The interaction between soil, water and bed or slope protection

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ABSTRACT: The interaction between soil, water and structure is discussed for a bed and slope protection. It is shown that for various types of structures, and various boundary conditions, this interaction can be approached in a uniform way. The hydraulic conditions at the boundaries of the structure and along the non-protected parts of the bed result in groundwater flow. Hydraulic conditions and groundwater flow determine the loading on the protection. The structure itself influences the groundwater flow by its flow resistance and may sometimes also influence the load by deformation. Phenomena that may be relevant depending on the situation are summarized and methods are described for determining which of these phenomena should be taken into account in a specific situation when modelling soil, water and structure in order to determine the stability.

1 INTRODUCTION

Until recently the design of a bed or bank protection was mainly based on experience.

Unique circumstances (the Oosterschelde Storm Surge Barrier), new materials (for example geotextiles) and increased loading (in navigation fairways) require a different approach. Therefore the Dutch Public Works Department (Rijkswaterstaat) of the Ministry of Transport and Public Works commissioned Delft Hydraulics and Delft Geotechnics to perform research programmes on the loading and the failure mechanism of bed and slope protections. This paper presents some of the results of these research programmes. Some specific situations and problems are discussed more in detail in three separate contributions to this conference:

- The permeability of closely placed blocks on gravel (M. Klein Breteler, A. Bezuijen)
- Stability against sliding of flexible revetments (K.J. Bakker, P. Meijers)
- Scale effects in modelling the stability of asphaltic bed protections (J.L.M. Konter, W.G. de Rijke)

Different protection types are available, for example asphalt layers, concrete blocks, concrete slabs, all kinds of mattresses, gabions and layers of cemented gravel. These can be applied on different types of soil (gravel, sand, clay) for different types of loading (current, waves), see Figure 1.

Notwithstanding this variety, an understanding of the behaviour of water, soil and structure can be approached in one uniform way. This approach is described in the following paragraph. The approach consists of three steps, each of which deals with the behaviour of one of the three components: water, soil and structure. These steps are separately discussed in the Chapters 3, 4 and 5.

Methods are described to determine which phenomena should be taken into account in

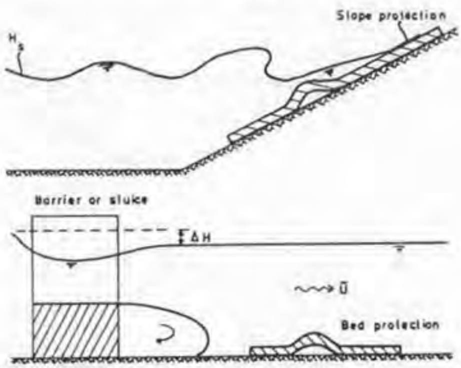

Fig. 1 Examples of slope or bed protection

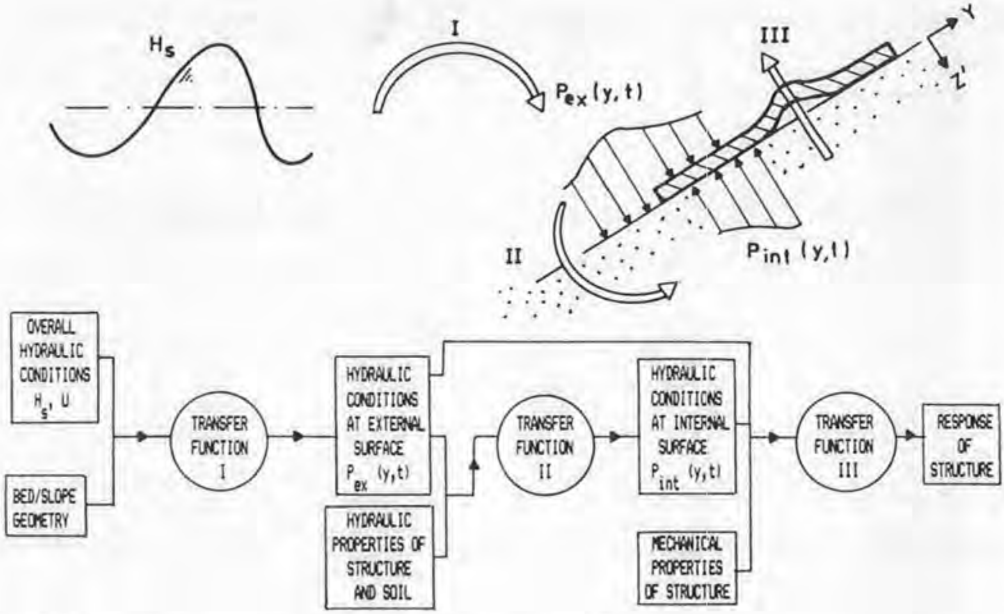

Fig. 2 Schematic presentation of the three Transfer Functions

which situations. An example is presented in Chapter 6. Other examples are given in the paper of Konter and de Rijke.

2 GENERAL APPROACH

The phenomena which may be relevant can be divided roughly according to the three components of the system: water, soil and structure. The interaction between these components can be described using three Transfer Functions (see Fig. 2):

I. The Transfer Function from the overall hydraulic conditions, e.g. wave height H, mean current velocity U, to the hydraulic conditions along the external surface, i.e. the boundary between free water and the protection or soil, e.g. external pressure P_{ex}.

II. The Transfer Function from the hydraulic conditions along the external surface to those along the internal surface, i.e. the boundary between protection and soil. The hydraulic conditions along the internal surface can be described as the internal pressure P_{int}.

III. The structural response of the protection to the loads along both surfaces.

Information about these functions can be obtained by means of measurements in nature and (scale) model tests. If quantitative knowledge of the physical phenomena involved is available, or if there is enough to hand experience then mathematical models or empirical formulae can also be applied. All these ways of obtaining information are dealt with in the present paper and are referred to as "models".

All three Transfer Functions can be described in one model, or individually in three separate models, depending on the type of structure and the loading. In the example presented in Chapter 6, the approach followed involves three separate models. Konter and de Rijke discuss other examples illustrating the single model approach in their paper.
The distinction between the three functions serves here mainly as a framework to describe the different phenomena that are important for the modelling.

