the Constitution introduced in 1848 did represent a certain break with the past. It separated the Water Board from the local district council and put it on the same footing. It defined the Water Board's function, namely to work in the interests of good water management. Water Boards were to draw up and enforce by-laws to this specific end only.

Their place in society was not brought up for discussion again till after the Second World War. For centuries the Water Boards had quietly carried on with their work. They were now subjected to the search-lights of public opinion and Parliamentary debate. In modern Dutch society water had become a crucial factor in many aspects of life such as industry and recreation and their attendant problems. Some considered the Water Boards to be obsolete institutes, unfit to tackle the tasks of the twentieth century. Some objected to the care for water being placed in the hands of people who owed their position on the Boards solely to their material interests in accordance with the age-old coupling of material interests and taxation to a voice in their management.

As so often occurs in the Netherlands, the immediate reason for the whole discussion was the increasing inability of the polder landowners to bear the brunt of the financial burden of water management and water defence. It resulted in the appointment of a Commission to study the place of the Water Boards in society. The Commission was given a brief which included considering their abolition. Its conclusion, published in 1971, was "that the care for local and regional water management should continue to be the task of bodies specialized in such care, in other words, the Water Boards. However, the Commission laid down a number of conditions for their continued existence. The amalgamation into larger authorities better able to carry out their tasks was to be continued.
A new structure for the imposition of Water Board levies was to be introduced, which ensured that all those having an interest in the Water Boards" work should be subject to a

levy. The Board was to represent a wider section of society. Lastly, all the coastal defences were to be placed under the authority of the Water Boards. In general, the government acceded to the Commission's recommendations, thereby confirming the place of the Water Boards in society.

Authority over a dike -- the practical aspects

The Water Boards exercise authority over 1300 kilometres of sea dikes and 900 kilometres of river dikes. It is difficult to estimate exactly the length of all the "boezem" water embankments and dikes under Water Board control, it runs into the thousands. A Water Board is a functional democracy. In other words it fulfils a well-defined function, the care for water and water management in its district. The age-old coupling of material interest and the payment of levies to a say in the authority guarding those interests still applies. Those who have an interest in security against flooding and, therefore, well-maintained dikes, which includes practically all of us, must pay for their upkeep. Those who pay have a right to a say in the use of that money.

The executive of a Water Board is elected by the interested parties. There are, therefore, no elections on a one-man/one-vote system such as when a local, provincial or national government is elected, but on a one-acre/one-vote system. The number of parties with an interest in the Water Board's work has grown in the course of the years as more and more people move into the countryside to live. Where once the control of water was solely of interest to farmers and landowners, the owners of business and industrial property and people in their homes are now just as much involved. "Property" is no longer just a matter of land and the Boards now include representatives from all types of property-owners in their area.

The very nature of dike systems prescribes the type of body to be put in authority over them. The national government has never been able to develop the insight nor the administration needed to manage the great variety of dikes and embankments in the Netherlands. Neither have the Provinces been able to do so either. The dikes in the area of a Province do, it is true, mostly comprise a closed system, but again the task of attending to the details of dike maintenance is mostly too complicated to allow of fast decisions at critical moments at a provincial level. The lowest rung of government in the Netherlands, the Municipality, is, on the other hand, too small. Dikes and dike systems take little heed of municipal districts and their boundaries. This leaves the job in the hands of the Water Board. The 1848 Constitution provided the Provinces with the authority to set up or abolish Water Boards. They may also regulate the apportionment of tasks to individual Boards in accordance with the needs of each particular area. Both in the case of defence as of management, these areas, and the work involved, must, as far as is possible, cover a complete unit of water management, such as a ring or system of dikes, or a polder or other reclaimed area.

A Water Board's regulations mostly include detailed provisions for the recruiting and the functioning of the dike "army". When danger threatens, the "troops", must be on alert, patrolling the dike and standing by with sand bags and equipment to stop breaks occurring. Members of the "army", recruited from among the ordinary citizens of the Water Board's district, must be instilled with the need to defend their families, community and country and have an intimate knowledge of the lie of the land. The water defences are of such vital importance for the country's safety that the Water Boards may never allow their attention to

