Profile development of a nourishment behind a submerged breakwater

Master's thesis at the Delft University of Technology

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Faculty of Civil Engineering
Division of Hydraulic and Geotechnical Engineering
PROFILE DEVELOPMENT OF A NOURISHMENT
BEHIND A SUBMERGED BREAKWATER

COMPARISON OF DIFFERENT PROFILES
For the breakwater with crest at NAP=-2m

By: Theo Winter

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PREFACE

This paper is the master’s thesis of Theo Winter at the Delft University of Technology.

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ABSTRACT

In this report the profile development of a nourishment behind a submerged breakwater is investigated. This is done using the UNIBEST-TC program and a more qualitative approach. UNIBEST-TC is a cross-shore profile development simulating program developed by Delft Hydraulics.

It appears that the following three processes, resulting from the presence of the breakwater, are important:

1. Change in wave height
2. Change in wave period
3. Transition zone width or time lag

A literature review is undertaken to understand the causes and the effects of the first process. Using the resulting equations, the UNIBEST-TC program is verified for the process of wave transmission over and through a submerged breakwater. Due to the fact that only impermeable breakwaters can be implemented in the program, deviations occur. For conventional breakwaters the agreement is quite good. For reef breakwaters it is not accurate enough. Due to the fact that a conventional submerged breakwater will be used, the approximations as computed with UNIBEST-TC will suffice.

The cause and results of the second process are qualitatively more or less known. However, more quantitative data are needed.

The third process is very important because it causes a spatial difference between the point of breaking and the position of maximum return flow. It determines the amount and effect of scour just behind the breakwater. It seems that these effects are slightly overpredicted by the program. Model tests and extensive measurements will be needed to verify this assumption.

All three processes together result in a steeper slope of the nourishment behind the breakwater, as compared to the reference slope. The amount of shoreline retreat is computed for a nourishment behind breakwaters with the crest located at NAP -2 m and NAP -4 m. A stability analysis on the primary armour layer as used on these two breakwaters was carried out. Using the results of that analysis, a rough estimate of the costs for both breakwaters is derived. Adding these figures to the costs needed for the nourishments behind the breakwaters, a comparison is made between these two protected nourishments and an unprotected nourishment. Both result in a more or less stable coast enlargement of 1000 m.

The conclusion is that the protected nourishment is economically a better option than the unprotected one. Due to the fact that the overall costs for the two protected nourishments do not differ much, the one with the lower crest (NAP -4 m) is recommended.
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ABSTRACT

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1.1: General introduction

In recent years the idea of expanding the Netherlands by artificially moving the coastline seaward, has gained more and more interest. The distances over which this moving has to take place are rather large; in the order of kilometers. Therefore a very large amount of sand is needed per alongshore meter, especially if this enlargement is to be carried out by an unprotected beach nourishment. Such an unprotected beach nourishment is shown in Figure 1.1 as well as the original, reference profile. If the wave climate does not change and the nourishment sand has the same diameter as the original sand, the profile shape of the nourishment will be the same as the original, reference profile shape. In other words, the whole cross-shore profile is moved seaward over a distance of 1000 m. A beach width enlargement of 1000 m is achieved in Figure 1.1 by using an amount of approximately 19500 m$^3$ sand per alongshore meter. In other words, if an enlargement of the beach of 1 km is needed over an alongshore distance of for instance 10 km, a total amount of 195 million m$^3$ sand is needed. This is indeed a large amount. Do note that the scales of the two axes as featured in Figure 1.1 are different. The vertical axis gives values in meters whereas the horizontal axis states values in kilometers.

![Figure 1.1 A cross-shore profile with an unprotected nourishment](image)

The idea behind this study is to minimize the needed amount of sand by using a submerged breakwater behind which a beach fill is placed. In that way the lower, most seaward part of the nourishment becomes superfluous. This superfluous part is the shaded area in Figure 1.2. This figure also features a submerged breakwater. Due to the small horizontal scale the breakwater is represented by a vertical line.

1.2: Aims of this study

If the breakwater could be placed at a certain intersection point with the unprotected nourishment as displayed in Figure 1.2, and the same profile just behind the breakwater
would be the result, the same enlargement of the beach width is achieved. However, this is not the case. The breakwater influences the wave height and wave period. These wave characteristics influence the cross-shore transport and thus the shape of the profile. As a result the profile of the nourishment behind the breakwater will not be the same as the profile of the unprotected nourishment. This is schematically shown in Figure 1.3. The dotted lines (a and b) are possible profile shapes which deviate from the unprotected nourishment shape (c). As mentioned earlier, the profile of the unprotected nourishment is basically the same as the original, reference profile (c').

This leads to the first aim of this study: to determine in what way the shape of the profile of a nourishment behind a submerged breakwater differs from a more or less equilibrium reference profile shape. In this reference profile the breakwater is not present.
Figure 1.4 The definition of the attachment point A

Figure 1.5 The amount of shoreline retreat (R) resulting from a change in position of the attachment point A (over a distance Y)

Figure 1.4 features a submerged breakwater with a nourishment behind it. The attachment point of the nourishment to the breakwater is indicated by A. This attachment point will probably not be located at the crest of the breakwater due to scour influences and the influence of the permeability of the top layer of the breakwater.

If however the attachment point is located at a greater depth, the shoreline will retreat compared to the original situation. This is shown in Figure 1.5. If point A is moved to a greater depth over a distance Y, the shoreline will retreat over a distance equal to R. That results in a smaller beach enlargement, namely the original enlargement minus the value of R. This leads to the secondary aim of this study: to be able to predict the amount of
shorline retreated resulting from the position of the attachment point of the nourishment to the breakwater.

The third aim of this study consists of a rough comparison between the costs of on the one hand the coastal enlargement through a nourishment placed behind a submerged breakwater, and on the other hand through an unprotected nourishment.

1.3: Type of breakwater

According to Van der Meer (1990b) three types of low crested breakwaters can be distinguished. To distinguish between two of these types use is made of the so called freeboard ($R_e$). This freeboard is the distance between the top of the crest of the breakwater and the water level (see also Figure 1.6). The freeboard has a negative value if the breakwater is submerged. The first type is the so called dynamically stable reef breakwater. This is a homogeneous structure without a core or filter layer present. The idea behind such a reef breakwater is that its form changes due to the wave attack. This is illustrated in Figure 1.6 by the difference between the initial profile and the damage profile.

![Figure 1.6 Cross-section of a reef breakwater (laboratory scales), after Ahrens](image)

Due to the rather large permeability of the reef breakwater type it is not very well suited for substituting the toe of a nourishment since the sand will move through the breakwater too easily.

The second type is the statically stable low crested breakwater with a freeboard greater than zero ($R_e > 0$). These structures are more stable than non-overtopped structures, because part of the wave energy passes over the breakwater. They do resemble them very closely though, see Figure 1.7.

The breakwater will be located in front of the coast causing together with the nourishment a beach widening. Since the beaches in the Netherlands are a main touristic feature and a lot of watersports take place, it is inconceivable to use a crest located any higher than at NAP -2 m.
This leads to the conclusion that the breakwater with a freeboard greater than zero is not the right type for this study.

That leaves the third type, namely the statically stable submerged breakwater with a negative freeboard ($R_c < 0$). This type is characterized by overtopping by all the waves and its stability increases rapidly if the crest height decreases (Losada et al [1992]). An example of this last type is shown in Figure 1.8. This is the type of breakwater which will be used in this study.

Figure 1.8 Statically stable submerged breakwater ($R_c < 0$), according to Van der Meer (laboratory scales)

1.4: Lay-out

In order to fulfil the three aims as stated in section 1.2, the occurring physical processes will have to be understood. The submerged breakwater will influence both the wave height and the wave period. Since these wave characteristics determine the amount of cross-shore transport and thus the shape and slope of the profile, this influence is important. It is expected that the influence of the breakwater will become less if the crest is located further below the mean water level.
To be able to simulate the profile development, use is made of the UNIBEST-TC program as developed by Delft Hydraulics. This is a dynamic profile model developed to be used for cross-shore processes. Since the program is primarily meant to be used on gradually changing bottom profiles, problems may occur when implementing an abrupt transition such as a submerged breakwater. The fact that only impermeable structures can be implemented, as well as the fact that the model is only calibrated for spilling- and not for plunging breakers, might cause problems. Therefore it is important that the general ideas behind, and the schematizations and formulae within, the program are understood. This is discussed in Chapter 2.

In order to concentrate on only the cross-shore effects, a certain location has to be chosen which does not suffer from a substantial longshore transport gradient. This cross-sectional area should be as stable as possible. This choice of location is presented in Chapter 3, as well as the processes and boundary conditions which influence the cross-shore profile. Most of these boundary conditions will be implemented in the UNIBEST-TC program.

The natural occurring profile and its schematization as used in the UNIBEST-TC program are given in Chapter 4. This Chapter also features the more or less equilibrium profile which is used as reference profile.

A literature review was undertaken on the subject of wave transmission over and through breakwaters. The amount of wave height reduction due to the presence of the breakwater is indicated by the value of the transmission coefficient, $K_t$. The complete review on this transmission coefficient is given in Appendix G. In Chapter 5 only the relevant resulting equations are reprinted. A description of how the submerged breakwater influences the wave period is also given. By using the resulting equations of Chapter 5, it is possible to verify the UNIBEST-TC program for the process of wave transmission over and through a submerged breakwater.

A comparison between different transmission determining formulae as well as a comparison between the theoretical transmission values and the ones as computed with the UNIBEST-TC program is stated in Chapter 6. In Chapter 7 artificial beach nourishments and their approximate costs per cubic meter are discussed. The needed amount of sand for a specific beach widening of 1000 m is also computed. In Chapter 8 a choice regarding the placement and the dimensions of the breakwaters to be used is made. The stability of the primary armour is discussed and the costs per breakwater per alongshore meter are computed.

Important effects resulting from the presence of the submerged breakwater as compared to the situation where the breakwater is not present, are discussed in Chapter 9. The resulting profiles behind the breakwater are also outlined. The shoreline retreat as a function of the vertical coordinate of the attachment point A (see Figure 1.5), is presented in graphical form. It is computed by using the UNIBEST-TC program and by using a more qualitative approach. This approach is used because some doubts exist about the results as computed with the UNIBEST-TC program. A rough cost comparison is made in Chapter 10 between the two types of beach enlargement. In Chapter 11 problems which might arise due to the alongshore transport modification resulting from the presence of the protected nourishment are discussed. In the last Chapter 12 conclusions are drawn and recommendations are made.
CHAPTER 2: UNIBEST-TC

2.1: Cross-shore profile models

The UNIBEST-TC program plays an important role in this study. It is important that the underlying ideas as well as the used schematizations are understood. This Chapter deals with these ideas and schematizations. The most important formulae are stated. For more specific formulae reference is made to various literature as cited and to Appendix A, which is a reprint of the UNIBEST-TC User's manual (1992).

Four types of models can be distinguished according to Roelvink and Hedegaard (1993). These models will be mentioned and briefly described in this paragraph.

- Descriptive models

These are very useful as qualitative models. These models can point out typical topographies and transitions for a wide range of natural beaches. The most important parameters which determine the cross-shore morphology have been identified in these models. For quantitative purposes however, they cannot be used, unless a wide range of possible outcomes may be accepted. Another serious drawback is that results of man-made changes in the profile cannot be predicted.

- Equilibrium profile models

It is accepted that for certain conditions cross-shore profiles tend to develop towards an equilibrium. As long as longshore gradients are of no significant influence and the time scale is such that an equilibrium can be reached, these models are useful for (sandy) beaches and dunes.

- Empirical profile evolution models

These type of models determine the difference between the existing profile and the theoretical equilibrium profile. The latter is based on an equilibrium model as mentioned in the above paragraph. The assumption is made that the amount of sediment transport is a direct function of the difference between these two profiles. The more they differ from each other the greater the transport will be. Unfortunately a certain number of empirical coefficients are needed which have to be determined for each specific project. This of course limits the overall applicability of the models. They do not need a lot of computing capacity, however.

- Dynamic profile models

Various different processes which determine the development of a profile are taken into account in these models. The degree of sophistication varies per model. The main advantage of such a model is that the location of a specific profile does not influence the applicability. The drawback is that there are still a lot of processes not well understood in these models. Hopefully this will improve in time. The number of computations needed is rather large.
The UNIBEST-TC program belongs to the last type of models. An elaboration of this type is given in Section 2.2. Since UNIBEST-TC is developed to be used as a purely cross-shore model, the variations in longshore direction will not be taken into account in the following section. It has to be mentioned here that for instance Stive and Roelvink (1988) attempt to couple this cross-shore model to a horizontal two dimensional model and derive interesting results for a case with an alongshore gradient in the sediment transport.

2.2: A dynamic profile model

The parameters which influence the sediment transport are the seaward boundary conditions (such as the wave height \(H\), wave period \(T\), and surge levels), the sediment properties (grain size \(D_{50}\), the original shape of the cross-shore profile (depth \(h\)), tide velocities and water level deviations, and a possible longshore current (Roelvink and Stive [1989a]). Due to the fact that alongshore variations will not be taken into account, this alongshore current will be constant in the cross-shore area considered, i.e. there will be no alongshore gradient.

Using the sediment volume continuity equation:

\[ \frac{\delta z_b}{\delta t} + \frac{1}{1-n} \frac{\delta \bar{q}}{\delta x} = 0 \]  

(2.1)

with for the time-averaged cross-shore volume transport rate:

\[ \bar{q}(x) = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} U(x,z,t) C(x,z,t) \, dz \, dt \]  

(2.2)

with

\[ n = \text{porosity of the bed material} \]  

[-]  

\[ U = \text{horizontal velocity} \]  

\[ [m/s] \]  

\[ C = \text{volume concentration of sediment} \]  

[-]

the change of the bottom level, \(z_b\), can be computed after a certain time step \(\Delta t (\Delta t = t_2 - t_1)\). This procedure can be repeated as many times as necessary. It is important that during a time step the boundary conditions do not vary. The basic procedure is outlined in Figure 2.1.

It is much too complicated to solve (2.2) for all time and length scales, since a complete time-dependent solution is needed. Therefore a schematization has to take place. First of all the time scales are separated in the following way: time scales of variation of the mean parameters of the wave field \((U)\), of wave groups and long waves \((u_{lg})\), of wind waves \((u_{wh})\) and of turbulence \((u')\). A similar time scale dependent split up of the concentration can be made.

Since only products of the same time scale are not equal to zero when time-averaged, equation 2.2 changes into equation 2.3.

Still further schematization has to take place. It is in this stage that different models are
The basic concept is the following (Roelvink [1988]): the distribution of the wave height over the profile is calculated by the wave modelling program ENDEC (Section 2.3). The currents resulting from the waves (flow field) are a function of local parameters of the irregular wave field like the wave energy, the dissipation and mass flux above the mean trough level (Section 2.4 Hydrodynamic aspects). Using the thus derived flow parameters in a transport model (Section 2.5 Sediment transport model), the transport distribution along the cross-shore profile can be calculated.

By using the continuity equation as given by formula 2.1, the profile changes are then determined. The bottom level is computed numerically after a certain time step and the whole procedure is repeated again (this is shown in Figure 2.1 by the arrow from the lowest, fourth box going up to the second box). In this Chapter 2 the most important overall formulae are stated as well as the underlying ideas behind the different models. For some more specific formulae the reader is referred to Appendix A as well as to various literature as cited.

2.3: ENDEC

ENDEC stands for ENergy DECay. It is primarily designed to take account of the effect of the depth-induced breaking in near shore regions. Two coupled differential equations, the wave energy balance and the mean momentum balance, are solved to obtain a distribution of the wave heights and of the wave induced set-up over the cross-shore profile.
ENDEC is primarily based on the energy model as derived by Battjes and Janssen (1978). In this model the assumption is made that for each depth \( h \), a maximum possible wave height \( H_m \) is defined and that all the waves which are breaking (or are already broken) at this depth \( h \) have a height equal to \( H_m \). To be able to compute the root mean square of all wave heights, \( H_{rms} \), the following (simplest form) of the energy balance is used. The rate of energy dissipation per horizontal area, \( D_b \), is used as a sink:

\[
\frac{\delta P_x}{\delta x} + D_b = 0
\]  
(2.4)

with

- \( P_x \) = onshore energy flux per unit width \([\text{W/m}]\)
- \( E C_g \)
- \( E \) = wave energy \([\text{J/m}^2]\)
- \( E = \frac{1}{8} \rho g H_{rms}^2 \)
- \( C_g \) = group velocity for \( f = f_p \) \([\text{m/s}]\)

If \( Q_b \) represents the local fraction of breaking waves then the following relation is derived:

\[
\frac{1 - Q_b}{\ln Q_b} = -\left( \frac{H_{rms}}{H_m} \right)^2
\]
(2.5)

The value of \( H_m \) is computed using the following formula based on Miche's criterion for the maximum height of periodic waves.

\[
H_m = 0.88 \ k^{-1} \tanh(\gamma kh/0.88)
\]
(2.6)

The value of \( k \) is derived from (being the positive real root):

\[
(2 \pi f_p)^2 = g k \tanh kh
\]
(2.7)

with

- \( f_p \) = peak frequency of the energy spectrum \([\text{s}^{-1}]\)
- \( g \) = gravitational acceleration \([\text{m/s}^2]\)

The \( \gamma \) in formula 2.6 was used to account for influences of bottom slope and incident wave steepness. Battjes and Stive (1985) found that \( \gamma \) did not vary significantly with the beach slope but did vary slightly with the deep water (as denoted by the subscript \( \text{op} \)) wave steepness, \( s_{op} \). They derive the following expression for \( \gamma \) under the condition that \( \alpha = 1 \) (for the definition of \( \alpha \) see formula 2.10):

\[
\gamma = 0.5 + 0.4 \tanh(33 s_{op})
\]
(2.8)

with

- \( s_{op} = H_{rms,\text{op}}/L_{op} \) \([-]\)
- \( L_{op} = g/(2 \pi f_p^2) \) \([\text{m}]\)

This is graphically displayed in Figure 2.2. In UNIBEST-TC both \( \alpha \) and \( \gamma \) have to be specified by the user. Only one value per run can be specified. This value has to be a mean value since the deep water wave steepness might change during the run time.
Battjes and Janssen (1978), following Le Méhauté (1962), estimate the energy dissipation rate in a breaking wave to be more or less the same as in a bore of the same height connecting two uniform flow regions. This is displayed in Figure 2.3. On the left hand side of the picture a single, steady bore is featured, on the right hand side one out of a sequence of broken waves on a beach.

Due to the fact that the flow conditions are not uniform, an order of magnitude relation (and not an exact formulation) follows from this modelling. The rate of energy dissipation per unit horizontal area, $D_b$, is determined as in equation 2.9.

$$D_b = \frac{1}{4} f \rho g \frac{H^3}{h}$$

(2.9)

with $f = \text{wave frequency}$ [$s^{-1}$], $\rho = \text{mass density of water}$ [kg/m$^3$]

If equation 2.9 is to be used for random waves, $f$ is replaced by $f_p$, $H$ is replaced by $H_m$ since the dissipation is only the result of the broken waves, and the chance of occurring of these waves is implemented by adding $Q_b$. The ratio $H_m/h$ was omitted since it was assumed to be near to unity in the area of most dissipation. This results in formula 2.10.

$$D_b = \frac{1}{4} \alpha Q_b f_p \rho g H_m^2$$

(2.10)

with $\alpha = \text{coefficient of order 1}$ [-]

So $H_{ma}(x)$ can now be determined for a given depth profile and given incident wave parameters, since if $\gamma$ is computed from the deep water wave steepness according to
formula 2.8, the value for $\alpha$ is equal to 1. Equation 2.4 has been extended for use in UNIBEST-TC by adding the possible components $V$ (alongshore directed depth-averaged velocity) and $\theta_b$ (incidence angle of the waves) to account for changes due to refraction. An extra term to include the bottom dissipation is also added, the $D_f$:

$$D_f = \frac{1}{8} \rho \, f_w \, \pi^{-1/2} \left( \frac{\omega \, H_{rms}}{\sinh (kh)} \right)^3$$

with $f_w = \text{friction factor}$, $\omega_r = \omega - k \sin \theta_w \, V$, $\omega = 2\pi/T$.

So the total energy balance as implemented in UNIBEST-TC is given by:

$$\frac{\delta}{\delta x} \left( \frac{E \, C_g \, \cos \theta_w}{\omega_r} \right) = - \left( \frac{D_b + D_f}{\omega_r} \right)$$

The mean momentum balance is given by Battjes and Janssen (1978) as:

$$\frac{\delta S_{xx}}{\delta x} + \rho \, g \, h \, \frac{\delta \eta}{\delta x} = 0$$

with the radiation stress component $S_{xx}$ defined by:

$$S_{xx} = \left( \frac{1}{2} + \frac{2 \, k \, h}{\sinh 2kh} \right) \, E$$

with $d = \text{bottom depth below SWL}$, $\eta = \text{wave-induced set-up}$, $h = d + \eta$.

In UNIBEST-TC the definition for the radiation stress also includes the incidence angle of the waves (Appendix A).

Both Battjes and Janssen (1978) and Battjes and Stive (1985) state that the wave height prediction across the surf zone is predicted well if compared to test results, especially when using the $\gamma$-equation as given by formula 2.8. However, it is also mentioned that although the maximum set-up of the mean water level is predicted well, the transition between set-down and set-up is consistently predicted too far seaward.

The transition zone, in which rapid wave decay without an increase in energy dissipation is featured, also called spatial- or time lag, is extensively discussed in Section 9.2.4. Svendsen (1984) describes this transition zone as a region with a nearly horizontal water level from just inside the breakpoint to the beginning of a steep water level gradient. In

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1Van der Meer (1990a) notes that the ENDEC model underpredicts the breaking significant wave height on a very gentle foreshore. However, since this does not apply to the situation studied in this report, it doesn’t have to be taken into account.
this Section 2.3 only the essence as featured in the program is discussed. Roelvink and Stive (1989b) suggest that the organized wave energy has to be converted into turbulent energy which is not dissipated immediately. That is the reason why they propose to use a storage term by implementing a \( k \)-equation defined as equation 2.15.

\[
\rho \beta_f \frac{\delta}{\delta x} (k_i \kappa c) = D_b - \rho \beta_d k_1^{3/2} \tag{2.15}
\]

with

- \( k_i \): depth mean time-averaged turbulence intensity \([m^2/s^3]\)
- \( c \): wave propagation speed \([m/s]\)
- \( \beta, \beta_d \): coefficients of order 1

The first term of equation 2.15 is the storage term, the second is the production term and the third consists of the actual turbulent dissipation \( D \). So the \( H_{ms} \) value is computed from equations 2.12 using (2.10) and (2.11) and when using that value the parameter \( k_i \) as well as the turbulent dissipation can be solved from 2.15. In UNIBEST-TC formula 2.15 is used with \( c = c \cos \theta_w \). The value of the actual turbulent dissipation is needed to determine the return flow as will be outlined in Section 2.4.

### 2.4: Hydrodynamic aspects

When looking at equation 2.3, it is clear that four different time scales are present, each with its own determining parameters and effects. To start with the last term; generally speaking the effect of turbulent fluctuations of the horizontal velocity is so small compared to the other terms, that it is neglected. That leaves three different time scales for the horizontal velocity, each of which is modelled in UNIBEST-TC in a different way.

The first and largest time scale features the return flow under breaking waves which transports sediment suspended in the vertical. This flow is modelled using the formulations as presented by De Vriend and Stive (1987). The second, smaller time scale, is related to wave groups and the generated long waves accompanying these wave groups. Since a slow variation in orbital velocity amplitude causes a variation in concentration on the same time scale, this term might be significant. It is modelled according to equations found in Roelvink and Stive (1989b). The third and smallest time scale (since the fourth term is omitted) features wave asymmetry and lag effects within a certain wave period. The orbital velocity near the bottom resulting from short waves determines the possible onshore wave asymmetry transport. It is modelled using the model RFWAVE and described in the user manual by Klopman (1989). All three will be described briefly in this paragraph.

To determine a value for the wave mean cross-shore current (secondary current) use is made of the so called three layer model of De Vriend and Stive (1987). This secondary current or undertow compensates for the shoreward mass flux above the wave trough level. It results from the imbalance between the set-up and radiation stress. Stive and Wind (1986) show that the difference between this imbalance in the shoal zone and in the surf zone is large. Following Stive and Wind (1986), De Vriend and Stive (1987) model the effects of the surface layer through a shear stress at trough level \( (\tau) \) which compensa-
tes for the momentum decay above it, and by the condition that the net undertow compensates the mass flux in the surface layer. This shear stress is proportional to the ratio of wave energy dissipation and wave phase speed, $c$. The shear stress is built up of contributions resulting from the non-breaking wave part ($\tau^{(\text{nb})}$) and of the breaking part ($\tau^{(\text{br})}$) of the waves.

$$\tau = \tau^{(\text{nb})} + \tau^{(\text{br})} = \rho \nu_t \frac{U_{orb}^2}{c} \sinh(2kh) + \left(\frac{1}{2} + \frac{7}{h} \right) \frac{D}{c} \quad (2.16)$$

with 
- $\nu_t$ = eddy viscosity [m$^2$/s] 
- $U_{orb}$ = near bottom oscillatory velocity amplitude [m/s] 
- $L_b$ = wave length at breaking [m]

The eddy viscosity in the bottom layer is an order of magnitude lower than in the middle layer. Initially the value of $D$ was taken from the wave prediction equation 2.4 ($D_{\text{b}}$). Roelvink and Stive (1989b) showed that using the $D$ value out of formula 2.15 gave improved results when undertow calculations are compared to test results. The factor $(1/2 + 7(h/L_b))$ was dropped in the equation used in UNIBEST-TC, in accordance with recent views on the shear stress at trough level.

Using the momentum balances for the middle and lower layer (Appendix A), the mean flow, $U$, can be solved. Five conditions are needed since the momentum balance is a second-order differential equation and the wave averaged surface elevation is also still not known. The first three conditions follow from the shear stress at trough level, the no-slip condition at the bottom and the earlier mentioned integral mean flow condition to compensate for the net mean flow above trough level (Appendix A). The two remaining conditions result from patching the velocities and shear stresses at the boundary between the middle and lower layer. This results in a value for the secondary current.

Due to the grouping of short waves in a random wave field, long waves are generated. Roelvink and Stive (1989b) found that these waves have a certain effect on the transport of sediment and derived a certain expression to include these effects. It is assumed that the wave group-related features of a random wave field can be represented by a bi-chromatic wave train with an accompanying bound long wave.

Using a transfer function and the condition that the random wave must have the same surface variance as the schematized wave train, the near bottom time-varying flow resulting from the interaction between the short- and long wave flow is given by:

$$u_{bi} = \hat{u}_m \cos[(\omega_p)t] + \hat{u}_n \cos[(\omega_p + \Delta \omega)t] + \hat{u}_l \cos[(\Delta \omega + \phi)t] \quad (2.17)$$

where $\Delta \omega$ is equal to $1/5\omega_p$ and the subscript $m$ and $n$ refer to the short waves whereas $l$ refers to the long wave. $\hat{u}_n$ and $\hat{u}_m$ are computed with linear wave theory from the derived surface elevation amplitudes, whereas $u_l$ is calculated using a long wave approximation (Appendix A). In the case of a complete bound long wave situation the value for $\phi$ (phase shift between the long wave and short wave envelope) is equal to $-\pi$. It appears that seaward from the surf zone the correlation between the long wave and short wave
envelope is slightly negative. This is due to the influence of bound long waves. The correlation is positive inside the surf zone.

The oscillatory velocity moments near to the bottom resulting from short waves are computed using the model RFWAVE, based on a Fourier approximation of the stream function method as developed by Rienecker and Fenton (1981). It is a high order, non-linear wave theory using wave energy as the input. These orbital velocities are important because they determine the strength of the onshore transport resulting from wave asymmetry (Stive and Roelvink [1988]).

2.5: Sediment transport model

UNIBEST-TC has evolved from the models OSTRAN and CROSTRAN. The first model was proposed by Stive and Battjes (1984). In this model the mean flow contribution was seen as the most important one, and the transport equation was then simplified to equation 2.18.

\[ q(x) = B \int_{0}^{h+n} \overline{U} \overline{C} \, dz \]  \hspace{1cm} (2.18)

\text{with} \quad B = \text{coefficient (0.25-1.0)} \quad [\text{-}]

The coefficient B incorporates all the deviations resulting from the neglecting of certain terms. However, B should be close to unity if a sensible application is required. This model works well in situations of initial storm erosion. On large time scales, as often needed in morphology, a dynamic equilibrium exists between both the seaward and the shoreward directed transport, so that it is not right to use one specific transport direction. This resulted in the CROSTAN model, a so called time varying total load model (Stive [1986]).

In this model it is assumed that the vertical velocity profile is more or less constant in the area where significant concentrations are present. It is also assumed that the concentration can adapt instantaneously to changes in velocity (following Bailard [1981]) and that most of the transport takes place in a small layer near the bottom. The resulting transport equation is featured as 2.19.

\[ q(x) = B_2 \frac{1}{t_2-t_1} \int_{t_1}^{t_2} \overline{U_z}(t) \overline{L}(t) \, dt \]  \hspace{1cm} (2.19)

\text{with} \quad B_2 = \text{coefficient} \quad [\text{-}]
\quad \overline{U_z}(t) = \text{depth-mean near bottom velocity} \quad [\text{m/s}]
\quad \overline{L}(t) = \text{total amount of sediment} \quad [\text{m}^3/\text{m}^2]

Since an instantaneous response exists, equation 2.19 can be rewritten as equation 2.20.
Making a distinction between the bed load- and suspended transport, Bailard (1981) derived formulae for the total transport which consisted of a linear combination of velocity moments (since the total load is a power function of the near-bottom velocity), the local bottom slope ($\delta z_b/\delta x$) and sediment parameters, given by:

$$q(t) = C_1 |u(t)|^n + C_2 |u(t)|^m \delta z_b/\delta x$$

(2.21)

with $C_1, C_2 = \text{constant coefficient}$

$n = 2, 3$

$m = 3, 5$

Of the formulations only the cross-shore component is used for the time dependent morphological computations, in which a complete interaction between the bottom topography and the hydraulic processes exists. For the exact definition of equation 2.21 as used in UNIBEST-TC the reader is referred to Appendix A.

Since time averaging of equation 2.21 is needed, so called odd and even moments appear and have to be evaluated. The odd moments are $< |u| |u| >$ and $< |u| |u|^3 >$ whereas the even moments are defined as $< |u|^3 >$ and $< |u|^5 >$. The brackets $<$ indicate time averaging. In the UNIBEST-TC program the determination of the value of these moments is basically the same as the process as outlined in Roelvink and Stive (1989b). Only the angle between the waves and possible currents has been included.

The basic outline of the computation of the velocity moments is as follows. The bottom flow velocity is assumed to be decomposed in a mean $\bar{u}$ and time varying component $\ddot{u}$, i.e.:

$$u(t) = \bar{u} + \ddot{u}(t)$$

(2.22)

The time varying component $\ddot{u}$ contains the variation on the time scale of wave groups as well as of the individual waves. By using a Taylor series expansion with the assumption that either the mean velocity is small compared to the time varying velocity or vice versa, the odd and even moments can be rewritten in both directions. This procedure is outlined in detail in Appendix A.

The $\bar{u}$ term is represented by the value of the wave induced undertow as calculated by the method of De Vriend and Stive (1987) as outlined in Section 2.4. The time varying component $\ddot{u}(t)$ is decomposed into two contributing factors. The first is a short wave

\[ q(x) = B_2 \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} f[u_x(t)] \, dt \]  

(2.20)

\[ 2 \text{The bed load consists of grains which are permanently in contact with other grains or with the bottom, whereas the grains featured in the suspended transport are kept in suspension by turbulent diffusion.} \]
component varying on a time scale of the individual wave \( u_i \) and the second, long wave component \( u_l \), varies on the time scale of the wave group. In other words:

\[
\bar{u} = u_i + u_l \tag{2.23}
\]

Under the assumption that \( u_i << u_l \) and \( u_i \) is uncorrelated to \( |u_i|^2 \) and \( |u_l|^3 \), the odd central flow moments can be rewritten as for instance:

\[
<\bar{u} | \bar{u} |^3> = <u_i | u_i |^3> + 4<u_i | u_i |^3> \tag{2.24}
\]

The \( <u_i | u_i |^3> \) term is only not equal to zero if an asymmetry about the horizontal plane exists, caused by the non linearity of the waves. These first terms are computed using a Fourier approximation of the stream function (Rienecker and Fenton [1981]) which is applied to the fraction of non breaking waves \((1-Q_b)\). A saw tooth model is used for the breaking waves. This is calculated by using RFWAVE as mentioned in Section 2.4.

The \( 4<u_i | u_i |^3> \) term is only non-zero if a correlation exists between the velocity variance \( u_i^2 \) and the slowly varying velocity \( u_l \). Reference is made to Section 2.4 in which the bound long waves are discussed and which features an expression for \( \bar{u}_l \) (2.17). It appears that in the case of long bound waves the following equation can be used:

\[
4<u_i | u_i |^3> = < | u_{bi} |^3 u_{bi}> \tag{2.25}
\]

The other velocity moments are expanded in a similar way. In order to save computing time, tables are used of the velocity moments as a function of two dimensionless parameters which are defined by equation 2.26.

\[
\begin{align*}
H' &= H_{rms}/h \quad [-] \tag{2.26.a} \\
T' &= T_p\sqrt{(g/h)} \quad [-] \tag{2.26.b}
\end{align*}
\]

The bottom level changes are computed using equation 2.1 in the following form:

\[
\frac{\delta z_b}{\delta t} + \frac{\delta S_x}{\delta x} = 0 \tag{2.27}
\]

\textit{with} \quad S_x = \text{cross-shore transport, including pores} \quad [m^2/s]

The equations are solved simultaneously by an implicit method. The values which the user has to specify in order to run the program are outlined in the following paragraph.