3 WAVES AND CURRENTS

Waves and currents (or "free water flow") define Transfer Function I, and the following points should be taken into account when selecting the most appropriate model:

- Water motion phenomena at the external surface depend not only on the type of hydraulic condition (wind wave, ship-induced wave or current) but also on the

Table I

TYPE OF OVERALL HYDRAULIC CONDITIONS		STATUS OF WATER MOTION	PHENOMENA AT EXTERNAL SURFACE	CHARACT. PERIOD AND DIMENSION	APPROCH ON MODEL TYPE	FIG.
WAVES	WIND INDUCED	Non-breaking wave on a horizontal bed	Horizontal velocities and accelerations	1 s 1 m	Analytical formulae from general propagating wave theories (Airy, Stokes, Cnoidal etc.) (US Army Corps of Eng. 1984)	3.1
		Propagating breaking wave on a horizontal bed		1 s 1 m	Numerical models (e.g. BEACH, Vinje and Brevig, 1981)	3.2
		Breaking wave on a slope $1 < \cot \alpha < 6$	Wave impact pressure peaks	0.001 s 0.01 m	Detailed, high frequency large scale wave impact measurements (Delft Hydraulics, Delft Geotechnics, 1986)	3.3
				0.1 s 0.1 m	Numerical models (not yet operational)	
				0.1 s 0.1 m	Semi-analytical semi-empirical models (Stive, 1984)	3.4
			Cyclic velocity and pressure variations	1 s 0.1 m	Large or small scale physical model measurements	
				1 s 1 m	Numerical models (Operational within specific limitations) (Klopman, 1987)	3.5
				1 s 1 m	Empirical formulae describing static pressure distribution (Banach, 1987)	3.6
		Non-breaking wave	run-up and run-down velocities. Cyclic pressure variations	1 s 1 m	Numerical models (Klopman, 1988) Analytical formulae from modified propagating wave theory (Stoker, 1957)	
	SHIP-INDUCED	Combination of current non-breaking and breaking waves	Pressure variations due to - front wave - water level depression - transversal stern wave - secondary waves	1 s 1 m	Empirical formulae based on extensive physical model investigations (PIANC, 1987) <u>Note</u>! Breaking wave phenomena may be treated as wind waves	3.7 A 3.7 B
CURRENTS			water velocity and pressure variations due to - return current - screw race	1 s 1 m		
	OTHER	Current	velocity and pressure variations around local bed form	- 0.1 m	scale model or potential flow (in acceleration zone) (e.g. de Groot and Konter, 1984)	
		Turbulence	velocity and pressure variations	0.1 s 0.1 m	mathematical descriptions, scale models or measurements in nature (Flokstra, 1986)	

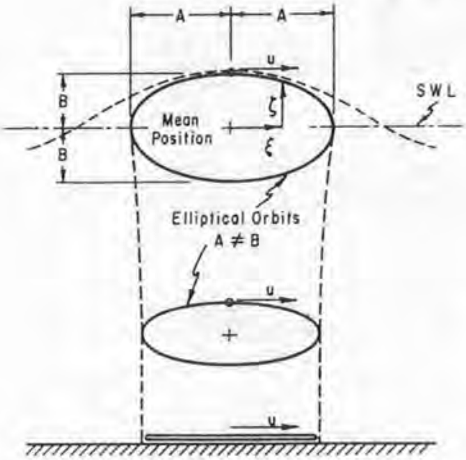

Fig. 3.1 Anatylical description of Propagating

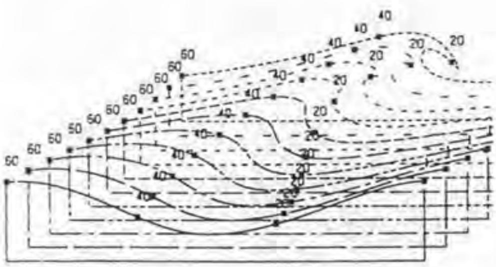

Fig. 3.2 Numerical description of propagating wave

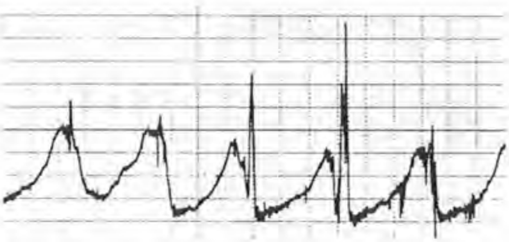

Fig. 3.3 Detailed wave impact measurements

status of the water motion (propagating or breaking wave, accelerating or decelerating flow).

- The type of subsoil, the type of structure and the stability aspect or failure mechanism under consideration determine which phenomena are relevant.

- The most appropiate model types for Transfer Functions II and III depend, among other things, on the characteristi time period and length dimension of the water motion at the external surface tha causes the considered loading.

The types of hydraulic conditions, the possible status of the water motion, the phenomena and the time period and length dimensions are briefly summarized in Tab 1 and Figure 3. Examples of the various physical, numerical and analytical model available are also given in the table wi some literature references.

Each phenomenon at the external surface can be characterized by various time periods and length dimensions. The large characteristic period and the largest dimension often represent most of the energy and have the greatest influence c stability. The smaller periods and dimensions may nevertheless be important depending on the particular situation. T more refined the knowledge required abou the phenomenon the smaller is the smalle dimension or period of interest and the more refined the model that should be applied.

This is indicated in Table I "characteristic period and dimension" ar "approach or model type".

The largest period and the largest dimension will be indicated by T and L i the present paper. The longest period fc wave phenomena is the wave period, excep for wave impact forces for which T ≈ 0.3 The largest dimension for waves on a horizontal bed is the wave length, for waves on a slope the wave height. The largest dimension of velocity and pressu variations around a local bed form equal the dimension of the bed form. The large dimension of channel flow turbulence equals the water depth. The largest dimension of turbulence downstream of a structure is the most characteristic dimension of the structure: for example, pile diameter, diameter of opening or waterdepth. The longest period of turbulence is the ratio between the largest dimension and the mean velocity.