their duties to slacken. The supervision of their activities and the manner by which they are elected are, therefore, the subject of detailed provisions in their regulations.
Just as any other form of government, the Water Board has the authority to pass by-laws with which the inhabitants of its district must comply. The by-laws are backed up by penalties. The police may be called in to enforce them. The Water Board may levy taxes to finance its work. All these activities are supervised by the Province. Any citizens who consider themselves unjustly or unlawfully treated by the Water Board can appeal to the Province or to the Crown (i.e. Council of National Ministers). The highest authority of a Water Board rests m the General Executive Board, often known as the "General Meeting" or "United Meeting". The General Executive Board must approve the budget and decides the rate of taxation or "levy", draws up and passes by-laws and must give its approval for large-scale works. It also often elects the members of the Water Board's executive committee. The "dijkgraaf", the "reeve", who chairs the Board, is mostly a Crown-appointed official, exemplifying the importance attached to this job by the highest authority in the land of ensuring the safety of the country. The importance of the "reeve" may be compared to the position of the mayor, another Crown appointee and head of a Municipality. The "reeve" acts as the link between the General Board and the administrative officials working for the Board.

The Board's everyday work is carried on by an administrative and a technical department. The administrative department prepares policy and gives legal and financial direction to all the work of water management to be carried out by the technical department. The technical department cares for the design, execution and management of dikes, pumping stations, sewage treatment plants, roads and buildings. Lastly, it must again be said that the construction, management, and improvement of sea and river dikes make exceptional demands on the body in charge of them. Since none of the great water defences are stronger than the links between them, these bodies must constantly be on the alert and can never let down their guard. The strength of the chain of defences is never greater than their weakest link. The demands made by the surge of the tides and the power of the winds on the skill of the Boards' engineers are one side of the care Water Boards must devote to their dikes. The beauty of the countryside of which the dikes are a part, the historic importance of many of the buildings built on and against them, and the exceptional plant and animal communities associated with their environment demand wise management. It is clear that the present-day Water Boards are well-equipped to meet all of these demands. But, above all, it must never be forgotten that, however beautiful the landscape, however picturesque the houses, farms and churches, and delicate the beauty of flowers and plants, they all owe their existence to the strength of the dike in whose lee they nestle. The none too distant past has made dramatically clear what will happen if we fail to give safety the highest priority. "Dam or be damned" is the old saying and never a truer word was spoken.

16.2 Guarding the dikes

However stoutly built the dikes may be and however well the experts may have computed the stress dikes would be subjected to in exceptional circumstances, it is still men who must remain on the alert to guard them when gales rage. Theory is never more than an approximation of reality. However engineers may tax their imaginations, nature always succeeds in doing the unexpected. So whenever storms, spring tides, a sudden thaw or other calamities threaten, men must be on the alert to guard the dikes. The guarding of the dikes is governed by strict rules. This is hardly surprising in view of the interests of the population, their goods, beasts and chattels, which are at stake.

The difference between guarding river and sea dikes is practical. Gales and the floods associated with them often blow up out of nothing. Flood conditions on the rivers, however, are more easily predicted, since flood-water surges must first travel all the way through Germany or Belgium before they reach Lobith or Maastricht and enter the Netherlands. A flood surge passing through Cologne for example still needs two days to arrive at Lobith.

sea-water

The sea flood alert system has only been in operation m the Netherlands since 1921. Before then the local dike authorities--then, as now, mostly the Water Boards-- relied on their own intuition and experience when deciding whether the weather gave cause to sound the alarm. In the days before telephones, church-bells were tolled to alert the population to danger. However, after the 1916 floods a greatly improved coastal defence alert system was developed on a national scale The Ministry of Public Works (Rijkswaterstaat) was put in charge of the organization, creating a Flood Warning Service which works in close operation with the Meteorological Service (KNMI), The Telegraph and Telephone Service (PTT) and the Broadcasting Service.

The Meteorological Service uses computer models to forecast the increases in astronomic tide levels and warns the Ministry in The Hague about 10 hours before the high tide is expected. The Flood Warning Service at the Ministry initially warns the Water Boards and other bodies in charge of water defence works. As high tide level increases threaten to become still higher, other bodies, such as the provincial government and the army, are warned.

If the high tide level is expected to be more than half a metre above or below the usual level then this is broadcast in the official weather report on the radio. The coastline is divided into five sections, Schelde, West Holland, Den Helder, Harlingen and Delfzijl, and a forecast is broadcast for each section. Circumstances along the coast differ, and, moreover, high tide at Delfzijl occurs 10 hours later than at Vlissingen (Flushing). Although the 1970 closure of the Haringvliet as part of the Delta Plan reduced tidal influence upriver, the decision not to close the Nieuwe Waterweg, the waterway giving access to Rotterdam and upstream, means that the never dikes in the low-lying areas around Dordrecht and up to Gorinchem on the Waal and Nieuwegein on the Lek are still subject to the influence of tides and gales pushing up from the North Sea. When coinciding with flood water coming down the rivers, the situation along the river dikes in this area can still be extremely hazardous. Consequently, a sixth (river) sector, Dordrecht, was added to the 5 coastal sectors.