\textbf{2.6: User specified parameters}

The different input parameters as well as the values which have been most commonly used in this report are mentioned in this Section 2.6. The specific part of this report where the initial settings can be found are printed between brackets.
Time series of the offshore boundary conditions:

- water level $h_0$  
- root mean square wave height $H_{\text{rms}}$  
- deep water wave angle $\varphi_0$  
- peak period $T_p$  
- long shore tide-induced velocity

The run-constant parameters are set to:

- wave decay model:  
  \[ \alpha_c = 1.00 \]  
  \[ \gamma = 0.75 \]  
  \[ f_w = 0.01 \]  

- secondary flow model:  
  \[ \text{rkls} = 0.05 \] [m]

- transport model:  
  \[ \epsilon_b = 0.1 \] [-]  
  \[ \epsilon_s = 0.02 \] [-]  
  \[ w = 0.03 \] [m/s]  
  \[ \tan \varphi = 0.63 \] [-]  
  \[ D_{50} = 0.000225 \] (Section 3.6) [m]  
  \[ \rho_s = 2650 \] [kg/m$^3$]  
  \[ n = 0.4 \] [-]

The \text{rkls} stands for the bottom roughness. Both the $\epsilon_b$ and $\epsilon_s$ are efficiency factors used in the Bailard transport equations. The $w$ stands for the fall velocity of the sediment. It has been calculated using the following formula taken from Van der Velden (1990):

$$\log(1/w) = 0.4758(\log D_{50})^2 + 2.1795(\log D_{50}) + 3.1925$$ (2.28)

One last remark has to be made. In principle the CROSTRAN model from which the UNIBEST-TC program has evolved, is derived for the case of spilling breakers. As mentioned in Roelvink and Hedegaard (1993), concentrated wave energy losses in the case of plunging waves are not included. As will be shown in Chapter 6 in which a comparison is made between the transformation coefficient, $K_t$, as derived by using UNIBEST-TC and the one according to various formulae of different researchers, the program copes quite well with these losses since the results do not deviate much from each other.
3.1: Introduction

In order to be able to use the UNIBEST-TC program as outlined in Chapter 2, certain boundary conditions have to be known. These conditions are treated in this Chapter 3.

As already mentioned in Chapter 1, a certain reference profile is needed in order to make a comparison between different profiles possible. This reference profile is a schematized profile resulting from computations with the UNIBEST-TC program. In order to be able to compute such a schematized profile, a certain initial profile has to be chosen first. This is schematically shown in Figure 3.1. This initial profile is a certain natural occurring cross-shore profile somewhere along the closed Dutch coast. This part of the Dutch coast is treated in Section 3.2.

![Diagram showing the relation between initial profile, UNIBEST-TC program, and reference profile.]

Figure 3.1 Relation between the initial (natural occurring) profile and the reference profile

The more stable the initial (natural occurring) profile as implemented in the UNIBEST-TC program is, the sooner the schematized profile will reach a more or less equilibrium state. So it is important that the used initial profile is as stable as possible. This means that a certain location has to be chosen which does not suffer from a substantial longshore transport gradient. This choice of location is outlined in Section 3.3.

The other boundary conditions which have to be known consist of a certain wave climate (treated in Section 3.4), the influence of the tide (treated in Section 3.5) and the diameter of the sand (treated in Section 3.6). Sediment transport by wind cannot be taken into account when using the UNIBEST-TC program. However, as will be shown in Section 3.7, it is an important phenomenon and should thus be kept in mind when inspecting results from computations using the UNIBEST-TC program. Section 3.8 deals with possible changes of the hydraulic conditions.

3.2: The Dutch coast

The submerged breakwater is to be placed somewhere between Den Helder and Hook of Holland. This part of the Dutch coast is called the "closed Dutch coast" since it is not composed of islands like the Waddenzee or a combination of islands and peninsulas as Zeeland. This part of the Dutch coast has a concave form. In the most northern (Noord Holland) and southern parts (Delfland) erosion takes place whereas in the middle part (Rijnland) accretion occurs. Disturbances in this general picture are found at the harbor.
entrances of Scheveningen and IJmuiden. The boundaries of this part of the coast are formed by De Nieuwe Waterweg in the South and Het Marsdiep in the North. Especially in the North a lot of sediment erodes from this area.

The complete Dutch coast is divided in rather large coastal stretches called "kustvakken", which are numbered from 1 (situated at Rottemeroog) till 17 (situated at Cadzand) and smaller cross-shore sections, called "raaien". There are about 3000 of these cross-shore sections with a distance between them of 200-250 m and which are situated perpendicularly to the coast as much as possible. This sectional system is the basis of coastal measurements which are carried out every year and once every five years. The yearly measurements are called the JARKUS-measurements and are done for a strip starting at 200 m shoreward of the foredune or "zeereep" until 800 m seaward of the so called Benchmark-line or "Rijksstrandpalenlijn". The foredune is formed by the first coherent range of dunes shoreward of the dunefoot. Every five years more extensive measurements (the so called "doorlodingen") are carried out. During such an extensive measurement a much wider strip is measured, up to about 2500 m seaward of the R.S.P.-line. However, these extensive measurements are only carried out in cross-sectional areas every kilometer, so only once in every five JARKUS-sections. A schematic display of a cross-shore profile is given in Figure 3.2.

![Figure 3.2 Schematic presentation of a cross-shore profile](image-url)

The coast-height measurements, during which use is made of stereophotogrammetry, are carried out by the Survey Department of Rijkswaterstaat. The coast-depth measurements are carried out under the supervision of the regional management of Rijkswaterstaat. This is done using ships which are equipped with a totally automatic depth measuring and position system.

1The Benchmark-line or "Rijksstrandpalenlijn", equal to the R.S.P.-line, is a line of reference which is marked by benchmarks which consist of piles driven into the beach every kilometer. These piles ("strandpalen" or "R.S.P.") are numbered from 0 (Den Helder) to 118 (Hook of Holland) for the closed Dutch coast.
3.3: Cross-shore profile at R.S.P. 84

Groenendijk and Roelvink (1992) show that a cross-shore profile computed with UNIBEST-TC using as initial profile the natural occurring profile at raai (R.S.P.) 84, turns into a dynamic equilibrium rather fast. As can be seen in Appendix B this profile at R.S.P. 84 is located between Noordwijk and Katwijk. The contrast between using this profile and using as initial profile a more southward (Terheyde) or northward (Egmond) located profile is large. In the latter cases the time needed to reach a more or less equilibrium profile is almost twice as large as in the first case (when the profile occurring at R.S.P. 84 is used). As mentioned earlier in Section 3.1, the reference profile, which will be used in further computations, has to be as stable as possible. If this is not the case then determining the right cause of the deviations in results of computations will be very difficult because there are a lot of processes involved.

Groenendijk and Roelvink (1992) only use waves which approach the coast perpendicularly. In the present study that is also the case, since a lot of doubts exist about the way that the longshore current, initiated by waves approaching the coast with a certain angle, is modelled in UNIBEST-TC. In Chapter 4 the results of using a wave climate with directions are shown and discussed however, using the same profile which is used for the other computations. If wave directions are taken into account, it is expected that if a profile located at the middle part of the closed Dutch coast is used as initial profile (in the computations with UNIBEST-TC), a dynamically stable reference profile will sooner be reached than if as initial profile a more northern or southern located profile is used.

This presumption is backed by research performed by De Ruig (1989). In this paper the JARKUS-measurements and the extensive measurements from the period 1963-1986 have been combined to obtain an overall picture of the sediment balance of the Dutch coast. It is shown that the coastal part between Katwijk and Noordwijk is relatively one of the most stable parts of the whole closed Dutch coast.

![Figure 3.3 Sand balance of the Dutch coast, according to De Ruig](image)
Figure 3.4 Erosion in the surf zone and accretion above the N.A.P. line, according to De Ruig

Figure 3.3 shows that at km 84 (equal to R.S.P. 84) a slight accretion takes place. In this Figure 3.3 the term "erosie" stands for erosion and the term "sedimentatie" is equal to accretion. If the cross-shore profile is divided in three different parts, the dune and beach part accretes a little, the surfzone erodes slightly and the foreshore accretes minimally. This is shown in Figure 3.4 in which the surfzone and the foreshore are displayed as one, called the "brandingszone/vooroever". The dune and beach part are indicated by "duin & strand". The general picture is a rather stable one. It features a slight accretion for the total profile. Another consideration to use the profile situated at R.S.P. 84 is that the influence of the longshore transport is smaller there compared to the more southward and northward located profiles. Roelvink and Stive (1989a) show that the longshore transport on the foreshore first decreases when going from Hook of Holland to Noordwijk, and after that increases when going from Noordwijk to Den Helder. Stroo (1991) also comes to this conclusion. Since the attention in this paper is primarily focussed on the cross-shore transport, the influence of the longshore transport should be as small as possible.

2The longshore transport on the foreshore is influenced by both the tides as well as the waves. The combination of these two effects is much more important than the individual effects (Roelvink and Stive (1989a) and Van der Velden (1990)). In the surf zone the wave induced transport is the most important factor, whereas in deeper water the tide driven transport component dominates.
3.4: Wave climate

3.4.1: Origin of data

As mentioned in Section 3.1 a certain wave climate has to be used in order to carry out computations with the program UNIBEST-TC. This wave climate may consist of time series of the wave height, the wave period and the wave direction\(^3\).

![Figure 3.5 Position of measuring stations on the Dutch part of the continental shelf](image)

Eight measuring stations are placed on the Dutch part of the continental shelf as displayed in Figure 3.5. To obtain a total wave climate the measurements from the stations Goeree (LEG), Noordwijk (MPN) and Eierland (ELD) have been collected and revised for a period of ten years by Roelvink and Stive (1989a). From the collected data a certain wave climate is determined for each station. If a deep water wave climate is needed for a certain cross-shore section along the Dutch coast, the wave climates from the two neighboring stations are used. In this paper use is made of the deep water wave climate from the station Noordwijk (MPN), see Table 3.1. The wave climates from ELD and LEG have deliberately not been added to the one from MPN to compute a mean wave climate because the cross-shore profile at R.S.P. 84 is very near MPN. The expected occurring deep water wave climate will thus not differ much from the one measured at MPN. It has to be mentioned that during the determination of the wave climates of ELD,

\(^3\)A clear definition of "a wave climate" doesn't exist. Sometimes one refers to a distribution of the wave heights as a wave climate without taking the wave period or the wave directions into account. Such a wave climate is then used to enable predictions of certain chances of exceedance of wave heights, usually using some kind of extrapolation. More about this subject is found in Battjes (1990), especially in Chapter 7. So the kind of wave climate needed is a direct result of the purpose it has to fulfil.
MPN and LEG, use was made of known data from the two neighboring stations whenever the data were not complete.

3.4.2: Wave heights

The wave heights given in the wave climate are the zeroth moment wave heights, the $H_{m0}$. This wave height is computed with the help of the zeroth moment $m_0$ from the wave energy density spectrum ($m_0$ equals the area of the spectrum) in the following way:

$$H_{m0} = 4 \sqrt{m_0} \quad (3.1)$$

with the following definition:

$$m_n = \int_0^\infty f^n E(f) df \quad (3.2)$$

with $H_{m0}$ = zeroth moment wave height [m]

$m_n$ = $n^{th}$ moment of the spectrum [m$^n$]

$f$ = frequency [Hz]

$E(f)$ = wave energy density spectrum [m$^2$/Hz]

However, not the $H_{m0}$ but the $H_{rms}$, the root mean square wave height, has to be used in UNIBEST-TC. This $H_{rms}$ is equal to the root of the mean value of the square of the occurring wave heights, as the name already implies. A rather simple relation between the root mean square- and the zeroth moment wave height exists, namely:

$$H_{rms} = \frac{H_{m0}}{\sqrt{2}} \quad (3.3)$$

with $H_{rms}$ = root mean square wave height [m]

---

4 The data/information consists of three-hourly wave data obtained by measurements. These series are incomplete, however. The mean registration density for the measurements of LEG/MPN/ELD in the period 1979-1986 when only taking the wave heights into account amounts to 67%. If both the wave height as well as the wind direction are taken into account, the registration density becomes lower, namely 49%. However, the wave height exceedance chances for both registration densities (with and without direction) are almost the same. This means that the wave climate with wave direction is probably also a rather good representation of reality for that certain period. A more detailed consideration on registration densities can be found in Roskam (1988), Chapters 2 and 3.
Often this $H_{\text{no}}$ is used as the significant wave height, $H_r$. If however this significant wave height is also thought to be the same as the mean of the highest one-third of the waves, the $H_{1/3}$, deviations may arise\(^5\). In the remaining part of this paper whenever the phrase "wave height" is mentioned, the $H_{\text{no}}$ is intended, unless otherwise stated.

The above mentioned is summarized in Table 3.1 which states the used wave climate without direction occurring at MPN. The values and relations between the wave periods are outlined in the following paragraph. The abbreviation w.d. stands for water level deviation as opposed to the reference level NAP. The column Perc. states the percentage of the time during which the mentioned wave occurs. The total of that column is not equal to 100% because the waves resulting from winds coming from the directions between 61° and 180° are not taken into account in this Table 3.1. The reason for this is that these offshore winds will not generate any waves of importance so near to the coast.

### 3.4.3: Wave period

In the third column of Table 3.1 the period given is indicated by $T_{m01}$. This is the mean period derived from the wave energy density spectrum in the following way:

$$
T_{m01} = \frac{m_0}{m_1} \quad (3.4)
$$

with $T_{m01}$ = mean wave period \([s]\)

To use the UNIBEST-TC program another wave period has to be determined, namely the peak period, $T_p$. This is the period which occurs at the maximum value of the wave energy density spectrum. It is however at least questionable to use the peak period and not the mean period to characterize the kind of spectrum which is used. It is possible for two very different spectra to have almost the same peak period. The values of the mean period do differ in such a situation, however. The cause of this is that an extra element may occur in the spectrum; swell. In Figure 3.6 this is shown. Swell is a wave field which is not influenced anymore by the windfield which generated it. This may be because either the windspeed has become less or because the wave field due to its propagation speed has left the area which was influenced by the windfield. Normally both

---

\(^5\)The relation $H_{\text{no}} = 4.004 \left(m_0\right)^{1/2} = 4 \left(m_0\right)^{1/2}$ is a direct result from the assumption that the sea state ("zeegang" in Dutch) is a Gaussian process with a narrow spectrum. If however the additional assumption is made that $H_{1/3} = H_{\text{no}}$ then formula 3.1 would also be valid for $H_{1/3}$. This is sometimes not the case however. Forristall (1978) uses an empirical distribution and finds a value 0.942 times the one from the Rayleigh distribution, i.e. he finds the constant 3.77 (pp.2357). Goda (1985) recommends the constant 3.8. When looking at situations with very shallow water the constant might even become greater than the value 4. In short, by definition formula 3.1 is valid for $H_{\text{no}}$ but it isn’t necessarily valid for $H_{1/3}$. The "significant wave height" is often used for $H_{1/3}$. In this paper use will be made of the $H_{\text{no}}$ whenever the wave height (or the zero-moment wave height) is mentioned to avoid misunderstandings. If the significant wave height is mentioned use will be made of the $H_{1/3}$. 

25
Table 3.1 Wave climate without direction

in the first and in the second case the wave field will be at least a day old. Since older wave fields are characterized by lower frequencies, the spectrum will move to the low frequency side and the period will become larger. When a wave field has just been generated by the wind it is called sea state. In Figure 3.6 examples of spectra with both a swell- and sea state component are shown. Sometimes the combination can be distinguished (c), sometimes this is not so easy (e).

![Figure 3.6 Different kinds of spectra](image-url)
The conclusion that the peak period is not a good representative parameter seems to be justified. Since for the input of UNIBEST-TC a certain value of this period is needed however, a certain relation between the peak period and the mean period has to be found. Generally speaking this relation will not be the same for different situations, but will depend on the shape of the spectrum considered.

A lot of different relations have been derived by various researchers. These relations will be mentioned first and then summarized in Table 3.3. Goda (1985) finds from empirical observations a certain range, namely $T_p = 1.16$ till 1.37 times $T_{m02}$. A specific mean value of $T_p = T_{m02}/0.81$ is given by Goda (1978) which is determined using a large number of observations. Note that Goda uses the $T_{m02}$ value for the mean wave period. Van der Meer (1988) uses for a Pierson Moskowitz-spectrum (also noted as PM-spectrum) the relation $T_p = T_{m02}/0.85$ and for a very narrow JONSWAP-spectrum (also noted as JW-spectrum) the relation $T_p = T_{m02}/1.0$. Stam (1988) finds as a mean relation for the JW-spectrum $T_p = T_{m02}/0.82$. Daemrich et al (1985) use, also for the JW-spectrum, the mean relation $T_p = T_{m02}/0.83$ and for a PM-spectrum $T_p = T_{m01}/0.77$. The mean relation as used by Rijkswaterstaat is given by Hokke and Roskam (1986) as $T_p = T_{m01}/0.78$.

Since the relation between $T_{m01}$ and $T_p$ is needed and a lot of research has been focussed on $T_{m02}$ the relation between $T_{m01}$ and $T_{m02}$ is also needed. Using the standard formula 3.5 for the JW-spectrum (Hasselman et al [1973]) and determining the values of $m_0$, $m_1$, and $m_2$ the parameters $T_{m01}$ and $T_{m02}$ were computed.

$$E(f) = \gamma(f) \propto g^2(2\pi)^{-4} f^{-3} \exp[-\frac{5}{4} \left(\frac{f}{f_m}\right)^{-4}]$$ \hspace{1cm} (3.5)$$

with the following formula for the peak-enhancement function $\gamma(f)$:

$$\gamma(f) = \gamma_0 \exp\left(-\frac{1}{2} \left(\frac{f}{f_m}\right)^2\right) \hspace{1cm} (3.6)$$

with $\sigma = 0.07$ if $f < f_m$

$\sigma = 0.09$ if $f \geq f_m$

For three values of $\gamma_0$ the ratio $T_{m01}/T_{m02}$ was computed, see Table 3.2, as well as the ratio $T_{m01}/T_p$. Note that for $\gamma(f) = 1$ the JW-spectrum is the same as the PM-spectrum. Table 3.2 as well as the information mentioned above was used to create Table 3.3.

The decision was made to make use of the JW-spectrum because this represents the typical occurring North Sea spectrum rather well. To determine the mean value the highest (1.02) and the lowest (0.78) are omitted. The value 1.02 is omitted because this isn't a value occurring at a "normal" JW-spectrum with $\gamma_0 = 3.3$ but at a very narrow

---

This mean period $T_{m02}$ is not the same as the previously mentioned mean period as defined by $T_{m01}$. The value for $T_{m02}$ is also computed from the wave energy density spectrum but in a different way, namely $T_{m02} = \sqrt{m_0/m_2}$.
<table>
<thead>
<tr>
<th>Gamma</th>
<th>$T_{m01}/T_p$</th>
<th>$T_{m02}/T_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.86</td>
<td>1.052</td>
</tr>
<tr>
<td>3.3</td>
<td>0.90</td>
<td>1.039</td>
</tr>
<tr>
<td>20.0</td>
<td>0.96</td>
<td>1.017</td>
</tr>
</tbody>
</table>

Table 3.2 Values for the ratio $T_{m01}/T_{m02}$ and $T_{m01}/T_p$ for different values of $\gamma_0$ according to the JONSWAP formula

<table>
<thead>
<tr>
<th></th>
<th>$T_{m02}/T_p$</th>
<th>$T_{m01}/T_p$</th>
<th>$T_{m02}/T_p$</th>
<th>$T_{m01}/T_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>JW</td>
<td>PM</td>
<td>JW</td>
<td>PM</td>
</tr>
<tr>
<td>Van der Meer</td>
<td>1.0(narrow)</td>
<td>1.02</td>
<td>0.85</td>
<td>0.89</td>
</tr>
<tr>
<td>Daemrich</td>
<td>0.83</td>
<td>0.86</td>
<td>0.77</td>
<td>0.81</td>
</tr>
<tr>
<td>Hokke</td>
<td></td>
<td>0.78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stam</td>
<td>0.82</td>
<td>0.85</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Goda</td>
<td>0.81</td>
<td>0.84</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.3 Values of the ratio $T_m/T_p$

spectrum. To determine this value of 1.02 use was made of Table 3.2 with $\gamma_0 = 20$. The lowest value of 0.78 was not taken into account because its value is distinctly not in comparison with the other values and some doubts exist about its reliability. So in UNIBEST-TC use is made of the following relation:

$$T_p = \frac{T_{m01}}{0.85} \quad (3.7)$$

3.4.4: Wave direction

The wave direction can also be implemented in UNIBEST-TC. It appears that the wave direction is not necessarily the same as the wind direction. The wave direction is not only influenced by this wind direction but also by currents, bottom topography and already present older wave energy. Research has been done by the Dienst Getijdewateren from Rijkswaterstaat to determine the relation between wind and wave direction, see Hokke and Roskam (1986), Roskam (1988) and Roskam (1992). In this last report use is made of data achieved by using the so called wavec-buoys (specially designed to measure wave directions) which are in use since 1985.

The general conclusion is that at very low spectral frequencies the wave direction is almost always northward due to the influence of the swell coming from the North. At
very high frequencies the mean wave direction does not deviate much from the wind direction. The latter is the case because when considering cases with high frequencies one is looking at "young" sea state which is still very much influenced by the same wind field which generated it. When looking at frequencies between these two extremes wave directions resulting from changing wind fields are present. For the specific case of MPN the relation as shown in Figure 3.7 is valid.

![Figure 3.7 Comparison of wave- and wind direction at MPN, according to Roskam](attachment://image.png)

At MPN the differences between the wind- and the wave direction are rather large, also compared to other measuring stations. This is caused by the fact that MPN is so near to the coast (which results in a small fetch, refraction, tidal currents) and because of the earlier mentioned swell from the North. Only the mean relation for the high frequency range ("hoog-frequente band", number (15)) shows some resemblance to the wind direction. In order to be able to compute a mean wave direction which is coupled to a certain wind direction class of 30°, use is made of the relation given by Roskam (1992), for the mean value of the whole spectrum, i.e. range 12 presented in Figure 3.7. This results in Table 3.4. Again no use is made of the sector between 61° and 180°.

The direction of the waves is not taken into account at first. In Chapter 4 however, where the initial profile is determined, the influence of taking the wave direction into account on the profile development is shown and discussed (Section 4.3).

3.5: Tides

3.5.1: Introduction

As a result of the changing gravitational attraction of the moon, sun, and other astronomical bodies acting on the rotating earth, a periodic rising and falling of the sea level, called tide, occurs. In front of the Dutch coast the tide has a period of 12 hours and 25 minutes. This is the result of the fact that the lunar day is about 50 minutes longer than the solar day and that there are two high/low waters (or tides) every tidal or lunar day. During new moon and full moon the earth, moon and sun are lined up in the same direction and as a
result the influence of both the sun and the moon act in the same direction. This is called spring-tide. A situation called neap-tide occurs during first- or last quarter when the influences weaken each other. The influence of the moon is about twice as large as the influence of the sun (Pugh [1987]).

The tide can be regarded as a wave which propagates out of the Atlantic Ocean into the North Sea mainly through the strait between Scotland and Norway. Within the North Sea the tide is amplified as a result of resonance effects. As a result of the Coriolis force an amphidromic system exists in the North Sea. In such a system the tide moves around a certain number of points (In the North Sea this number is equal to three) where the amplitude of the movement is minimal, in an anti-clockwise manner (Pugh [1987] and Kalkwijk [1975]).

Such an amphidromic point can also be found in the southern part of the North Sea. As a result of that the tide along the Dutch coast moves from the South to the North. This is the reason why for instance Noordwijk will experience high tide later than Hook of Holland, see also Figure 3.8. To get an idea of the average tidal range along the Dutch coast Figure 3.9 is presented. The tide can be divided into the vertical tide and the horizontal tide. The first will be discussed in Section 3.5.2 and the second in 3.5.3.

---

7The greatest difference between high and low tide occurs during spring-tide. In that situation the high tide is higher and the low tide lower as compared to the mean situation.

8The Coriolis force is an imaginary force which compensates for the fact that calculations are carried out in reference to the rotating earth. As a result of this Coriolis force each particle moving in the Northern Hemisphere will have an apparent acceleration to the right of the direction in which it is moving. In the Southern Hemisphere the apparent acceleration is to the left of the direction of flow.
Figure 3.8 Amphidromic system

Figure 3.9 Mean level of low- and high tide along the Dutch coast
3.5.2: Vertical tide

The variation of the water level is called the vertical tide. This vertical tide can be computed using a method called the harmonic analysis. According to this method the tide is built up out of a finite number of harmonic components using the following formula (Schureman [1988] pp.123 and further, and Ippen [1966] Chapter 4):

\[
h_x(t) = h(0) + \sum_{n=1}^{N} f_n H_{n,x} \cos(\omega_n t + (V_0 + u)_n - g_{n,x})
\]  

(3.8)

with:

- \( h_x(t) \) = water level at time point \( t \) [m]
- \( h_0 \) = mean water level [m]
- \( f_n \) = correction factor for 18.6-yearly cycle [-]
- \( H_{n,x} \) = amplitude [m]
- \( \omega_n \) = angular speed [°/hr]
- \( g_{n,x} \) = local phase-angle [°]
- \( N \) = total number of harmonic components [-]
- \( (V_0 + u)_n \) = mean tide phase at \( t = 0 \), including a correction term \( u \) for the 18.6-yearly cycle [°]

subscript:

- \( n \) = belonging to the harmonic component no. \( n \)
- \( x \) = local

Each component has its own period, phase-angle and amplitude. The value for \( \omega_n \) is constant for each harmonic component and is universally valid. The value for \( (V_0 + u)_n \) is also universally valid but changes in time. The values for \( H_{n,x} \) and \( g_{n,x} \) are, as is indicated by the subscript, locally valid but do remain constant in time however.

When computing the vertical tide one starts with the astronomic components such as M2 or S2 for instance. As a result of non-linear phenomena which influence the tide as it propagates through shallow seas, higher frequencies are introduced. An example of this is the influence of the bottom friction on the water flow. If the velocity function is of the sinus kind, it can be shown with the help of a Fourier approximation that terms with higher frequencies are generated. Other frequencies can also be generated through mutual non-linear interactions between tide components. For example, the component MS4 is a result of the interaction between M2 and S2.

To implement the vertical tide in UNIBEST-TC, a distribution of the occurrence frequencies of the water level variation is needed. For this purpose the program TWGETIJ was written (in Turbo Pascal). The listing is given as Appendix C. A variable number of tide components along with the matching parameters can be used as input in this program. The resulting water level variations are then computed with a certain time step between successive computations. The value of this time step can be determined by the user. These computations are carried out for the period of one year. It is very easy to

\[9\] According to Schureman (1988) these values change every year but remain constant during a specific year. As will be shown further on this is not exactly true. The differences are however most of the time very small.
alter the program (only one constant has to be changed) in order to be able to use a computing period of two or more years. However, the accuracy only increases by a fraction, not enough to make a mentionable difference in the outcome. The computed water level variations are then stacked together in classes of 1 cm. The final result of the program consists of a table consisting of the occurring frequencies and cumulative frequencies (in percentages) for every height class. The results can be presented in a graph, such as is displayed by Figure 3.10.

![VERTICAL TIDE](image)

**Figure 3.10 Graphical presentation of the results computed with TWGETIJ**

For the sake of convenience the number of classes has to be limited. As a result the width of the classes is set at 25 cm which results in a total of eleven classes. This is shown as a bar diagram in Figure 3.11 and is printed in Table 3.5.

An important question is how many harmonic components are needed to achieve a satisfactory result. First of all the most influential local components have to be determined. Ippen (1966) suggests to use the ten most influential components in the computations.

Since the program does all the computing it was decided to make use of the 31 components which are printed in Table 3.6. These 31 were selected because these are the ones used in the prediction of the tide in front of the Dutch coast as given in the Dutch tide tables ("Getijtafels voor Nederland 1991" [1990]).
Figure 3.11 Bar graph displaying the height classes

<table>
<thead>
<tr>
<th>Class [cm]</th>
<th>Middle of class [cm]</th>
<th>Percentage [%]</th>
<th>Cumulative Perc. [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>-125 till -100</td>
<td>-112.5</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>-100 till -75</td>
<td>-87.5</td>
<td>4.24</td>
<td>4.49</td>
</tr>
<tr>
<td>-75 till -50</td>
<td>-62.5</td>
<td>17.76</td>
<td>22.25</td>
</tr>
<tr>
<td>-50 till -25</td>
<td>-37.5</td>
<td>19.48</td>
<td>41.73</td>
</tr>
<tr>
<td>-25 till 0</td>
<td>-12.5</td>
<td>12.07</td>
<td>53.80</td>
</tr>
<tr>
<td>0 till 25</td>
<td>12.5</td>
<td>10.60</td>
<td>64.40</td>
</tr>
<tr>
<td>25 till 50</td>
<td>37.5</td>
<td>9.97</td>
<td>74.37</td>
</tr>
<tr>
<td>50 till 75</td>
<td>62.5</td>
<td>10.05</td>
<td>84.42</td>
</tr>
<tr>
<td>75 till 100</td>
<td>87.5</td>
<td>9.34</td>
<td>93.76</td>
</tr>
<tr>
<td>100 till 125</td>
<td>112.5</td>
<td>5.09</td>
<td>98.85</td>
</tr>
<tr>
<td>125 till 150</td>
<td>137.5</td>
<td>1.15</td>
<td>100.00</td>
</tr>
</tbody>
</table>

Table 3.5 Tide classes with occurring percentages
<table>
<thead>
<tr>
<th>Type</th>
<th>((V_{0}+U)_{n}) [°]</th>
<th>(f_{n}) [-]</th>
<th>(H_{n,x}) [m]</th>
<th>(\theta_{n,x}) [°]</th>
<th>(\omega_{n}) [°/hr]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SA</td>
<td>98.5</td>
<td>1.000</td>
<td>10.1</td>
<td>219.1</td>
<td>0.041</td>
</tr>
<tr>
<td>SM</td>
<td>134.0</td>
<td>0.996</td>
<td>3.0</td>
<td>49.3</td>
<td>1.016</td>
</tr>
<tr>
<td>Q1</td>
<td>197.3</td>
<td>1.041</td>
<td>3.5</td>
<td>131.1</td>
<td>13.399</td>
</tr>
<tr>
<td>O1</td>
<td>50.0</td>
<td>1.041</td>
<td>10.5</td>
<td>188.6</td>
<td>13.943</td>
</tr>
<tr>
<td>P1</td>
<td>171.5</td>
<td>1.000</td>
<td>4.0</td>
<td>346.5</td>
<td>14.959</td>
</tr>
<tr>
<td>K1</td>
<td>179.9</td>
<td>1.026</td>
<td>7.5</td>
<td>193.7</td>
<td>15.041</td>
</tr>
<tr>
<td>3MS2</td>
<td>318.0</td>
<td>0.989</td>
<td>2.0</td>
<td>329.7</td>
<td>26.952</td>
</tr>
<tr>
<td>MNS2</td>
<td>239.4</td>
<td>0.993</td>
<td>2.0</td>
<td>198.0</td>
<td>27.424</td>
</tr>
<tr>
<td>NLK2</td>
<td>104.8</td>
<td>1.207</td>
<td>2.5</td>
<td>36.3</td>
<td>27.886</td>
</tr>
<tr>
<td>MU2</td>
<td>94.2</td>
<td>0.996</td>
<td>8.5</td>
<td>219.1</td>
<td>27.968</td>
</tr>
<tr>
<td>N2</td>
<td>13.4</td>
<td>0.996</td>
<td>10.9</td>
<td>85.4</td>
<td>28.440</td>
</tr>
<tr>
<td>NU2</td>
<td>306.8</td>
<td>0.996</td>
<td>3.1</td>
<td>72.0</td>
<td>28.513</td>
</tr>
<tr>
<td>MPS2</td>
<td>217.5</td>
<td>0.996</td>
<td>1.5</td>
<td>189.3</td>
<td>28.943</td>
</tr>
<tr>
<td>M2</td>
<td>226.0</td>
<td>0.996</td>
<td>73.1</td>
<td>108.7</td>
<td>28.984</td>
</tr>
<tr>
<td>λ2</td>
<td>325.2</td>
<td>0.996</td>
<td>3.0</td>
<td>123.3</td>
<td>29.456</td>
</tr>
<tr>
<td>2MN2</td>
<td>78.7</td>
<td>0.989</td>
<td>7.0</td>
<td>306.8</td>
<td>29.528</td>
</tr>
<tr>
<td>S2</td>
<td>0.0</td>
<td>1.000</td>
<td>18.5</td>
<td>173.1</td>
<td>30.000</td>
</tr>
<tr>
<td>K2</td>
<td>179.4</td>
<td>1.044</td>
<td>5.5</td>
<td>173.0</td>
<td>30.082</td>
</tr>
<tr>
<td>2SM2</td>
<td>134.0</td>
<td>0.996</td>
<td>3.0</td>
<td>36.1</td>
<td>31.016</td>
</tr>
<tr>
<td>2MK3</td>
<td>272.2</td>
<td>1.019</td>
<td>1.0</td>
<td>223.5</td>
<td>42.927</td>
</tr>
<tr>
<td>MK3</td>
<td>45.9</td>
<td>1.022</td>
<td>0.5</td>
<td>286.7</td>
<td>44.025</td>
</tr>
<tr>
<td>3MS4</td>
<td>318.0</td>
<td>0.989</td>
<td>2.5</td>
<td>260.3</td>
<td>56.952</td>
</tr>
<tr>
<td>MN4</td>
<td>239.4</td>
<td>0.993</td>
<td>6.5</td>
<td>146.9</td>
<td>57.424</td>
</tr>
<tr>
<td>M4</td>
<td>92.0</td>
<td>0.993</td>
<td>18.6</td>
<td>172.4</td>
<td>57.968</td>
</tr>
<tr>
<td>MS4</td>
<td>226.0</td>
<td>0.996</td>
<td>11.5</td>
<td>230.6</td>
<td>58.984</td>
</tr>
<tr>
<td>MK4</td>
<td>45.5</td>
<td>1.041</td>
<td>3.5</td>
<td>234.1</td>
<td>59.066</td>
</tr>
<tr>
<td>2MN6</td>
<td>105.4</td>
<td>0.989</td>
<td>2.0</td>
<td>187.4</td>
<td>86.408</td>
</tr>
<tr>
<td>M6</td>
<td>318.0</td>
<td>0.989</td>
<td>4.5</td>
<td>214.0</td>
<td>86.952</td>
</tr>
<tr>
<td>2MS6</td>
<td>92.0</td>
<td>0.993</td>
<td>4.5</td>
<td>268.7</td>
<td>87.968</td>
</tr>
<tr>
<td>M8</td>
<td>184.0</td>
<td>0.986</td>
<td>2.5</td>
<td>280.7</td>
<td>115.936</td>
</tr>
<tr>
<td>3MS8</td>
<td>318.0</td>
<td>0.989</td>
<td>4.0</td>
<td>137.9</td>
<td>116.952</td>
</tr>
</tbody>
</table>

Table 3.6 The components used in the harmonic analysis
The \( \omega_n, g_{nu}, \) and \( H_{nu} \) as stated in Table 3.6 have been taken from the Dutch tide tables (1990). Because of the fact that the values occurring at R.S.P. 84 are not directly given, they were computed using the values given for Hook of Holland and IJmuiden, the two neighboring places with known values. In the computing of the values as given in Table 3.6 the differences in distances between these three places were taken into account.