4 FLOW THROUGH PROTECTION AND ADJACENT GROUND

4.1 Summary of modelling requirements ar flow phenomena

Transfer Function I has been dealt with Chapter 3 and the phenomena characterist for the resulting hydraulic conditions along the external surface have been

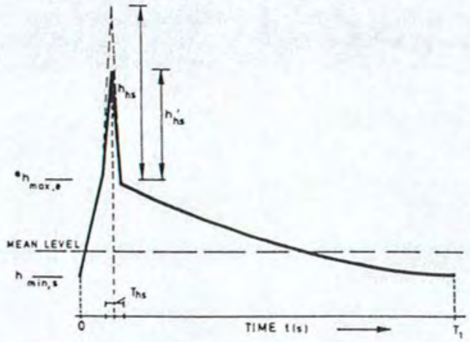

Fig. 3.4 Semi-empirical description of wave impacts

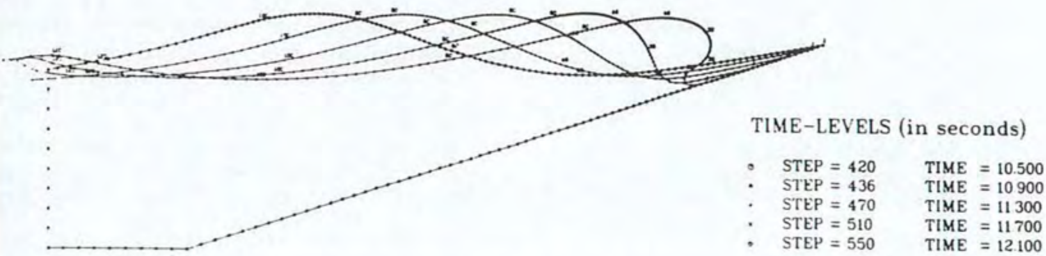

Fig. 3.5 Numerical description of cyclic pressure variations

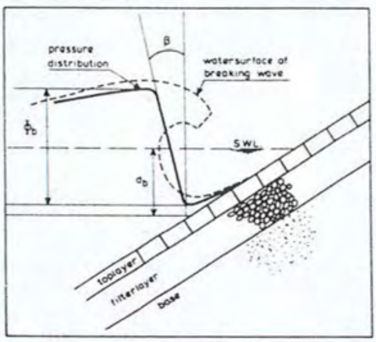

Fig. 3.6 Semi-empirical description of static pressure distribution

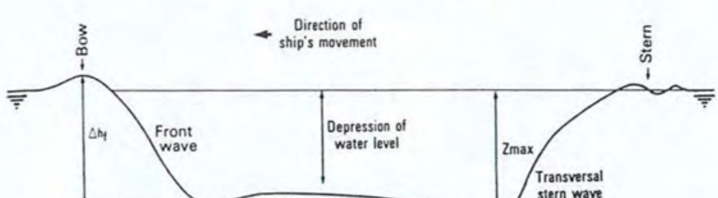

Fig. 3.7A Profile of water surface adjacent to a moving ship

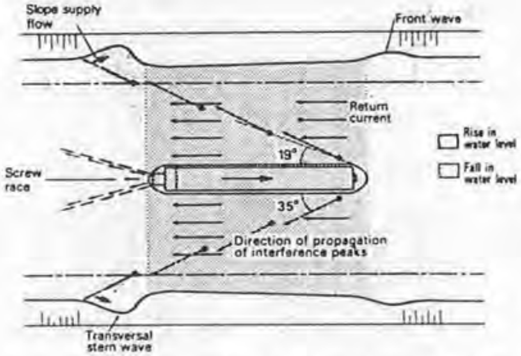

Fig. 3.7B Components of ship induced water motion

presented for each situation. Transfer Function II is discussed in the present chapter and describes the relationship between the external and the internal hydraulic conditions. The flow through the protection is considered to be a special part of the groundwater flow.

Modelling of groundwater flow requires the correct representation of boundary conditions, geometry, forces (momentum) and continuity.

The following phenomena have been considered and the means by which they can be determined in particular situations presented:

- Boundary conditions: characteristic period and dimension of hydraulic conditions along the external surface (Chapter 3)
- Geometry: one, two or three-dimensional groundwater flow (Par. 4.2)
- Forces: flow resistance determined by laminar or turbulent flow (Par. 4.3) and flow resistance of protection compared to that of the soil (Par. 4.4)
- Continuity: the role of phreatic storage (Par. 4.5) and the role of elastic storage (Par.4.6).

The following two statements concerning the correct modelling of the forces and the continuity can be made for nearly all types of bed or slope protection:

- Only hydraulic gradient and flow resistance are important in the momentum equation and acceleration (inertia) needs to be taken into account.
- The effect of groundwater discharge to or from the free water on the free water flow (Transfer Function I) can be neglected (an exception is discussed in Chapter 5).

These statements make it possible to reduce the requirements concerning the

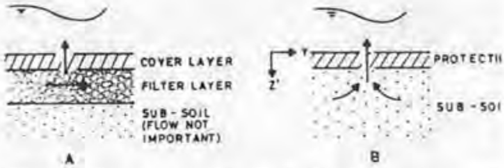

Fig. 4 One-dimensional and two-dimensiona flow. Characteristic situations.

forces to only one single requirement: th ratio of the flow resistances in the mode and in nature should be constant, irrespective of location and time. Similarly there is only one continuity requirement: "the ratio between discharge in the model and in nature should be constant". The complications which arise if the statements cannot be made are discussed by Hölscher et al. 1988.

This single requirement concerning the forces can be further reduced in many cases depending on the flow resistance an turbulence phenomena discussed in Par. 4. and 4.4.

4.2 One, two or three-dimensional flow

In every situation all three dimensions will have some influence. In most situations however the influence of at least one dimension can be neglected.

The assumption of one-dimensional flow can often be justified if a thin filter layer separates the coverlayer of the protection from a relatively impermeable subsoil, see Bezuijen et al. 1987. The flow can be considered to be "one-dimensional" if the flow in the filter layer is parallel to the protection (y-direction) and the flow through the coverlayer perpendicular to it (z'-direction). see Figure 4.A.

Bakker and Meijers demonstrate in their paper on stability against sliding of revetments that very practical solutions can be found if the assumption of one-dimensional flow can be justified.

A typical two-dimensional groundwater flow situation occurs if there is no filter layer present, see Figure 4.B.

4.3 Flow resistance and turbulence

As mentioned in Par. 4.1, the single requirement for modelling forces is, that the ratio between the flow resistances at different locations and at different points of time should be the same in model and nature. If the flow is completely laminar, as is always the case in sand, this requirement is easily met. However, if the flow is turbulent or partly turbulent, complications arise in both physical scale models and in mathematical models.

A PHYSICAL SCALE MODEL is often very useful for solving the Transfer Functions I, II and III simultaneously. Froude scale is required for Transfer Function I and often for Transfer Function III. This scaling requirement can be combined with the single requirement for Transfer Function II, but not in all circumstances. Scaling down causes a reduction of the Reynolds number. If the flow in nature is completely turbulent (coarse rubble) no problems arise, provided that the Reynolds number related to the grain diameter remains larger than about 1000 in the model (Dailey and Harleman 1966, Ch. 9-3). This can often only be realised if the grains are scaled down less than proportionally.