When issuing warnings, standard telegram texts are used. Either a limited or an extended alert is issued. A limited alert telegram is issued if the predicted high tide level is no higher than usually occurs once a year. In the Vlissingen sector, for example, this is put at N.A.P. (the national standard ordinance datum, approximately mean sea level) plus 3.10 metres. This level must be N.A.P. plus 3.50 metres (at least) for an extended alert to be issued. Tides as high as this occur, on average, once every five to ten years. If possible, the telegrams are

sent to the Water Boards and national, provincial, and local bodies five to six hours before the high tide is expected. The telegram gives advice on the type of guard which should be mounted and the high tide level expected at one or more coastal stations. In the case of a limited alert, then the Water Board and provincial guard posts are manned. When an extended alert is issued, then the central provincial command centres are fully manned. These function as communication centres for the Water Boards which, of course, need to have all the information at their command as soon as it is available in order to take all the measures necessary in their sector.
In turn, the Water Board has its own central command post where the members of the Board, under the leadership of the dike-reeve, are present when danger threatens. Once the Water Board has received a telegram, the dike guard organization springs into action. Each Board has its own organization which can draw on years of experience in dealing with wind and water. Once they have been alerted, patrols leave to guard their "own" section of dike day and night if need be. They keep in touch with the command post by radio or telephone. Apart from reporting on the level the water may have reached, their duties include keeping a close watch on the body of the dike itself -- heavy drift-wood logs may be battering its surface, mole and rabbit burrows may give the water a hold, to name but two of the dangers threatening the dikes. Materials which may be needed to avert disaster such as sand-bags, timber and tools are kept in depots at the foot of the dike. Should the situation become really serious then the Water Board can bring up its heavy equipment such as bull-dozers and drag-lines, or requisition it from local contractors.

The dike defence "army", recruited from among the inhabitants of the Board's district, can also be alerted. They can, for example, also be allotted to patrol duties. There are usually more than enough volunteers so the Board does not need to make use of its statutory powers to "call up" citizens for dike guard service, though it may have to if a sudden emergency threatens disaster.

The Meteorological Service computer also processes data from North Sea oil-rigs, which are equipped with instruments recording the height of waves and the level of the tides, wind speed and direction, temperature and atmospheric pressure. These instruments automatically broadcast the data round the clock to a shore station. Data is also received at The Hague on tide levels from several stations on the east coast of England.

the rivers
The first warning system for flood water levels along the river dikes was set up in 1798. Horsemen rode from place to place along the dikes to keep the dike-reeves informed of the water levels. This was by far the most efficient warning system before the advent of telegraph and telephone. Flags and cannon-shot were also used. This system was set up under the Emergency River Correspondence Act.

Ice floes present the greatest hazard to the river dikes. Pack-ice jams could simply push the dikes aside. The capacity of a frozen river to carry water is far smaller anyway, and when pack-ice occurred in combination with flood water because the thaw had set in earlier in Germany or Belgium, then the flood water often overflowed the dikes. This confronted the defenders with yet another hazard as the water could then scour the dike away on the landward side causing it to collapse. Such situations often occurred or threatened to occur in the past. Gangs of men once used to tackle the ice with picks, saws and poles to break up the jam. In the twentieth century the army has sometimes been called in to clear jams with dynamite. However, because of the thermal and other pollution, especially from industrial cooling water, and perhaps because winters have become milder, no such problems have occurred since 1967.

It is extremely important that the dike authorities know what water levels they can expect. Consequently, the Ministry of Public Works has the water level reports for the whole length of the Rhine, its tributaries and the Meuse broadcast on the radio and published in the newspapers daily.

If the water level at Lobith has risen above NAP+ 14 metres and a level of NAP+ 15 metres is expected, then the Emergency River Correspondence Act is invoked, and the never flood alert system comes into action. On the Meuse, the alert is given when the Maastricht level reaches NAP+ 45 metres.

When the Flood Warning Service in Arnhem puts out a warning for coastal sectors, the responsibility for the actual guarding of the dikes remains in the hands of the competent authorities. On the rivers, however, Ministry of Public Works engineers are given special powers over all dikes for which the Emergency River Correspondence Act has been invoked by the Minister. This occurs whenever the area of the great rivers is under immediate threat because of the high flood water levels. The last time it happened, however, was in 1948 and covered the rivers Rhine, Waal and Meuse.