The values of \( f_n \) and \( (V_0+u)_n \) have initially been taken from Schureman (1988). It seems however that some mistakes and/or inaccuracies are present in these data. This may be the result of two possibilities; firstly computational errors and secondly the fact that Schureman states that these values remain constant during a specific year, which assumption is not exactly valid. To overcome this problem all values have been computed again using the program ASCON\(^{10}\) from Delft Hydraulics. If the values of Schureman (1988) did not agree with the ones computed with ASCON (for the middle of the year) they were replaced by the latter. The values of \( f_n \) are practically all the same, but the values of \( (V_0+u)_n \) can deviate quite a lot from each other.

### 3.5.3: Horizontal tide

The movement of water as a result of the changes in water level caused by the vertical tide is called the horizontal tide. The method used to compute the longshore- and the cross-shore horizontal tide velocity is basically the same as the method outlined in Section 3.5.2 for the calculation of the vertical tide. Again a certain number of components is used in a cosinus function. In this report the horizontal tide is not taken into account however.

### 3.6: Soil composition

Computations are carried out using sand with a diameter \( D_{50} \) (DSED) of 225 \( \mu \text{m} \). This value is given in "Handboek zandsuppleties" (1988) for R.S.P. 84. Van Bemmelen (1988) gives a slightly higher mean value but the same value of 225 \( \mu \text{m} \) appears in the last part of the measurement series.

### 3.7: Sediment transport by wind

Not only as a result of the currents caused by the tide and the waves does sediment transport occur, the wind also causes a direct transport. Especially during storms this transport can be rather large. Again the total transport can be divided into a cross-shore- and a longshore transport.

It appears that the present range of dunes functions as an important catchment area of sand ("zandvang" in Dutch). Between Den Helder and Hook of Holland an accretion takes place (De Ruig [1989]). This accretion is almost completely limited to the dune face because the marram grass catches the sand. Practically no sand transported by wind returns by wind from the dune face back to the beach due to the sheltered position of the

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\(^{10}\)ASCON 2.01 is a part of the programm GETIJSYS developed by Delft Hydraulics. By using ASCON values of \( (V_0+u)_n \) and \( f_n \) can be computed for a certain harmonic component and a certain date which can be specified by the user.
toe of the dune where the sand accumulates. Under severe storm conditions some of the accumulated sand is transported back to the beach however. This is a result of the effect of high storm surge levels and large wave heights.

Reference is made to Figure 3.4 for the accretion of the dune and beach strip at R.S.P. 84. It seems that this strip gains a yearly average of 7.5 m$^3$ sand per alongshore meter. UNIBEST-TC cannot cope with sediment transport by wind. As a result this sediment transport will not be taken into account in the computations. However, it has to be mentioned that as a result of this wind driven transport in reality, as opposed to the profile as computed with UNIBEST-TC, a sink is present at the location where the NAP-line intersects the beach.

3.8: Future hydraulic conditions

Using the currently available knowledge on the global warming problem and the expected temperature rise in the future as reference, it is very likely that the current hydraulic conditions will change. This change not only refers to the rise of the mean sea level (coupled to the subsidence of the bottom, "bodemdaling in Dutch"), but also to factors like a possible change in the average wind force and the wind direction, an enlargement of the tidal difference and the possibility that the chance of occurring of storms will become greater.

The treatment of this problem is not within the range of this study however. The purpose of this paragraph is only to show the reader that these changes may play an important role in the future. Because of the fact that not only the morphological development around the breakwater but also the stability of the breakwater itself is a direct function of these factors, they will have to be taken into account in an eventual extensive feasibility study. Reference is made to De Ronde and Vogel (1988), which is also the source of Table 3.7. This Table is given here to show that the possible changes cannot easily be neglected since a substantial change of the current hydraulic conditions may occur.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>A-scenario</th>
<th>B-scenario</th>
<th>C-scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean sealevel</td>
<td>+20 [cm]</td>
<td>+60 [cm]</td>
<td>+85 [cm]</td>
</tr>
<tr>
<td>Windforce</td>
<td>0 [%]</td>
<td>0 [%]</td>
<td>+10 [%]</td>
</tr>
<tr>
<td>Winddirection</td>
<td>0 [°]</td>
<td>0 [°]</td>
<td>+10 [°]</td>
</tr>
<tr>
<td>Average high tide</td>
<td>+20 [cm]</td>
<td>+65 [cm]</td>
<td>+90 [cm]</td>
</tr>
<tr>
<td>Average low tide</td>
<td>+20 [cm]</td>
<td>+55 [cm]</td>
<td>+80 [cm]</td>
</tr>
</tbody>
</table>

Table 3.7 Possible future changes in important parameters, as a result of different scenarios, as given by De Ronde and Vogel

With the phrase "Scenario", a consistent set of parameters is meant. The A-scenario consists of the current values, the B-scenario consists of the most probable future values and the C-scenario consists of the most unfavorable values. These last values are determined by using the expected mean value (as used in the B-scenario) and adding the
standard deviation. The chance that the future circumstances will be even more unfavorable than the ones resulting from the C-scenario is equal to 5 till 15%. The different scenarios have been linearly interpolated between 1990 and 2090.
CHAPTER 4: INITIAL PROFILE AND REFERENCE PROFILE

4.1: Natural initial profile

As outlined in Section 3.3 the profile at R.S.P. 84 is chosen because of its relative stability as opposed to other more southern or northern profiles. In order to compute a reference profile with UNIBEST-TC a certain initial profile had to be chosen and implemented in the program. The natural profile occurring at R.S.P. 84 is this initial, "first" profile.

In this Section 4.1 the initial, "first" profile is discussed. In the following Section 4.2 the reference profile as computed with the UNIBEST-TC program is treated. The deep water wave angles are not taken into account in that Section. This is done in Section 4.3.

In order to obtain sufficient data for this natural profile, use is made of different kinds of literature. First of all Groenendijk and Roelvink (1992) was used because the profile of Noordwijk is given and results of earlier runs done with UNIBEST-TC on this certain profile are known and presented. Since this profile is very schematized and not exactly occurring at R.S.P. 84, additional information was requested at the "Dienst Getijdewaterr" from Rijkswaterstaat. A floppy disc containing data from the yearly JARKUS-measurements from 1983 as well as the results from two more extensive measurements, the so called "doorlodingen", one in 1980 and one in 1985, was received. The JARKUS measurements and the "doorlodingen" give sufficient detailed data in the area of most interest, this roughly being the area from the NAP-line until about 2500 m seaward from that line. This data is used to create a profile which can be used in UNIBEST-TC. However, since the wave climate which is going to be used was measured at MPN (Section 3.4.1) where a depth of approximately NAP -18 m occurs, the obtained profile lacked a description of the deeper parts.

It is necessary to use this deeper part of the profile in the computations, because the wave climate measured at NAP -18 m will not be exactly the same as the wave climate measured as it passes the NAP -12.5 m line, which is roughly the outer edge of the profile defined by the extensive measurements. The deeper part of the profile is obtained from De Ronde and De Ruijter (1986), which gives a rather detailed Figure of a cross-shore profile situated at Noordwijkerhout running to 22000 m seaward of the shoreline. It has to be mentioned that Noordwijkerhout is not exactly R.S.P. 84 but about 5 km northward of it (see also appendix B). Since no strange phenomena or coastal changes occur between these two profiles the assumption was made that the deep water profile of R.S.P. 84 will resemble the deep water profile of Noordwijkerhout rather closely.

This leads to the so called "first" profile, the natural profile (schematized) at R.S.P. 84 in 1983, as shown in Figure 4.1.

The area of most interest is the area where the submerged breakwater will probably be built. This is roughly the area between the shoreline and about 3000 m seaward of that line. This is displayed in Figure 4.2 which is a blow-up of a part of Figure 4.1. From here on attention in figures will be primarily focussed on the range that is shown in Figure 4.2, so that profile changes can be more easily seen than when showing the complete profile.
The profile described in Section 4.1 and displayed in Figure 4.1 was implemented in UNIBEST-TC along with the wave climate (without direction) mentioned in Section 3.4 and summarized in Table 3.1. The variation of the water level as a result of the vertical tide is taken into account by using the results of the program TWGETIJ as described in Section 3.5.2 and more specifically mentioned in Table 3.5. As stated earlier the horizontal tide effects are not taken into account. Neither is the wave direction initially. In Section 4.3 results from UNIBEST-TC runs with wave direction are outlined however. The 50% grain diameter (DSED) is taken as 225 micrometer (Section 3.6). The settings of other variables and parameters are discussed in Section 2.6.
The time step of the calculations was set at 0.5 days. This gives a good enough accuracy to work with, especially since the interest lies in long periods of about 10 years. A smaller time step is of course possible but the computing time will also increase as a result of that. Waves resulting from the wind directions between 61-180° are not taken into account (neither here nor in the wave climate of Section 4.3). This results in two possibilities, either simulating these directions by using a wave height, $H_{rms}$, of 0 or not taking into account the days at which these wind directions occur. UNIBEST-TC cannot work with a wave height of 0 because in one of the formula the wave height appears in the denominator. The conclusion is that the days when these wind directions appear will be neglected. This will not change the outcome of the program since UNIBEST-TC calculates the bottom changes at each time step immediately using the results of the previous time step and all input variables. If the wave height is supposedly 0 no bottom changes will be calculated, so that the same result is obtained by not using this particular time step with a wave height equal to 0. This has a positive side effect; the computing time becomes less. Therefore a year in the computations is no longer represented by the standard of 365 days, but by 278.5 days, which represents 76.4% of 730 half days. This percentage follows from Table 3.1. Thus the wave height ($H_{rms}$), peak period ($T_p$), and water level variation ($h_0$) were implemented in the program by tables formed by 557 elements and having a recurrence interval of 278.5 days.

The waves with their own period and coupled variation of the reference level NAP (as a result of the wind set-up) are randomized before they are written to the tables. To make sure all the possible waves would indeed occur within a certain year the randomizing consists of taking a random element out of the complete collection and not putting it back afterwards. The result of this is that the total number of elements of the total collection decreases by one each time a random element is chosen. An example of a random collection of waves is shown in Figure 4.3. This is the wave climate which is used as the deep water wave climate for all the computations carried out in the remaining part of this Chapter.

![Wave Heights Hrms at R.S.P. 84](image)

Figure 4.3 Deep water wave climate at R.S.P. 84
The vertical tide variations are also randomized independent of the randomizing of the waves and afterwards coupled to a certain wave. The final deviation of the sea level compared to the reference level NAP is then built up of the sum of the wind- and wave induced variation and the vertical tide variation.

After a number of test runs the final settings and modifications were set and the profile of Figure 4.4 was obtained. As expected (Groenendijk and Roelvink [1992]) the near shore sandbar is completely flattened and cannot be traced at the end of the ten years. The seaward sandbar is also flattened but some indications of its existence can still be found, although it moves offshore. Around SWL the steepness of the profile becomes less and the dune-foot retreats about 30 m.

Figure 4.4 Profile development after 10 years

To get an idea about the change of volume and thus of the stability of this profile Figure 4.5 is created. This figure shows the development of the volume per alongshore meter of the most shoreward part of the profile, Area 6 (for the boundaries of this Area 6 see Table 4.1). This figure clearly shows the initial strong onshore sediment transport which is a direct result of the difference between the actual profile and the theoretical equilibrium profile (Section 2.1). As this difference becomes smaller so does the transport.

As can be seen in Figure 4.5 the difference in volume per year at the end of the run is still rather large (17.6 m³/m'), so further investigation is required. The 10 years profile of Figure 4.4 is used as initial profile for a run of another ten years to see how the profile would further develop. This resulted in Figure 4.6 and a comparison of the two profiles is given in Figure 4.7. As can be seen in both pictures the shoreline moves still further offshore and the seaward sandbar is still present in the profile but also keeps on moving further offshore. When regarding the difference between the 10 and 20 years profile roughly speaking the area between \( X = 13150 \) m and \( X = 14300 \) m erodes whereas the area shoreward of the point \( X = 14300 \) m accretes. So the total profile between NAP -12 m and NAP 1 m tends to be steepening a bit. However, the area between the "offshore" sandbar and the NAP 1 m line does accrete but the steepness doesn't change; this part of
the profile just moves seaward with the same rate for all parts. To achieve further insight Figure 4.8 is made, for the same Area 6 (for boundaries see Table 4.1) which was used in Figure 4.5 so that a direct comparison between these two graphs is possible.

The periodic influence of high waves with moderate periods ($T_p = 9.2$ and $9.4$ s.) resulting in high offshore sediment transport can be clearly distinguished. It is also evident that although the volumetric change per year becomes less as time goes by it doesn’t reach a zero growth point. Since the area roughly for $14300 \text{ m} < X < 15000 \text{ m}$ doesn’t become steeper as mentioned earlier and the rate of annual volumetric change of Figure 4.8 becomes less, it is evident that the rate of annual volumetric change of Area 5 just seaward of Area 6 will become greater. In order to determine if the profile is more

Figure 4.5 Time series of volume

Figure 4.6 Profile development after 20 years
or less dynamically stable it is divided in six areas. The vertical boundaries of these areas are printed in Table 4.1. Each area also has a horizontal boundary at an imaginary line below the sea bottom. Area 5 and Area 6 together roughly form the surf zone according to the definition of De Ruig (1989), see also Section 3.3. Area 2 up to Area 4 form the "foreshore". Since the interest isn't in the total volume of the areas, but in the change of these volumes per year, Table 4.2 is made. For each Area the volume at the end of each year is computed and subtracted from the value of the year before. In this way the volumetric change ($\Delta V_i$, with $i = 11$ to 20) per year is computed.

Note that although the volumetric change of Area 1 seems to be rather large, this area has a width of about 20 times the width of the other areas (Table 4.1). The assumption that as
Table 4.1 Areas with their boundaries

<table>
<thead>
<tr>
<th>AREA 1</th>
<th>$0 &lt; X &lt; 12175$</th>
</tr>
</thead>
<tbody>
<tr>
<td>AREA 2</td>
<td>$12175 &lt; X &lt; 12775$</td>
</tr>
<tr>
<td>AREA 3</td>
<td>$12775 &lt; X &lt; 13375$</td>
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<tr>
<td>AREA 4</td>
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<tr>
<td>AREA 5</td>
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</tr>
<tr>
<td>AREA 6</td>
<td>$14575 &lt; X &lt; 15000$</td>
</tr>
</tbody>
</table>

Table 4.2 Volumetric change per year per Area per alongshore meter ($\Delta V_i$)

the $\Delta V_i$ of Area 6 decreases the $\Delta V_i$ of Area 5 increases seems to be justified. In the beginning sand is transported through Area 5 to Area 6. As this Area 6 gets larger, the sand is deposited more and more in Area 5.

Regarding the surf zone (Area 5 and 6 together), it seems that during the last 7 years an accretion of about 18.8 m$^3$/m$^2$ per year occurs. If the cross-shore transport due to wind effects (7.5 m$^3$/m$^2$ as stated in Section 3.7) is taken into account this results in a net value of 11.3 m$^3$/m$^2$ accretion per year. In reality, as was shown in Section 3.3, the surf zone erodes very slightly. This difference may be the result of a lot of different factors. First of all the presence of a longshore current is not taken into account. The possible existence of a longshore transport gradient will of course alter the shape of the profile, as well as the cross-shore transport and thus the accretion or erosion. Secondly the used wave climate is a schematized climate. All the waves in a certain wave height class have been given the same mean period for instance, whereas in reality rather large differences in
wave periods may occur (Groenendijk and Roelvink [1992]). Since the used wave period has a large influence on the outcome of the computations this might also be a cause of the difference. Thirdly the deep water wave angle is not taken into account. It has not been taken into account because UNIBEST-TC doesn’t model the effects of this wave angle very well, as will be shown in the following paragraph. Furthermore one must realize that the UNIBEST-TC program is a model of the complex reality and deviations of that reality will automatically occur due to the schematization of the natural processes involved.

When taking the foreshore, Area 2 up to and including Area 4, into account, the conclusion can be drawn that over the years a more or less constant erosion of about 13.3 m³/m² per year takes place. This is not surprising since the sand which causes the accretion in the surf zone must have its origin somewhere nearby in the same profile since there is no longshore transport gradient. The same reasons for the deviation of reality which were mentioned for the surf zone hold true for the foreshore.

The change of volume for the different areas is caused by the cross-shore transport, Sx. The cross-shore transport at X = 13800 m is shown in Figure 4.9, for the period of one year.

![Figure 4.9 Cross-shore transport (Sx) at approximately NAP -8.0 m.](image)

The value X = 13800 m is chosen because the depth there is 8.29 m below NAP, which is very near to NAP -8 m, a significant point often mentioned in Dutch coastal defence papers. The influence of the high waves coupled to high water levels and moderate wave periods (< 9 s, which together cause the strong offshore transport) is clear, especially if Figure 4.9 is compared to Figure 4.3. Note that the transport during storms is much greater than the annual mean value. The annual mean value of the first year, as displayed in Figure 4.9, is 7.03 E-7 m³/m² whereas for instance the offshore transport during the storm of day 123.5 is 82 E-7 m³/m². The calculated transport for the complete 10 - 20 years period is shown in Figure 4.10. A very minor, hardly noticeable, decrease in the mean value of the transport occurs. The difference in the annual mean transport values of the first and the last year amounts to 1.4%. The mean yearly value of the whole 10 - 20
years period is 6.98 E-7 m³/m’s.

It was decided that the UNIBEST-TC 20 years profile was stable enough to use it as a reference profile. So it is stated that in the remaining part of this thesis use will be made of the profile as given in Figure 4.6 as the reference profile, unless the UNIBEST-TC program could cope well with wave directions. Unfortunately, this is not the case as will be outlined in the following paragraph.

4.3: UNIBEST-TC profile resulting from a wave climate with direction

Because in reality the waves don’t all approach the coast perpendicularly but will have different angles of approach, a certain longshore current will develop which will influence the cross-shore profile. To get an idea of the influence of the angle of approach of the waves on the longshore sand transport the CERC-formula is given (formula 4.1), as well as Figure 4.11 (both from Van der Velden [1990]). This figure clearly shows the influence of the angle of approach, φ₀, on the longshore sediment transport, Sₓ. As can be seen the longshore sediment transport is not symmetrical about the line φ₀ = 45° because the velocity of the waves, c_b, also varies with the angle of approach. The peak value usually occurs somewhere between 40° and 45°.

CERC-Formula (Coastal Engineering Research Centre):

\[ S_y = 0.020 \ H_0^2 \ c_b \sin(\phi_0)\cos(\phi_0) \]  \hspace{1cm} (4.1)

with \( S_y \) = longshore sediment transport \( [m^3/s] \)
\( H_0 \) = deep water significant wave height \( [m] \)
\( c_b \) = wave velocity at the breaker line \( [m/s] \)
\( \phi_0 \) = deep water wave angle \( [^\circ] \)

As explained in Section 3.4.4 the wave-angle isn’t necessarily the same as the wind
direction. As a result Table 3.4 was given. It has to be noted that the direction of the wind is given in degrees clockwise. This means for instance that a direction of 270° corresponds to a Western wind. UNIBEST-TC however uses counter clockwise directions, so the directions mentioned in Table 3.4 have to be rewritten. Since the coast at R.S.P. 84 has an angle of -30 degrees (in the UNIBEST-TC definition) all the incoming deep water wave directions have to be increased by 30 degrees. This results in Table 4.3, where the chance of occurring of a specific wave determined by its value of the wave height and deep water wave angle is given as a percentage. For UNIBEST-TC the values of $H_{\text{max}}$ have to be transformed to values of $H_{\text{max}}$ as outlined by formula 3.3.

All the parameters and variables that have to be used in the UNIBEST-TC program are set to the same values as the ones used in Section 4.2. So the only difference is that now the angles of approach of the waves are taken into account. This was done by randomizing the directions given in Table 4.3 in the same way as was done with for instance the wave heights in the previous paragraph. However, since each wave direction has different probabilities for the same wave height, this also had to be taken into account. This resulted in 57 different combinations of wave heights and deep water wave angles. These were randomized and randomly coupled to the deviation of the NAP level in the same way as is mentioned in Section 4.2. The final profile obtained after 10 years is shown in Figure 4.12.

A direct comparison with the profile obtained without taking the wave angles into account is given in Figure 4.13. It is clear that a rather strange profile develops. First of all the profile shoreward of $X = 14500$ m is very steep and secondly there appears to be a very abrupt transition between this very steep part of the profile and another very flat part seaward of $X = 14500$ m. This abrupt transition is probably caused by the influence of the deep water wave angle on the cross-shore transport. The longshore transport is shown for $X = 14700$ m in Figure 4.14. Since it is not possible to "lose" an amount of sand through the cross-shore boundaries of the profile considered, it is assumed that if the deep

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Figure 4.11 Influence of the deep water wave angle ($\phi_d$) on the longshore sediment transport ($S_L$), after Van der Velden
Table 4.3 Percentage of occurring of waves defined by the wave height (H_m0) and the deep water wave angle, \( \varphi_0 \)

<table>
<thead>
<tr>
<th>H_m0 [m]</th>
<th>39 [°]</th>
<th>43 [°]</th>
<th>27 [°]</th>
<th>4 [°]</th>
<th>345 [°]</th>
<th>331 [°]</th>
<th>322 [°]</th>
<th>313 [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>2.270</td>
<td>2.318</td>
<td>1.197</td>
<td>1.213</td>
<td>1.278</td>
<td>0.953</td>
<td>1.385</td>
<td>1.596</td>
</tr>
<tr>
<td>0.75</td>
<td>4.200</td>
<td>6.035</td>
<td>3.476</td>
<td>2.071</td>
<td>2.348</td>
<td>1.811</td>
<td>2.464</td>
<td>2.861</td>
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<tr>
<td>1.25</td>
<td>2.087</td>
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<td>2.585</td>
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<td>2.409</td>
<td>1.239</td>
<td>1.298</td>
<td>1.666</td>
</tr>
<tr>
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<td>1.782</td>
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<td>1.513</td>
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<td>1.057</td>
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<td>0.454</td>
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<td>0.678</td>
<td>0.295</td>
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<td>0.052</td>
<td>0.026</td>
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<tr>
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<td>0.000</td>
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<td>0.000</td>
<td>0.009</td>
<td>0.087</td>
<td>0.070</td>
<td>0.035</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Figure 4.12 Profile after 10 years computed by UNIBEST-TC while taking the wave direction into account

Water wave angle causes a certain mean alongshore transport, the amount of sand that leaves the area is replaced by the same amount of sand entering it. In other words, there is no alongshore transport gradient present.

The periodic influence of storms with specific directions can clearly be distinguished as well as the fact that the mean transport deviates just a little bit from zero, in fact the
mean value is equal to 8.9 E-6 m³/m's. It seems that the annual value first increases and after about two years starts to decrease. This is a direct result of the development of the profile. The local profile slope changes in time. As time continues the profile also tends to move seaward. As a result the depth at X = 14700 m becomes less. So in the beginning most of the longshore transport is situated shoreward of X = 14700 m, but as the local bottom slope changes and the profile moves seaward, the breaker line also moves seaward, and the maximum value of longshore transport also moves that way. At a certain time this maximum value is located at X = 14700 m (2 years) and after that it moves on towards smaller values of X.
The difference between the two profiles as shown in Figure 4.13 is a direct result of the fact that the cross-shore transport is influenced by the deep water wave angles. This is caused by the fact that waves which do not approach the coast perpendicularly will cause a certain alongshore directed current. Since the velocity is a vector, the wave induced alongshore current vector, \( V \), as well as the wave induced bottom flow vector, \( U \), have to be added. Since in the transport formulations use is made of higher order velocity terms, the resulting vector (\( U_r \) in Figure 4.15) has to be raised to at least the second power, i.e. \( U_r^2 \). As can be seen in Figure 4.15 this results in an onshore directed transport, since \( B > A \).

If the original situation consisted of a symmetrical movement, then due to the wave induced alongshore current a net onshore transport will result. This means that in the situation where deep water wave angles are taken into account, the resulting mean transport has to consist of a larger onshore directed value than in the original case, where the waves approach the coast perpendicularly.

The resulting cross-shore transport for the first year is shown in Figure 4.16. The mean value amounts to 8.08 E-7 m³/m's in the first year, so compared to Figure 4.9 this is indeed an increase in value. This supports the above mentioned as to how the cross-shore transport is influenced by the deep water wave angle of approach.

The conclusion is that UNIBEST-TC does not model the effects of the longshore transport on the cross-shore transport well enough to use it for prediction purposes. This is also backed by Roelvink (personal communication) so that it is decided that in the remainder of this study all attention will be focussed on using the UNIBEST-TC program without using the direction of the waves. In other words, the 20 years profile of Figure 4.6 in Section 4.2 will be used as reference profile and the deep water wave angles of approach will be neglected.
Figure 4.16 Cross-shore transport at $X = 13800$ m, taking the deep water wave angles $\varphi_0$ into account.
CHAPTER 5: BREAKWATER INFLUENCE ON WAVE HEIGHT AND PERIOD

5.1: Introduction

A submerged breakwater will influence both the wave height and the wave period. These wave characteristics influence the cross-shore transport and thus the profile shape behind the breakwater. So it is important to know in what way and to what extent these wave characteristics are influenced by the presence of the submerged breakwater.

A literature review on wave height transmission over and through breakwaters was undertaken. Since the subject interested the author, reef breakwaters as well as breakwaters with a positive freeboard are also investigated. The complete review is given as Appendix G. Only the relevant formulae needed for a conventional submerged breakwater are reprinted in this Chapter 5.

These resulting formulae (given in Section 5.2) will be used in Chapter 6 to verify the results on wave transmission as computed with the UNIBEST-TC program. There are basically two kinds of transmission formulae, one which only uses the parameter $R_c/H_i$ (for the definition of these parameters see Figure 5.1) and another one which takes more parameters into account. In order to see how much influence such an increase in parameters has in the determination of the transmission, both types will be outlined in Section 5.2 and used in Chapter 6.

In Section 5.3 the influence of the conventional submerged breakwater on the wave period is discussed. Qualitatively the process is understood rather well. Quantitatively much less is known, however.

5.2: Transmission formulae for a conventional submerged breakwater

The degree of reduction of the wave heights as a result of the presence of the breakwater is given by the transmission coefficient, $K_t$. The definition for this transmission coefficient is as follows:

$$K_t = \frac{H_i}{H_t}$$  

(5.1)

with $H_i$ = transmitted wave height [m]

$H_t$ = incoming wave height [m]

The different parameters which influence the transmission coefficient are all displayed in Figure 5.1.

The specified parameters are:

- $R_c$ = crest freeboard [m]
- $B$ = crest width [m]
- $h_c$ = height of the structure [m]
- $h$ = water depth in front of the structure [m]
- $D_{n50}$ = nominal diameter of the primary armour rock [m]
Figure 5.1 Important parameters determining the wave transmission

\[ H_i = \text{incoming wave height} \quad [m] \]
\[ H_t = \text{transmitted wave height} \quad [m] \]
\[ T_p = \text{peak period} \quad [s] \]
\[ T_{mo} = \text{mean period} \quad [s] \]
\[ S_{op} = \text{wave steepness} \quad [-] \]

The wave steepness \( S_{op} \) is determined as follows:

\[
S_{op} = \frac{2 \pi H_s}{g T_p^2} \quad (5.2)
\]

with \( H_s = \text{significant wave height at the toe of the structure} \) (mean of highest one third of the waves) \[ m \]

-Formula of Van der Meer (1990b)

Van der Meer (1990b) used a lot of data from different research to derive a relatively simple overall formula which gives the transmission coefficient as a function of only the relative freeboard. The following formula was derived, with the comment that a large amount of scatter has to be accepted:

For: \(-2.0 < R_c/H_s < -1.13\) \[ K_t = 0.8 \] (5.3.a)

For: \(-1.13 < R_c/H_s < 1.2\) \[ K_t = 0.46 - 0.3R_c/H_s \] (5.3.b)

The scatter is a direct result of the simplicity of the formulae. The parameters peak period \( (T_p) \), cross-sectional area \( (A_c) \), crest width \( (B) \) and the nominal diameter \( (D_{50}) \) are not taken into account. Figure 5.2 shows formula 5.3 with an upper- and lower boundary of \( K_t \pm 0.15 \), which is the 90% confidence level. The standard deviation amounts to \( \sigma(K_t) = 0.09 \).

The high transmission coefficient values for \( R_c/H_s > 1 \) are caused by very low wave heights relative to the used stone diameter. It is clear that in reality for very large negative values of \( R_c/H_s \) the value of the transmission coefficient should approach 1. For very large positive values of \( R_c/H_s \) the value of the transmission coefficient should approach 0 if the structure is completely impermeable. If that is not the case it is feasible that the transmission coefficient remains at a constant small value not equal to 0, since transmission as a result of flow through the structure will remain possible.
Van der Meer (1990b) states that a more in depth study is needed in which also the influence of the other factors as mentioned above (\(T_p, B, A, \) and \(D_{n50}\)) has to be studied, in order to minimize the scatter and to improve the reliability of the formulae. This study, along with a new test series, was carried out by Daemen (1991).

### Formula of Daemen (1991)

Daemen (1991) separated the freeboard \((R_c)\) and the incoming wave height \((H_i)\). To be able to use dimensionless parameters, use is made of the nominal diameter of the primary armour, \(D_{n50}\). Taking also both the peak period (via the deep water wave steepness, \(s_{op}\)) and the crest width, \(B\), into account, the following formula was derived:

\[
K_t = a \frac{R_c}{D_{n50}} + b
\]  
\[(5.4.a)\]

\[
a = 0.031 \frac{H_i}{D_{n50}} - 0.24
\]  
\[(5.4.b)\]

with for a conventional breakwater:

\[
b = -5.24s_{op} + 0.0323 \frac{H_i}{D_{n50}} - 0.0017(B/D_{n50})^{1.84} + 0.51
\]  
\[(5.4.c)\]

This formulae is presented in Figure 5.3. As can be seen in this figure there are distinct maximum and minimum values of \(K_t\). These are given as:

![Figure 5.2 Formula of Van der Meer with the 90% confidence level](image)
for a conventional breakwater: $K_{t,\text{min}} = 0.075 \quad K_{t,\text{max}} = 0.75$

Formula 5.4 is valid for:

$$1 < H_l/D_{n50} < 6 \quad \text{and} \quad 0.01 < s_{op} < 0.05$$

The standard deviation of formula 5.4 is $\sigma = 0.05$. This means that the 90% confidence level can be given as $K_t \pm 0.08$. Compared to formula 5.3 of Van der Meer where the standard deviation was $\sigma = 0.09$ this is a much better result. Formulae 5.4 can be used outside the given boundaries but the reliability will be lower according to Daemen. The upper boundaries are physically bound, because if $H_l/D_{n50} > 6$ then instability of the structure will occur and if $s_{op} > 0.05$ wave breaking will occur.

![Wave transmission graph according to Daemen](image)

**Figure 5.3** Wave transmission graph according to Daemen

It has to be mentioned again that a much more detailed description of the formulae 5.3 and 5.4 as well as other transmission formulae can be found in Appendix G.

### 5.3: Comparison of incident and transmitted period

The wave period influences the cross-shore transport and thus the shape of the profile of the nourishment behind the submerged breakwater. That is why it is important to know in what way and to what extent the wave period is influenced by the submerged breakwater. That is outlined in this Section 5.3.

It is important that a distinction is made between the different kind of periods that can be used. These periods are the significant period, $T_s$, the peak period, $T_p$, and the mean period, $T_{m02}$ or $T_{m01}$. Regarding the mean period it has to be mentioned that most research concerning transmission has been done using $T_{m02}$ and not $T_{m01}$. However, there is a clear division into two groups, on the one hand the $T_p$, and on the other $T_s$ and $T_{m02}$.

It appears that the ratio between the incoming peak period and the transmitted peak period...
is always almost 1 (Hearn [1987] and Van der Meer [1990b]). This means that the peak-frequency (the frequency where the maximum value of the spectrum is located) does not change when the waves travel over the breakwater.

This is definitely not the case for $T_p$ and $T_{m02}$. This is to be expected since these periods vary if the wave height varies. Hearn (1987) shows the importance of the shift between transmission by flow and by overtopping, at $R = 1.0$. The parameter $R$ is defined by $R_c/H_{m0}$ (see also Appendix G). For $R < 1$ the ratio of the transmitted mean period over the incoming mean period, $T_{m02}/T_{m02,i}$, is smaller than one. The conclusion is that either higher harmonic components are introduced when the waves travel over the breakwater or that longer waves are more effectively reflected for these values of $R$. For $R > 1$ the opposite is the case. This can be seen in Figure 5.4, which is one out of a series of four, each for a different value of $T_p$. Seelig (1980) finds that for submerged breakwaters the wave energy shifts to higher harmonics, which is in accordance with the findings of Hearn. Van der Meer (1990b) also supports the theory that $T_{m02,i}/T_{m02} < 1$ for values of $R_c \leq 1$, see also Figure 5.5. For the sake of comparison: $R = R_c/H_{m0}$.

One might think that because UNIBEST-TC uses the peak period the difference in wave period as the waves pass the breakwater does not have to be accounted for. As was explained in Section 3.4.3 however, the peak period used is derived from the mean period, either $T_{m01}$ or $T_{m02}$, so it is very important to know in what way and why the mean period changes.

Neither Hearn, Seelig or Van der Meer try to explain this phenomenon theoretically. Beji and Battjes (1993) do. They state that the wave breaking process itself is not the most important factor. The most important physical mechanism is the amplification of the bound harmonics during the shoaling process, and their release in the deeper region just behind the breakwater. The wave breaking process dissipates a part of the overall wave energy but doesn't influence the overall shape of the spectrum other than just scaling it down. When the waves are shoaling, the bound harmonics are amplified. Once the waves propagate into deeper water at the leeward side of the bar or the submerged breakwater, a de-shoaling process takes place, resulting in harmonic decoupling. This decoupling has the same effect as "releasing" these bound harmonics so that these waves are then decomposed into several smaller amplitude waves. This results in re-distributing the total energy, thus determining the final shape of the spectrum, towards the "shorter wave side", i.e. towards the higher harmonics. The longer the incoming waves are, the more drastically this effect will be due to the fact that a longer wave will develop a bigger tail wave (a bound harmonic) during shoaling than a shorter wave. Thus the decoupling and the redistribution of energy will also be greater.