If the flow in nature is not completely turbulent, or is only turbulent at some locations or intermittently, exact scaling can be very complicated. However, it is often sufficient to establish model scales on the basis of proportionality between the hydraulic resistances of only the most critical parts of the construction at the most critical points of time. In some cases it is even sufficient to make the resistance in a part of the model, e.g. the protection, very large or very small compared to that of another part, e.g. the subsoil, as is discussed in Par. 4.4.

The complication caused by turbulence in MATHEMATICAL MODELS has to do with the fact that the relation between hydraulic gradient and hydraulic resistance is no longer linear. The modelling may often be achieved by introducing both a linear and a quadratic term according to the Forchheimer Equation, see Hannoura and Barends (1981), Klein Breteler and Bezuijen (1988), and Bakker and Meijers (1988). The last contribution indicates that an analytical solution can only be found by linearization of this equation. Modelling in numerical models requires special measures like iterative procedures, see Hannoura and Barends (1981) and Hjortnaes-Pedersen et al. (1987).

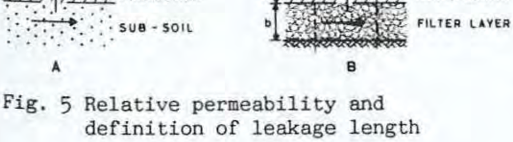

Fig. 5 Relative permeability and definition of leakage length

4.4 Flow resistance of protection compared to subsoil

"Turbulence" is one of the factors involved in the correct modelling of forces. A second follows immediately from the fact that forces can be modelled correctly by meeting one single requirement which relates to the ratio between the flow resistances at different locations (and at different points of time). Two "locations" are of special importance: the protection itself on the one hand and the subsoil on the other.

Special attention should therefore be paid to the ratio between the flow resistance of the protection perpendicular to the surface and the flow resistance of the subsoil parallel to the surface, see Figure 5.A.

This relationship can be expressed by a length dimension, the so-called "leakage length" Λ.

This can be shown most clearly in the case of one-dimensional flow in a filter layer. The leakage length Λ is defined as the length of the piece of protection, l, in which the flow resistance through coverlayer and filter layer are the same.

The flow resistance may be defined as the ratio between the head difference and the discharge per unit length. A linear or linearized relationship between the head gradient and the specific discharge, according to Darcy, with Darcy coefficients of k' for the coverlayer and k for the filter material, yields:

RESISTANCE PIECE OF COVERLAYER = $D/k'l$
RESISTANCE PIECE OF FILTER LAYER = l/kb

The resistances are equal if

$$l = \Lambda = \sqrt{kDb/k'}$$

In the case of sloped protection the "leakage length" λ is often defined as the vertical component of Λ: $\lambda = \Lambda \sin \alpha$.

The practical importance of the leakage length can best be illustrated by some examples.

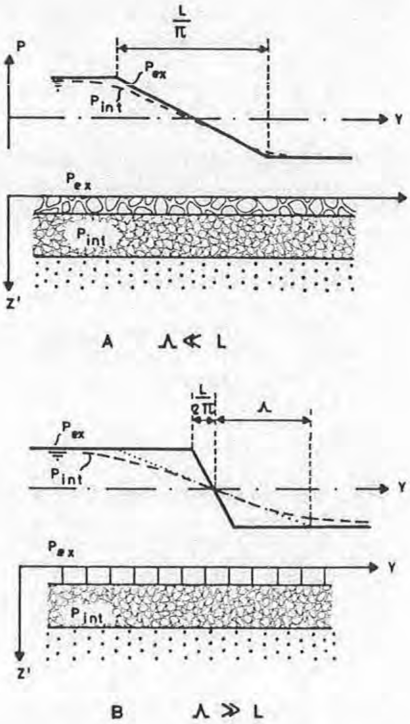

Fig. 6 Internal pressure distribution as a function of Λ/L

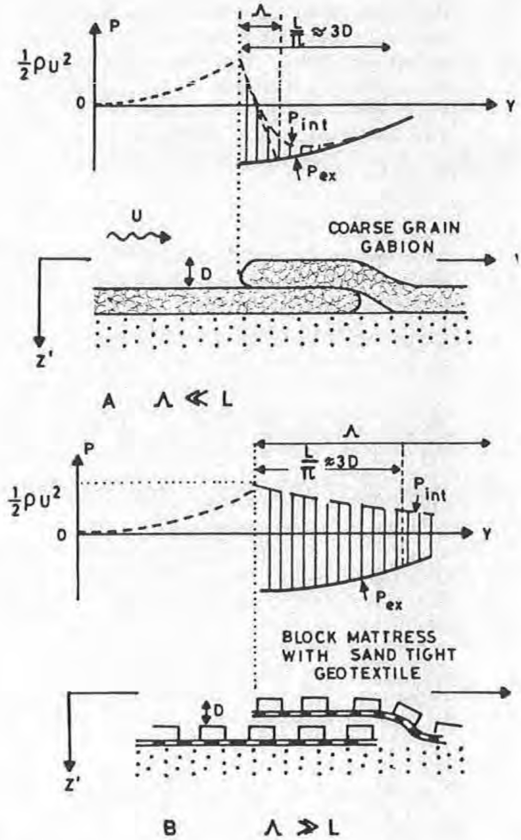

Fig. 7 Pressure difference around the edge of bed protection in currents as a function of Λ/L

The first example, Fig. 6, concerns a continuous bed protection on a filter layer loaded by an external pressure (P_{ex}) which varies along the external surface. The internal pressure (P_{int}) can easily be found for one-dimensional flow and linear resistances by using a simple exponential function.

However, the following conclusions are valid if the flow is in more dimensions and the resistance non-linear.

- If the leakage length, Λ, is much smaller than the largest characteristic dimension of the external pressure, L, so if Λ << L (Fig. 6A), P_{int} is nearly equal to P_{ex}. The small difference between these pressures strongly depends on both Λ and L. The steepest gradient of P_{int}, however, hardly depends on Λ, only on L.
- If Λ >> L (Fig. 6B), however, Λ and L do not have a great influence on the pressure difference. The pressure difference is only determined by the amplitude of the external pressure. In contrast the steepest gradient of P_{int} is completely determined by Λ.

The second example, Figure 7, concerns the pressure difference around the edge of a bed protection in a current, see de Groot and Konter (1984). If Λ << L, the difference between P_{ex} and P_{int} is again small. If Λ >> L, however, the force on the edge is much larger and there is no need to know the exact value of Λ.

The examples clearly show that the requirements for modelling Transfer Functions I or II can often be simplified considerably, if one of the dimensions Λ or L is small or simply large compared to the other.

An important assumption, made implicitly, is that flow through the coverlayer is homogeneous. This is not strictly correct for a protection of blocks with interstices. It is nevertheless acceptable as long as the length and width of the blocks are small compared to the leakage length Λ, see Klein-Breteler and Bezuijen (1988).