The Water Board has powers to call up the inhabitants when high water threatens--and the Act has been invoked. The Board recruits this so-called "dike army" from among those people who, on the grounds of their residence in the Board's district, are eligible, according to the Water Board regulations, for service. Preference is given, just as on the coast, to volunteers, people who know the lie of the land, and who because of their ties with their surroundings, have no need to be convinced of the importance of security against flooding. In principle all male residents aged between 18 and 60 have to report for duty, but a Water Board will rarely resort to compulsion, again just as on the coast.

High flood water levels accompanied by gales can create especially hazardous situations because of the waves and the extra increase caused by the wind-blown water. Parts of the dike can subside because the water seeps through the ground and comes to the surface on the landward side, thus undermining the dike. Rabbits, moles and musk-rats play a role too in undermining dikes. All these factors serve to illustrate the importance of the patrols mounted by the dike guards at.times of high water. Drift-wood such as rotten pollard willows can seriously damage the dikes. The patrols must try and remove heavy logs as quickly as possible before they batter the vulnerable slopes of the dike. If necessary, traffic can be banned from the dike road to prevent the dike from becoming too heavily strained when under pressure from floodwater. Should the situation get really serious then the dike-reeve is empowered to requisition lorries, bulldozers and any other heavy equipment considered necessary as well as to call up the "dike army".

Fortunately, good management, maintenance and improvement of the country's flood defences have saved the coast and the beautiful riverside country from inundation for many years now. We may not, however, allow this success to lull us to sleep. Nature remains unpredictable and always continues to break its own records.

16.3 set-up of a management system

A management scheme for a sea-defence system, but also for revetments and bottom-protection has to be suitable for the next two items:

* Satisfaction of the responsible public authority (the regional council, a waterboard, etc.) that maximum value for money is achieved (social output).
* Critical reflection on the own performances and output.

It appears to be almost impossible to identify and value the social output of sea-defence works in an explicit way. Therefore one may do this in an implicit way in a so-called REC-assessment model. REC stands for "Risk, economic and conservation".

Together with a database (or register) with all relevant data of the sea-defence and a management policy plan, this model provides the basic information for the maintenance plan, the annual budget estimates, the annual account and justification reports.

<u>data-base/register</u>

This register contains all physical qualities of the administered objects and related aspects inside the influence zone:

* the boundaries of the influence zone. This zone includes those areas of adjoining sea bed and land where processes are linked in some way to the behaviour of the defence works.
* a description of the as-built situation and the actual situation, including longitudinal and cross-sections, geotechnical profiles, etc.
* a list of issued licenses and permits.
* an ownership and farming-out register.
* a map with all cables and pipelines owned by public utilities, oil companies, etc.
* a damage record.
* a record of executed maintenance.
* boundary conditions (hydraulic, geotechnical, traffic, etc).

<u>management policy plan</u>

Control of the coastline and coastal activities may lay with different authorities and regulatory bodies. In this case it may be difficult to fix responsibility, control and accountability throughout the considered coastal area, or to decide among proposed uses when questions of permitting, engineering, maintenance and financial responsibilities arise. However it may be, a management policy plan is needed to link the objectives of the political arena and users of the coastal zone with those of the defence works management.

The plan should contain the following elements:

* A statement on the scope of the plan with the links to all relevant laws and regulations. Application of legislation depends upon the policies of the government,

departments and other regulatory bodies (local, state, federal, private) and the grant-aid or other funding associated with these policies.

* The procedure to be followed at the adoption and amending of the plan by the responsible administration.
* A statement on the management policy outlines. Policy options are: maintain existing coastline, set back defence line, allow overflow of dikes, allowable risk factor for flooding. Management and engineering options are: create, modify, reinstate or do nothing.
* A definition of the influence zone for the defence work (see previous section). There is no legal definition, so this has to be determined from a pure technical and managerial point of view.
* A review of all relevant functions and uses. Examples are:

  ** Environment:
    - flood defence
    - conservation
    - outfalls

  ** Industry
    - power stations
    - ports
    - dredging
    - reclamation

  ** Leisure
    - beach recreation
    - recreational fishing

  ** Land Use
    - agriculture
    - habitation
    - roads, rail-roads

  The original functions may have changed and still may change. Many single-purpose agencies promote specialized activities and client groups with no forum available to reconcile their competing interests. All coastal uses can be identified with one or more conflicts. The political arena has to deal with weighing these conflicting values.
* A changing climate will influence the threat to coastal areas, both directly by a change in the hydraulic boundary conditions, as indirectly by a change in sediment transport in front of the sea-defence.
* The management and maintenance strategy and tactics to be followed for the short and long term, including procedures for the consultation of other disciplines. Also monitoring to be performed during extreme conditions and scenarios in case of calamities have to be included

<u>REC-assessment model</u>

Management and maintenance will be difficult to justify in narrow benefit-cost terms. Therefore it is proposed to make an evaluation in an implicit way.