Since there is very little data available only a rough approximation can be made of this ratio of the transmitted mean period over the incoming mean period. The numeric values of Figure 5.5 are known by the author, the ones from Figure 5.4 unfortunately not. Three additional figures like Figure 5.4 (which are not reproduced here but can be found in Hearn [1987])) will also be taken into account.

In Figure 5.4 a pattern can be distinguished for $R \leq 0.5$. It seems that if the $R$ gets a larger negative value, the ratio $T_{m02,i}/T_{m02}$ gets closer to the value one. If $R$ becomes smaller than approximately -4, it may be assumed that the influence of the breakwater has
become negligible and thus the ratio is equal to unity. The data displayed in Figure 5.5 from Van der Meer (1990b) offers a different picture. Taking the numerical values of that Figure 5.5 into account, the opposite of what occurs in the figures from Hearn takes place, i.e. if Re gets a larger negative value, the ratio $T_{m02,i}/T_{m02,p}$ becomes smaller. The reason for this is not clear. Since the validity range of these data is however very small ($-0.9 \leq R_e/H_{m0} \leq -0.4$) and all of the needed values for the ratio $T_{m02,i}/T_{m02,p}$ are outside this range (Chapter 9), it will no longer be used. So the remaining data are the graphs as given by Hearn (1987). If values of the ratio $T_{m02,i}/T_{m02,p}$ are needed they will be determined using these graphs, if possible deriving a mean value. Of course this depends on the validity range of the graphs. Examples are outlined in Section 9.2.2. Since so little data is available there is no use in trying to determine a more specific relation than the one as given by the graphs.
CHAPTER 6: COMPARISON OF THE TRANSMISSION COEFFICIENT

6.1: Introduction

In this Chapter 6 the UNIBEST-TC program is verified for the process of wave transmission over and through breakwaters. This verification is necessary because if the program cannot cope with wave transmission, it cannot be used for the simulation of a situation with a nourishment behind a submerged breakwater. A certain breakwater is implemented in the program. Using a specific wave climate the wave height in front (H_i) and behind (H_r) the breakwater are computed using the UNIBEST-TC program. This is done by letting the program run in the usual way, as in a case without the breakwater present. However, a lot of computing points have to be set on the breakwater in order to be able to achieve an acceptable degree of accuracy. By doing this, the transmission coefficient (K_t), as computed with UNIBEST-TC (or more specifically, with ENDEC), can be determined since K_t is equal to the transmitted wave height divided by the incoming wave height. This value is then compared with transmission coefficient values computed with the formulae as stated in Chapter 5.

A comparison between the two transmission formulae as given in Chapter 5, is also carried out. This is done in order to see what the difference in computed transmission values is, if more parameters are taken into account. In this Chapter 6 all attention is focussed on the conventional submerged breakwater type. However, a comparison of wave transmission coefficients for a reef breakwater has also been carried out. The results of this comparison can be found in Appendix H.

To see how the program UNIBEST-TC or more specifically ENDEC, reacts to a breakwater, use is made of the possibility to implement a fixed bottom for a part of the profile (Zf-table). The original idea behind this possibility of labelling a part of the profile as fixed was that if laboratory measurements in flumes were to be simulated or verified, the bottom of the flume would remain stable, i.e. would not change, during the computations. However, it can also be used for simulating a breakwater. An important reservation has to be made at this point, namely that when using the program in this way only totally impermeable breakwaters can be simulated. This may cause deviations in results when compared to other theoretical computations.

Since the primary interest in this chapter lies in the specific transmission of the earlier mentioned wave climate (Section 3.4) coupled to the vertical tide variations and not in the morphologic behavior of the profile, use was made of the possibility of carrying out all the computations on the initial profile (ibod=0). In this way no bottom changes will be calculated so every wave will be influenced by exactly the same bottom profile, which then includes the breakwater.

The input consisted of the ten possible wave heights with their accompanying wind set-up (Table 3.1) coupled to the eleven different vertical tide water level variations (Table 3.5), which adds up to a total of 110 possibilities. Since it is of no importance in what sequence these waves are imported no randomizing is necessary.

Three different breakwaters (also called dams) were investigated which showed the relative influence of different values for the freeboard as well as the effect of different
placements (as denoted by the horizontal breakwater axis coordinate \( X \ m \)). The first breakwater was placed on the foreshore well outside the surfzone, with its axis at \( X = 13325 \ m \). The second breakwater was placed just inside the surfzone, at \( X = 14100 \ m \). Both breakwaters have a crest height at NAP -2 m, so that the influence of the \( X \)-coordinate on the comparison of the transmission coefficient could be investigated. The crest height of NAP -2 m is chosen because it guarantees a negative freeboard under all circumstances (the greatest negative value of the sum of the vertical tide and the wind set-up is equal to -1.335 m and the wave set-down at both places has a maximum negative value of -0.062 m). The crest height of NAP -2 m is still near enough to the surface to influence at least the bigger waves.

To determine the influence of the crest height a third breakwater was used at the same location of the first, but now with the crest height at exactly NAP.

The formulae used to determine values of the transmission coefficient compared to the ones computed with UNIBEST-TC are mentioned in Section 5.2. The incoming wave heights were determined by using the ENDEC module and using the results obtained for the resulting wave heights at the locations of the toe of the different breakwaters.

A run without a breakwater present was used to obtain values of the wave set-up or set-down (\( \eta \)) at the location where the axis of the breakwater was planned in later computations. This value is needed for the determination of the freeboard. It might be more appropriate to use a value of the set-up obtained from runs in which the breakwater is implemented but since that value changes rapidly during the passage of a wave over the breakwater and it was not very clear which value should be taken (mean or maximum) this was not done. Furthermore this set-up value is relatively small compared to for instance the vertical tide variation, so that the influence of this decision on the computed value of the transmission coefficient will be minimal.

Since this chapter is mainly focussed on comparing transmission coefficient values, the three breakwaters mentioned were not extensively studied on their stability or economics of shape. Stability computations are carried out in Chapter 8 when a final choice has been made which type of breakwaters are to be used and where they will be placed to support the nourishment behind it. A few preliminary calculations were done however with the program package BREAKWAT (more about this program can be found in Chapter 8) so that the primary armour layer consisting of rocks would be stable. The value of the diameter, \( D_{50} \), as well as the crest width is needed in the formula of Daemen (1991), so these computations were necessary.

To determine what kind of rocks should be used in the computations the "requirements for standing gradings" from the CUR report 154 (1991) was used. It appeared from the computations with BREAKWAT that rocks out of weight class 3000-6000 kg with a \( D_{50} \) of \( \pm 1.18 \) m did not cause instability whereas stones from the next lighter class, 1000-3000 kg with a \( D_{50} \) of \( \pm 0.90 \) m could cause instability when regarding at breakwater one and two. The Shore Protection Manual (1984) advises a crest width of at least three times the primary armour stone diameter. Since these were only preliminary computations, it was decided that a crest width of about 4 m would suffice for both dam 1 and dam 2. Dam three was given a crest width of 5 m because here the heavier class stones gave a much bigger safety against instability compared to the lighter ones since the crest is much
closer to the water level. In that case the rocks are more violently attacked by the waves. Again the point is stressed that in this chapter the only interest lies in the comparison of computed transmission coefficient values and not in morphologic changes or in the question whether or not the outline of the dam is the most feasible one.

A description of the values computed for the different breakwaters/dams is listed in Appendix D. This appendix consists of the results of the program TWKTBERT.PAS which was constructed (in Turbo Pascal 6.0) in order to compute values of the transmission coefficient in a faster way compared to the use of a spreadsheet.

All the relations needed to compute these values have been outlined in previous chapters except the one between the significant wave height, $H_{\text{sig}}$, and the mean wave height, $H_{\text{m0}}$, at the seaward toe of the structure. This relation is needed for the formula of Van der Meer and for the determination of the fictitious wave steepness at the toe of the breakwater. Since the $H_{\text{m0}}$ is known through the computations with UNIBEST-TC so is $H_{\text{m0}}$. Hokke and Roskam (1986) state that $H_{\text{sig}} = H_{\text{m0}}$ with the comment that "$H_{\text{m0}}$ is usually a few centimeters higher". In Roelvink (1993) it is found that $H_{\text{sig}} = H_{\text{m0}}$ as long as the percentage of broken waves ($Q_b$) is smaller than approximately 20%. Since this is the case for all three breakwaters this relation was used.

The value for the freeboard, $R_c$, is computed as follows:

$$ R_c = h_c - h $$

with $h_c$ = height of the breakwater \([m]\)$

$h$ = depth at the toe of the breakwater \([m]\)$

and the value for $h$ is computed as follows:

$$ h = d + \Delta h_t + \Delta h_{\text{wind}} + \eta $$

with $d$ = depth compared to NAP. \([m]\)$

$\Delta h_t$ = vertical tide water level variation \([m]\)$

$\Delta h_{\text{wind}}$ = wind set-up \([m]\)$

$\eta$ (eta) = wave set-up \([m]\)$

In the following paragraphs the results for the three different breakwaters will be presented and discussed. Conclusions on the comparison of the different $K_t$ values are given in Section 6.5. In the following paragraphs use will also be made of the symbol $K_t$ to denote the transmission coefficient.

6.2: Submerged breakwater outside the surfzone

This breakwater (first dam) was placed with its axis at $X = 13325$ m. The crest height is located at NAP -2 m, which gives a breakwater height of 9 m, since the bottom there is located at NAP -11 m. The width of the crest was set at 4 m and the width at the bottom is set at 50 m. For a graphic presentation see Figure 6.1 and 6.2. The results of the comparison of $K_t$ values are displayed in Figures 6.3 and 6.4 and 6.5.
Considering Figure 6.3 where the $K_i$ values of Van der Meer ($K_{d}$) and Daemen ($K_{a}$) are compared it seems that they agree rather well. Only at values of $K_i$ lower than about 0.65 Van der Meer tends to give a structural higher value. The limiting effect of the maximum $K_i$ value of 0.75 of Daemen is obvious. There is a rather large amount of scatter however, which is due to the difference in the number of parameters and the way in which they are taken into account in the different formulae. Since the Daemen formula uses more parameters this is probably the more precise one.

Another result of the use of different parameters is that when the same breakwater is used but now with a primary armour rock diameter of 1.45 m instead if 1.18 m and a crest width $B$ of 5 m instead of 4 m, the values of $K_i$ according to Daemen would be influenced whereas the values of Van der Meer remain constant. If computations using these changes are carried out it appears that the values of $K_i$ according to Daemen become smaller so that the datapoints displayed in Figure 6.3 would shift to the left. This amounts to a less agreeing effect, especially since the low values of $K_i$ according to Van der Meer
The comparison between values of $K_t$, computed with UNIBEST-TC (also denoted by $K_t$ UNIBEST) and the ones mentioned above is shown in Figure 6.4. The overall picture is that the $K_t$ UNIBEST in general has a lower value than the other two, although for higher values of $K_t$ this difference becomes smaller. This is the direct result of the fact that the breakwater as represented in the program is impermeable as mentioned in Section 6.1, whereas both Van der Meer and Daemen used structures with a certain degree of permeability. It is evident that when the waves are influenced by the breakwater the permeability will also influence the transmission. This is especially true when for instance a high wave coupled to a high negative tide variation is regarded. That is why the differences between the computed $K_t$ values with UNIBEST-TC and the ones from Van der Meer and Daemen increase as the $K_t$ values decrease.
Another point of interest is how to use the maximum values for the Van der Meer and Daemen formulae. In the pictures presented in this paragraph and in Section 6.3 and Section 6.4 use is made of the maximum and minimum values as well as the boundaries for which the formulae are valid as given by these authors. Whenever a parameter determining a value of $K_t$ was outside this range, the transmission coefficient $K_t$ was given the value 100, see also Appendix D. As a result these values are not shown in the pictures. This does present a problem however because it is unrealistic to let the transmission stall at a certain point when it is evident that a certain higher value should be used. As is seen in Figure 6.4 the Van der Meer formula does not give any more $K_t$ values once the $K_t$ value computed with UNIBEST-TC is greater than 0.9. The Daemen formula however continues to present the same maximum value of 0.75 even near to where the maximum value of 1 is computed with UNIBEST-TC. Either the ranges are set too wide and should be narrowed down, or a different formula has to be used for these high $K_t$ values. This discussion is continued in Section 6.5.

All in all the agreement is quite good. It might be possible to get better results by using a breakwater with a freeboard which is a little bigger when computing $K_t$ values with UNIBEST-TC compared to the one which is used to compute $K_t$ values with the formulae from Daemen and Van der Meer. In this way the data displayed in Figure 6.4 would move to the right so that a better agreement is reached. Of course the structural difference between the deviations of high and low values of $K_t$ would still exist.

6.3: Submerged breakwater inside the surfzone

This breakwater also has a crest width of 4 m, a crest height at NAP -2 m and a primary armour consisting of rocks with a diameter equal to 1.18 m. The axis is situated at $X = 14100$ m. Since the bottom is located at NAP -7 m the height of the breakwater equals 5 m. The width at the base of the breakwater is set at 50 m. This breakwater is shown in Figures 6.5 and 6.6 as Breakwater 2.

The comparison between the $K_t$ values from Van der Meer and Daemen is shown in Figure 6.7. Compared to the ones computed with dam 1 the values of Van der Meer are
slightly higher for the same $K_t$ values according to Daemen. In other words, the data-points have shifted upwards in the Figure. When inspecting the results as printed in appendix D the reasons for this are threefold. Firstly, the wave height at the toe of the structure is generally lower when using the same deep water wave heights as in Section 6.2 because of the wave set down. Secondly since the set-up used is computed for a situation without the dam present (Section 6.1), it has a larger negative value, so that the freeboard is slightly smaller. Thirdly the wave steepness also changes because the wave height at the toe of the breakwater is smaller, i.e. the wave steepness is also smaller. When the influence of these three factors is combined it leads to the following conclusion: up till step 55 values for both $K_{t1}$ and $K_{t2}$ are lower and from step 56 till 110 these values are higher compared to the ones computed in Section 6.2. Before step 56 the $K_t$ values are lower because in the case of Van der Meer the negative change of $R_s$ is bigger than the negative change in $H_{sig}$ so that $R_s/H_{sig}$ is smaller and hence $K_{t1}$ is smaller. From step 56 onwards this process is reversed, so that the $K_t$ gets bigger. The value of $K_t$ according to Daemen is influenced in a similar way although now the change in wave steepness also
plays a (minor) role. Before step 56 the negative change of the \(K\), according to Van der Meer is less than the negative change in the \(K\), according to Daemen so that the ratio \(K_{\text{Van der Meer}}/K_{\text{Daemen}}\) gets bigger. After step 55 this is still the case but now due to the fact that the value of the \(K\), according to Van der Meer grows faster than the one according to Daemen.

Figure 6.8 shows the comparison between the \(K\) values as computed with UNIBEST-TC and the ones computed according to Van der Meer and Daemen. When comparing this Figure 6.8 to Figure 6.4 it is obvious that in general the agreement is better. It is also clear that in this case the data points according to Van der Meer are indeed plotted higher than the ones according to Daemen, as should be the case as mentioned above.

![Figure 6.8 Comparison of values of the transmission coefficient, \(K\), computed with UNIBEST-TC and with the formulae of Van der Meer and Daemen for the second dam](image)

The complete range of data points has shifted to the right which implies that in general for the same values of \(K_{\text{1}}\) and \(K_{\text{2}}\) the \(K_{\text{UNIBEST}}\) is higher. The explanation for the \(K_{\text{UNIBEST}}\) being higher is that since the incoming waves are now lower they will be less influenced by the breakwater compared to the first case. The lowering of the wave heights plays a more important role than the very slight lowering of the freeboard caused by the parameter eta. Evidently the UNIBEST-TC program reacts more fiercely to these changes than the formulae of Van der Meer or Daemen. Again reference is made to Appendix D where it can be seen that the \(K_{\text{UNIBEST}}\) (which represents the \(K\), as computed with UNIBEST-TC) has a larger value for every step when compared to the first breakwater. At every step the relative growth of \(K_{\text{UNIBEST}}\) is bigger than the relative growth of \(K_{\text{1}}\) and \(K_{\text{2}}\). This explains why the data points have shifted to the right. Due to the fact that up to and including step 55 the \(K_{\text{1}}\) and \(K_{\text{2}}\) are smaller, whereas the \(K_{\text{UNIBEST}}\) is bigger, and after step 55 all three \(K\)'s are higher as compared to the first situation, an additional change in the data points placement occurs. This change is small compared to the general shift to the right however.

6.4: Breakwater with crest height at NAP outside the surfzone

Since this breakwater has a crest height located at NAP and is placed in exactly the same
location as dam 1, the need for bigger primary armour stones was clear. That is why the diameter of the primary armour rock equals 1.45 m and the crest width amounts to 5 m. The base width of the dam is set at 50 m. This breakwater is shown in Figures 6.9 and 6.10.

Since the breakwater crest is located at NAP, it is now possible to experience situations with a positive freeboard, i.e. $R_s > 0$. A wider range of $K_\xi$ values is now expected and not only the maximum but also the minimum values of the formulae of Van der Meer and Daemen have to be taken into account. The comparison between $K_{u1}$ (Van der Meer) and $K_{a2}$ (Daemen) is shown in Figure 6.11.

As outlined before the range is indeed much larger and consists of nearly the total possible range for the Daemen formula; $0.075 < K_i < 0.75$. The agreement is quite good although the same trend as is observed in both the two previous cases is present, namely that at low values of the transmission coefficient the $K_i$ according to Van der
Figure 6.11 Comparison of computed values of the transmission coefficient, $K_t$, according to the formulae of Van der Meer ($K_{vl}$) and Daemen ($K_d$) for the third dam.

Figure 6.12 Comparison of values of the transmission coefficient, $K_t$, computed with UNIBEST-TC and with the formulae of Van der Meer and Daemen for the third dam.

Meer deviates more from the $K_t$ according to Daemen than at high values. It also seems that the $K_{vl}$ tends to be a little higher than the $K_d$. This is probably caused by the use of bigger stones as is outlined in Section 6.2. The data compared to the values computed with UNIBEST-TC ($K_{u,n}$) are shown in Figure 6.12.

The large scatter at the value zero from $K_{u,n}$ is caused by the fact that at positive values of the freeboard the question if the breakwater is permeable or not plays an important role. Especially if the negative vertical tide variation is large and the wave heights are small the $K_{u,n}$ is always near to 0 whereas the $K_{vl}$ and $K_d$ give certain larger values. The general deviation resembles the first case (Figure 6.4), but now the maximum value has a much smaller influence on the general picture.

It seems that since in this case the influence of the permeability of the breakwater is
greater than in the previous, second case, the difference between the \( K_{t,u} \) and the \( K_{t} \) and \( K_{\alpha} \) is also greater.

### 6.5: Conclusions on the comparison

The fact that in UNIBEST-TC only a breakwater with zero permeability can be implemented, plays an important role. The logical consequence is that when the freeboard \( (R_c) \) has a positive value or a small negative value (compared to the incoming wave height) a rather large deviation occurs between the \( K_{t,u} \) (\( K_t \) as computed with UNIBEST-TC) and the \( K_{t} \) (\( K_t \) according to Van der Meer) and \( K_{\alpha} \) (\( K_t \) according to Daemen). Especially at a \( K_{t,u} \) value of 0 caution has to be maintained (Figure 6.12). This deviation becomes drastically lower if the \( R_c \) gets a larger negative value, i.e. the crest height is further below the water level, or the incoming wave height becomes smaller.

The difference between computed values of \( K_{t} \) and \( K_{\alpha} \) seems to be rather small, although a structural deviation between high and low values of \( K_t \) exist. At low values of \( K_t \) the value of \( K_{t} \) tends to be structurally higher than \( K_{\alpha} \), whereas at high values of \( K_t \) this effect is less present. There is also a large influence of the used primary armour rock diameter because whereas \( K_{\alpha} \) does react to a change in rock diameter, the \( K_{t} \) does not. This influence has to be determined for each case individually.

The question what to do with certain maximum values of \( K_t \) still remains. To gather some insight in this problem the Figures 6.4/6.8/6.12 have been reprinted in this paragraph but now omitting the limiting maximum/minimum values as well as the boundaries given by Daemen (1991) with reference to the wave steepness and the ratio wave height over rock diameter, \( H/H_{50} \). These figures are reprinted as Figures 6.13/6.14 and 6.15.

In all three figures a large scatter for high \( K_{t,u} \) values (featured on the horizontal axis) appear in the data points of the \( K_{t_2} \) (represented by the squares). These are mainly caused by very low wave heights in comparison to the rock diameter. Van der Meer (1991) states that this kind of values should not be used because they are out of the validity range. Taking that into account, it seems that although the deviations become larger as the \( K_t \)
The general conclusion is that a transmission coefficient computed with UNIBEST-TC in the case of a conventional breakwater offers a good approximation of the transmission coefficient according to either Van der Meer or Daemen. If it would be possible to introduce a certain permeability for the breakwater in the program the similarity would even be better. In the present situation one has to pay special attention when low values of the transmission coefficient are computed, especially at values of the freeboard near to 0, since the permeability of the breakwater plays its biggest role then.

To determine how the profile will develop if a conventional breakwater and a nourishment behind it are present, two different approaches can be followed. The first consists of using the whole profile and computing the change of the waves as they pass the breakwater with UNIBEST-TC. The peak period has to be scaled down however (Section 5.3). This will also influence the seaward part of the profile next to the breakwater because in this way the whole profile will be affected by the change of the period. The second...
approach consists of carrying out the computations on just a part of the profile, consisting of the breakwater and the further shoreward part. In this approach the waves will have to be scaled down too, due to the transmission over (and through) the breakwater. This can be done by using as the incoming wave height the wave climate given by the UNIBEST-TC computations at the toe of the breakwater and then applying the formula from Daemen, without the maximum values. The formula of Daemen is taken because it is more precise than the one from Van der Meer and because a certain degree of permeability is used whereas UNIBEST-TC can only cope with zero-permeability. For the very low wave heights it is better to use the formula of Van der Meer since the formula of Daemen is no longer valid then. When the data are not in the validity range of either formula use can be made of transmitted wave heights as calculated with UNIBEST-TC. If the breakwater is completely impermeable then it might be best to use only the transmitted wave heights as computed with UNIBEST-TC. This will be further discussed in chapter 9.
CHAPTER 7: ARTIFICIAL BEACH NOURISHMENT

7.1: Introduction

An artificial beach nourishment, from here on abbreviated to nourishment, usually consists of supplying a certain coastal area with an amount of sand in order to call a halt to the retreat of this coast. This erosion may be the result of the combined influences of tides, currents and waves or man-made changes along the coastline. As opposed to the United States where artificial beach nourishment projects were carried out even before the Second World War, only after about 1970 large nourishments have been carried out in the Netherlands. Due to the increased knowledge of morphological processes and hydraulic matters, the nourishments which have been carried out during the last twenty years have all but one lived up to their expectancy, being the supposed number of years (the life span, usually between 5 and 10 years) that the nourishment has to consist out of a minimal area of sand (Technical Report No. 11 [1989]). So the development of a certain artificial beach nourishment scheme can be predicted with a fairly high degree of accuracy. Due to the dynamic nature of a nourishment, monitoring and observation will be necessary during the complete life span in order to see if the nourishment behaves as expected.

In Section 7.2 the type of nourishment to be used in this study is determined. In the following Section 7.3 the importance of the used borrow sand is outlined. It is shown that the diameter of the borrow sand compared to the diameter of the native sand plays an important role in the overall profile shape. An indication of the costs per cubic meter nourishment sand is given in Section 7.4. A distinction is made between the costs for a nourishment of relatively small proportions, as usually carried out along the Dutch coast, and a nourishment consisting of very large amounts of sand (see Chapter 1) as in this study.

Section 7.5 features an unprotected nourishment needed for a beach widening of approximately 1000 m. The amount of needed sand is computed and using the cost values as stated in Section 7.4, the total cost is derived. This cost value will be used in Chapter 10, in which a cost comparison is made between a protected nourishment and the unprotected one as featured in Section 7.5.

7.2: Types of nourishments

In general a complete beach nourishment project can be divided in three parts, namely the fill area, the borrow area and the way the sand is transported between these two areas. All three elements influence both the economical and technical aspects of a certain project. First of all the decision which type of beach fill should be used, has to be made. According to the Manual on Artificial Beach Nourishment, also mentioned as CUR Report 130, (1987), four types in the cross-shore position can be distinguished. For the positions reference is made to Figure 7.1.

1) Location / Type A: The placed sand is not attacked by waves unless a severe storm causes a large part of the dunes to erode.

2) Location / Type B: The sand is placed at the dune face and will occasionally be attacked by waves during moderate storm surges.
3) Location / Type C: The placed sand is continuously influenced by waves and currents since it is located on the beach. Only the uppermost part of the beach fill will not be continuously reshaped.

4) Location / Type D: The sand is deposited in the offshore zone and will also be continuously influenced and reshaped as a result of the wave- and current forces.

Since in this report only cross sections of the coast are dealt with, the variation in along-shore position will not be outlined in detail. This along-shore question mainly refers to whether placing the sand at exactly the required position or making a stock-pile of sand and letting the natural hydraulic forces distribute it along the coast. A very detailed description of artificial beach nourishments from the design process to the executing and monitoring, can be found in the earlier mentioned CUR Report 130 (Manual on Artificial Beach Nourishment [1987]). A rough translation of this Manual in Dutch is given by the Dutch "Handboek Zandsuppleties" (1988). A third publication also resulted from this CUR study, namely the Background Information on Artificial Beach Nourishment (1986). Whereas the English volumes deal more specifically with the scientific aspects and backgrounds of nourishments in general, the Dutch volume is intended for practical use in specific Dutch situations.

The objective of the beach nourishment will determine its type. Since in this report that objective consists of expanding the present beach width, the nourishment will be a mixture of Types C and D. If however this coast enlargement gives rise to building on this new strip of land, then due to safety reasons a new range of dunes will have to be constructed on the seaward edge of this new strip. For now emphasis is only put on the expanding of the beach and not taking eventual future building plans into account, so that the beach fill will have to be placed on locations C and D as displayed in Figure 7.1. At the seaward edge the nourishment will be supported by the submerged breakwater. As a comparison the development of a nourishment without the presence of a breakwater is featured in Section 7.5.

Reference is made to CUR Report 130 for the available methods of execution and different types of dredging equipment. One important remark has to be made at this stage however. Since the submerged breakwater will be built before the dredging of the beach fill starts, the material used for the beach fill must be able to cross the breakwater. Dependent on the crest height of the submerged breakwater this poses limitations on the dredging- and transportation equipment which can be used.
7.3: Borrow area

The area where the sand is coming from is treated here separately because of two reasons. The first is that the grain diameter of the sand which comes from this certain borrow area is one of the most important parameters which (apart from the incident wave processes and sources or sinks of sediment in the specific area), influences the beach slope (Swart [1991]). The second being the fact that if this borrow area is located further away from the beach fill, the transport costs will be much higher and thus the cost per m$^3$ beach fill will increase.

The grain size of the borrow sand should be equal to or larger than the grain size of the native sand if possible. The reason for this is graphically displayed in Figure 7.2. The top picture shows the case where the borrow sand is coarser than the native sand, i.e. $w_2 > w_1$. The new beach slope will be steeper so less borrow material is needed to obtain a certain enlargement of the beach strip as opposed to when finer borrow sand is used. This last case is featured in the bottom half of Figure 7.2.

![Figure 7.2 Influence of the grain diameter on the average slope of the beach fill, according to Handboek Zandsuppleties](image)

The theoretical background of this Figure 7.2 follows from scale relationships as derived by Vellinga (1986). In this report results from extensive tests on dune erosion are discussed. The following relation is derived (no. 138 in Vellinga [1986]):

$$n_1 = (n_w)^{-0.56}$$  \hspace{1cm} (7.1)

with 

$\begin{align*}
  n &= \text{ratio of prototype value over model value} \\
  l &= \text{length} \\
  w &= \text{fall velocity of the sediment}
\end{align*}$

Formula 7.1 can be rewritten as:

$$l_2 = (w_1/w_2)^{0.56} \times l_1$$  \hspace{1cm} (7.2)
with subscript

\[ \begin{align*}
1 & = \text{original value} \\
2 & = \text{new value}
\end{align*} \]

Figure 7.3 displays the differences in profile slopes when different values of \( D_{50} \) (grain diameter) are used. Figure 7.3 gives the same (quantitative) results as the (qualitative) ones as shown in Figure 7.2.

The conclusion is that it is important to know in what way the borrow sand grain size is related to the original grain size. The development of a less steep profile if borrow sand is used with a smaller diameter than the diameter of the native sand, is supported by a report on test series carried out at Delft Hydraulics, namely a report on a situation near Lido di Ostia (Van Overeem [1989]). Generally speaking, if the diameter became smaller, the profile became less steep (this is also found in Van der Velden [1990]). It is advised to use a coarser type of borrow sand if possible.

The problem of efficiency and costs relates directly to the choice of borrow site. If the site is located further away from the beach fill the transportation costs will increase but a possible influence of this borrow area on the beach fill will become less. The Dutch government states that the borrow area has to be located further offshore than the 20 m depth line or, if this depth line is located more than 20 km away from the shore line, this 20 km line equals the minimum seaward value. The reason behind this rule is that the dredging of the borrow material will thus have no effect on the morphology of the coast. The transportation costs are rather high however, and it might be economically a better option to use borrow areas more shoreward than the present ones, even if the erosion of the new beach profile increases a bit due to these new, more nearby, locations. This has to be determined for each specific case individually.

A study performed on this subject by Ribberink and Roelvink (1989) which was used in the Technical Report No. 10 (1989), gave as results that the effects of a borrow pit should be separated into direct- and indirect effects. The first are the effects as on wave heights for instance, and the second, indirect effects, are the more long term morphological effects. It is shown that no direct effects should be expected in the surf zone if the borrow pits are located seaward from the 10 m depth line. The indirect effects on the
coast line of the closed Dutch coast (as outlined in Section 3.2) will not be noticed in at least 200 years if the borrow pit is located seaward of the 16 m depth line. So it seems that the boundary could indeed be specified as the 16 m depth line, instead of the presently used 20 m depth line. However, since the actual occurring processes have all been schematized in the study of Ribberink and Roelvink (1989), it is stated in Technical Report No. 10 that these results should only be interpreted in a qualitative way and as such do not justify a change in government policy as long as no large scale tests and accompanying measurements have been carried out.

7.4: Costs

In Technical Report No. 11 (1989) the costs per m³ sand nourishment for Dutch conditions have been determined. These costs vary depending on the place of the borrow pit and the place (and type) of the nourishment. A mean value of costs for the closed Dutch coast between Hook of Holland and Den Helder is derived. The point is stressed that these are mean values. Per area an investigation in the costs of likely nourishments has been carried out and the mean value of all these nourishments is computed. These "likely" nourishments are the ones necessary to keep the coast line at the same place as it was in 1989, i.e. all the erosion has to be compensated by nourishments.

Since the location of the submerged breakwater coupled to the nourishment is at R.S.P. 84, these cost-figures should be representative for the studied situation. Three different scenarios for the borrow pit location have been used to determine the costs. The first scenario consists of the presently used locations as outlined in Section 7.3. This means that dredging is only allowed seaward of the 20 m depth line or the 20 km line. The second scenario consists of alternative locations. In this case that means dredging just outside the NAP -10 m depth line. Finally the third scenario consists of the present locations but now using a higher calculation value. This higher value might be the result of having to use dredging equipment which is more expensive in usage as the ideal dredging equipment for the specific job for instance. This results in the first three values as printed in Table 7.1.

These cost figures are derived for cases where relatively small amounts of sand are needed compared to the needed volumes in this study. That is why a fourth cost value is given in Table 7.1 (f 5.00/m³). This value is based on the assumption that due to the large amounts of sand needed, the costs per cubic meter will be much lower. The specific value of f 5.00/m³ results from previous studies on large scale nourishments (Van de Graaff, personal communication).

When a nourishment on or behind the dunes is carried out, the amount of f 3.90 has to be added to the values of Table 7.1. This is not of primary interest for this paper however, as outlined in Section 7.2.

As can be seen in Table 7.1 there is a large difference in values. The costs as printed are nearly all inclusive, which for instance means that VAT, dredging losses, execution costs, etc. have all been included. Not included are planning costs, monitoring costs, costs resulting from a grain diameter of the borrow sand smaller than the diameter of the original sand, and evaluation costs. It has to be mentioned however, that as a result of the market mechanism of free enterprise the costs may deviate. Furthermore the assumption
Different scenarios for the borrow pit location | Costs for a beach fill within the closed Dutch coast area (per cubic meter)
---|---
Present locations | f 9.30
Alternative locations | f 4.80
Present locations with a higher calculation value | f 10.70
Present locations with very large amounts of sand | f 5.00

Table 7.1 Nourishment costs in f/m³ (Dutch guilders per cubic meter) according to Technical Report No. 11

is made that the needed amount of sand does not result from other activities. It is solely excavated for beach nourishment purposes.

The costs which will be used in the comparison between the different coast enlargement projects are the last costs as mentioned in Table 7.1 (f 5.00/m³).

### 7.5 Beach widening of 1000 m

To obtain a widening of the beach width of 1000 m by a nourishment, a large volume of sand has to be deposited. If the borrow sand has the same diameter, D₅₀, and the same grain size distribution as the native sand, and the incident wave climate stays the same, the slope of the new profile will resemble the slope of the original profile very closely. As outlined in Section 7.2, the beach part of the nourishment is thought only to be meant for recreational purposes for now and does not need to be placed at a high level above NAP. Furthermore no new range of dunes will be taken into account. The level of the beach height is determined at NAP +2 m gradually rising to NAP +2.5 m at the point where the dune face starts. This level is chosen since it is high enough above moderate wind- and wave set-up. The remark is made that this level is an arbitrary one and could be modified to a higher value.