4.5 Continuity: the role of the phreatic surface

One of the requirements for modelling the groundwater flow is "correct representation of continuity", see Par. 4.1. Attention should be paid to the role of the phreatic surface, particularly in the case of a slope protection and unsteady loading. In this case the phreatic surface fluctuates. It can be shown, however, that the fluctuation of the phreatic surface in the subsoil is very small compared to the fluctuation of the external water level if

$$\frac{k\ T\ \sin^2\alpha}{n\ \lambda} << 1$$

in which n is the porosity of the subsoil. This is generally the case with slope protections by attacked waves.

Another phenomenon is internal set-up, the term used to describe the increase in level of the internal phreatic surface above the average level of the external phreatic surface (Hölscher et al. 1988). Sellmeijer (1982) has shown that this internal set-up is of minor importance for slope protection stability.
Internal set-up is present only if $H/\Lambda >> 1$. In that situation the condition mentioned above: $\Lambda << L$ is often satisfied and the difference between P_{int} and P_{ex} is small.

In a slope protection the decisive parameter for the calculation of the internal pressure is Λ, not the position at the phreatic surface.

4.6 Continuity: the role of internal storage

Cyclic loading causes elastic and possibly plastic deformation of the soil skeleton and therefore variation in the internal storage. The plastic deformation is the subject of special studies on liquefaction due to cyclic loading, (Lindenberg and van der Weide, 1988) and will not be discussed here. The attention is focussed here on elastic internal storage.
The groundwater pressure distribution at a certain moment will not only be a function of the momentaneous load, but also of time history, if internal storage is important.

The question in which circumstances elastic storage is important can be answered by considering the following length scale, which is partly a function of a characteristic of the hydraulic conditions on the external surface and partly a function of the properties of the subsoil:

$$L_{es} = \sqrt{T\ c_v}$$

in which L_{es} is the length dimension for elastic storage, T is the characteristic time period of the external pressure and c_v is the consolidation coefficient of the subsoil, defined as:

$$c_v = \frac{k}{\gamma_w} \Big/ \left(\frac{n}{K_w} + \frac{1}{K + \frac{4}{3}\ G}\right)$$

in which k is the Darcy permeability of the subsoil, γ_w the specific weight, n the porosity, K_w the compression modulus of the water, K the compression modulus of the grain skeleton and G the shear modulus of the grain skeleton.

The physical meaning of L_{es} can be explained by the following two examples, derived from Yamamoto et al. (1978) and Verruijt (1982):

The first example, Figure 8, is a homogeneous soil with compressible pore water and incompressible soil skeleton, loaded at the surface by a harmonically varying water pressure. The water pressure in the soil can be characterized by a phase shift and by an exponential reduction of the amplitude with depth. The characteristic length of the exponential function is $L_{es}/\sqrt{\pi}$.

The second example, Figure 9, is a homogeneous soil with incompressible pore water and compressible soil skeleton, loaded at the surface by a harmonically varying grain pressure, the pore water at the surface remaining constant. There is also a water pressure phase shift in the soil and the amplitude of this water pressure increases with depth, tending eventually to become equal to the amplitude of the load σ_o. The difference between soil pressure and pore pressure decreases exponentially with depth, again with $L_{es}/\sqrt{\pi}$ as characteristic length.

Yamamoto et al. (1978) and Verruijt (1982) have worked out more complicated examples in which the external load not only fluctuates harmonically in time but also in space, with wave length L. From their analytical solutions the following conclusions could be derived, which are also valid when the hydraulic conditions on the outer surface have no harmonic character:

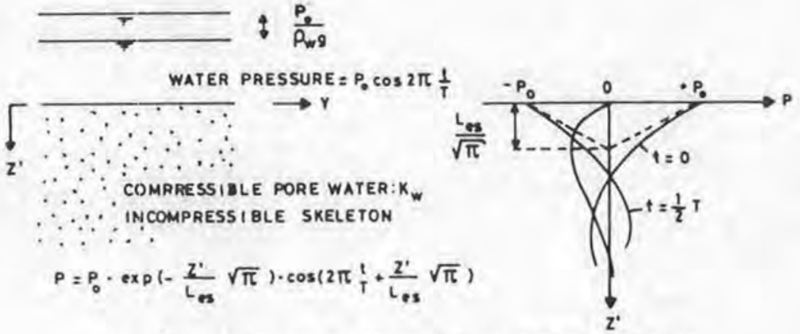

Fig. 8 Groundwater pressure due to elastic storage - compressible pore water

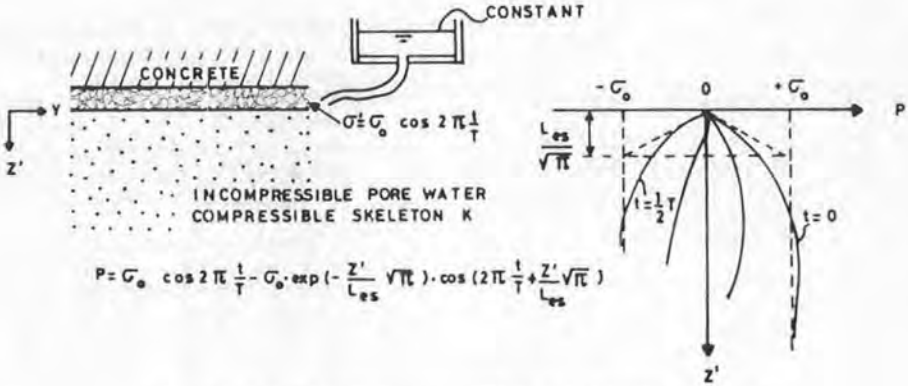

Fig. 9 Groundwater pressure due to elastic storage - incompressible pore water

- Elastic storage is not important if $L_{es} \gg L$. A consequence for bed and slope protection is that elastic storage does not play a role in fine gravel or coarser material because the high permeability produces a high consolidation coefficient and consequently a high value of L_{es}.

- Elastic storage is not important if $\Lambda \ll L$ and $K_w \gg K + \frac{4}{3}G$, i.e. if the coverlayer is very permeable, so that the subsoil is loaded by water pressure variations only and the pore water is relatively incompressible. In this case the external pressure variations are completely absorbed by the pore water.

- Elastic storage is not important if $\Lambda \gg L$ and $K_w \ll K + \frac{4}{3}G$, i.e. if the coverlayer is very impermeable so that the subsoil is loaded by grain-pressure variations only and the soil skeleton is relatively incompressible. In this case, the external pressure variations are completely absorbed by the grain skeleton and the pore pressure remains unchanged.