The following figure shows the flow scheme for a model on risk-, economic- and conservation assessment, that is suited to visualize the change in social output value when certain measures are taken (for example the increase of potential flood risk when the maintenance budget is cut down).

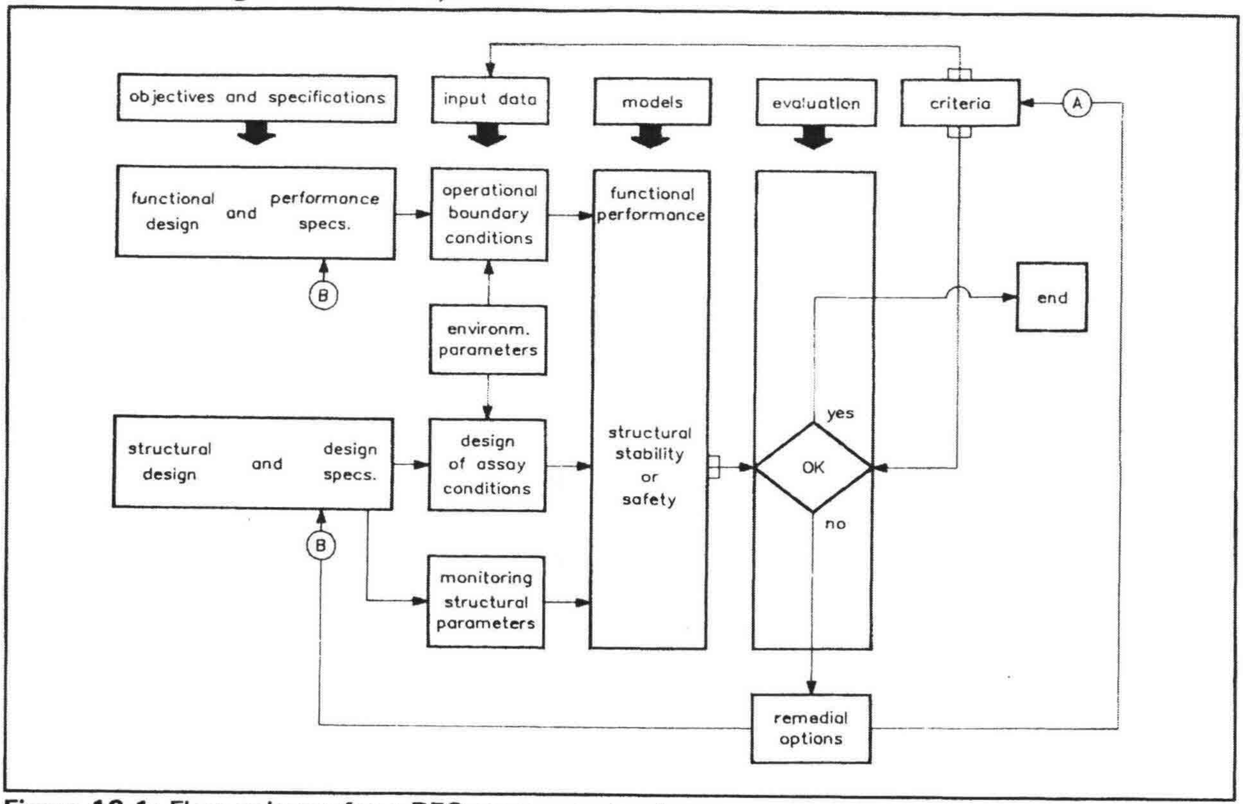

Figure 16.1: Flow scheme for a REC assessment scheme

Essentially, the method contains the following steps:

- Definition of objectives and specifications of the structures.
- Definition of criteria for performance, structural integrity, boundary conditions, accepted risk level, finances and conservation.
- Assessment of input data: structural (resistance) parameters and environmental (surcharge) parameters.
- Models to describe the behaviour of the structures.
- Model to evaluate the results and to define remedial options.