The question arises till what depth the nourishment should be carried out. A 'safe' procedure is to move the entire cross-shore profile over the required distance (in this case 1000 m) seaward. This was earlier shown in Figure 1.1 in Chapter 1. So in this case the depth till which the nourishment is carried out is equal to 17.2 m. The amount of sand needed is 19400 m³ per alongshore meter. The total costs per alongshore meter are thus equal to f 97000.-(19400 * f 5.-). This total cost value will be used in the cost comparison as outlined in Chapter 10.

Since the coast is not eroding in the studied situation, maintenance will not be necessary. However, monitoring will have to take place in order to see if the behavior of the beach fill is indeed more or less the same as predicted.

Another, more theoretical depth value can also be derived. Hallermeier (1981) divided the
cross-shore profile into three parts and defined two depths, \( d_1 \) and \( d_2 \), as the boundaries for the middle, so called shoal zone. This is shown in Figure 7.4.

The landward boundary, \( d_1 \), is the maximum depth at which seaward transport and erosion resulting from extreme yearly wave conditions takes place. This is also the seaward limit of noticeable seasonal profile changes (shaded area in Figure 7.4). The seaward boundary, \( d_2 \), of the shoal zone is the seaward limit of the wave influenced profile. This means that it is the maximum depth at which sand is still moved (on a plain beach) by the median wave condition. Hallermeier (1981) proposes the following relation:

\[
d_1 = 2.28 H_s - 68.5 \left( H_s^2 / g T^2 \right)
\]  

(7.3)

The \( H_s \) used should be an extreme wave height exceeded only twelve hours per year. The formula 7.3 has been rewritten specially for Dutch conditions. According to Handboek Zandsuppleties (1988) the final derived formula valid for typical Dutch conditions is as follows:

\[
d_1 = 1.75 \left( H_s \right)_{0.137\%}
\]  

(7.4)

with \( d_1 \) = depth below low water \( H_s \) = significant \( H_{1/2} \) wave height with an annual exceedance probability of 0.137 %, in this case:

\[
(H_s)_{0.137\%} = 5.5 \quad [m]
\]

The second, lower boundary is given by:

\[
d_2 = 1.5 \sim 2 \times d_1
\]  

(7.5)

If the same sand is used as the native kind, then a total mean value of \( d_m = 14.5 \) m can be found for this specific case. It has to be mentioned that these values are crude approximations. A run was done using the UNIBEST-TC program in which the borrow sand diameter was the same as the diameter of the native sand \( (D_{50} = 225 \mu m) \). The same diameter is used since the expectation is that the borrow sand will be composited of this sand having this nominal grain diameter. Due to the fact that very large amounts of sand are needed, it will probably not be possible to specifically use coarser borrow sand. The nourishment was carried out till a depth of 14.5 m.

Results of this run are shown in Figure 7.5. As a result of the earlier mentioned (Section
Figure 7.5 Profile development for the unprotected nourishment carried out till a depth of approximately 14.5 m.

4.2) annual mean onshore transport of 18.8 m$^3$/m per year in the top part of the profile a slight accretion occurs. A sand bar is formed and the shoreline is moved seaward over a distance of 31 m in this 10 year period. The total amount of sand needed in this case is equal to 15300 m$^3$/m. This situation seems stable. However, one has to bear in mind that the largest wave in the used wave climate has a $H_{rms}$ of 3.36 m. If larger waves are used, as will occur in reality during very severe storms, the profile will be influenced more. Sand from the upper part of the profile will be transported to the lower part of the profile. It may even be deposited below the depth of 14.5 m. In that case additional, smaller nourishments will be needed to maintain the shoreline at the present location. Since it is not exactly clear in what way this profile will develop, use is made of the earlier mentioned nourishment which was carried out till a depth of 17.2 m.
CHAPTER 8: BREAKWATER STABILITY

8.1: Introduction

The choice regarding the type of breakwater to be used, is already made in Chapter 1. It is a statically stable submerged breakwater. The meaning of the phrase "statically stable" is outlined in this Section 8.1. Also outlined in this Section is the damage level, $S$. This parameter $S$ is important because it will be used in the stability analysis of the primary armour layer as outlined in Section 8.2.3. In order to be able to use the formulae from Daemen (Chapter 5) for the determination of the transmission coefficient (Chapter 6), the nominal diameter of the rocks as used in the primary armour layer has to be known.

The breakwaters which will be used in Chapter 9 for the protected nourishments are described in Section 8.2.2. Knowing the dimensions as well as the position of these breakwaters, a stability analysis can be carried out. The needed nominal diameter of the rocks of the top layer is derived in Section 8.2.3. In Section 8.2.4 some remarks are made about other design characteristics of the breakwater. These are not treated in detail since designing the breakwater in detail is not an aim of this study.

A rough approximation of the costs of the two breakwaters which are used in Chapter 9, is outlined in Section 8.3. This cost estimate is needed for the cost comparison between an unprotected nourishment and a protected one as featured in Chapter 10.

To determine if a certain breakwater resembles a statically stable structure, use is made of the following parameter; the stability number $N_s$:

$$ N_s = \frac{H_s}{\Delta D_{n50}} \quad (8.1) $$

With $H_s = \text{significant wave height}; \text{ in this case:}$

- $H_s = 4 \sqrt{m_0} \quad [m]$  
- $\Delta = \text{relative mass density} = (\rho_r/\rho_w)-1 \quad [-]$  
- $\rho_r = \text{mass density of the rock} \quad [kg/m^3]$  
- $\rho_w = \text{mass density of water} \quad [kg/m^3]$  
- $D_{n50} = \text{nominal diameter of the primary armour rock}$  
  units, computed by:  
  - $D_{n50} = (M_{50}/\rho_w)^{1/3} \quad [m]$  
  - $M_{50} = \text{median mass of unit given by 50% on the mass}$  
    distribution curve \quad [kg]$

Statically stable structures are those where only minor or no damage at all is allowed during design conditions. The individual armour units must be able to withstand the forces generated by the waves upon them during these conditions. The amount of damage is determined by the number of armour units which are displaced. A rough classification of this type is given by $1 \leq H/\Delta D \leq 4$. Examples of statically stable structures are for instance a caisson or a conventional rubble mound breakwater.

If a structure has a stability number greater than 5, it is classified as dynamically stable.
During design conditions the profile of such a structure changes due to the displacement of armour units as a result of the wave forces. This displacement goes on until an equilibrium is reached, i.e. until the transport capacity along the profile is reduced to a very low level. The characterization of this type is determined by the design profile, as opposed to the characterization of the statically stable type, which is determined by the design damage. An example of this dynamically stable type is the earlier mentioned reef breakwater from Ahrens (1987), see Figure 1.6. The breakwaters used to support the nourishment are of the statically stable type, however.

The amount of damage is described by a certain erosion area (for instance around SWL) related to the size of the armour stones. In this way a dimensionless parameter is presented which is independent of the size or form of the structure. In other words, the damage level parameter $S$ is defined by (Van der Meer [1988]):

\[ S = \frac{A_e}{D_{n50}^2} \]  

(8.2)

with $S$ = damage level

$A_e$ = erosion area around SWL

$D_{n50}$ = dimensionless parameter

$S$ can be seen as the number of cubic stones with an edge of $D_{n50}$ eroded within a $D_{n50}$ wide strip of the structure. Generally speaking the actual number of displaced primary armour units will be less than $S$ because of the fact that this actual number depends on the shape and the grading of the stones as well as on the porosity. Figure 8.1 illustrates the definition of the damage level $S$.

![Graphical presentation of the parameter S, according to Van der Meer](image)
The values of S which should be considered as acceptable are dependent on the slope angle of the breakwater. Generally speaking for slopes between 1 : 1.5 and 1 : 2 the initial damage is defined by $S = 2$, intermediate damage is defined by $S = 3-6$ and failure occurs if $S > 8$. If the slope becomes less steep, the acceptable values of S become larger.

### 8.2: Stability of a conventional submerged breakwater

#### 8.2.1: General introduction

A few general applicable design rules will be mentioned here which refer to the crest width (B), to how far the primary armour layer should extend below SWL, to the thickness of consecutive layers and to the number of rocks per surface area. All of these design rules are taken from the Shore Protection Manual (1984). A schematic presentation of these parameters is given in Figure 8.2.

![Figure 8.2 Different parts of a breakwater](image)

In principle the width of the crest of the submerged breakwater will be either as minimal as possible (in order to minimize the costs), or be determined in a certain way to obtain a maximum reduction of wave transmission. The latter within limits of course, due to the fact that the whole idea is to minimize the total costs by substituting the toe of the nourishment by the breakwater. The total costs of the breakwater may thus not be higher than the total costs of the toe of the nourishment. The minimum width which has to be applied is given by the formula 8.3.

$$B_{\text{min}} = 3 k_\Delta D_{n50}$$

(8.3)

with $B_{\text{min}} =$ minimum crest width $[m]$

$k_\Delta =$ layer coefficient (see Table 8.1) $[-]$

The thickness of the layers is determined by the number of layers, the $D_{n50}$ and a certain
coefficient \( k_A \) which depends on the type of material being used (either rock or tetrapods or cubes for instance). This yields the following formula:

\[
r = n \cdot k_A \cdot D_{n50}
\]

(8.4)

with \( r \) = layer thickness \([m]\), \( n \) = number of layers (normally 2 for rock) \([-]\)

The required number of units per \( m^2 \) consists of the layer thickness \( r \), multiplied by a factor taking the porosity into account, and divided by the \( D_{n50} \) raised to the third power.

\[
N_r = r \cdot (1 - P/100) \cdot (D_{n50})^{-3}
\]

(8.5)

with \( N_r \) = required number of units per \( m^2 \) \([m^2]\)

\( P \) = average porosity of the cover layer in percentage (see table 8.1) \([-]\)

These formulae are valid for rather narrow gradings. If rip rap or wider graded material is used the required number of stones is not so easily determined and use has to be made of the volume of the rock on the structure (Van der Meer [1992]).

<table>
<thead>
<tr>
<th>Armour unit</th>
<th>( n )</th>
<th>Placement</th>
<th>( k_A )</th>
<th>( P ) (in %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quarrystone (smooth)</td>
<td>2</td>
<td>Random</td>
<td>1.02</td>
<td>38</td>
</tr>
<tr>
<td>Quarrystone (rough)</td>
<td>2</td>
<td>Random</td>
<td>1.00</td>
<td>37</td>
</tr>
<tr>
<td>Quarrystone (rough)</td>
<td>&gt;3</td>
<td>Random</td>
<td>1.00</td>
<td>40</td>
</tr>
</tbody>
</table>

Table 8.1 Values for the layer coefficient \( k_A \) and the porosity \( P \), according to the SPM (1984)

The armour units should extend down slope to an elevation below minimum SWL equal to at least the design wave height when \( h > 1.5 H_s \), i.e. if the wave height is not limited by the water depth. If \( h < 1.5 H_s \), the armour units should extend to the bottom and be supported by a toe.

To determine the stability of the primary armour use will be made in Section 8.2.3 of stability formulae as derived by Van der Meer (1988 and later). These formulae substitute the Hudson formula (SPM [1984]).

The reasons for not using the Hudson formula are as follows. First of all this formula is intended for conditions in which the crest of the structure is high enough to prevent major overtopping. Since the purpose of this chapter is to determine the stability of a submerged breakwater this is a major drawback. Furthermore only regular waves have been used and only limited data is available on breaking waves (SPM [1984]). There is no description of
the damage level and neither the wave period nor the storm duration is taken into account. There might also be scale effects since most of the tests were conducted at small scales (Van der Meer [1992]). Van der Meer (1988 and later) conducted an extensive series of model tests and derived formulae in such a way that the above mentioned limitations are no longer valid.

8.2.2: Breakwater dimensions

The two breakwaters which will be used in Chapter 9 to support the nourishment are outlined in this Section. The crest of the first breakwater is located at NAP -2 m. This is the minimum depth for the crest location as outlined in Chapter 1. The exact crest width (B) will be determined in Section 8.2.3. The depth just in front of the breakwater is equal to 9.15 m. This first breakwater is featured in Figure 8.3 as Breakwater A. Figure 8.4 shows the second breakwater, Breakwater B, with the crest located at NAP -4 m. The depth just in front of this breakwater is equal to 10 m.

![Breakwater A with crest at NAP -2m](image)

*Figure 8.3 Schematic presentation of the breakwater with the crest located at NAP -2 m*

![Breakwater B with crest at NAP -4m](image)

*Figure 8.4 Schematic presentation of the breakwater with the crest located at NAP -4 m*
It was decided not to use a breakwater with a crest located deeper than the crest of Breakwater B. The reason for this is that the expected costs for the amount of extra sand needed behind such a breakwater would be greater than the amount of money saved on the construction costs of this lower breakwater.

8.2.3: Primary armour layer

The relative crest height, the spectral stability number and the allowed damage level are the parameters which determine the stability of the top armour layer. Van der Meer (1990b) gives the following formula:

\[ h_c/h = (2.1 + 0.1 S) \exp(-0.14 N_s^*) \] (8.6)

with

\[ h_c = \text{crest level relative to the sea bed} \] [m]
\[ h = \text{water depth} \] [m]
\[ S = \text{damage level (see formula 8.2)} \] [-]
\[ N_s^* = \text{spectral stability number, defined by:} \]

\[ N_s^* = \frac{H_s}{\Delta D_{n50}} s_p^{-1/3} \] (8.7)

with

\[ s_p = \text{local wave steepness} \] [-]

If a certain damage level is chosen and if the wave height, wave period, crest height and water level are known, the \( \Delta D_{n50} \) can be calculated. Of course it is also possible to determine the maximum wave height which the structure can endure if use has to be made of specific rocks. One can either use design curves as displayed in Figure 8.5 or make use of the program BREAKWAT, in which the formulae 8.6 and 8.7 are implemented.

Van der Meer (1992) advises not to use very wide gradings (\( D_{45}/D_{15} > 2.5 \)) for armour layers on a structure with an impermeable core. This advice is given because it is difficult to obtain a good gradation all along the structure and a large scatter in tests on stability occurs when using these very wide gradings. This means that the reliability of the formulae is distinctly lower than when using narrow gradings.

Certain design conditions have to be determined. If the breakwater is seriously damaged, a lot of sand from the beach fill behind the breakwater will be lost. If in the future the newly acclaimed land will be used for more than just recreational purposes, this is not allowed to happen. So a rather severe design condition has to be chosen. Four exceedance chances were determined for which a certain surge level (s.l.) and a certain wave height were estimated. The surge levels are determined using the water level exceedance graph as stated in the Delta commission Report (1960). This graph is reproduced in Appendix F. With another graph from TAW (1984), which is also featured in Appendix F, the expected values of the significant wave height accompanying each surge level are determined. Since the value of the peak period is also needed, the following relation between the significant wave height and the mean period, \( T_m01 \), from Hokke and Roskam (1986) is used:
Spectral stability number.

Figure 8.5 Design curves for submerged breakwaters, after Van der Meer

$$T_{mo1} = 1.1(H_s)^{0.31}$$

with $H_s = \text{significant wave height (} H_{1/3} \text{)}$ [cm]

Using the relation between the peak period and the mean period as given by formula 3.7, the peak period ($T_p$) is determined. The value for the root mean square wave height ($H_{ms}$) is needed for computations with UNIBEST-TC. Referring to formula 3.3 a relation between $H_s$ and $H_{ms}$ is needed. The deep water wave relation $H_{ms0} = 1.05 H_s$ of both Goda (1985) and Forristall (1987) is used. All these relations and derived parameters are grouped together in Table 8.2. The P stands for the exceedance chance per year.

<table>
<thead>
<tr>
<th>P</th>
<th>s.l. [m]</th>
<th>$H_s$ [m]</th>
<th>$H_{ms}$ [m]</th>
<th>$T_p$ [s]</th>
<th>$s_{op}$ [-]</th>
<th>$\gamma$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$5.10^{-2}$</td>
<td>2.95</td>
<td>6.50</td>
<td>4.84</td>
<td>9.6</td>
<td>0.034</td>
<td>0.82</td>
</tr>
<tr>
<td>$1.10^{-2}$</td>
<td>3.50</td>
<td>6.85</td>
<td>5.10</td>
<td>9.8</td>
<td>0.034</td>
<td>0.82</td>
</tr>
<tr>
<td>$1.10^{-3}$</td>
<td>4.25</td>
<td>7.50</td>
<td>5.59</td>
<td>10.1</td>
<td>0.035</td>
<td>0.83</td>
</tr>
<tr>
<td>$1.10^{-4}$</td>
<td>5.00</td>
<td>8.10</td>
<td>6.03</td>
<td>10.3</td>
<td>0.036</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Table 8.2 Exceedance chances per year (P) with the accompanying surge levels (s.l.), wave heights ($H_{ms}$ and $H_s$) and peak periods ($T_p$)

To determine the local wave height at the toe of the breakwater, use is made of the UNIBEST-TC program, more specifically of the ENDEC-module. Since the waves as featured are much higher than the previously used waves in the computations, the mean $\gamma$ value of 0.75 may not be appropriate. So for each specific wave height the deep water wave steepness was determined in order to compute the value for $\gamma$ which has to be
implemented in the program. This computing is done by using the earlier mentioned equation 2.8 as derived by Battjes and Stive (1985). By using this equation the value for $\alpha$ which also has to be specified in the program, can be kept at the value 1. The values for the deep water wave steepness ($s_{op}$) and $\gamma$ are also featured in Table 8.2.

The surge levels as stated in Table 8.2 include the maximum high tide, which is equal to 1.5 m. It is possible that large waves occur whereas the surge level is still rather low, for instance at the beginning of a storm (Van der Velden [1990]). In that case a high wave (although more influenced by the now lower depth compared to the situation with the total surge level present) will probably cause more damage than in the case of a high surge level. For each exceedance chance three possible water levels were investigated. The first level is equal to the surge level as printed in Table 8.2. The second level is equal to the surge level minus the tidal range (2.75 m). The third level is set at zero, i.e. a water level at NAP. These water levels are given as $h_0$ in Table 8.3, which also features the values of the wave heights ($H_{rms}$) as determined through computations with ENDEC.

<table>
<thead>
<tr>
<th>$H_{rms}$ Deep water [m]</th>
<th>$h_0$ [m]</th>
<th>$H_{rms}$ crest at NAP -4 m. [m]</th>
<th>$H_{m0}$ crest at NAP -4 m. [m]</th>
<th>$H_{rms}$ crest at NAP -2 m. [m]</th>
<th>$H_{m0}$ crest at NAP -2 m. [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.84</td>
<td>2.95</td>
<td>3.40</td>
<td>4.81</td>
<td>3.34</td>
<td>4.72</td>
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<td>4.84</td>
<td>0.20</td>
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<td>4.84</td>
<td>0.00</td>
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<td>4.10</td>
<td>2.80</td>
<td>3.96</td>
</tr>
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<td>5.10</td>
<td>3.50</td>
<td>3.54</td>
<td>5.01</td>
<td>3.48</td>
<td>4.92</td>
</tr>
<tr>
<td>5.10</td>
<td>0.75</td>
<td>3.08</td>
<td>4.36</td>
<td>2.98</td>
<td>4.21</td>
</tr>
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<td>5.10</td>
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<td>2.94</td>
<td>4.16</td>
<td>2.83</td>
<td>4.00</td>
</tr>
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<td>5.59</td>
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<td>3.70</td>
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<td>4.67</td>
<td>3.21</td>
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<td>4.26</td>
<td>2.90</td>
<td>4.10</td>
</tr>
<tr>
<td>6.03</td>
<td>5.00</td>
<td>3.97</td>
<td>5.61</td>
<td>3.91</td>
<td>5.53</td>
</tr>
<tr>
<td>6.03</td>
<td>2.25</td>
<td>3.50</td>
<td>4.95</td>
<td>3.42</td>
<td>4.84</td>
</tr>
<tr>
<td>6.03</td>
<td>0.00</td>
<td>3.07</td>
<td>4.34</td>
<td>2.96</td>
<td>4.19</td>
</tr>
</tbody>
</table>

Table 8.3 The computed wave heights ($H_{rms}$ and $H_{m0}$) at the toe of the two breakwaters

When using the BREAKWAT program the user has to specify the peak period, amongst other variables. The computations are then automatically carried out for a significant wave height of half the depth in front of the structure. Since that results in a too large wave height compared to the one as computed with ENDEC, it was decided to directly use the equations as implemented in the program instead of the program itself. These equations are the previously stated formulae 8.6 and 8.7.
A further advantage of using these formulae directly instead of using BREAKWAT, is that the results can now be presented in tables instead of just in a graph. There is a drawback, however. The values as mentioned in Table 8.4 should not be regarded as the absolute right values. Van der Meer (personal communication) states that this is precisely the reason why the BREAKWAT program only produces the results in graphical form with a large spreading (shaded area). Thus the printed values in Table 8.4 should be regarded as indicators. Model tests are necessary to determine the more or less absolute safe stone diameter or mass coupled to a certain exceedance frequency of both the wave height and storm surge level.

However, for now use is made of formulae 8.6 and 8.7. The depth in front of the breakwaters (h) is equal to the water depth as stated in Section 8.2.2 (9.15 respectively 10 m) plus the values of \( h_0 \) as stated in Table 8.3. The crest level relative to the sea bed (\( h_c \)) is equal to 7.15 and 6 m respectively. The slope angle of both breakwaters is set at 1:2. As mentioned in Section 8.1, initial damage occurs if the damage level, \( S \), is equal to 2 (Van der Meer [1992]). From computations it follows that for both breakwaters and for all four exceedance chances the water level equal to NAP causes the need for the largest primary armour rocks. If the water level is equal to the surge level, smaller rocks are needed. The results of the computations for these two water levels are printed in Table 8.4. The results of computations using a water level equal to the surge level minus the tidal range, are not given since these results lie in between the ones as printed in Table 8.4.

<table>
<thead>
<tr>
<th>P [( \cdot )]</th>
<th>( h ) [m]</th>
<th>( D_{50} ) [m]</th>
<th>( M ) [kg]</th>
<th>( h ) [m]</th>
<th>( D_{50} ) [m]</th>
<th>( M ) [kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.10^{-2}</td>
<td>12.10</td>
<td>0.94</td>
<td>2188</td>
<td>12.95</td>
<td>0.81</td>
<td>1384</td>
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<td>5.10^{-2}</td>
<td>9.15</td>
<td>1.05</td>
<td>3068</td>
<td>10.00</td>
<td>0.86</td>
<td>1706</td>
</tr>
<tr>
<td>1.10^{-2}</td>
<td>12.65</td>
<td>0.95</td>
<td>2248</td>
<td>13.50</td>
<td>0.82</td>
<td>1447</td>
</tr>
<tr>
<td>1.10^{-2}</td>
<td>9.15</td>
<td>1.07</td>
<td>3266</td>
<td>10.00</td>
<td>0.88</td>
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</tr>
<tr>
<td>1.10^{-3}</td>
<td>13.40</td>
<td>0.97</td>
<td>2392</td>
<td>14.25</td>
<td>0.84</td>
<td>1582</td>
</tr>
<tr>
<td>1.10^{-3}</td>
<td>9.15</td>
<td>1.11</td>
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<td>10.00</td>
<td>0.92</td>
<td>2035</td>
</tr>
<tr>
<td>1.10^{-4}</td>
<td>14.15</td>
<td>0.98</td>
<td>2489</td>
<td>15.00</td>
<td>0.86</td>
<td>1667</td>
</tr>
<tr>
<td>1.10^{-4}</td>
<td>9.15</td>
<td>1.14</td>
<td>3947</td>
<td>10.00</td>
<td>0.94</td>
<td>2201</td>
</tr>
</tbody>
</table>

Table 8.4 Primary armour mean rock diameter (\( D_{50} \)) and mean unit mass (\( M \)), resulting from a certain exceedance chance (\( P \))

Due to the fact that the nourishment will probably function as a basis for new housing or hotel projects in the future, the highest value of the mentioned stone mass in Table 8.4 should be used for the primary armour. This corresponds to an exceedance chance of 1 E-4 per year, which will be considered as the design condition.
Since a certain degree of gradation must always be accepted, the mean values as derived for both breakwaters can be regarded as either mean- or a lowest values. If regarded as a mean value then during design conditions intermediate damage ($3 \leq S \leq 5$) will probably occur since smaller rocks are also present. Due to the fact that the design condition is already rather severe, this will not be taken into account any further.

In conclusion: the primary armour units of the breakwater with the crest at NAP -2 m should have an approximate mean mass of 4000 kg, which corresponds to a nominal diameter of 1.15 m. The standard heavy grading class of 3000-6000 kg as mentioned in CUR Report 154 (1991) can be used. For the breakwater with the crest at NAP -4 m, the mean mass value amounts to approximately of 2200 kg which is the same as a diameter of 0.94 m. For this purpose the standard heavy grading class as stated by CUR Report 154 (1991) as 1000-3000 kg can be used.

8.2.4: Various design parameters

Since the exact design of the cross-section of the used breakwater is not of primary importance in this study, only the basic characteristics will be outlined in this Section. A rough cost estimate can be made using these characteristics so that a comparison with an unprotected nourishment is possible in Chapter 10.

The crest width of Breakwater A (Figure 8.3) is set at 5 m. Since the total cross-sectional coastal area considered is very large relative to this crest width, it is decided to make use of crest width values which are multiples of one meter. The computational grid used by the UNIBEST-TC program will otherwise not be able to take these values into account. This width of 5 m results from using a crest width of four times the nominal diameter of the primary armour rocks. Use is made of the mean nominal diameter of the standard 3000-6000 kg grading class, which amounts to approximately 1.20 m.

For Breakwater B (Figure 8.4) use can be made of the standard grading class of 1000-3000 kg. The mean diameter is equal to 0.90 m, hence the crest width is set at 4 m.

Using formula 8.4 the primary armour layer thickness can be determined. For the number of layers (n) the standard value for rock of 2 is chosen. Since the layer coefficient has a value of unity (Table 8.1), the layer thickness is equal to two times the rock diameter. This amounts to approximately 2.40 m for the breakwater with the crest at NAP -2 m, and 1.80 m for the second breakwater.

Following the basic rules given by SPM (1984) which are displayed in Figure 8.2, the mean mass of the rock used in the secondary layer of Breakwater A amounts to 450 kg. For Breakwater B the secondary mean mass amounts to 200 kg. The minimal mean mass value for the core is equal to approximately 12 kg respectively 5 kg for both breakwaters.

A filter layer is necessary between the sea bed and the breakwater. Due to the limited depth it might be better to use a geotextile instead of a granular filter. This is something which has to be checked out in more detail if a possible feasibility study is carried out.

Since the sand of the nourishment behind the breakwater is not allowed to pass through the breakwater (with the exception of the top part of the crest formed by the primary
armour layer), a certain sand-tight layer has to be constructed at the shoreward side of the breakwater. This might be done by using a geotextile or by using consecutive layers of gravel. This is also something which has to be looked upon in more detail in a possible extensive feasibility study.

### 8.3: Costs

To be able to compare a beach enlargement by a non-protected nourishment to one with a submerged breakwater present, a rough approximation of the costs of such a breakwater is needed. In Chapter 7 the costs per m$^3$ nourishment sand were stated. A same kind of cost-value exists for rubble mound breakwaters. In De Ruig en Roelse (1992) it is stated that for rocks used as armour layers a price of $f\, 50.$- per ton mass may be used. For fill material a price of $f\, 34.$- per ton mass has to be used. These prices are again almost all inclusive, just as the ones mentioned in Section 7.4 for the nourishment.

These prices do not include the costs of a filter construction between the sandy bottom and the breakwater. Since a rough approximation is needed, the total bulk volume per meter of the breakwater will be multiplied by the factor $f\, 50.$- per ton mass. That amounts to $f\, 83.50$ per m$^3$ rock, if the porosity factor ($P$) is taken into account as featured in Table 8.1. Since the primary armour of the breakwater with the crest at NAP -4 m consists of smaller rocks than the ones used for the other breakwater, the cost for this breakwater with the crest at NAP -4 m will probably be estimated too high.

The height of the breakwaters has been extended with one meter down into the sea bottom to account for deviations of the mean bottom level in alongshore direction. A rough estimation of the total costs per alongshore meter breakwater amounts to

For the breakwater with crest at NAP -2 m.: $f\, 14500.$-

For the breakwater with crest at NAP -4 m.: $f\, 10500.$-

The part which is not included in these total overall costs, is the amount of money needed to make the breakwater sand-tight (except for the primary armour layer) as well as the costs for a filter construction. This has to be kept in mind when using these two cost estimates in a comparison.
CHAPTER 9: SUBMERGED BREAKWATER WITH A NOURISHMENT

9.1: Introduction

In this Chapter 9 the profile development of a beach fill behind a submerged breakwater is described. Two breakwaters with a nourishment behind them are featured, one with the crest located at NAP -2 m and one with the crest located at NAP -4 m. These two breakwaters were previously outlined as Breakwater A and Breakwater B in Chapter 8. The costs of these protected nourishments will be compared to the costs of an unprotected one in Chapter 10.

Reference is made to Figure 1.3 in Chapter 1 which featured different possible profile shapes of the nourishment behind the breakwater. As mentioned in Chapter 1, the influence of the submerged breakwater on the wave height and the wave period plays an important role. Both the wave height and the wave period influence the cross-shore transport and thus the resulting profile shape.

A third important factor is the so-called transition zone width or time lag. This lag plays an important role in the development of the profile. It determines the spatial distance during which the production of energy (the "released" energy from wave breaking) is not immediately totally dissipated (not immediately used for the wave induced return flow). As a result even though the waves may break on the front slope or on the crest of the breakwater, the peak of the dissipation may occur behind the breakwater. This results in scour effects behind the breakwater.

One of the aims of this study is to determine in what way the shape of the nourishment behind the submerged breakwater differs from the reference profile shape (see Chapter 1). This reference profile shape is the shape of the 20 years profile as outlined in Chapter 4. To achieve more insight in the determining factors (change of wave height, change of wave period, transition zone width), each of these will be treated separately first in Section 9.2.

In Figure 9.1 a schematic display of the situation is given. A certain reference profile is present. Part of this profile is given as profile SQ. On this reference profile a submerged breakwater is built. Behind this breakwater a nourishment is placed in order to achieve a beach widening of 1000 m. At first the slope of this nourishment (slope RP) is considered to be the same as the reference slope (SQ).

Point R is located at the same depth as point S. Point R is located just behind the crest of the breakwater whereas point S is located on the reference profile without the breakwater present. If the wave climate occurring at point R would be the same as the wave climate occurring at point S, the slope of the nourishment behind the breakwater (slope RP) would be the same as the reference slope (slope SQ). As mentioned in Chapter 1 and in the beginning of this Section 9.1, this is not the case. Both the wave height and the wave period are influenced. Thus the expected resulting slope of the nourishment behind the breakwater (slope RP) will not be the same as the reference slope (slope SQ).

In Section 9.2.1 the differences between the wave heights occurring at point R (with the breakwater present) and at point S (without the breakwater present) are discussed. This is
Figure 9.1 Reference points R and P on a profile with the breakwater present, and reference points S and Q on the reference profile without the breakwater present

done for both breakwaters. Results of these differences on the shape of the profile are shown. The same procedure is followed in Section 9.2.2 for the wave period. In Section 9.2.3 the combined influences of the change in wave height and the change in wave period on the resulting shape of the profile are featured. This resulting profile shape would be the shape of the nourishment behind the breakwater if the transition zone would not play an important role too. This role is discussed in Section 9.2.4.

In Section 9.3 the results of computations with the UNIBEST-TC program on the protected nourishments are shown. The protected nourishments feature an initial coast enlargement of 1000 m.

Since it is not completely clear to what extent the results of computations using the UNIBEST-TC program are correct, another, more qualitative approach, is used in Section 9.4. Doubts about the results of the computations result from the fact that the breakwater as implemented in the program is impermeable, whereas in reality the primary armour layer will be permeable to some extent (Chapter 8). Doubts also exist about the way the transition zone width is modelled in the program. This qualitative approach is also used to determine the profile development of the beach fill behind the submerged breakwater, more specifically, the shape of the resulting profile behind the breakwater. The results of these Sections are reviewed in Section 9.5, conclusions are drawn and recommendations are made.

9.2: Important effects

9.2.1: Change of wave height

As a result of the presence of the breakwater the wave heights will change as the waves pass over the breakwater. One of the conclusions of Chapter 6 was that the transmission coefficient computed with UNIBEST-TC compared rather well with the theoretical values
as given by the formulae from Daemen (1991). There is a structural difference due to the fact that the breakwater as implemented in UNIBEST-TC is completely impermeable, whereas the formulae from Daemen have been derived for breakwaters with a certain degree of permeability. So the computed values of the transmitted wave heights at point R in Figure 9.1, might be a bit lower than the ones occurring in reality, but this difference decreases rapidly as the breakwater crest becomes more submerged. All in all the conclusion seems justified that the values for the transmitted wave heights as computed with UNIBEST-TC for a submerged breakwater will give a good approximation of reality.

The purpose of this Section is to see how the shape of the nourishment behind the breakwater compares to the profile shape without the dam present, as a result of differences in wave heights. The most important and interesting parameter is the bottom slope. If the wave height changes, this bottom slope will change, and thus the general form of the nourishment. If the profile shape of the nourishment changes, the volume needed to achieve a certain enlargement of the beach width will also change.

As outlined in Section 9.1, the wave heights occurring at point R (with the breakwater present) and at point S (without the breakwater present) are compared in this Section. Reference is made to Figure 9.1 for points R and S. First this is done for Breakwater A (the breakwater with the crest located at NAP -2 m), and later on this is done for Breakwater B (the breakwater with the crest located at NAP -4 m). Both breakwaters have been discussed earlier in Chapter 8.

The results for Breakwater A are graphically presented in Figure 9.2. The horizontal axis of Figure 9.2 features the wave height at point S, whereas the vertical axis features the wave heights occurring at point R. The used deep water wave heights are the ones as mentioned in Table 3.1. These wave heights have been coupled to the vertical tide variations just as was done in Chapter 6. Using the UNIBEST-TC program the wave heights occurring at point S (H_s) on the reference profile (without the breakwater present) were determined. The wave heights occurring at point R (H_r) were also determined using the UNIBEST-TC program. To indicate that a certain parameter value belongs to this first case of the Breakwater A, the subscript A is added. It appears that in general the waves are higher if the breakwater is present. To be more precise, the higher waves (with a deep water wave height, H_max, greater than 0.884 m) are distinctly higher whereas the low wave heights (deep water wave height, H_min, equal to either 0.177 or 0.530 m) in general are lower. The reasons are twofold: the expected values of the H_r,A will be greater than H_s,A since the transition between approximately NAP -9 m and NAP -2 m takes place over a much shorter horizontal distance in the case of the breakwater (slope of 0.3) than in the 20 years profile (mean slope of 0.007). As a result the waves will experience less influence of the bottom (like bottom friction and the
breaking of the higher waves) and will thus be higher. The higher waves support this theory. The reason that the lower waves are lower is that in this case the low $H_{a,A}$ values are in the region of wave height growth just before the breaking point is reached.