In all these cases time history is not important and the groundwater pressure distribution is completely determined by the instantaneous hydraulic conditions on the external surface, as far as phreatic storage is also unimportant

In certain other cases elastic internal storage should be taken into account. The importance of the length dimension L_{es} in these cases is illustrated in the example of the pressure distribution in loosely packed fine sand under a rubble slope protection in a canal during the water level depression caused by a passing ship. Probably $L_{es} \ll L$ and, if much air is present in the pores, $K_w \ll K + \frac{4}{3}G$. The pore pressure distribution in the sand will be similar to that shown in the example in Figure 8 and sketched in Figure 10. At a small depth of L_{es} below the sand surface the groundwater pressure remains nearly constant, even though the coverlayer is very permeable. This may lead to a dangerous situation since the soil pressure and therefore the grain pressure decrease considerably, which

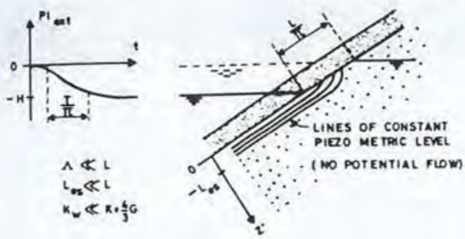

Fig. 10 Groundwater distribution in sand under rubble slope protection during the passing of a ship

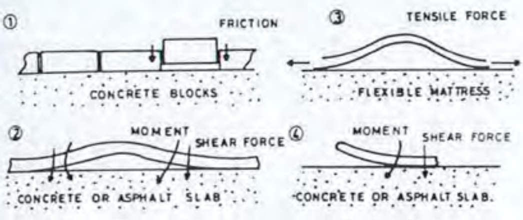

Fig. 11 Types of deformation caused by lift

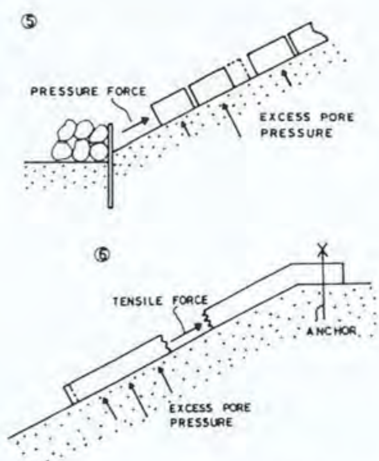

Fig. 12 Types of deformation caused by shearing

could easily cause the slope protection to slide (Schulz and Köhler 1986).

5 STRUCTURAL DEFORMATION

5.1 Stiffness of the protection

Transfer Functions I and II, Figure 2, are discussed in Chapters 3 and 4. These functions determine the load on the structure. Transfer Function III concerns the response of the structure, i.e. the protection, to this load. The possible influence of this response on the other Transfer Functions ("interaction"), an important aspect of this function, is discussed in the present chapter. This type of influence is only present if the load-induced deformation of the protection is large.

The groundwater flow may be influenced during the deformation by additional flow into the open space created between subsoil and the protection being uplifted. The free water flow (waves and/or current) may be influenced in a later stage, after deformation has created a considerable change in the surface geometry of the bed. The importance of these influences is mainly determined by the stiffness of the protection. A survey of the types of deformation of various protections and the corresponding stiffnesses is given in Figures 11 and 12.

The influence of the deformation of the protection on free water flow and groundwater flow is only relevant if a large deformation is needed to mobilize a considerable force in the protection for example, with Type 3, or if the force in the protection remains constant with increasing deformation, for example with Type 1.

Special cases are Types 2 and 4 if the material asphalt is used. These types satisfy both conditions provided the duration of the load is sufficiently long (plastic deformation). It should be noted that the influence is only significant in cases of "uplift", and not in cases of "shearing".

5.2 The influence of protection deformation on groundwater flow

Type 4 deformations hardly influence directly the groundwater flow and therefore P_{int} will only change after a considerable change of geometry. In case of other types of deformation, Types 1, 2 or 3, the uplift causes a reduction in load because the flow into the space between subsoil and the protection requires a reduction of P_{int}.

This reduction is only important if the deformation is sufficiently fast to induce discharge into the open space of at least the same order of magnitude as the groundwater discharge parallel to the protection.

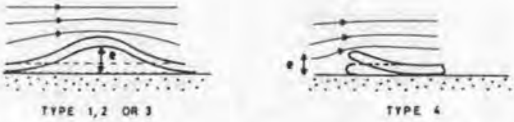

Fig. 13 Streamline concentration caused by protection

The open space discharge is related to the parallel groundwater discharge by the dimensionless parameter:

$$\frac{e_{max}}{T\ k\ i_{max}}\ ,\ \text{no filter layer, or}$$

$$\frac{B\ e_{max}}{b\ T\ k\ i_{max}}\ ,\ \text{with a filter layer}$$

in which:

e_{max} - maximum uplift of coverlayer
i_{max} - maximum gradient of pressure head below coverlayer
B - width of deformation

The influence of deformation can be neglected if the parameter is much smaller than unity.

The way in which the influence of deformation can be taken into account is described by Burg et al. (1980) and by Bezuijen et al. (1987).

5.3 The influence of protection deformation on free water flow

In the previous paragraph attention is paid to the stabilizing influence of a deforming protection on groundwater flow.

The effect of a certain deformation on the free water flow is discussed below. This influence is nearly always unfavourable for stability. The influence is caused by a change in the geometry of the bed or the slope which induces a reduction of P_{ex} by a concentration of the streamlines, see Führböter (1986) and Figure 13.

The influence can be neglected if the maximum deformation e_{max} is small compared to the dimension of the characteristic bed form. In this case the protection thickness D can be taken as the characteristic bed form dimension, and the influence may be neglected if $e_{max}/D \ll 1$.

With a Type 4 deformation, see Fig. 11 and

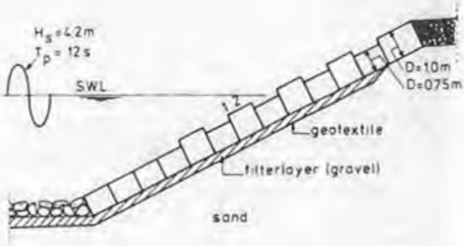

Fig. 14 Danish coast revetment

13, the load-increasing effect of the deformation is not compensated by any load-reducing effect. This implies an explosive increase in the total load. This case is worked out by Konter and de Rijke in their contribution "Scale effects in Modelling the Stability of Asphalt Bed Protection".