The objectives are based upon the functional and structural requirements and the corresponding performance and design specifications. All this information can be found in the data-base and the management policy plan. The structural design and/or assay specifications may be expressed in terms of a probability of failure or as a damage level. The criteria are a further quantitative definition of the functional and design or assay requirements. One has to distinguish criteria on the input-side of the model (agreed boundary conditions, budgets, maintenance level, etc.) and on the evaluation-side of the model (social and technical output, such as a maximum risk level, minimum recreational capacity, maximum impact on vegetation, etc.). All criteria have to be made clear in the management policy plan. Budgets follow from the budget estimates after approval by the responsible authority.

The surcharge input data follow from the input criteria, the resistance input data from the data base and monitoring of those structural parameters which effect the performance and the stability. The annual budget estimate and the maintenance plan give an insight into the realizable maintenance level that may influence the resistance of the structure.

The models used to describe the behaviour have to be selected in accordance with the objectives. These models are based upon a description of the most important failure modes and relate the environmental parameters to the probability of failure or minimum appreciation lev of each model. A number of these models are to be found in the other papers of this course.
The model used for the evaluation of the results can be a (multi-) criteria analysis, supplemented by a more sophisticated model to Optimize the effect of changes in structural or environmental Parameters.

Remedial options, in case the evaluation reveals no o.k. status, are:

A. Adjust criteria, what usually means: accept an higher risk level, accept a lower leisure quality, accept a lower level of naturalness etc., or: increase management budgets.
B. Modify the functional and/or structural design.

Decisions on these options have to be made by the responsible authorities; resulting new policy has to be incorporated into the management policy plan.

16.4 periodical testing of dikes

16.4.1 general conceptions

According to the REC-model the maintenance plan forms a major instrument for the management of a coastal structure. In this chapter the elements of the maintenance plan with emphasis on the inspection and monitoring system will be worked out for a dike system.

The main function of a dike is to protect inland area from flooding. In addition the dike may perform other functions such a road-transport, agriculture, and very often dikes have also an ecological function. The dike-system should be designed, constructed and maintained in such a way that the relevant functions will be fulfilled during the required life-time.

A dike system forms part of a total water retaining system. consisting of different type of structures. If one part of the system fails, flooding of the inland area may occur. In order to guarantee an acceptable safety level under extreme loading conditions the contribution of the total length of the dike sections to the overall probability of failure of the potential flooding area amy not exceed a specified value.

Each of the dike sections is composed of a number of structure components, termed as elements. These elements can be related to the detailed water retaining function as derived from the main function.

Examples are:

dike element function

slope revetment: hydraulic erosion control
dike crest: control overflow and overtopping
slope: ensure geotechnical stability.

The elements can be described by a number of characteristic condition parameters, mainly geometrical and material (=strength) parameters. For example the dike crest can be described just by the geometrical parameters height and width of the crest, whereas the description of a slope revetment necessitates specifications of the type, geometry, weight an strength of the successive layers of the revetment.

In common engineering practice of dike design for each of the relevant condition parameters functional requirements have to be specified that should meet the overall safety requirements of the total dike system. After construction of a new dike of enlargement of existing dikes, the actual value of the condition parameters should at least satisfy the specified range.

However, during the service life of the dike a number of deterioration mechanisms may occur that will affect the strength of the structural elements. I other words these deterioration processes may change the values of the condition parameters.
Examples of these processes are gradual settlement of the dike crest due to consolidation of underlying soft soil layers, washing out of finer particles of a stone revetment due to wind, rain and temperature effects and damage effects of slopes due to biological activities such as vegetation and animal burrowing.

These alterations caused by deterioration mechanisms, which will be recognized as characteristic damage patterns, can be of such significance that the dike-safety will no longer be sufficient. This means that the actual quality of the condition parameters may decrease to a level at which the extreme loading condition cannot be sustained.

```
deterioration        damage         alteration of          ultimate
mechanism    ====>   pattern  ==>   condition      ======> failure
                                    parameter              mechanism
```

Figure 16.2: relationship of damage patterns and failure mechanisms

The above figure shows the relationship of the damage patterns and the deterioration mechanisms (at daily loading conditions) on one hand and the ultimate failure mechanisms (at extreme loading conditions) on the other hand.

By means of an adequate maintenance and control system of the dike the managing authority aims to secure that the actual quality of the relevant condition parameters do not decrease to a level lower that the acceptable failure limit.
For this purpose the actual state of the condition parameter has to be assessed by periodical inspection and monitoring of the dikes. In addition based on these observations an supported by theoretical interpretations predictions have to be made of the expected behaviour in the

near future. From these two conditions, i.e. the actual state and the expected behaviour, the necessary maintenance measures can be planned.
These measures may vary from "do nothing (=zero option)" or "increase inspection frequency" till "repair or replacement of structural elements".