It appears that the ratio of wave height at point R over wave height at point S, $H_{a,A}/H_{a,A}$, has a mean value of 1.074. The larger wave heights will be increased by more than 7.4%; for the smaller wave heights even a decrease is found.

To see what the result of that on the bottom slope is, the development of the 10 year profile was monitored for a 10 year run with a new wave climate, $H_2$, as input. This wave climate was determined by multiplying the deep water wave height by the earlier defined ratio $H_{a,A}/H_{a,A}$. This ratio is thus used as a multiplier. If the 10 years profile is used as initial profile for a 10 year run using the original ($H_1$) wave climate as defined in Table 3.1, the 20 years, reference profile is the result. This was shown in Chapter 4. So a direct comparison between the outcome of the run using the changed wave climate ($H_2$) and the reference profile (defined as "Original $H_1$" in Figure 9.3) is now possible and shown in Figure 9.3.

![Figure 9.3 Differences in profile development when changing the wave climate](chart)

The overall profile till a depth of approximately NAP -12 m becomes less steep. This is as expected since in general a profile will become less steep if the wave heights become larger (Van der Velden [1990]). The shoreline has receded over a distance of 50 m compared to the 20 years profile shoreline. To show how the difference in wave height effects the cross-shore transport Figure 9.4 is given. When compared to Figure 4.9, which shows the cross-shore transport for the 20 years, reference profile, it is clear that the offshore transport peaks occur more frequently and have a larger absolute value. The mean value of Figure 4.9 consisted of 7.03 E-7 m$^3$/m's whereas the mean value of Figure 9.4 is equal to -9.7 E-7 m$^3$/m's, so it has the opposite sign. Regarding the point $X = 13800$ m in Figure 9.3 it is clear that this should be the case because now (the $H_2$ profile) erosion takes place shoreward and accretion takes place seaward of this point, i.e. the mean value of the cross-shore transport should have a negative value.
The area of most interest is the area from the shoreline till a depth of NAP -2 m. This area becomes slightly steeper if compared to the reference profile.

To confirm the above stated the same procedure was carried out for the breakwater with the crest height at NAP -4 m, i.e. Breakwater B. Parameters which refer to this second breakwater are given the subscript B. Since this breakwater is placed more offshore than the previous one, the lower wave heights as occurring at point S, \( H_{s,B} \), will not be in the area where wave height increase due to shoaling takes place. As a result the values of \( H_{r,B} \) should all have a greater value than \( H_{s,B} \), i.e. the ratio of wave heights at point R over wave heights at point S, \( H_{r,B}/H_{s,B} \), will be greater than unity for all the waves. This is indeed the case and displayed in Figure 9.5. The mean value of the ratio \( H_{r,B}/H_{s,B} \) is equal to 1.068. However, since now the ratio > 1 for all the wave heights, the larger wave heights will be increased by a factor of approximately 6.8%. So although the mean value of the multiplier is nearly the same as in the case of Breakwater A, the resulting profile will differ. This is shown in Figure 9.6, which gives the comparison between the profiles as computed with modified wave climates as outlined for the case of Breakwater A. Note that the scale is different (X-axis) from the one used in Figure 9.3. This is done to get a clear picture of the difference which occurs at the shoreline. This difference at the NAP level amounts to 20 m, so that the H3 wave climate causes a receding shoreline of 30 m as compared to the reference profile, in which the original (H1) wave climate was used. In this case the area of most interest is the area above a depth of NAP -4 m. As in the
Figure 9.6 Comparison of cross-shore profiles resulting from the two different wave climates (H2 and H3) computed with the ratio $H_{1,n}/H_{1,n}$

previous case, the profile in this area becomes steeper than the reference profile, although less so as in the case of Breakwater A.

Figure 9.3 and 9.6 may be seen as estimates which are a little on the conservative side. The multiplier (consisting of the ratio $H_{1}/H_{1}$) is used to create a new deep water wave climate, whereas it is measured in shallower water. Due to influences as bottom friction the now higher waves will be more influenced than the original waves. As a result it would be more accurate to use wave height values slightly higher than the ones determined with the multiplier.

In conclusion: the profile slope till a depth of approximately NAP -12 m becomes less steep. This was expected since the wave heights at point R are larger than the wave heights at point S (for location of these points see Figure 9.1). In the areas of most interest (for Breakwater A till a depth of NAP -2 m and for Breakwater B till a depth of NAP -4 m) the profile becomes slightly steeper though. The conclusion can be drawn that the resulting nourishment behind both breakwaters will have a slightly steeper bottom slope than the slope of a nourishment without the breakwater present, due to the change in wave heights.

9.2.2: Change in period

The used wave period has a large influence on the profile development, as already mentioned in Chapter 4. Groenendijk and Roelvink (1992) show that high waves with moderate periods (peak periods smaller than approximately 9 s) will cause strong offshore transport, whereas high waves with large periods result in strong onshore transport. This is especially the case if the water level is high due to the tide and wind set-up. Results of computations carried out with UNIBEST-TC also show this influence of the period. When two waves are considered with identical deep water wave height but different periods, the one with the higher period and thus being the longer wave will cause significant less
offshore transport than the shorter one.

The reasons for this influence of the period can be explained as follows. If one compares the form of two waves in shallower water, and the period of one wave is larger than the period of the other wave, certain differences exist. A wave will be more asymmetrical if the period is larger. This means that the top of the crest of the wave will be more peaked and located further away from the mean level than the trough.

As a result of this asymmetry the earlier mentioned (Chapter 2) onshore transport takes place, since the oscillatory flow moments are non-zero (Stive and Roelvink [1988]). If the waves become more asymmetrical, so will the onshore transport. An often used parameter to indicate the degree of asymmetry is the Ursell parameter, defined by (Battjes [1986]):

\[ U = \frac{(H \cdot L^2)}{h^3} \]  

(9.1)

The larger this Ursell parameter becomes, the more asymmetry occurs. Since the wave length, L, (and hence the period) appears in the numerator of the Ursell parameter, this explains the fact that a larger period results in a larger asymmetry of the wave. And if the asymmetry is larger, so will the onshore transport.

In Section 5.3 the theoretical background of the change in mean period and thus of the used peak period is explained. Reference is made to Beji and Battjes (1993). To determine the ratio of the transmitted period \( T_{m,t} \) to the incoming period \( T_{m,i} \) use is made of the graphs as presented by Hearn (1987). An example of one of these graphs is given as Figure 5.4 in Chapter 5.

To determine the ratio of the transmitted period over the incoming period, \( T_{m,t}/T_{m,i} \), with the help of these graphs, the mean value of the freeboard divided by the wave height, \( R/H_{m0} \), has to be known first. This value is determined by taking all the possible wave height-water level combinations into account. The combinations with the smallest waves (\( H_{m0} = 0.25 \) m) were left out during this procedure because they consisted of very large values and thus had a far too great effect on the mean value of \( R/H_{m0} \) as compared to all the other combinations. Using this mean value a certain value of the ratio \( T_{m,t}/T_{m,i} \) was computed. For the two specific cases the ratio amounted to the following:

Breakwater A, with crest at NAP -2 m: \( T_{m,t}/T_{m,i} = 0.73 \) (mean value)
Breakwater B, with crest at NAP -4 m: \( T_{m,t}/T_{m,i} = 0.87 \) (mean value)

To investigate the influence of this change in wave period on the development of the profile, the same procedure as in Section 9.2.1 was carried out. Figure 9.7 shows the results.

As can be seen this change of wave period affects the profile development more than the change in wave height. The reference profile is defined in Figure 9.7 as "Original period \( T_{r} \)."

The conclusion is that if the wave period becomes smaller, the general profile till a depth of approximately NAP -12 m will become less steep. The smaller the value of the wave period, the less steep this general profile will become. The area of most interest for both
breakwaters shows again a slight steepening of the profile, just as in the case of the change in wave height. For breakwater A the amount of steepening is larger than for Breakwater B.

9.2.3: Combined effect of the change in wave height and period

In this section the combined effects of the change in wave height (Section 9.2.1) and of the change in period (Section 9.2.2) on the shape of the profile are shown.

For the case of Breakwater A this results in Figure 9.8. The area of most interest, being the area from the shoreline till a depth of NAP -2 m, becomes steeper. To be able to see this more clearly, a part of Figure 9.8 is shown in more detail in Figure 9.9.

The horizontal distance between the points U and W in Figure 9.9 is equal to 182 m, whereas the horizontal distance between points V and X is equal to 147 m. This is indeed a steepening of this part of the profile. In the ten years the shoreline retreats over a distance of 117 m if compared to the reference profile. It seems that the influence of the change of both the wave height and period amounts to slightly less steepening of the resulting profile than the sum of the effects of these individual changes. This is probably the result of mutual interaction of these wave parameters on the different transport processes which take place.

The results for Breakwater B are shown in Figure 9.10. Again a steepening in the area of most interest occurs. In this case this area is the area above a depth of NAP -4 m. The steepening is less than the steepening in the case of Breakwater A due to the fact that the wave characteristics are less influenced by this second breakwater.
9.2.4: Time lag or transition zone width

In this Section 9.2.4 the time lag or transition zone width is discussed. Due to the fact that some theory on the subject is necessary and that the reader might lose track if all the information is presented without sub-divisions, use is made of sub-sections. In this way the reader can decide whether or not certain parts of the Section are too theoretical to take into account.

Introduction

The definition of the transition zone is presented first. As mentioned in Section 9.1, the
Figure 9.10 Comparison between the reference profile and the profile resulting from the changed wave characteristics caused by Breakwater B (Profile 2)

The main result of this transition zone is that although the waves may break on the front slope of the breakwater, the peak value of the wave induced return flow is located behind the breakwater. This results in a large scour effect. If a normal, gradually changing profile is used (without a breakwater present), the UNIBEST-TC program models the transition zone rather well (Roelvink and Stive [1989b]). However, due to the fact that the breakwater is implemented and thus the profile changes in an abrupt manner (not only causing an abrupt increase but also an abrupt decrease of the depth), the modelling of this transition zone in UNIBEST-TC for this specific case might not be completely correct.

In order to be able to decide whether or not the modelling in the UNIBEST-TC program is correct, theory on the subject of the transition zone is necessary. This is presented in sub-section "Theory on the NRS (1990) approach". This approach is one of the most recent approaches on the subject. Certain problems arise when using it in the studied situation however. A possible solution to these problems and an order of magnitude value for the transition zone width for the case of a submerged breakwater is presented. This value is also supported by various literature. The way in which the order of magnitude value is derived is given in Appendix I.

The way in which the transition zone width is modelled in the UNIBEST-TC program can be found in the sub-section "UNIBEST-TC approach". A comparison between the order of magnitude value of the transition zone width and this value as computed with the UNIBEST-TC program is presented in sub-section "Conclusions".

Definition

The transition zone, sometimes also referred to as the outer region, is one of the three areas in which the surfzone (where waves are breaking on an plane beach slope) can be divided. The other two are the inner zone and the swash zone. Svendsen (1984) describes this transition zone as a region with a nearly horizontal water level from just inside the
breakpoint to the beginning of a steep water level gradient (which is the result of the wave set-up). At the shoreward end of this zone a new bore-like wave with different kinematics has evolved from the wave type before breaking. Basco and Yamashita (1986) describe this evolving process in the transition zone as a transformation from oscillatory, irrotational motion to highly rotational, turbulent motion.

The remarkable feature of this zone is that rapid wave decay is present but there is not an increase in energy dissipation. Since the mean water level is nearly constant, so is the radiation stress. As a result wave-induced currents will only start to develop at the shoreward end of this transition zone. This influence of the transition zone was also described as a time lag by Roelvink and Stive (1989b). They showed that including such a time lag between the production and dissipation of energy, improved the agreement between measured and predicted values of the wave-induced return flow considerably.

**Theory on the NRS (1990) approach**

Nairn, Roelvink and Southgate (1990) (also mentioned as NRS [1990]) derive an empirical expression for the width of this transition zone. The width can be determined with the help of the ratio between the depth at the inner limit of the transition zone \(d_1\) to the depth at breaking point \(d_b\) which is determined as follows:

For \(\xi_{bb} \geq 0.05\): \[
\frac{d_1}{d_b} = 0.47 \xi_{bb}^{-0.275} \tag{9.2.a}
\]

For \(\xi_{bb} < 0.05\): \[
\frac{d_1}{d_b} = 1 \tag{9.2.b}
\]

with \(\xi_{bb} = \frac{m_b}{(H_b/L_b)^{0.5}} \tag{9.2.c}\)

where

- \(\xi_{bb} = \) surf similarity parameter at breakpoint
- \(m_b = \) bottom slope in the vicinity of the breakpoint
- \(H_b = \) wave height at breaking
- \(L_b = \) wave length at breaking

Formulae 9.2 are valid for both plunging and spilling breakers since Basco (1985) showed that both types feature the same development process from the just broken wave into a turbulent bore. Thus the start of the inner breaking region is characterized by the fact that regardless of the initial type of breaking (plunging or spilling), the form of the bores at this point are similar. The scale on which this happens is different however, and that is implemented in Formulae 9.2 by the surf similarity parameter which differs for both types.

In accordance with the definition from Svendsen the width is defined by NRS (1990) as the distance from the breakpoint to the abrupt change in slope of the mean water level. This definition is illustrated by Figure 9.11.

As can be seen in Figure 9.12 the ratio \(d_1/d_b\) becomes less (and thus the width becomes larger) if the surf similarity parameter increases.

To achieve an analytical expression which describes the influence of the transition zone...
on random wave processes, the two approaches from Svendsen (1984) and Roelvink and Stive (1989b) are combined. Svendsen suggested that part of the energy production is first converted into forward momentum flux in the roller. This influence was substituted both in the energy and the momentum balance. Since this substituting consisted of an extra term in which the roller area is directly related to the local wave height, the value of the set-up changes but the transition width lag effect is not present. This is due to the fact that the spatial distribution of the set-up remains directly linked to the wave energy decay.

Roelvink and Stive (1989b) use an equation with a storage term because they suggest that the wave energy is first converted into turbulent kinetic energy which is not immediately dissipated. As a result of this storage term, the dissipation lags behind the production. The results showed that the distribution of the set-up was not influenced however, as opposed to for instance the return flow as mentioned earlier.

The combination of these two approaches leads to an expression in which the roller serves as storage of kinetic energy (the second term in formula 9.3) and thus a lag effect is present. The energy balance equation is given by NRS (1990) as:

$$\frac{\delta}{\delta x} EC_x + \frac{\delta}{\delta x} E_x C = -2 \beta g E / C$$  

(9.3)

with $E = \text{mean wave energy}$ [$J/m^2$], $C_x = \text{group velocity}$ [m/s], $C = \text{phase velocity}$ [m/s], $E_x = \text{roller energy} = \rho kh$ [$J/m^2$], $\rho = \text{density of water}$ [kg/m$^3$], $k = \text{depth- and time-mean turbulence intensity}$ [$m^2/s^2$], $h = \text{water depth}$ [m], $\beta = \text{mean slope under the roller}$ [-]

Tests revealed that the agreement between predicted and measured values agreed much better when taking the lag into account (NRS [1990]).
The transition zone width was defined earlier (in other words) as the distance between the peak of the dissipation (D in UNIBEST-TC) and the peak of the production (P in UNIBEST-TC). Regarding Figure 9.13 it appears that the ratio $d/d_b$ is almost entirely dependent on the beach slope. A change in wave steepness ($s_{op}$) only results in a very small change in ratio $d/d_b$.

It is clear that a time lag between the production and the dissipation exists and plays an important role in many processes, including the generation of the wave-induced undertow (Roelvink and Stive [1989b]). Since this undertow, through the associated sand transport, is important for the development of the cross-shore profile behind the submerged breakwater, the value of the width of the transition zone has to be known. NRS (1990) state that when using random waves (which is the case in this report) formulae 9.2 can be used for each individual wave because mutual interaction between the waves may be ignored in the surfzone. So a comparison between the lag as computed with UNIBEST-TC (difference between peak values of P and D) and as given by formulae 9.2 is possible. However, it is mentioned in NRS (1990) that caution is advised when using high values of $\xi_{wb}$ in formulae 9.2 since indications exist that the width of the transition zone is then overestimated.

The idea behind formulae 9.2 is that if the ratio $d/d_b$ can be calculated from known values, the value of the inner limit of the transition zone, $d_i$, is known. The difference between $d_b$ and $d_i$ is then divided by the value of the bottom slope in order to know the transition zone width. The assumption is made that the value of the bottom slope remains more or less the same in this transition area.

### Problems when using the NRS (1990) approach

This last assumption is not true in the case of the submerged breakwater. Three problems arise; the first is that the testing range of formulae 9.2 lies between bottom slope values of 0.005 and 0.05. So the slope of the breakwater ($m_d = 0.3 - 0.5$) is not within this range. Secondly twice abrupt changes in the bottom slope appear, the first one at the point where the seaward slope of the breakwater reaches crest height, and the second one where the crest changes into the shoreward slope. Thirdly the depth is not monotonically decreasing but an increase of the depth just behind the breakwater is found, before a decrease appears again.

1The peak of the dissipation indicates a large water level increase and the peak of the production defines the location of the maximum gradient in the wave height distribution, i.e. the breaking point.

2This assumption is made due to the fact that the shallow water restrictions are valid and so the larger waves will have a velocity proportional to the depth. Therefore the possibility of one wave overtaking the other is limited.
Order of magnitude value of the transition zone width

In Appendix I possible solutions to overcome the three mentioned problems are presented. Results from other literature are also used to achieve an order of magnitude value of the transition zone width.

The assumption is made that the slope of the front face of the breakwater plays a more important role than the slope of the crest when determining the transition zone width using the NRS (1990) approach. The value of this width will then be rather small. If a deep water wave is considered with a $H_{m0}$ of 2.25 m having a mean period ($T_{m0}$) of 6 s, computations show that the value of the transition zone width should be in the order of 10 m. This is supported by research of Basco (1985) and of Smith and Kraus (1991). The point is stressed that this follows from a more or less qualitative approach and empirical results so that further investigation as well as model tests are necessary to confirm this order of magnitude value as well as the made assumption.

UNIBEST-TC approach

In the UNIBEST-TC program use is made of the time lag computation as presented in Roelvink and Stive (1989b). As outlined before this equation (the energy balance is reprinted here as formula 9.4) results in a time lag between the production and dissipation so that the wave induced return flow prediction is more accurate. NRS (1990) stated however, that since the set-up as computed using formula 9.4 is not significantly influenced by this lag although that should be the case, this equation does not describe all the occurring processes accurate enough. They propose the earlier mentioned formulae 9.2 and 9.3.

From a morphological point of view the fact that the return flow is modelled quite accurately is much more important than the modelling of the set-up. This is the case because the return flow is the most important determining parameter of the cross-shore transport.

However, it has to be kept in mind that in UNIBEST-TC use is made of the following equation (earlier stated in a different form as formula 2.15) which could be improved as far as the set-up is concerned by formulae 9.2 and 9.3:

$$-\frac{\delta}{\delta x}EC_t - \rho \beta_f \frac{\delta}{\delta x}khC = \rho \beta_d k^{3/2}$$  \hspace{1cm} (9.4)

with $\beta_f$ and $\beta_d$ coefficients of order 1

The different parameters as used in formula 9.4 were previously discussed in Chapter 2. The first term in formula 9.4 including the - sign is the production, $P$. The second term is the storage term and the third term is equal to the actual dissipation, $D$. The first term will have a positive value when moving towards the coast. The second term, which is positive in the area of initial decay of wave energy, will have a negative sign. Thus this second term will indeed serve as a storage term since now the value for $D$ will be smaller than $P$ and hence lag behind. The value of $\delta k/\delta x$ is positive in the area of initial wave
energy decay but will become negative further on in the process.

Qualitatively speaking the dissipation at a certain point is computed from the production at that point minus a certain amount of storage plus a certain amount of production resulting from storage at previous points. So it is important to know in what way this storage term changes as the wave propagates over the breakwater. If the storage term has a larger value then so will the transition zone width.

Due to the numerical nature of the program, the computations as carried out with UNIBEST-TC will not only use the slope of the front face of the breakwater in order to determine the transition zone width, but at each grid point the local bottom slope will be used. Since at the shoreward side of the breakwater the depth increases, this might pose a problem for the computation of the dissipation since the storage term also features the values of the change of depth, $\delta h/\delta x$, and the change of wave propagation speed, $\delta c/\delta x$. These two terms now change from a negative to a positive sign due to the depth increase. Whether or not the storage term as a whole has a positive or negative sign, depends on this change of $\delta h/\delta x$ and $\delta c/\delta x$ relative to the change in $\delta k/\delta x$.

If the value of the depth mean time-averaged turbulence intensity, $k$, is still increasing or near to its maximum value, then the storage term will always be positive and hence the dissipation ($D$) will be smaller than the production ($P$). If the value of $k$ is decreasing then $D$ can still be smaller than $P$, although also $D \geq P$ is now possible which results from a negative value of the storage term. The conclusion can be drawn that the values of the dissipation at the back slope (shoreward side) of the breakwater will be smaller compared to a situation with a monotonically decreasing depth.

In order to make a comparison possible between the earlier stated order of magnitude value and the value of the transition zone width as computed with UNIBEST-TC, an example is given. The same wave ($H_m = 2.25$ m and $T_{m0} = 6$ s) is used on a profile with a submerged breakwater and a heap of sand in front of it. Reference is made to Figure 9.14 and 9.15.

Figure 9.14 shows the bottom profile for which the comparison between the production and dissipation is shown in Figure 9.15. Since the deep water wave height is rather large, the wave will start breaking on the slope of the heap of sand just in front of the breakwater. This explains the first peak in the production. The second peak of the production results from the breakwater slope. Here, as on the slope on the heap of sand, the storage term causes a smaller value of the dissipation than of the production. The difference in these values is smaller now due to the fact that the increase of the $k$ value becomes less and because of the influence of a smaller depth value.

The fact that the maximum value of the dissipation is indeed significantly smaller than the maximum value of the production indicates the importance of the storage term in the computations.

The spatial difference between the peak value of the production (the second peak) and the dissipation appears to be very small, 3 – 4 m. One must keep in mind that two peaks of the production are present which confuses the matter of the definition of the transition zone width. The difference between the first peak of the production and the peak of the
dissipation is equal to 17 m. It appears that the general range of the transition zone width as defined by the difference in peak values of production and dissipation is quite good.

The value of the dissipation at the deepest part behind the breakwater (X = 13657 m) is greater than the value of the production. There is also a surplus of production from previous X coordinates that has to be transformed through dissipation. As a result one can see that over a relatively large area just behind the breakwater the dissipation exceeds the production. Because of the fact that this area is relatively large, the general conclusion can be that although the value of the transition zone width as defined as the difference between the P and D peak values, is small, a large part of the energy is stored and
dissipated in a later stage. This is a result of the fact that the storage term has a relatively small negative value on the back slope of the breakwater. This can also be seen as a time lag or a second transition zone width.

This last time lag exceeds the first one. Unfortunately it is not possible to see what a certain sort of mean value of these two time lags may be by checking out the wave set-up ($\eta$) distribution (definition of Svendsen [1984]). The set-up is not modelled correctly in the UNIBEST-TC program since it is not influenced by a certain time lag.

Conclusions

It seems at first that the transition zone width as computed with UNIBEST-TC (according to the definition as being the difference between the peak values of $P$ and $D$), conforms well with the small order of magnitude value of the transition zone width as given before. However, the large area of larger values for the dissipation than for the production behind the breakwater confuses matters. The reason for this large area is a result of equation 9.4 but it is not certain that it is completely correct. As mentioned before, model tests and extensive measurements will be needed to verify the outcome.

For now the conclusion seems justified that for the relatively larger (peak) values of the production and the dissipation the schematization as used in UNIBEST-TC might be correct. For the smaller values occurring just behind the breakwater it is not clear whether these are also correctly modelled.

One has to bear in mind that the program was primarily built to cope with profiles with gradually changing topography and not with abrupt changes as in this situation due to the presence of the breakwater. In any case, results of computations using the UNIBEST-TC program with an implemented submerged breakwater will have to be examined critically.

9.3: Coast enlargement of 1000 m

9.3.1: Nourishment behind a breakwater with crest at NAP -2 m

The starting point consists of an enlargement of the beach width by 1000 m. The same slope as the reference slope was used for the beach fill behind the breakwater to see in what way the profile would develop. Furthermore it was decided to start the beach fill right at the shoreward edge of the breakwater to get an idea of how fast a scour hole would develop.

In Chapter 6 (Section 6.5) two approaches were outlined which could be used to determine how a profile will develop if a conventional submerged breakwater and a nourishment behind it are present. The first approach consisted of using the whole profile and computing the change of the waves as they pass the breakwater with UNIBEST-TC. The peak period has to be scaled down however (Section 5.3). This will also influence the seaward part of the profile next to the breakwater because in this way the whole profile will be affected by the change of the period. The second approach consisted of first using the UNIBEST-TC program to obtain the wave heights at the seaward toe of the breakwater. Using a transmission formula (such as the one from Daemen (1991) for instance), the transmitted wave heights at the shoreward side of the breakwater can be computed.
Using these wave heights and the scaled down periods as input, the UNIBEST-TC program can be used again to see in what way the profile of the nourishment behind the breakwater develops.

Due to the fact that the UNIBEST-TC program has to start at a deep water level and needs a certain horizontal distance to get all the schematizations in order, the second approach didn’t work. Instabilities were the result, caused by either mathematical errors or by the appearance of unrealistically large sandbars at the seaward edge. So it was decided not to go on with this second approach, but to use the first one, as outlined above. This means that the whole profile will be taken into account with the breakwater present.

The values of the used wave periods were scaled down according to the method outlined in Section 9.2.2. Since the change of the wave height resulting from the presence of the breakwater is computed rather well when using the UNIBEST-TC program, no changes were made to the used wave heights. As a result the effects of the change of the wave heights might be slightly underestimated due to the earlier mentioned fact that in the program only impermeable breakwaters (Chapter 5) can be implemented.

The breakwater which is used has been outlined in Chapter 8 as Breakwater A. The dimensions and depth values are stated in Section 8.2.2. The bottom profile is the reference, 20 years profile as computed in Chapter 4. At the seaward side of the toe of the breakwater use is also made of this profile.

Due to numerical stability problems the optimum values of the time- and spatial step size were set at 0.1 days and 2 m for the breakwater region. Just in front of and behind the breakwater use was made of a spatial grid of 5 m and further away of 25 m. Due to the fact that the maximum number of grid points is limited to 149 (UNIBEST-TC user's manual [1992]) and a total horizontal distance of 15 km has to be covered, some very large steps were used (10 times a dx of 1000 m) at the seaward side of the investigated 15 km area. This spatial grid gave satisfactory, accurate enough results. Unfortunately the computing time increased rapidly due to the lowering of the time step from 0.5 to 0.1 days. Since the maximum executable number of time steps is equal to 9999 and a simulated year consists of 278.5 days (Chapter 4), the maximum number of complete years per run is equal to 3. So instead of a run time of 10 years as used to investigate the results of the two unprotected nourishments in Section 7.5, use is now made of a run time of 3 years. Since the life span of a beach nourishment project usually lies between 5 and 10 years (Chapter 7), an additional runtime of 3 years (4-6 years total) was investigated in order to see if maintenance is necessary after approximately 6 years and if that is the case, how many cubic meters per alongshore meter are needed.

The result of the first run is shown in Figure 9.16. It is immediately clear that a large amount of material is transported from the shoreward side of the breakwater to the seaward side where it accumulates and forms a large heap of sand. As a result of this the shoreline retreats over a distance of 89 m. Since the material can not be transported through the breakwater because it is implemented as completely impermeable (this has been verified with test runs), the transportation of the sand only takes place over the breakwater. The way in which the volume behind the breakwater changes in time is shown in Figure 9.17.
Figure 9.16 Profile development of the nourishment behind Breakwater A for the first three years

Figure 9.17 Volume development in the first 3 years behind Breakwater A

In the first year a relative large amount of sand is transported over the breakwater. In the following years the mean amount of lost sand per year (as defined by the volume of sand transported from the shoreward to the seaward side of the breakwater) decreases but doesn't approach the value zero. The latter would mean that an equilibrium profile is formed. The development of the mean value of the volume behind the breakwater is printed in Table 9.1.

The values for year 1 till 4 were obtained from the results of the first run. The profile resulting from this run was then used as the starting profile for the following run. However, the heap of sand in front of the breakwater was removed so that at the
beginning of the fourth year more energy is able to pass over the breakwater and hence causing a larger return flow. Thus a larger erosion is the result since the waves do not break in front of the breakwater on that heap of sand as they did at the end of the third year of the first run. That explains why the $\Delta V$ values as stated in Table 9.1 suddenly increase ($88 - 97$) instead of continuously decrease. Taking this into account it is clear that the amount of lost sand decreases as time continues but still consists of a large volume at the end of the sixth year ($69 \, m^3/m$).

This seems rather strange. It is expected that a certain, more or less stable profile will develop behind the breakwater. This ongoing large transport might be the result of the way in which the transition zone width is modelled in the UNIBEST-TC program. If the transition zone width is given a too large value, a too large scour influence will be the result (Section 9.2.4). So the amount of lost sand is probably exaggerated.

The reduced volumes as given in Table 9.1 are the values as shown in Figure 9.17 but subtracted by the volume which is occupied by the reference profile above the horizontal reference level and by the volume which is taken up by the part of the breakwater. In other words, the reduced values consist of the amount of sand needed to form a beach fill behind the submerged breakwater with characteristics as shown (for two time points) in Figure 9.16.

The development of the profile can be explained as follows. Due to the transition zone width as outlined in Section 9.2.4, the resulting cross-shore return flow just behind the breakwater causes a strong erosion in the beginning. As time continues the depth behind the breakwater becomes larger and the transportation of sand over the breakwater becomes less. This is a result of the fact that a larger vertical as well as horizontal distance has to be crossed. Due to the changing of the wave height and period (the results of which were discussed in Sections 9.2.1, 9.2.2 and 9.2.3), the cross-shore transport just behind the breakwater and on the upper part of the profile is no longer onshore but has changed into offshore transport. So the deeper part just behind the breakwater is filled.

### Table 9.1 Differences in the volume behind Breakwater A

<table>
<thead>
<tr>
<th>Year</th>
<th>Volume at the beginning of the year [m$^3$/m]</th>
<th>Reduced volume [m$^3$/m]</th>
<th>Difference in reduced volume ($\Delta V$) [m$^3$/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15670</td>
<td>8286</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>15431</td>
<td>8047</td>
<td>239</td>
</tr>
<tr>
<td>3</td>
<td>15317</td>
<td>7933</td>
<td>114</td>
</tr>
<tr>
<td>4</td>
<td>15229</td>
<td>7845</td>
<td>88</td>
</tr>
<tr>
<td>5</td>
<td>15132</td>
<td>7748</td>
<td>97</td>
</tr>
<tr>
<td>6</td>
<td>15052</td>
<td>7668</td>
<td>80</td>
</tr>
<tr>
<td>7</td>
<td>14983</td>
<td>7599</td>
<td>69</td>
</tr>
</tbody>
</table>
with sand having its origin more shoreward and since this sand is then transported over
the breakwater the shoreline will retreat.

Regarding Figure 9.17 certain regions can be distinguished during which the volume
hardly changes (nearly horizontal parts of the graph) as well as periods where a rapid
change in volume appears. The first period consists of wave action resulting from small
waves which will cause offshore transport from the shoreline to the breakwater, but will
not cause significant transport over the breakwater. The latter is caused by the higher
waves. As the depth just behind the breakwater becomes larger, higher waves which
possess a larger amount of energy are needed to obtain the same transport over the
breakwater as when the depth was smaller. Again, that is the main reason that the mean
values as given in Table 9.1 decrease.

Another factor in this decreasing process is the fact that from a certain time onwards the
larger waves will also break on the heap of sand in front of the breakwater. As a result
less energy will be left to cause a return flow behind the breakwater, although due to the
time lag that effect seems to be rather small.

The beach fill volume never increases. This means that even at high water levels the
waves are influenced by the breakwater in such a way that the onshore transport resulting
from asymmetry (Section 9.2.2) is never larger than the offshore transport resulting from
the wave induced undertow. Furthermore the amount of shoreward transported sand
would have to cross over the breakwater. This would mean that the flow causing the
transport must be strong enough to transport the sand from just above the bottom level in
front of the breakwater (NAP - 9.15 m) to the crest height at NAP - 2 m.

Since the transport of sand over the breakwater keeps on going, although at a smaller rate
as time continues, the equilibrium profile for the given changed wave climate is not
reached. Regarding Figure 9.16 three slope types can be distinguished behind the
submerged breakwater. The first is the steep slope around the shoreline. The second is the
less steep slope for approximately 13800 m < X < 13900 m, which transforms into the
third steeper slope just behind the breakwater. This second slope is slightly less steep than
the original slope (displayed as the initial profile). Due to the fact that the seaward
transported sand from the second region is still being transported over the breakwater,
which causes the steeper third part of the slope, the equilibrium profile from shoreline to
breakwater cannot develop and the shoreline will continue to retreat. This is shown in
Figure 9.18.

In the second run the shoreline retreats over a distance of 48 m. This leads to a total
retreat of 137 m compared to the initial profile in six years. As can be seen in Figure
9.18 the depth just behind the breakwater gets larger although it is clear that in the last
three years the depth as well as the volume decreases less than in the first three years as
displayed in Figure 9.16. The slope does not change, though.