They conclude that in most cases deformation is not acceptable under any circumstances. This enables modelling to be simplified considerably.

6 AN EXAMPLE OF MODELLING

The way in which the three Transfer Functions can be used in the design of a placed block revetment is discussed in the present chapter. In order to prevent dune erosion a decision was taken to make a placed block revetment at some locations on the west-coast of Denmark, see Figure 14. The following combination of physical and mathematical models was selected to solve the three Transfer Functions and find the most economic dimensions.

The deep water wave conditions were first transformed to wave conditions near the shore by using the Delft Hydraulics one-dimensional ENDEC computer program. The design incident wave near the shore could be characterized by significant wave height H = 4.2 m and spectrum peak period Tp = 12 s. The pressure distribution, $P_{ex}(y,t)$, along the slope was measured in a physical scale model in a wave flume with irregular waves. According to Chapter 3, the largest dimension and period of the pressure along the external surface could be taken as L = H = 4.2 m and T = Tp = 12 s. In this way Transfer Function I could be determined.
Transfer Function II was solved twice, first to determine the stability of the individual blocks, second to review the

stability against sliding of the slope as a whole. In the first case the STEENZET/1 numerical model (Bezuijen et al. 1987) was used; in the second case the STEENZET/2 numerical model (Hjortnaes-Pedersen et al. 1987). The choice of these models was based on the following considerations.

The stability of the blocks is influenced mainly by the flow in the filter layer, and hardly influenced at all by the flow in the much less permeable subsoil. A one-dimensional model, like STEENZET/1, is therefore suitable for individual block stability (par. 4.2). The pressure distribution in the subsoil, however, is important for the stability against sliding and a two-dimensional model, like STEENZET/2 is required to study overall slope stability.

The flow in the gravel filter layer will be partly turbulent. A carefully linearized Darcy-coefficient therefore had to be used in the STEENZET/1 model. STEENZET/2 is able to deal with non-linear flow resistance. The leakage length is smaller than the above mentioned dimension of the external pressure ($\lambda \approx 0.5$ m). Both the value of the leakage length and the pressure distribution $P_{ex}(y)$ must therefore be known accurately (Par. 4.4). A schematised pressure distribution, as used in the analytical model described by Burger et al. (1988) and Bakker & Meijers (1988) is not available for the shallow water waves in this situation. Therefore numerical models are chosen in which the measured wave pressures can be used and thus an accurate pressure distribution $P_{ex}(y)$ is obtained.

Phreatic and elastic storage are not important for the flow in the filter layer (Par. 4.5 and 4.6). Elastic storage, however, greatly influences the pressure distribution in the sand, because its length dimension is smaller than the length dimension of the external pressure: L_{es} = is about 1 m. This was an important reason for selecting the STEENZET/2 model to study stability against sliding.

Transfer Function III had to be found in order to study: the behaviour of the blocks and possible sliding. The wave pressures P_{ex} and the pore pressures in the filter layer P_{int} determine the loading of the blocks. Since this loading is temporarily larger than the weight of the blocks, the lifting of the blocks was also calculated taking into account the inertia forces and the reduction of P_{int} due to the deformation (Par. 5.2).

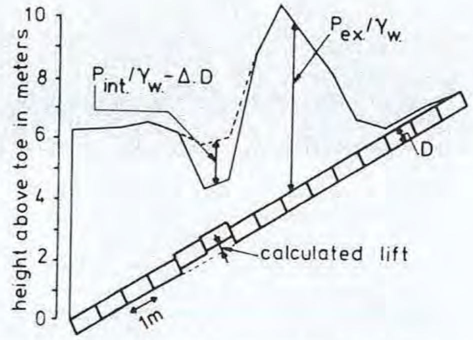

Fig. 15 Measured external head, calculated internal head and calculated lift of the blocks

The STEENZET/1 program is used to calculate the stability of the blocks because it has a special routine for calculating this soil-structure interaction. The result of the calculations at a critical moment are shown in Figure 15.

The movements of the blocks were only calculated in order to get an idea about the safety margin, only in extreme conditions will small movements of one block in a cross-section be acceptable. It is very likely that no movements will occur in the actual structure. The movements will probably be hindered by the clamping forces, which were not taken into account in the calculations.

The safety against sliding was studied with the help of calculations based on the method proposed by Bishop, taking into account the pore pressure distribution assessed with STEENZET/2.

7 CONCLUSIONS

The interaction between soil, water and bed or slope protection can be studied using an approach in which the following three Transfer Functions are defined:

I. From the overall hydraulic conditions, e.g. H, U, to the hydraulic conditions along the external surface, e.g. P_{ex}.

II. From the external surface hydraulic conditions to the conditions along the internal surface, e.g. P_{int}, and

III. The structural response to loads on both surfaces.

The modelling requirements depend on the phenomena that are important in a specific situation. The relative importance of these phenomena can be estimated as follows:

- The groundwater flow can be modelled as one-dimensional flow if a filter layer is present below the coverlayer (Par. 4.2, Fig. 4).
- If $\Lambda \ll L$ i.e. if the hydraulic resistance of the coverlayer is small and the characteristic dimension of P_{ex} is large, then both Transfer Functions I and II must be modelled precisely to find the pressure difference across the coverlayer; Transfer Function II is of very little importance if the steepest gradient of P_{int} is to be found (Par.4.4).
- If $\Lambda \gg L$, then only parts of the Transfer Functions I and II (amplitude of P_{ex} and internal set-up) need to be modelled precisely, to find the pressure difference across the coverlayer; Transfer Function II should be modelled precisely to find the steepest gradient of P_{int} (Par.4.4).
- Precise modelling of the forces in Transfer Function II (groundwater flow) in many cases only requires proportionality of the flow resistance at the most relevant locations at the most relevant points of time (Par.4.3).
- Elastic storage is not important if:
  - $L_{es} \gg L$, or
  - $\Lambda \ll L$ and $K_w \gg K + \frac{4}{3} G$, or
  - $\Lambda \gg L$ and $K_w \ll K + \frac{4}{3} G$

(Par.4.5)

- Uplifting of the coverlayer hardly influences the groundwater flow (Transfer Function II) if (Par.5.2):

$e_{max} \ll T\, k\, i_{max}$ (without filter layer)

$B\, e_{max} \ll b\, T\, k\, i_{max}$ (with filter layer)

- Uplifting of the coverlayer hardly influences the free water flow if $e_{max} \ll D$ (or whatever the characteristic dimension of the bed form is; Par. 5.3).