The so-called preventive condition-based maintenance strategy distinguishes three limits of quality levels of condition parameters (see figure below).

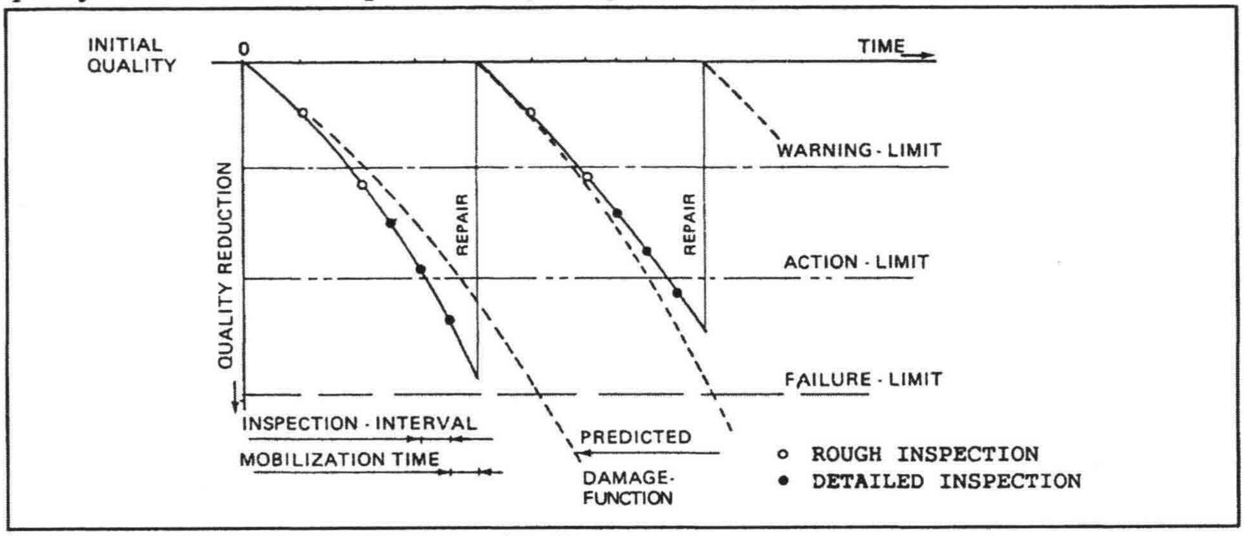

Figure 16.3: preventive condition-based maintenance

These three limits are:

- warning limit: quality level at which a more intensive control of the condition parameter is needed (higher inspection frequency).
- action limit: quality level at which repair measures should be prepared and carried out before the failure limit has been reached.
- failure limit: quality level that is just acceptable from the safety requirement. If the condition decreases below this level the dike system will not provide sufficient safety.

The margin between the action limit and the failure limit will depend on the inspection frequency and the mobilization time for the execution of repair measures.
An optimum maintenance and control strategy will be obtained by considering the minimum costs of repair and inspection, on the condition that the probability of exceeding the failure limit is sufficiently low.

16.4.2 inspection and monitoring strategy
The inspection strategy should provide information on:

- what (which damage pattern characteristics),
- where (locations and depth),
- when (how often),
- how (which measuring method) and
- by whom (which organization)

data should be collected and evaluated.
Monitoring is assumed to form a part of the total inspection and will not be discussed separately.

It is not possible to give a detailed description of the inspection strategy because it depends very much on the actual situation. Alternatively some general remarks will be given as a guideline for the set up of an inspection system.

The principle of the inspection strategy should be based on a phased inspection with respect to the amount of detail. Starting from a rough inspection a more detailed inspection may be required depending on the results of the preceding inspection.

A distinguish should be made between the periodical inspection of the damage patterns (deterioration mechanisms) according to the maintenance plan on one hand and the incidental inspection after the occurrence of special loading conditions (extreme mechanisms) on the other hand. The last type of inspection should be performed after each severe loading condition (such as an extreme storm surge) and will generally starts with a rough visual inspection of the total dike length.
The periodical inspection will be discussed in more detail by considering the before mentioned definition of the inspection strategy.

The answer to the question which data on which location have to be inspected will primarily follow from the type of damage patterns to be controlled. Each damage pattern has its own observable characteristics. For example the quality of a grass on clay revetment can be characterized by the presence of uncovered spots (that is the area where the grass is absent). The quality with respect to consolidation of the soft soil layers can be characterized by the settlement of the dike crest height. Some damage patterns have to be controlled only at a specific location that is representative for the total dike cluster, while other damage types should be controlled for the total dike length.