However, as mentioned in Section 9.2.4, it is not clear whether or not the wave induced
return flow resulting from the dissipation is modelled correctly at the right X values since
the area in which the dissipation (D) is greater than the production (P) might be too large.
Whether or not the transportation of the sand over the breakwater is also modelled
correctly, especially whether or not the bottom friction forces resulting from the "fixed
bottom profile" being the breakwater, acting on the return flow, are correct also remains questionable. It appears that the flow over the breakwater seems to be correctly modelled (Roelvink, personal communication), as far as the influence of the slopes of the breakwater are concerned. The kind of friction developed by the large rocks which form the primary armour layer, will however be greater than the friction forces resulting from a smooth concrete bottom. So probably the friction is modelled at a too low value. The fact that the primary armour rocks are rather large (Chapter 8), will result in the sand being able to move through the breakwater in reality. This is not the case in the UNIBEST-TC program since the implemented breakwater is impermeable. This has to be kept in mind when looking at the results of computations.

So the question whether or not the transition zone width is modelled correctly remains. If the area, in which the dissipation is larger than the production, is indeed too large, then the correct option would be a smaller area and thus a higher, more concentrated value of the dissipation, D. Depending on the amount of concentration (i.e. the "peakedness" of the distribution of D) there might be a moment when the depth behind the breakwater reaches a certain value at which the X coordinate is greater than the specific X coordinate at which the dissipation is not sufficiently greater than the small values of the production. The deepening of the beach fill just behind the breakwater at that point will stop for certain wave characteristics. The resulting cross-shore current will not be strong enough to transport the sand over the breakwater in that case. The offshore transport occurring at the higher part of the beach fill might still cause erosion if the equilibrium profile is not yet reached. That amount of eroded sand will first be transported towards the breakwater and then be transported over the breakwater since it causes the depth to decrease. However, due to the fact that a more or less maximum depth for the area just behind the breakwater exists, an equilibrium profile might develop in this case so that a more stable situation can be found.

On the one hand it seems that the results from the runs using UNIBEST-TC are more or
less correct, especially considering the mathematical approximations used in the program which appear to give sufficient accurate results. On the other hand the possibility exists that the spatial distribution of the return flow just behind the breakwater is overpredicted and as such causes a transport over the breakwater which goes on too long, i.e. the depth just behind the breakwater might be too large. This is again a direct result of the modelling of the transition zone width as outlined in detail in Section 9.2.4. The same recommendation as was made in that Section holds true in this one; model tests and extensive measurements are necessary to achieve more insight in this phenomenon at the back slopes of breakwaters. As mentioned earlier, the modelling of the friction forces resulting from the primary armour as well as the fact that the breakwater is implemented as completely impermeable, cause some doubts too.

In Section 9.4 another way of predicting the retreat of the shoreline is used. This is a more qualitative approach in which the results of the Sections 9.2.3 as well as 9.3.1 and 9.3.2 are used. It is not possible to use the program UNIBEST-TC in such a way that the time lag is not taken into account or is minimized without also altering other important characteristics.

One might propose the idea of using a breakwater in the computations with a much larger crest width than the prototype one (this idea was indeed carried out), but this also results in a much larger decrease of wave heights so that the cross-shore transport behind the breakwater is also influenced in a large way.

9.3.2: Nourishment behind a breakwater with crest at NAP -4 m

The development of the beach fill behind the used breakwater is shown in Figure 9.19.

![Figure 9.19 Profile development of the nourishment behind Breakwater B for the first three years](image)

The breakwater used is the Breakwater B as outlined in Section 8.2.2. It placed at a depth of NAP -10 m. The slopes are constructed as 1:2. The main reasons for the development
of this profile are the same ones as outlined in the previous paragraph. Due to the fact that the crest of the breakwater is located further below SWL than in the previous case of Breakwater A, the difference in wave heights and in periods are also smaller. As a result the original onshore transport on the upper part of the profile for the situation without the breakwater present, is still changed into an offshore transport, but the change is less. This means that the mean offshore transport has a lower absolute value in this case than in the case of the Breakwater A. The change in volume behind the breakwater is displayed in Figure 9.20.

![Figure 9.20 Volumetric changes of the area behind Breakwater B](image)

Three differences between Figure 9.20 and Figure 9.17 become apparent. The first is that the decrease is less smooth in Figure 9.20 and more abrupt changes take place. The fact that less energy is dissipated on top of the breakwater (and thus due to the time lag becoming effective behind the breakwater) as a result of the lower crest height, not only causes less transport in general (difference 2) but also causes these abrupt changes. Higher waves are needed to provide enough energy dissipation to develop a sufficient strong return flow for transportation over the breakwater. As mentioned in Section 9.2.1 the higher waves are less influenced by the lower breakwater. That is also why the periods where zero (or nearly zero) transport takes place, are longer. This can easily be seen in Figure 9.20 since these periods are characterized by the (nearly) horizontal parts. The third difference is that onshore transport over the breakwater does take place now, although on a very small scale. This can be seen in Figure 9.20, if one looks very carefully at for instance \( t = 100 \) or \( t = 720 \) days. This results from the fact that the crest height is now located significantly lower and that the period is influenced less which causes onshore transport due to the asymmetry effect of the waves. The change in volume is stated in Table 9.2.

The first four volume values result from the first run. For the initial profile of the second run use was made of the end profile of the first run. The heap of sand in front of the breakwater was in this case not removed. The reason for this is to see if the sudden increase in \( \Delta V \) values as featured in Table 9.1 would not occur in this case.
<table>
<thead>
<tr>
<th>Year</th>
<th>Volume at the beginning of year [m³/m]</th>
<th>Reduced volume [m³/m]</th>
<th>Difference in reduced volume (ΔV) [m³/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20306</td>
<td>9640</td>
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<td>99</td>
</tr>
<tr>
<td>4</td>
<td>19933</td>
<td>9267</td>
<td>76</td>
</tr>
<tr>
<td>5</td>
<td>19871</td>
<td>9205</td>
<td>62</td>
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<td>6</td>
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</tr>
<tr>
<td>7</td>
<td>19766</td>
<td>9100</td>
<td>50</td>
</tr>
</tbody>
</table>

Table 9.2 Differences in the volume behind Breakwater B

reason why the ΔV values as stated in Table 9.2 decrease in an orderly manner. Again a certain zero level is not reached but the amount of lost sand at the end of the sixth year is less than in Section 9.3.1, although it is still rather large, namely 50 m³/m. The retreat of the shoreline amounts to 43 m after 3 years and to a total of 78 m after 6 years compared to the initial profile.

In Section 9.4 the other, qualitative prediction approach is also used for this breakwater. In Section 9.5 the different results will be grouped together, compared, and conclusions will be drawn on that comparison.
9.4: Qualitative approach and comparison of profile shapes

9.4.1: Introduction

In this Section a more qualitative approach is used to determine the extent of the possible effects of a submerged breakwater on the nourishment behind it. This qualitative approach is used because doubts exist about the results of computations with the UNIBEST-TC program. It seems that the erosion behind the breakwater goes on too long in these results. Furthermore the primary armour layer will be permeable in reality, whereas it is modelled as totally impermeable in the program.

For both Breakwater A (the breakwater with the crest located at NAP -2 m) and Breakwater B (the breakwater with the crest located at NAP -4 m), two possible profile shapes have been derived in the previous sections. In Section 9.2.3 slopes were derived for both breakwaters which resulted from the combined effects of the change in wave height and in wave period. This profile shape is called Profile 1 for Breakwater A and Profile 2 for Breakwater B. These would be the resulting profile shapes of the nourishments behind the submerged breakwaters if no scour effects were present.

However, scour effects are present due to the influence of the transition zone width. If these effects are taken into account then the resulting profile shapes for the nourishments behind the submerged breakwaters are given in Sections 9.3.1 and 9.3.2. For Breakwater A the resulting profile shape is called Profile 3 and for Breakwater B it is called Profile 4. So for both breakwaters, two profile shapes are possible, one not taking scour effects into account (Profiles 1 and 2 as featured in Section 9.2.3) and one that does take scour effects into account (Profiles 3 and 4 as featured in Sections 9.3.1 and 9.3.2).

One of the aims of this study is to be able to predict the amount of shoreline retreat resulting from the position of the attachment point of the nourishment to the breakwater. This was previously outlined in Chapter 1. Reference is made to Figures 1.4 and 1.5. Figure 1.5 is reprinted here as Figure 9.21.

Figure 9.21 features a submerged breakwater with a nourishment behind it. The attachment point of the nourishment to the breakwater is indicated by A. This attachment point will probably not be located at the crest of the breakwater due to scour influences and the influence of the permeability of the top layer of the breakwater. If however the attachment point is located at a greater depth, the shoreline will retreat compared to the original situation. This is shown in Figure 9.21. If point A is moved to a greater depth over a distance Y, the shoreline will retreat over a distance equal to R. That results in a smaller beach enlargement, namely the original enlargement minus the value of R. So a relation between the parameter R and the parameter Y has to be derived. That is done in Section 9.4.2 for Breakwater A and in section 9.4.3 for Breakwater B.

The distance R also depends on the shape of the profile of the nourishment behind the breakwater. The distance R is taken relative to the shoreline position of the reference, 20 years profile, if that would be used as the shape of the nourishment behind the breakwater with the attachment point (A) located right at the crest of the breakwater. This is outlined in Figure 9.22.
Slope AB is equal to the 20 years reference slope CD. However, it appears that the resulting shape of the nourishment behind the breakwater is steeper than slope AB. Thus the nourishment profile shape will be more like shape EB. If the nourishment is attached right at the crest of the breakwater (Z = -4 m), the shoreline does not retreat but advances. In this case the parameter R (R is equal to the distance EA) has a positive value. If however the attachment point is located at a greater depth, for instance at B', the shoreline will retreat over a distance equal to R'. In the latter case R has a negative value (in this case R is equal to AE').

Figures showing this relative shoreline retreat (R) as a function of the depth just behind
the breakwater (Z) are derived. In order to be able to do that, the different profile shapes have been approximated by polynomial functions. The exact equations of the approximations are printed in Appendix E.

9.4.2: Nourishment behind the breakwater with crest at NAP -2 m

The shape of the nourishment (taking the scour effects into account) behind Breakwater A as computed with UNIBEST-TC in Section 9.3.1, is basically the same whether taking the profile into account after 3 years or after 6 years. The chosen profile to be approximated (Profile 3) is the profile at the end of the third year. Since a large scour hole just behind the breakwater is present a fourth degree polynomial function is necessary to include that effect to a satisfactory degree. This is shown in Figure 9.23. Figure 9.23 also features the approximation of Profile 1 as derived in Section 9.2.3, i.e. the profile where scour effects are not taken into account.

As can be seen in Figure 9.23 the upper part of the profile coincides well with Profile 1. This is expected since the change of the wave characteristics due to the presence of the breakwater were implemented in the computations which resulted in Profile 1. However, seaward from the point \( X = 115 \) m till the breakwater, a large deviation occurs. This deviation has to be the direct effect of the presence of the breakwater. It is caused by the difference in production and dissipation of wave energy as discussed in Section 9.2.4. One might conclude that the effect of the scour behind the breakwater extends over a shoreward distance of about 135 m. The greatest difference between the original Profile 3 values and the Profile 1 approximation is located just behind the breakwater and amounts to 0.85 m.

To compare the different profile shapes Figure 9.24 is made. Both the initial (10 years profile) as well as the more or less equilibrium, 20 years reference profile are featured next to the original and approximated Profile 3 values.
For the breakwater with crest at NAP-2 m

Figure 9.24 Comparison of different profile forms, for the case of Breakwater A. Profile 3 is represented by "original value" and its approximation by "approximation".

There is practically no difference between the initial and the reference, 20 years profile form. This was previously shown in Figure 4.7 in Chapter 4. So indeed this 20 years profile can be regarded as a more or less equilibrium profile as far as the shape is concerned. As mentioned in Section 9.2.3, the Profile 1 approximation is steeper for this shallow part of the profile.

If Profile 1 is assumed to be correct and the scour hole is momentarily not taken into account, a graph displaying the shore line retreat (R1) as a function of the depth at which the nourishment is attached to the breakwater (Z) can be made. This shoreline retreat is taken relative to point A as featured in Figure 9.24, since this point indicates the position of the shoreline of the reference profile if directly attached to the crest of the breakwater. The followed procedure as well as the exact definition of R was given in Section 9.4.1. Reference is made to Figure 9.22. The shoreline retreat R1 is featured in Figure 9.25. Note that the effect of scour is not included in this figure.

The advancement of the shoreline if the nourishment is attached directly to the crest (Z = -2 m) is clear. One has to bear in mind that if a certain depth behind the breakwater is chosen, a certain value has to be added to the value of Z (if scour effects are to be taken into account), since a scour hole will be present. In other words, if one is interested in the retreat of the shoreline resulting from a depth of for instance 4 m just behind the breakwater, the Z value to be chosen will have to be -3 m if a scour hole depth of 1 m is expected.

The same kind of graph is made for the approximation of Profile 3 as computed with UNIBEST-TC. This approximation is done by a fourth degree polynomial function. If this approximation is used to predict the retreat of the shoreline (R2) then an extra scour hole with a width of 36 m and a largest depth of 0.45 m has to be taken into account. This scour hole was featured in Figure 9.23 as the difference between the approximation of Profile 3 and the original values. Figure 9.26 shows the retreat of the shoreline as...
Figure 9.25 Graph displaying shoreline retreat (R1) resulting from Profile 1 relative to point A in Figure 9.24, as a function of the depth just behind the breakwater (Z)

Figure 9.26 Graph displaying shoreline retreat (R2) resulting from Profile 3 relative to point A in Figure 9.24, as a function of the depth just behind the breakwater (Z)

computed with this approximation but without taking the extra scour hole into account.

For the value $Z = -2$ m an advancement of the shoreline is again present, although less than in Figure 9.25. This advancement is due to the relative steep part of the Profile 3 between the NAP line and a depth of 1 m. The earlier mentioned 0.45 m (the depth of the scour hole as featured in Figure 9.23) has to be added to the $Z$ values in Figure 9.26 to obtain the retreat of the shoreline.

The primary armour of the breakwater consists of large rocks with a mean diameter of
about 1.20 m (Chapter 8). This means that approximately the top 2.5 m of the breakwater will be permeable to some extent. So probably a scour hole will develop behind the breakwater till a depth of 4.5 m. Whether or not the scour will result in a greater depth is not precisely known. It is assumed that the upper limit of this development is formed by a depth of 5.5 m. At this depth the resulting forces of the waves on the sediment will probably not be strong enough to transport the sand over a vertical distance of 1 m or more.

From Figure 9.26 it appears that under these conditions $100 \leq R2 \leq 150$ if the extra scour hole depth of 0.45 m is taken into account. From here onwards the values of $R1$ will be stated as positive in the case of a retreat (in the figures these values are stated as negative). If the Profile 1 approximation is to be used, i.e. Figure 9.25, a certain value for a modified scour hole has to be determined. If the influence of the scour is expected to be locally concentrated, a deep hole with a comparatively small width will be the result. A thus derived scour hole depth of about 1.5 m results in $55 \leq R1 \leq 190$ m. Since the approximation of Profile 1 for the lower part of the profile is less steep than the approximation of Profile 3 (see Figure 9.23), these values for $R1$ are further apart than the values for $R2$. The distance of 55 m for $R1$ seems to be rather small. This is due to the assumption of a deep scour hole. To be on the safe side, it will not be used as minimum value of the shoreline retreat. For this minimum value the value of 100 m of $R2$ will be used.

The conclusion can be drawn that if the nourishment is attached directly to the crest of the breakwater at the beginning, the shoreline will retreat over a distance of at least 100 m compared to the reference profile (also attached to the crest) before a more or less stable profile is reached. This results from the modification of the wave characteristics due to the presence of this breakwater and from the permeability of the primary armour layer and from scour effects. Depending on the influence and the shape of the scour hole just behind the breakwater, the upper limit can be set at 200 m.

9.4.3: Nourishment behind the breakwater with crest at NAP -4 m

The same procedure as outlined in the previous paragraph is followed in this one. Figure 9.27 shows the profile taking scour effects into account (Profile 4) as computed with UNIBEST-TC in Section 9.3.2 behind the breakwater as well as its fourth degree polynomial approximation. The Profile 2 approximation is also shown in this Figure 9.27. Profile 2 is the profile as derived in Section 9.2.3.

The absolute deviations between the approximations just behind the breakwater are in the same order as in the case of Breakwater A. The upper part of the profile resembles the Profile 2 approximation quite well. The influence of the scour hole as featured by Profile 4 is even bigger. It stretches from the breakwater till about 250 m shoreward. The slope in this influenced region is however less steep than in the previous case.

Figure 9.28 shows the difference in profile shapes. The upper most part of the Profile 2 approximation is still a bit steeper than the reference, 20 years profile. If Profile 2 is assumed to be correct than the retreat of the shoreline ($R3$) as a function of $Z$ can again be computed and this is displayed in Figure 9.29. A certain depth has to be taken into account again for the formation of a scour hole just behind the breakwater.
The amount of shoreline retreat as resulting from the approximation of Profile 4 (R4) is shown in Figure 9.30. In this case no extra scour hole has to be taken into account since the approximated depth value just behind the breakwater does not deviate significantly from the profile depth resulting from the computations with UNIBEST-TC. This was earlier shown in Figure 9.27. The original values are nearly the same as the approximated values of Profile 4.

Taking the permeability of the top layer of the breakwater into account, the scour hole might develop till a depth of about 6 m is reached. This is the lower boundary of the primary armour layer. According to computations carried out with the UNIBEST-TC program, the depth just behind the breakwater does not reach a higher value than 5 m.
The following values for the shoreline retreat if compared to the reference profile are derived for this case: 147 ≤ R4 ≤ 172 m and 150 ≤ R3 ≤ 230 m. These larger values for the shoreline retreat are logical since the slope of the lower part of the profile behind
this Breakwater B is less steep than the one behind Breakwater A. For the determination of the R3 values a scour hole depth of 1 m is assumed. This assumed depth value is smaller than the depth of the scour hole just behind Breakwater A (which was determined as 1.5 m) since it is located at a greater depth.

In this case the conclusion is that if the nourishment is attached directly to the crest of the breakwater, the shoreline will retreat over a minimal distance of 150 and a maximum of 230 m (compared to the reference profile slope attached directly to the crest) before a stable situation is reached.

9.5: Comparison of results

The following conclusions result from the previous sections.

Due to the changing of the wave heights and of the wave period resulting from the presence of the submerged breakwater, the slope behind it will differ from the reference, more or less equilibrium slope. Two breakwaters are used, one with the crest located at NAP -2 m (Breakwater A), and one with the crest located at NAP -4 m (Breakwater B). In the area of most interest the profile becomes steeper. This area of most interest is the area between the NAP line and a depth of NAP -2 m for Breakwater A and a depth of NAP -4 m for Breakwater B. This results in a steeper profile for the beach fill behind both breakwaters than the reference profile. The slope behind Breakwater A is steeper than the slope behind the other breakwater, Breakwater B.

The overall results of the computations carried out with the UNIBEST-TC program are quite satisfactory. Doubts remain about the way the so called transition zone width is modelled (Section 9.2.4) and thus about the way the scour hole develops just behind the breakwater. It seems that the UNIBEST-TC program overpredicts this effect slightly. Model tests as well as extensive measurements will be necessary to be able to verify the way in which the profile develops. The fact that the breakwater is implemented in the program as impermeable also causes possible deviations. A third, minor influence which is not modelled correctly, is the friction resulting from the large primary armour rocks.

Due to the fact that some doubts exist about the results of computations with the UNIBEST-TC program, a more qualitative approach, based upon the slopes of the various profiles, is used in Section 9.4. Both the profiles as computed with the breakwater present as well as the ones computed without the breakwater present, but with using the changed wave characteristics resulting from the presence of the breakwater, are taken into account. This results in graphs displaying the amount of shoreline retreat, R, as a function of the depth just behind the breakwater (Z).

By using these graphs (and thus the underlying polynomial functions), the amount of shoreline retreat (R) until a more or less stable situation is reached, is computed. This amounts to the following approximate range, depending on the amount of scour just behind the breakwater, for the two protected nourishments:

- nourishment behind Breakwater A: 100 \leq R \leq 200 \text{ [m]}
- nourishment behind Breakwater B: 150 \leq R \leq 230 \text{ [m]}

A rough cost comparison is made in Chapter 10 using these results.
CHAPTER 10: COST COMPARISON

In this Chapter a rough cost comparison between an unprotected nourishment and a protected one is carried out. Both result in a more or less stable enlargement of the coast of 1000 m.

In Chapter 7 the approximate costs for a stable unprotected nourishment were derived. Reference is made to Section 7.5 in which the total costs were stated as $97000.- per alongshore meter enlargement.

For the protected nourishments two breakwaters were used, one with the crest located at NAP -2 m (Breakwater A) and one with the crest located at NAP -4 m (Breakwater B). Both breakwaters are discussed in Chapter 8. In Section 8.3 the costs for these breakwaters were derived. A rough estimation of the total costs per alongshore meter breakwater amounts to:

- for the breakwater with crest at NAP -2 m.: $14500.-
- for the breakwater with crest at NAP -4 m.: $10500.-

The part which is not included in these total overall costs, is the amount of money needed to make the breakwater sand-tight (except for the primary armour layer) as well as the costs for a filter construction. This has to be kept in mind when using these two cost estimates in the comparison.

In Section 9.4 the amount of shoreline retreat (relative to the reference profile), before a more or less stable situation behind the breakwater is reached, is computed. This is done for both protected nourishments. It amounts to the following approximate ranges:

- nourishment behind Breakwater A: $100 \leq R \leq 200$ [m]
- nourishment behind Breakwater B: $150 \leq R \leq 230$ [m]

The cost comparison is carried out in the following way. First of all the two breakwater-connected nourishments were moved seaward over the above stated amount of computed shoreline retreat, R. In this way all three coast enlargement alternatives result in a more or less stable beach widening of 1000 m.

By moving the two breakwaters more seaward, they will be more severely attacked by the now slightly higher waves. However, the chosen primary armour grading stays the same for both breakwaters. The needed volumes of sand for the nourishments behind the breakwaters were derived, and costs compared. This results in Table 10.1. Since the costs needed in order to make the breakwater sand-tight are not known, a question mark is present in that column.

When regarding the total costs, as featured in the last column of Table 10.1, two important conclusions can be drawn. The first is that the total cost of the nourishment coupled to a submerged breakwater is more or less the same for both breakwaters. This is especially true if the following remark as made in Chapter 8 is taken into account. Namely that the costs for the NAP -4 m breakwater are probably estimated too high relative to the cost as derived for the NAP -2 m breakwater. The second conclusion is
<table>
<thead>
<tr>
<th>Unprotected nourishment</th>
<th>Sand needed [m(^3)/m]</th>
<th>Cost of sand [f/m]</th>
<th>Cost of breakwater [f/m]</th>
<th>Cost of sand-tight layer [f/m]</th>
<th>Total costs [f/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>19400</td>
<td>97000</td>
<td>-</td>
<td>-</td>
<td>97000</td>
</tr>
<tr>
<td>Breakwater with crest at NAP-2m</td>
<td>8870</td>
<td>44350</td>
<td>14500</td>
<td>?</td>
<td>58850 +/-</td>
</tr>
<tr>
<td></td>
<td>9500</td>
<td>47500</td>
<td>14500</td>
<td>?</td>
<td>62000 +/-</td>
</tr>
<tr>
<td>Breakwater with crest at NAP-4m</td>
<td>10500</td>
<td>52500</td>
<td>10500</td>
<td>?</td>
<td>63000 +/-</td>
</tr>
<tr>
<td></td>
<td>10950</td>
<td>54750</td>
<td>10500</td>
<td>?</td>
<td>65250 +/-</td>
</tr>
</tbody>
</table>

Table 10.1 Costs of the different alternatives. The total costs (featured in the last column) are stated in Dutch Guilders per alongshore meter coast enlargement.

that the protected nourishments are economically a better option than the unprotected nourishment. The sum needed to make the breakwater sand-tight and the costs for a filter layer, will probably not amount to more than half of the total costs for the breakwater. This is not sure however. All in all it appears that a nourishment behind a submerged breakwater is economically a better option than an unprotected nourishment.
CHAPTER 11: LONGSHORE EFFECTS

In this Chapter a short remark on the effects of the breakwater on the longshore transport processes is presented. The submerged breakwater will not only influence the cross-shore transport, but also the longshore transport. Reference is made to the CERC-formula as stated in Chapter 4 as formula 4.1. It basically states that the longshore transport is a function of the incident wave angles, the wave heights and wave velocity.

The situation considered is as follows. A straight coastline with a submerged breakwater and depth contours parallel to it. Waves are approaching the coast under a more or less constant angle. There are no changes in this situation along the coast. This is displayed in Figure 11.1.

![Figure 11.1 Top view of a schematized part of a constant shoreline](image)

In this case no longshore transport gradient will be present due to the fact that the determining factors in all cross-sectional areas are equal. In reality the submerged breakwater will have a starting and a finishing point. If these points are not combined with a breakwater located perpendicularly at the coast, such as the ones located at IJmuiden or Scheveningen for instance, gradients will occur at these points. In Figure 11.2 two areas are indicated, area A and area B.

S1 represents the alongshore transport without the influence of the breakwater. S2 is equal to the alongshore transport influenced by the breakwater. S2 will be smaller than S1 because less energy is present behind the breakwater due to the breaking of the waves on this breakwater. When considering area A, it appears that the amount of sediment entering the area is greater than the amount leaving the area. This results from the fact that just after the nourishment has been completed, the following situation exists. In area A part of the nourishment is located perpendicularly to the original coastline over a distance of approximately 1000 m. This has the effect of blocking most of the alongshore transport. Thus a large accretion will occur in that area. The shoreline of this accretion will align itself with the wave crest pattern. Just behind point P erosion will occur. This results from the fact that although S2 is smaller than S1, the amount of sediment entering the area at point P is very small due to the just mentioned accretion in area A.

In the other area, area B, accretion and erosion occurs. Around point Q the accretion will
Figure 11.2 Top view of a coast with a submerged breakwater in front. Due to the influence of the breakwater, area A will accrete and area B will erode.

occur and around point R a large erosion will take place. This erosion results from the fact that the waves will be higher again once the area which is influenced by the breakwater is left, and due to the large distance between the coastline of the nourishment and the parallel original coastline. The resulting form of the coastline in area B has not been drawn in Figure 11.2, because a more in-depth study is needed for that. For now the most important conclusion is that a certain amount of sand will be deposited around point Q and that a large erosion will take place around R. Some of the sand deposited at Q might be transported to the eroding area by the cross-shore transport.

If the area at the end of the breakwater is only allowed to erode to a certain extent, measures will have to be taken. Examples are for instance a nourishment or a process called sand by-passing. In this case the accumulating sand will be transported to the eroding area. This will only work if the sand is not transported in the cross-shore direction over a too large distance. Structures such as groins, seawalls or jetties are not recommended since these will only move the problem further away from the initially eroding area. As such they do not solve the problem, as long as no changes occur in the shoreline or wave climate. The treatment of this problem is not within the range of this paper, however. The purpose of this Section is only to indicate that problems may occur as a result of the influence of the submerged breakwater on the longshore current.

One aspect still has to be mentioned. If a very severe storm with high water levels occurs, sand will be transported from the shoreline to deeper parts of the profile. Depending on the storm surge level and the occurring wave heights, sand might be transported all the way over the submerged breakwater. One can imagine that this will happen sooner in the case of Breakwater A than in the case of Breakwater B since Breakwater A is located nearer to the coast. If this transport over the breakwater takes place, sand is lost from the cross-shore profile part behind the breakwater. This sand will not be transported shoreward back over the breakwater again. It will have to be replaced artificially. This is something worth of checking out in more detail if an extensive feasibility study on one of the protected nourishments is carried out.
CHAPTER 12: CONCLUSIONS AND RECOMMENDATIONS

The previous Chapters lead to the following conclusions and recommendations.

- Referring to wave height transmission formulae:
  For conventional breakwaters the formulae according to Daemen (1991) are the most precise ones, since the most parameters are taken into account. Caution is advised when using transmission coefficient values near to the maximum and minimum stated values.

  For reef breakwaters the formulae from Hearn (1987) are advised. The formulae from Daemen (1991) give a nearly constant, very low maximum value of 0.6 for submerged breakwaters. This does not appear to agree with reality. Further investigation on the effect of the crest width of the submerged breakwater on the transmission is needed.

- In the case of a conventional submerged breakwater the transmission coefficient values as computed with UNIBEST-TC, agree quite well with the theoretical, calibrated ones as given by the formulae of Daemen. The possible concentrated wave energy losses in the case of plunging breakers are modelled to a satisfactory degree. All waves as used in this study will be of the type of a collapsing breaker on the breakwater, due to the steep front face of the breakwater. When a reef breakwater is considered, the agreement is less good, especially if the value of the freeboard amounts to a positive or zero value. It is advised to use the formulae from Hearn and not to rely on results of the UNIBEST-TC program in the case of a reef breakwater.

- In both the above stated cases, the fact that the breakwater as implemented in the UNIBEST-TC program is completely impermeable, plays an important role. That is the reason why in the case of a reef breakwater the agreement is less good. A reef breakwater has a larger degree of permeability than a conventional one. If it is possible to implement a third bottom option in the UNIBEST-TC program, besides the complete impermeable one and the sandy bottom one, this would improve results. This is something worth of checking out in the future.

From here on the breakwaters as mentioned in the conclusions are of the statically stable conventional submerged type. This is the type used in this study to support a nourishment behind it.

- The presence of the submerged breakwater causes a smaller transmitted period. Qualitatively this is more or less understood, but quantitatively less is known. More should be known about it. It appears that studies on this phenomenon are carried out at this moment.

- The influence of the deep water wave angle of approach on the cross-shore transport, is not modelled accurately enough in the UNIBEST-TC program. The results do show an expected increase in onshore transport, but the profile develops in a non-realistic way. That is why the decision was made to only use the waves as approaching the coast perpendicularly.

- Under the present conditions the costs for a nourishment in the closed Dutch coast area amount to $9.30/m^3$. For a very large scale nourishment ($= 200,000,000$ m$^3$ or more for
a project) a reduction of this price per m$^3$ is expected. For this study a cost of f 5.-/m$^3$ is used.

-When using a design probability of 1 E-4, a surge level of 5 m and a significant deep water wave height of 8.1 m have to be taken into account. A rough cost estimate reveals that a conventional submerged rubble mound breakwater with the crest located at NAP -2 m (Breakwater A), will cost approximately f 14500.- per alongshore meter. For the breakwater with the crest at NAP -4 m (Breakwater B), this amounts to f 10500.- per alongshore meter. An extra sum for making the breakwater sand-tight and for a filter layer has to be added to these two estimates.

-Three effects are important when considering a nourishment behind a submerged breakwater. These are the change in wave height, in wave period, and the so called transition zone width.

-The combined effects of the change in wave height and the change in wave period result in a less steep profile from the NAP-line till a depth of NAP -12 m, when compared to the reference profile. However, due to the fact that the profile part between the NAP-line and a depth of approximately NAP -4 m is steeper, the final result is a steeper profile of the nourishment behind the breakwater.

-The way in which the results of the transition zone width are computed within the UNIBEST-TC program, leaves some doubts. It seems that the width is slightly overestimated. This causes a too large influence of the resulting cross-shore return flow and thus a too large effect of scour. Model tests and extensive measurements will be necessary to verify this assumption.

-The fact that the implemented breakwater in UNIBEST-TC is totally impermeable is a drawback. The relatively large porosity of the top layer can not be taken into account. Also the larger friction forces resulting from the rocks in this top layer are not modelled correctly. This is probably only a minor influence, though.

-The results from the computations with the UNIBEST-TC program, together with a more qualitative approach, are used to determine the amount of shoreline retreat, R, (relative to the reference profile if attached directly to the crest of the breakwater) before a more or less stable situation is reached behind the breakwater. Depending on the expected amount of scour just behind the breakwater, a range is derived. This amounts to the following approximate range for the two protected nourishments:

\[
\text{nourishment behind Breakwater A: } 100 \leq R \leq 200 \quad [\text{m}]
\]
\[
\text{nourishment behind Breakwater B: } 150 \leq R \leq 230 \quad [\text{m}]
\]

-Using these results on the approximate amount of shoreline retreat, a rough cost comparison is made. Two important conclusions can be drawn. It appears that the total costs of a nourishment together with a breakwater are more or less the same for both breakwaters. So due to the fact that the lower breakwater will cause less problems regarding recreation, this (the breakwater with the crest located at NAP -4 m) is the better option of the two. The second conclusion is that the protected nourishment option is economically a better option than the option of the unprotected nourishment. Reference is
made to Table 10.1.