LIST OF SYMBOLS

B	Width of protection deforming	(L^1)
b	Filter layer thickness	(L^1)
c_v	Consolidation coefficient (definition Par.4.6)	(L^2T^{-1})
D	Coverlayer thickness	(L^1)
e	Deformation (lift) of coverlayer	(L^1)
e_{max}	Maximum uplift of cover layer	(L^1)
G	Shear modulus of grain skeleton	$(ML^{-1}T^{-2})$
H	Wave height	(L^1)
i_{max}	Maximum gradient of pressure head below coverlayer	$(-)$
K	Compression modulus of grain skeleton	$(ML^{-1}T^{-2})$
K_w	Compression modulus of soil water	$(ML^{-1}T^{-2})$
k	Darcy permeability of soil	(L^1T^{-1})
k'	Darcy permeability of coverlayer	(L^1T^{-1})
L	Largest length dimension of hydraulic conditions at external surface	(L^1)
L_{es}	Length dimension of elastic internal storage	(L^1)
n	Porosity	$(-)$
P	Pressure	$(ML^{-1}T^{-2})$
P_{ex}	Pressure along external surface of coverlayer and bed	$(ML^{-1}T^{-2})$
P_{int}	Pressure along internal surface of coverlayer	$(ML^{-1}T^{-2})$
t	time	(T)
T	longest time period of hydraulic conditions at external surface	(T)
T_p	Wave period	(T)
U	Mean current velocity	(L^1T^{-1})
y	Direction parallel to protection	(L)
z	Vertical direction	(L)
z'	Direction perpendicular to protection	(L)
α	Slope angle	$(-)$
γ_w	Specific weight of water	$(ML^{-2}T^{-2})$
Λ	Leakage length for slope $\Lambda = \Lambda \sin\alpha$	(L^1)
Λ	Leakage length Λ	(L^1)
σ	Ground pressure	$(ML^{-2}T^{-2})$
σ'	Grain pressure	$(ML^{-2}T^{-2})$

REFERENCES

Bakker, K.J. and Meijers, P. (1988) "Stability against sliding of flexible revetment" SOWAS Delft

Banach, L (1987) "Evaluation of measurements of the wave pressures on a slope". Delft Hydraulics Project No. H195.20. The Netherlands.

Bezuijen, A., Klein Breteler, M. and Bakker, K.J. (1987). "Design criteria for placed block revetments and granular filters". 2nd. Int. Conf. Coastal and Port Eng. in Dev. Countries, Beijing.

Burg, J.A., Groot, M.B.de and Graauw, A.F.F. de (1980). "Bed protection adjacent to the Barrier". Hydr. Aspects Coastal Structures. Delft University Press.

Dailey, J.W. and Harleman, D.R.F. (1966). "Fluid Dynamics" Addison-Wesley

Publishing Company, Inc. Reading (MA) USA.
Delft Hydraulics and Delft Geotechnics (1986) Hydro Delft Magazine No 73 Special Issue on the Delta Flume, The Netherlands
Flokstra, C. (1986). "Research on the structure of turbulence in the mouth of the Oosterschelde Orientation" (in Dutch), Delft Hydraulics, R 2070.
Führböter, A. (1986). "Hydrodynamische Belastingen der Sohlsicherung des Eidersperrwerkes" (in German), Bau ingenieur 61, 319-328. Springer Verlag.
Groot, M.B. and Konter, J.L.M. (1984). "Prediction of Mattress Stability in Turbulent Flow". Symp. Scale Effects in Mod. Hydr. Structures, Esslingen, September 1984, Ed. Kobus, Stuttgart, FGR.
Hannoura, A.A. and Barends, F.B.J. (1981). "Non-Darcy flow. A state of the art". Proc. EUROMECH 143 "Flow and Transport through Porous Media" (ed. Verruijt and Barends), A.A. Balkema Publ. P.O. Box 1675, Rotterdam, 1981.
Hjortnaes-Pedersen, A.G.I., Bezuijen, A. and Best, H., (1987). "Non-stationary flow under revetments using the Finite Element Method", 9th. Euro. Conf. Soil Mech. and Found. Eng., Dublin, Aug/Sept. 1987.
Hölscher, P., Groot, M.B. de and Van der Meer, J.W. (1988). :Simulation of internal water movement in breakwaters" SOWAS, Delft (1988).
Klein Breteler, M. and Bezuijen, A. (1988). "Permeability of closely placed blocks on gravel". SOWAS Delft.
Klein Breteler, M., Burger, A.M., Bezuijen, A. and Banach, L. (1988) "Design method for block revetments", 21st. Conf. Coastal Eng., Malaga, Spain.
Klopman, G. (1987) "Numerical simulation of breaking waves on steep slopes". ASCE speciality conference on coastal hydrodynamics. Delaware USA.
Klopman, G. (1988) "Application of the two-dimensional boundary element program BEACH to free surface potential flows on slopes". Delft Hydraulics Project H195.21. The Netherlands.
Konter, J.L.M. and de Rijke, W.G. (1988). "Scale effects in modelling the stability of asphalt bed protections", SOWAS, Delft.
Lindenberg, J.L.M. and Weide, J. van der. (1988). "Influence of wave pressure penetration of structures" SOWAS, Delft.
PIANC (1987) "Guidelines for the design and construction of flexible revetments incorporating geotextiles for Inland Waterways". Report of Working Group 4 Brussels, Belgium.
Schulz, H. and Köhler, H.J. (1986). "Use of geotextiles in hydraulic constructions in the design of revetments", 3rd. Int. Conf. Geotextiles, Vienna, Austria.
Stive, R.J.H. (1984) "Wave impact on uniform steep slopes at approximately prototype scale". IAHR Symposium on "Scale effects in modeling hydraulic structures" Esslingen, Federal Republic of Germany.
Stoker, J.J. (1957) "Water waves: The mathematical theory with applications". Interscience Publ. Inc. New York, USA
US Army Corps of Engineers (1984) Shore Protection Manual. Washington, USA.
Verruijt, A. (1982). "Approximations of Cyclic Pore Pressures Caused by Sea Waves in a Poro-elastic half-plane". Chapter 3 of "Soil Mechanics Transient and Cyclic Loads" ed. G.N. Pande and O.C. Zienkiewicz, 1982, John Wiley & Sons Ltd.
Vinje, T. and Bravig, P (1981) "Numerical Simulation of Breaking Waves" Adv. Water Resources 4, pp 77-82
Yamamoto, T., Koning, H.L., Sellmeyer, H. and van Hijum, E.."On the response of a poro-elastic bed to water waves", J. Fluid Mech., 87, 193-206, 1978.