Apart from the predictable damage patterns the inspection should include a rough (visual) observation of the dike geometry in order to detect possible damage patterns which have not been identified beforehand. Through this additional inspection the probability that unforeseen damage pattern may affect the safety significantly can be reduced substantially. In case these damage patterns are detected, first of all explanatory investigations should be performed to understand the causes of the damage.
To decide when the data on the damage patterns should be collected, in other words which inspection frequency is needed, the following aspects have to be taken into account: the expected velocity of damage increase in the next period (varying from a very gradual increase till a sudden progressive increase); the predictability of the damage increase, varying from

very good (= small uncertainty) till very poor (= great uncertainty); the actual condition relative to the warning, action and failure limits.

To illustrate the effects of these aspects two extreme combinations of damage patterns and associated inspection strategies are outlined in the following table.

Aspects of inspection strategy	Aspect of damage increase	
	progressive great uncertainty	gradual increase small uncertainty
Inspection method	detailed	rough
Inspection frequency	high	low
Action-failure limit	great margin	small margin

The decision how the data should be collected, that is to say which measure techniques should be applied, again depends strongly on the type of damage patterns to be controlled. In this context the availability and terrain accessibility of equipment for geotechnical site investigation may play a major role. For each situation a cost effective technique has to be selected.

A summary of the most applicable measurement techniques related to the damage patterns clustered per extreme mechanism are listed below.

Geometry dike
- conventional geodetic land surveys (for a.o. crest height)
- advanced techniques with high resolution (for local settlements)

Slope stability, seepage and piping
- inspection of seepage flow of inner slope during high water levels by visual observation or by (infra-red) air photogrammetry
- installation of observation wells and piezometers for the measurements of pore water pressures and the phreatic level in the dike during both normal and high water levels
- Visual inspection of inland sand-boiling wells

Erosion grass revetment
- visual inspection of uncovered area
- shallow borings and sampling of top layers

Erosion stone or block revetment
- infiltration test to determine permeability of top layers
- in-situ tensile test to determine local up-lift resistance of the individual stone or block
- geophysical measurement to detect local cavities underneath the top layer (by ground radar or acoustic methods)

- visual observation of the gaps between the stones and blocks or the detection of complete loss of these elements

Erosion of asphaltic revetment

- assessment of layer thickness from sampling or geophysical measurements
- laboratory testing on samples to determine durability effects
- in situ deflection measurements to determine the stiffness
- nuclear or electrical density measurements in bore holes to determine the quality of the foundation layer underneath the asphaltic top layer
- global visual inspection of superficial cracks

As shown in this list visual observation plays a major role in the inspection. However special attention should be given to those damage patterns which are not easy detectable from the outside geometry of the dike. An example for this type of damage is the presence of erosion channels underneath a top layer of a revetment. These type of channels may substantially reduce the strength of the slope revetment under extreme conditions, without showing any superficial damage under daily conditions. There is still a need to develop special measuring techniques to control this type of "hidden" damage patterns. Geophysical methods may contribute to the solution of this type of detection problems.

<u>Scheme for repair measures</u>
The safety control system should provide a set of measures to be taken if certain damage limits have been exceeded. This set enables the responsible managing authorities to make the correct decisions and to prepare and carry out the necessary corrections in time. All these measures are aiming to meet the safety requirements during the life time of the structure.

The type of measures to be taken can be divided as follows:

- additional inspection and investigations
- repair measures
- replacement of parts of dike elements
- complete replacement of the dike

In practice a great part of the periodical maintenance measures is dealing with the repair of the slope revetments. This correction may consist of a partial replacement of the revetment by a similar type or by a stronger type of revetment.
The maximum allowable mobilization time for this type of repair measures is generally very limited (in the order of weeks). The organization and material control (deposits) have to be directed to this purpose.

More severe correction measures have to be taken if the safety with respect to overtopping, total geotechnical stability or piping is insufficient. In those cases enlargement of the dike including increasing of dike crest height and flattening of the slopes is needed. These type of measures are used to be performed after a long planning period in the order of 25 to 50 years.

periodical testing

Complete replacement of the dike may be necessary if, due to new developments, the functional requirements whether the loading boundary have changed significantly. For this reason a regular review (in the order of every 5 years) of the boundary conditions and functional design requirements should be performed.

Despite regular maintenance measures the occurrence of very great damage including total failure of the dike and flooding of the inland area cannot be excluded completely. For this purpose a disaster plan should have to be developed. This plan provides next to technical emergency measures mainly organizational measures.

16.5 budgeting of maintenance

Text not yet available

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