- The presence of the submerged breakwater will also cause alongshore effects. Erosion effects on the downstream end of the breakwater will appear if a certain constant longshore sediment transport is present. Measures to overcome this problem will have to be taken.
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APPENDIX A

FORMULAE USED IN THE UNIBEST-TC PROGRAM

Reprint from the UNIBEST-TC user's manual (1992)
Courtesy of DELFT HYDRAULICS
LIST OF SYMBOLS

Wave propagation

\( E \) : wave energy
\( \rho \) : density of water
\( g \) : gravitational acceleration
\( H_{\text{rms}} \) : root mean square wave height
\( c_g \) : group velocity
\( \omega_r \) : relative wave frequency
\( k \) : wavenumber in direction of propagation
\( V \) : alongshore directed depth-averaged velocity
\( D_r \) : wave energy dissipation due to bottom friction
\( D_b \) : wave energy dissipation due to wave breaking
\( Q_b \) : fraction of breaking waves
\( H_m \) : maximum wave height
\( f_c \) : friction factor
\( D_{50} \) : 50% grain diameter

Turbulence model

\( \beta_c \) : coefficient of order one
\( \beta_d \) : coefficient of order one
\( k_t \) : turbulent kinetic energy

Cross-shore momentum equation

\( \theta_w \) : angle of incidence for waves
\( S_{\text{ss}} \) : radiation stress

Longshore momentum equation

\( i_L \) : the longshore water-level gradient due to the tide
\( A \) : calibration coefficient
\( f_c \) : friction factor due to steady current
\( \theta |_{H_{\text{rms}}} \) : amplitude of the orbital velocity for \( H_{\text{rms}} \)
Secondary current

- \( v_t \) : eddy viscosity
- \( \theta_b \) : near bottom oscillatory velocity amplitude
- \( h \) : water depth
- \( c \) : wave phase speed
- \( D \) : turbulent dissipation
- \( U \) : secondary current
- \( \omega, \Omega \) : orbital wave velocities
- \( <z_s> \) : wave averaged surface elevation
- \( m \) : mass flux due to breaking waves

Long waves

- \( G_m \) : transfer function [Sand, 1982]
- \( a_s = a_w \) : short wave amplitudes
- \( \xi_s \) : long bound wave amplitude
- \( \Delta \omega \) : beat frequency
- \( \omega_p \) : peak frequency
- \( u_l \) : long-wave velocity amplitude
- \( u_m \) : bichromatic velocity

Sediment transport

- \( x, y \) : two directions perpendicular to each other
- \( \xi_x, \xi_y \) : transport \([\text{m}^3/\text{m}^2/\text{s}]\)
- \( \vec{u} \) : instantaneous, total velocity vector near the bottom \([\text{m/s}]\)
- \( \xi_x, \xi_y \) : instantaneous velocity component in \( x \) and \( y \) direction respectively \([\text{m/s}]\)
- \( \tan \beta_x = \frac{\partial z_b}{\partial x}, z_b = \text{bottom level,} + = \text{upwards} \)
- \( \tan \beta_y = \frac{\partial z_b}{\partial y} \)
- \( \Delta \) : relative density of sediment \([-]\)
- \( N \) : ratio of sediment volume to total volume, bed material \([-]\)
- \( \tan \phi \) : angle of internal friction \([\text{rad}]\)
- \( w \) : fall velocity \([\text{m/s}]\)
- \( c_f \) : friction coefficient = \( \frac{1}{2} f_c \)
- \( \epsilon_b \) : efficiency factor bottom transport
- \( \epsilon_s \) : efficiency factor suspended transport
- \( a_h \) : amplitude of hor. orbital excursion \([\text{m}]\)
- \( r = 2.5 \times D_{90} \)

C.2
Velocity moments

\[ \bar{u} \] : mean velocity component
\[ u \] : wave velocity component
\[ u_s \] : short-wave velocity component
\[ u_l \] : long-wave velocity component
\[ T_p \] : peak period

Morphology

\[ z \] : bottom level
\[ S_x \] : cross-shore sediment transport

Formulations

The UNIBEST_TC module is a direct descendant of the models OSTRAN [Stive and Battjes, 1984] and CROSTRAN [Stive, 1986]. In Roelvink and Stive [1989], the model is tested against wave flume measurements and improved on some points. In this paper a more detailed description of the mathematical-physical formulations used in the UNIBEST_TC module is given than presented here.

Figure C.1 Definition of coordinate system and domain
Wave propagation model

The wave energy decay model of Battjes and Janssen [1978] is used. It includes the wave energy changes due to bottom refraction, shoaling, bottom dissipation and wave breaking. The current refraction due to the longshore velocity component in the short-wave propagation direction is included as well:

\[
\frac{d}{dx} \left[ c_s \cos \theta_w \frac{E}{\omega_r} \right] + \frac{D_b}{\omega_r} + \frac{D_t}{\omega_r} = 0
\]

where:

\[
E = \frac{1}{8} \rho g H^2 \sin \theta_w \frac{E}{\omega_r}
\]

\[
c_s = \frac{\partial \omega_r}{\partial k}
\]

\[
\omega_r = \omega - k \sin \theta_w V \quad \omega = 2\pi/T
\]

\[
\omega_r^2 = g k \tanh (kh)
\]

\[
D_b = \frac{1}{4} \rho g a_e Q_b (\omega/2\pi) H^2
\]

and:

\[
Q_b = \exp \left( - \frac{1 - Q_b}{(H_m/H_r)^2} \right)
\]

calibration coefficient \( a_e = O(1) \)

\[
H_m = (0.88/k) \tanh (\gamma kh/0.88)
\]

\[
D_t = \frac{1}{8} \rho f_w \pi^{-1/4} \left( \frac{\omega_r H_m}{\sinh(kh)} \right)^3
\]

and:

\[
f_w = \exp \left[ -5.997 + 5.213 \left( \frac{a}{r} \right)^{-1.56} \right]
\]

\( a_e \) is the amplitude of the hor. orbital excursion near the bottom

\( r \) is \( 2.5 \cdot D_{x0} \)
The turbulent dissipation is derived from the following equation:

\[ D = D_b - \rho \beta I \frac{\partial}{\partial x} (k_j h c \cos \theta_w) \]

where:

\[ D = \rho \beta_d k_z^2 \]

The cross-shore momentum equation which is used to calculate the wave induced set-up:

\[ \frac{d}{dx} S_{sw} + \rho g h \frac{d h}{dx} = 0 \]

where:

\[ S_{sw} = E \left( n(1 + \cos^2 \theta_w) - \frac{\kappa}{h} \right) \]

\[ n = \frac{kh}{2 \sinh(2kh)} \]

The wave angle with the shore normal is after Snel’s law:

\[ k \sin \theta_w = \text{constant} \]

The longshore momentum equation, which describes the balance between the driving forces of the longshore current due to the tide and waves and the bottom friction:

\[ \rho g h \frac{d}{dx} k \sin \theta_w = A \rho \frac{(1 + \sin^2 \theta_w)}{\sqrt{\pi}} \sqrt{(\frac{c}{f_j}) (\frac{h}{c})} \frac{d U}{dx} \]

**Secondary current**

The secondary current is due to the vertical non-uniformity of the driving forces in the nearshore zone. This is modelled according to the formulations given by Stive and de Vriend [1987]. They use a profile function technique in combination with a horizontally two-dimensional current to describe the three-dimensional current system in the coastal zone. The secondary current velocity is estimated using a three-layer concept.
The influence of the surface layer on the underlying layers is accounted for by an effective shear stress at trough level [Stive and Wind, 1986]. This compensates for the momentum decay in the surface layer due to viscous dissipation and momentum loss due to wave breaking. The effective shear stress is at wave trough level is given by:

$$\tau(t) = \rho \nu_t \frac{u_t^2 k}{c} \sinh(2kh) + \frac{D}{c}$$

The use of the turbulent dissipation instead of the wave energy dissipation results in a spatial lag between the wave breaking and the offshore directed secondary current.

The horizontal wave-average momentum balance for the middle layer in the cross-shore direction reads:

$$\frac{\partial}{\partial x} \left( \rho \phi \frac{\partial u}{\partial x} \right) = \frac{\partial}{\partial x} \left( \langle u^2 \rangle - \langle w^2 \rangle \right) + g \frac{\partial \langle z \rangle}{\partial x}$$

For the bottom layer the horizontal wave-average momentum equation in the cross-shore direction is given by:

$$\frac{\partial}{\partial x} \left( \rho \phi \frac{\partial u}{\partial x} \right) = \frac{\partial}{\partial x} \left( \langle u^2 \rangle - \langle w^2 \rangle \right) + g \frac{\partial \langle z \rangle}{\partial x} + \frac{\partial}{\partial z} (\langle u w \rangle)$$

where the last term on the right-hand side is no longer negligible compared to the Reynolds stress term given on the left-hand side of the equation. With the shear stress condition at the trough level and a no slip condition at the bottom, the solution for the secondary current is obtained by patching the velocities and shear stresses at the bottom boundary $z_b$. Using the integral condition of continuity:

C.6
\[ \int U \, dz = -\frac{m}{\rho} \]

where:

\[ m = \left(1 + Q, \frac{7kh}{2\pi} \right) \frac{E}{c} \]

and \( m \) represents the mass flux in the surface layer due to breaking waves yields the final expression for the secondary current.

**Long waves**

In the case of a random wave field the grouping of the short waves will generate long waves. The assumption is made that the wave-group related features of a random wave field may be represented by a bichromatic wave train with accompanying bound long wave. For the amplitude of the bound long wave is used:

\[ \xi_s = \frac{a_s n_s}{h} \]

The short wave amplitudes are given by:

\[ a_w = \frac{1}{8} H_w^2 - \frac{1}{2} \xi_s^2 \]

where the condition that the schematized wave train has the same surface variance as the random wave has been used. The individual velocities are obtained using linear wave theory.

For the long wave component is taken:

\[ u_l = \xi_s \frac{\sqrt{gh}}{h} \]

The near bottom time-varying flow due to short and long waves is given by:

\[ u_w = \dot{u}_w \cos(\omega_p t) + \dot{u}_s \cos(\omega_s + \Delta \omega) t + \dot{u}_l \cos((\Delta \omega) t + \phi) \]

The beat frequency \( \Delta \omega = \frac{1}{5} \omega_p \)

**Short-wave orbital velocity**

The orbital velocities near the bed due to short waves, which determine the strength of the onshore directed wave asymmetry transport, have been computed using the model RFWAVE, developed at Delft Hydraulics [G. Klopman, 1989]. It is based on the Fourier approximation of the stream function method as developed by Rienecker and Fenton [1981], using wave energy as input.

C.7
Sediment transport

The sediment transport is calculated according to the formulations given by Bailard [1981], of which only the cross-shore component is used for the time-dependent morphological computations. This formulation includes transport due to the combined actions of steady current, wave orbital motion and bottom slope effect. The Bailard transport model in 2 horizontal dimensions is given by:

\[
\begin{align*}
q_r &= \frac{c_r}{\Delta g N} \left( \frac{\varepsilon_n}{\tan \phi} \left[ -\frac{\tan \beta_x}{\tan \phi} < |\vec{u}|^2 \vec{u}_x > - \frac{\tan \beta_y}{\tan \phi} < |\vec{u}|^2 \vec{u}_y > \right] + \\
&+ \frac{c_r}{\Delta g N} \left( \frac{\varepsilon_s}{w} \left[ -\frac{\tan \beta_x}{\tan \phi} < |\vec{u}|^3 \vec{u}_x > - \frac{\tan \beta_y}{\tan \phi} < |\vec{u}|^3 \vec{u}_y > \right] \right) \\
q_t &= \frac{c_t}{\Delta g N} \left( \frac{\varepsilon_n}{\tan \phi} \left[ -\frac{\tan \beta_x}{\tan \phi} < |\vec{u}|^2 \vec{u}_x > - \frac{\tan \beta_y}{\tan \phi} < |\vec{u}|^2 \vec{u}_y > \right] + \\
&+ \frac{c_t}{\Delta g N} \left( \frac{\varepsilon_s}{w} \left[ -\frac{\tan \beta_x}{\tan \phi} < |\vec{u}|^3 \vec{u}_x > - \frac{\tan \beta_y}{\tan \phi} < |\vec{u}|^3 \vec{u}_y > \right] \right)
\end{align*}
\]

where:

- \( c_r = 0.5 f_u \)
- \( f_u = \exp \left[ -5.977 + 5.213 (a/n)^{1/n} \right] \)
- \( <> \) indicate averaging over time.

The longshore transport is defined as the component of the total transport vector in longshore direction.

The transport formulation contains several velocity moments. The time-averaged total near bed velocity is split up into a mean and a time varying velocity component:

\[ u = \bar{u} + \bar{u}(t) \]

where \( \bar{u}(t) \) stands for the velocity variation on the time scale of the wave groups and that of the individual waves. With these separate terms for the wave-orbital motion and the steady current the terms \( < |\vec{u}|^m u_i^k > \), with \( m = 2, 3, 5 \) and \( n = 0, 1 \) can be approximated by a Taylor series. In these series the angle between waves and currents has also been included.

Two Taylor expansions are possible: one with the orbital motion small compared to the steady current and vice versa.
The first yields the following expressions for the odd velocity moments in u-direction:

\[ <|u| u^2> = \bar{u}^3 + \bar{u} <\bar{u}^3> (1 + \cos^2 \phi) + <\bar{u}^3> \cos \phi \]

\[ <|u| u^4> = |\bar{u}|\bar{u}^4 + |\bar{u}|\bar{u} <\bar{u}^3> (3 + 9\cos^2 \phi)/2 + |\bar{u}| <\bar{u}^3> (3\cos \phi + \cos^3 \phi) + 
+ \frac{\bar{u}}{|\bar{u}|} <\bar{u}^3> (3 + 6\cos^2 \phi - \cos^4 \phi)/8 \]

the odd velocity moments in v-direction:

\[ <|u| v^3> = \bar{u} <\bar{u}^3> (2\sin \phi \cos \phi) + <\bar{u}^3> \sin \phi \]

\[ <|u| v^5> = |\bar{u}|\bar{u} <\bar{u}^3> (3\sin \phi \cos \phi) + |\bar{u}| <\bar{u}^3> (3\sin \phi + 3\sin \phi \cos^2 \phi)/2 
+ \frac{\bar{u}}{|\bar{u}|} <\bar{u}^3> (3\sin \phi \cos \phi - \sin \phi \cos^3 \phi)/2 \]

and the even velocity moments:

\[ <|u| d^2> = |\bar{u}|^2 + |\bar{u}| <\bar{u}^3> (3 + 3\cos^2 \phi)/2 + \frac{\bar{u}}{|\bar{u}|} <\bar{u}^3> (3\cos \phi - \cos^3 \phi)/2 \]

\[ <|u| d^4> = |\bar{u}|^4 + |\bar{u}|^3 <\bar{u}^2> (5 + 15\cos^2 \phi)/2 + |\bar{u}|^2 <\bar{u}^2> (15\cos \phi + 5\cos^2 \phi)/2 
+ |\bar{u}| <\bar{u}^2> (15 + 30\cos^2 \phi - 5\cos^4 \phi)/8 
+ \frac{\bar{u}}{|\bar{u}|} <\bar{u}^2> (15\cos \phi - 10\cos^3 \phi + 3\cos^2 \phi)/8 \]
Where:
\[ \varphi = \theta_w - \theta_e. \]

**Figure C.4 Velocity directions**

If the mean velocity component is small compared to the orbital velocity the Taylor expansion yields for the odd velocity moments:

\[
\langle |a| \rangle = \langle |a|^2 \rangle + \vec{u} \langle |a|^3 \rangle (3 \cos \varphi) + \vec{u}^2 (\cos \varphi)
\]

\[
\langle |a| \rangle = \langle |a|^2 \rangle + \vec{u} \langle |a|^3 \rangle (4 \cos \varphi) + \vec{u}^2 \langle |a|^4 \rangle (3 + 9 \cos^2 \varphi)/2 + \vec{u}^3 \langle |a| \rangle (3 \cos \varphi + \cos^3 \varphi)
\]

the odd velocity moments in \( v \)-direction:

\[
\langle |a| \rangle = \vec{u} \langle |a|^2 \rangle (\sin \varphi) + \vec{u}^3 (\sin \varphi)
\]

\[
\langle |a| \rangle = \vec{u} \langle |a|^3 \rangle (\sin \varphi) + \vec{u}^2 \langle |a|^4 \rangle (3 \sin \varphi \cos \varphi)
\]

\[ + \vec{u}^3 \langle |a| \rangle (3 \sin \varphi + 3 \sin \varphi \cos^2 \varphi)/2 \]

and the even velocity moments:

\[
\langle |a| \rangle = \langle |a|^2 \rangle + \vec{u} \langle |a|^3 \rangle (3 \cos \varphi) + \vec{u}^2 \langle |a|^4 \rangle (3 + 3 \cos^2 \varphi)/2
\]

\[
\langle |a| \rangle = \langle |a|^2 \rangle + \vec{u} \langle |a|^3 \rangle (5 \cos \varphi) + \vec{u}^2 \langle |a|^4 \rangle (5 + 15 \cos^2 \varphi)/2
\]

\[ + \vec{u}^3 \langle |a| \rangle (15 \cos \varphi + 5 \cos^3 \varphi)/2 \]

\[ + \vec{u}^4 \langle |a| \rangle (15 + 30 \cos^2 \varphi - 5 \cos^4 \varphi)/8 \]
where:
\[ \psi = \theta_0 - \theta_n. \]

A smooth transition from one formulation to the other is taken care of.

The orbital velocity component is split up into a short-wave and a long-wave component:
\[ \tilde{u} = \tilde{u}_s + \tilde{u}_l. \]

The long-wave velocity is assumed to be significantly smaller than the short-wave component. A second assumption is that there is no correlation between \( u_s \) and \( |u_l| \). With these assumptions we can write for the following odd velocity moment:
\[ <\tilde{u}|\tilde{u}|^2> = <u_s|u_s|^2> + 3 <u_l|u_l|^2> \]

Where the first term on the right is non-zero in the case of an asymmetry about the horizontal plane caused by the non-linearity of the short waves. As mentioned before, this part is calculated with RFWAVE.

The second term on the right is non-zero if there is a correlation between \( u_s \) and \( u_l \). This correlation is present in the case of long bound waves accompanying a short-wave group, resulting in a negative correlation. In that case the velocity moment is approximated by:
\[ 3 <u_s|u_l|^2> = <u_l^3> \]

where the bi-chromatic velocity component is calculated as described previously. The other velocity moments are expanded in a similar way. In order to reduce computing time, the results have been tabulated as function of two dimensionless variables:
\[ f \left( \frac{H}{h}, T, \sqrt{\frac{g}{h}} \right) \]

**Morphology**

The bottom level changes are computed from the mass balance:
\[ \frac{\partial z}{\partial t} + \frac{\partial S_z}{\partial x} = 0 \]
Map of the numbered cross-sectional areas ("raaien") along the closed Dutch coast.
APPENDIX C

LISTING OF TWGETI.PAS
program TWGETIJI(input, output);
uses crt;
const n = 500;
type plotdata1 = array[0..n] of real;
var T, RANGE_T, DT, V, TOT, HO : REAL;
P, U, R, S, Q, OKE, GETI_HOOGTE_MAX, GETI_HOOGTE_MIN,
INTERVAL_T, AANTAL_COMP, RANGE_H, L : INTEGER;
UIT : TEXT;
VU, F, H, G, O, INT_MIDDEN, AANTAL, GETI_HOOGTE, GETI_HOOGTE_A, X, Y,
PERCENTAGE, PERC_TOT : PLOTDATA1;
Z : CHAR;

{****************************}
{HOOFDPROGRAMMA}
BEGIN
{INVOEREN PARAMETERS}
CLRSCR;
WRITELN;
WRITE('AANTAL GETIJCOMPONENTEN:');
READLN(AANTAL_COMP);
WRITE('DE MINIMALE GETIJHOOGTE IS:');
READLN(GETI_HOOGTE_MIN);
WRITE('DE MAXIMALE GETIJHOOGTE IS:');
READLN(GETI_HOOGTE_MAX);
WRITELN('HET INTERVAL VAN DE HOOGTES BEDRAAGT 1 CM.');
WRITELN('HET INTERVAL VAN DE TIJD MOET ZIJN (IN MIN.):');
READLN(INTERVAL_T);
WRITE('WAT IS DE MIDDENSTAND ?');
READLN(HO);
ASSIGN(UIT, 'A:\INVOER.DAT');
REWriteln(UIT);
WRITELN(UIT, 'De waarden voor VU, F, H, G, OMEGA:');
WRITELN(UIT);
FOR Q := 1 TO AANTAL_COMP DO
BEGIN
WRITE('VOER IN VU['',Q,']');
READLN(VU[Q]);
WRITE('F['',Q,']');
END;

Appendix C-1
READLN(F[Q]);
WRITE('H'[Q],',',' Q',' [',']');
READLN(H[Q]);
WRITE('G'[Q],',',' Q',' [',']');
READLN(G[Q]);
WRITE('OMEGA'[Q],',',' Q',' [',']');
READLN(O[Q]);
WRITELN('DE WAARDEN VOOR VU,F,H,G,OMEGA');
WRITELN(VU[Q]:7:3,F[Q]:7:3,H[Q]:7:3,G[Q]:7:3,O[Q]:7:3);
WRITELN(uit,',' VU','[Q]',' F','[Q]',' H','[Q]',' G','[Q]',' OMEGA','[Q]');
WRITELN(uit,VU[Q]:7:3,' ,,F[Q]:7:3,' ,,H[Q]:7:3,' ,,G[Q]:7:3,' ,,O[Q]:7:3);
END;
close(uit);

DT:=INTERVAL_T/60;
RANGE_H:=(GETI_J HOOGTE_MAX - GETI_J HOOGTE_MIN);
RANGE_T:=(365*24*60)/INTERVAL_T;
WRITELN('TIJDSTAP IN UREN, HOOGTE-RANGE, Tijd-RANGE');
WRITELN(DT:7:3,RANGE_H:7,RANGE_T:7:2);

FOR P:=1 TO RANGE_H DO
BEGIN
  INT_MIDDEN[P]:=GETI_J HOOGTE_MIN + (P-O.5);
  AANTAL[P]:=0;
END;

FOR P:=1 TO RANGE_H DO
BEGIN
  T:=0;
  TOT:=0;
  V:=0;
  OKE:=2;
  WHILE OKE=2 DO
  BEGIN
    R:=1;
    GETI_J HOOGTE[0]:=0;
    FOR R:=1 TO AANTAL_COMP DO
    BEGIN
      GETI_J HOOGTE_A[R]:=F[R]*H[R]*COS((O[R]*T+VU[R]-G[R])*(PI/180));
      GETI_J HOOGTE[R]:=GETI_J HOOGTE[R-1] + GETI_J HOOGTE_A[R];
    END;
    GETI_J HOOGTE[R]:=GETI_J HOOGTE[R] + HO;
    WRITELN('DE TID, DE R, DE GETI_J HOOGTE');
    WRITELN(T:7:2,R:7,GETI_J HOOGTE[R]:7:2);
    X[1]:= INT_MIDDEN[1] - 0.5;
    Y[1]:= INT_MIDDEN[1] + 0.5;
    S:=1;
    WHILE GETI_J HOOGTE[R] >= Y[S] DO
    BEGIN
      S:=S+1;
      WRITELN('X[S]:',X[S],',',' Y[S]:',Y[S],',',' S:',S);
    END;
    X[S]:=Y[S];
    Y[S]:=Y[S] + 0.5;
    S:=S+1;
    T:=T+DT;
    TOT:=TOT+1;
    V:=V+1;
  END;

Appendix C-2
X[S]:= INT_MIDDEN[S] - 0.5;
Y[S]:= INT_MIDDEN[S] + 0.5;
END;
AANTAL[S] := AANTAL[S] + 1;
TOT := TOT+1;
T := T + DT;
V := V + 1;
IF V = RANGE_T THEN OKE := 1;
writeln('totaal aantal',tot,' waarnemingen');
END;
ASSIGN(UIT,'A:\UITVOER.DAT');
REWRITE(UIT);
WRITELN(UIT,'DE KLASSEMIDDENS MET BIJHOREND PERCENTAGE EN TOTAAL PERCENTAGE');
PERC_TOT[0]:=0;
FOR L:= 1 TO RANGE_H DO
BEGIN
PERCENTAGE[L] := (AANTAL[L]/TOT)*100;
WRITE(UIT,INT_MIDDEN[L]:7:2);WRITE(UIT,' ');
WRITE(UIT,PERCENTAGE[L]:7:2);WRITE(UIT,' ');
PERC_TOT[L]:=PERC_TOT[L-1]+PERCENTAGE[L];
WRITELN(UIT,PERC_TOT[L]:7:2);
END;
CLOSE(UIT);
END.

Appendix C-3
APPENDIX D

LISTING OF COMPUTED RESULTS ON K, COMPARISON
\[ X_1 \quad X_2 \quad \text{Crest high} \quad Dn50 \quad B \quad B/Dn50 \quad K_{t1} \quad K_{t2} \quad K_{t3} \quad K_{t4} \quad K_{t,u} \]

\[
\begin{array}{cccccccc}
13300.00 & 13350.00 & -2.00 & 1.18 & 4.00 & 3.390 & K_{t1} \quad : \quad \text{Van der Meer} \\
& & & & & & K_{t2} \quad : \quad \text{Daemen conventional} \\
& & & & & & K_{t3} \quad : \quad \text{Daemen reef breakwater} \\
& & & & & & K_{t4} \quad : \quad \text{Hearn} \\
& & & & & & K_{t,u} \quad : \quad \text{UNIBEST} \\
\end{array}
\]
<p>| 55 | 1.9E-01 | 4.1E-01 | 6.9E-01 | 8.0E-01 | 7.5E-01 | 6.0E-01 | 8.4E-01 | 8.9E-01 |
| 56 | 1.8E-01 | 3.9E-01 | 6.6E-01 | 6.2E-01 | 6.8E-01 | 6.9E-01 | 4.7E-01 | 7.7E-01 |
| 57 | 1.9E-01 | 3.9E-01 | 6.4E-01 | 6.5E-01 | 6.3E-01 | 6.0E-01 | 7.1E-01 | 5.2E-01 |
| 58 | 1.7E-01 | 3.9E-01 | 6.6E-01 | 6.8E-01 | 7.5E-01 | 6.0E-01 | 7.3E-01 | 5.7E-01 |
| 59 | 1.7E-01 | 3.9E-01 | 6.6E-01 | 7.1E-01 | 7.1E-01 | 6.0E-01 | 7.4E-01 | 6.1E-01 |
| 60 | 1.7E-01 | 3.9E-01 | 6.6E-01 | 7.4E-01 | 6.0E-01 | 7.6E-01 | 6.5E-01 |
| 61 | 1.7E-01 | 3.9E-01 | 6.6E-01 | 7.4E-01 | 6.0E-01 | 7.5E-01 | 6.9E-01 |
| 62 | 1.7E-01 | 3.9E-01 | 6.6E-01 | 8.0E-01 | 6.0E-01 | 7.6E-01 | 7.1E-01 |
| 63 | 1.7E-01 | 3.9E-01 | 6.6E-01 | 8.0E-01 | 6.0E-01 | 7.9E-01 | 7.4E-01 |
| 64 | 1.7E-01 | 3.9E-01 | 6.6E-01 | 8.0E-01 | 6.0E-01 | 7.9E-01 | 7.4E-01 |</p>
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The table above represents the values for different steps and models, with or without boundaries.
| Kruinhgt | Dn50 | B | | Without boundaries | With Boundaries |
|---------|------|----|------------------|------------------|
| 0.0000  | 1.4500 | 4.0000 |------------------|------------------|

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**Kt1** : Van der Meer  
**Kt2** : Daeen conventional  
**Kt3** : Daeen reef breakwater  
**Kt4** : Heann  
**Ktu** : Unibest
APPENDIX E

APPROXIMATION OF PROFILE SHAPES
The approximation of the different profiles as mentioned in Section 9.4, was done using a least-squares approximation method. Since all the approximations are of the polynomial form, Table E.1 is made which features the different values of the coefficients as presented in equation E.1:

\[ Y = a + bx + cx^2 + dx^3 + ex^4 \]  

(E.1)

<table>
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<tr>
<th>Profile</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
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Table E.1 Values for the coefficients used in the approximations

The lower part of Profile three was also approximated by a different function than the one stated in Table E.1. The function used was:

\[ Y = 0.581 - 0.0219x + 1.35 E-5 x^2 \]  

(E.2)

As example three figures are added which show the original function values and the approximation.

Figure E.1 Approximation of Profile 1

Appendix E-1
Figure E.2 Approximation of the original, 10 years profile

Figure E.3 Approximation of the lower part of the third profile
APPENDIX F

EXCEEDANCE GRAPHS
OVERSCHRIJDINGSFREQUENTIE PER JAAR

Figuur 3.0.1. Overschrijdingslijnen van de stormvloedstanden te Hoek van Holland 1859 t/m 1958

rangnummer in beschouwde periode

overschrijdingslijn beneden N.A.P. ± 3 m
werklijn boven N.A.P. ± 3 m
denkbare overschrijdingslijnen
Verwachtingswaarde van de significante golflengte als functie van het stormvloedpeil voor enkele locaties voor de Nederlandse kust [10].

DH = Den Helder
EG = Eierlandsche Gat
IJ = IJmuiden
Bo = Borkum
HvH = Hoek van Holland
VI = Vlissingen

*) buiten het plattelandsgebied
APPENDIX G

THE TRANSMISSION COEFFICIENT $K$. 
APPENDIX G: THE TRANSMISSION COEFFICIENT K.

G.1: Introduction

This Appendix G consists of a literature review on the subject of wave transmission over and through a breakwater. Apart from the conventional submerged breakwater type, as used in this study to support a nourishment, reef breakwaters as well as breakwaters with a positive freeboard are discussed.

The degree of reduction of the wave heights as a result of the presence of the breakwater is given by the transmission coefficient, \( K \). The definition for this transmission coefficient is as follows:

\[
K = \frac{H_t}{H_i}
\]

with \( H_t \) = transmitted wave height \( [m] \)

\( H_i \) = incoming wave height \( [m] \)

The different parameters which influence the transmission coefficient are all displayed in Figure G.1.

If the transmission coefficient has a value smaller than one, i.e. the transmitted wave energy is smaller than the energy possessed by the incoming wave, the driving component of the longshore current resulting from the waves will become less (when compared to the situation without the breakwater present). This is a result of the fact that this driving component is mainly determined by the amount of dissipation of the wave energy. Since the amount of energy has become less so will the dissipation. The cross-shore transport will also change as a result of the change in wave heights. This is caused by the fact that the wave height is of prime importance to the wave driven current which determines the transport of sediment. This can be shown by regarding the influence of the wave height on the wave driven cross-shore mean flow in the surf zone. Stive and Wind (1986) explain this phenomenon theoretically and also show various examples which support this theory.

In Section G.2 the literature review is given on research consisting of theory as well as experiments which are focused on the subject of wave transmission. It appears that a distinction between two types of breakwaters has to be made, namely the conventional submerged type and the reef type. Otherwise it is not possible to derive a formula which doesn't have a large scattering in the outcome.

G.2: Literature review

Various authors have published articles on the subject of wave transmission. A comparison between most articles is possible since most of the relevant details are included in these articles. This has been done by Van der Meer (1990b). A further improvement of
First a definition in Figure G.1 is given of most of the relevant parameters which have been used in different research. Secondly the relevant research will be discussed and presented in chronological order.

![Figure G.1 Important parameters determining the wave transmission](image)

The specified parameters are:

- \( R_c \) = crest freeboard
- \( B \) = crest width
- \( h_c \) = height of the structure
- \( h \) = water depth in front of the structure
- \( D_{n50} \) = nominal diameter of the primary armour rock
- \( H_i \) = incoming wave height
- \( H_t \) = transmitted wave height
- \( T_p \) = peak period
- \( T_{\text{m01}} \) = mean period
- \( s_{op} \) = wave steepness

The wave steepness \( s_{op} \) is determined as follows:

\[
s_{op} = \frac{2 \pi H_i}{g T_p^2}
\]

with \( H_s = \text{significant wave height at the toe of the structure (mean of highest one third of the waves)} [m] \)

**Powell and Allsop (1985)**

Powell and Allsop (1985) used the results of experiments carried out at Hydraulics Research and derived a general picture as displayed in Figure G.2.

Powell and Allsop use the parameter \( R_p^* = R_p / H_s \ast (s_{op}/2\pi)^{0.5} \) as their starting point in order to take the influence of the wave period, besides the influence of the freeboard, into account. The data displayed in Figure G.3 show that the form of the breakwater (the

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1 A brief summary of amongst other research of Van der Meer (1990b) and Daemen (1991) can be found in Van der Meer (1991), which is a publication of Delft Hydraulics.

Appendix G-2
cross-section) influences the transmission. In this figure various cross-sections are displayed numbered by Sect.i, with i equals 2 up to and including 8. The general trend is that if the cross-sectional area, $A_i$, or the breakwater crest width becomes larger, the transmission coefficient becomes smaller. This can be illustrated by comparing the values of Sect.8 and 5 final in Figure G.3. Sect.8 has a larger crest width (one and a half time as large) and the larger cross-sectional area than 5 final. Powell and Allsop present design curves for the transmission as shown in Figure G.4. The curve is influenced by, among many other variables, the porosity ($P$) of the breakwater. The upper curve is valid for $P = 0.5$ and the lower for $P = 0$, both of which can be seen as boundaries. In other words, the greater the porosity of the breakwater, the greater the transmission will be. For general design purposes a porosity value of 0.4 is recommended and also shown in Figure G.4.

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Goméz Pina and Valdés (1990) also point out the influence of the crest width on the transmission coefficient. They also show that values of the transmission coefficient may display a large scatter for certain set values of the incoming wave height, $H$, and the crest width as a result of model-scaling factors.
Daemrich et al (1985) have conducted experiments on breakwaters which were either completely submerged or had a freeboard of zero, i.e. the crest height at SWL. During this research it was found that tests using monochromatic waves didn’t give satisfactory results because the results seemed to be distorted. The point is stressed that random waves should be used, especially when comparisons are to be made. The results showed a clear influence of the peak period, especially if the freeboard was equal to zero. Since this also follows from other research and Ahrens (1987) shows this more clearly it will be treated further on.

The influence of the crest width is also clear as shown in Figure G.5. The transmission coefficient, $K_n$, is stated as the parameter CT. In other words, CT is equal to $K_n$. Nearly all the values of the transmission coefficient as a function of the significant wave height, $H_{1/3}$, for $B = 0.2$ m (on the left hand side of the figure) are structurally higher than the ones for $B = 1.0$ m. The influence of the crest width is a function of the depth at which the breakwater crest is located. This influence becomes less if the value of the freeboard becomes more negative. A large reduction of the transmission if the freeboard is equal to zero or has a small negative value can be found however if the width becomes substantially larger. For example it is found in the results of Daemrich et al that if the freeboard is equal to zero, a reduction of the transmission of 40% is achieved when expanding the crest width from 0.2 to 1.0 m. The influence of the roughness of the primary armour has also been studied. It does seem to have a small effect on the transmission but this effect becomes drastically smaller if the crest of the breakwater is located further below the mean water level.
Ahrens (1987) conducted research on reef breakwaters. A reef breakwater is a homogeneous structure without a core or filter layer present, see Figure G.6. The idea behind such a reef breakwater is that its form changes due to the wave attack. This is also illustrated in Figure G.6 by the difference between the initial profile and the equilibrium profile. The changed form, the equilibrium profile, has a larger stability factor than the initial profile.

It has to be mentioned that Ahrens did not use the wave height at the toe of the breakwater as the incoming wave height, $H_i$, but instead used the wave height measured in the situation without the breakwater present at the same location where the transmitted wave height, $H_t$, was to be measured. So the incoming wave height was measured first in the situation without the breakwater present, and afterwards the transmitted wave height was measured at the same location with the breakwater present. The main result is shown in Figure G.7.a. The large scatter for positive values of the relative freeboard, as defined by...
$R_v/H_{m0}$, is caused by small waves with approximately the same wave height as the diameter of the stones. The waves then propagate through the breakwater quite easily.

Results obtained by Ahrens have been used again by Van der Meer (1990b). Two important conclusions may be drawn. First of all it is clear that a larger peak period consequently results in a larger value for the transmission (Figure G.8). Secondly if the cross-sectional area is larger, then the value for the transmission coefficient becomes smaller, which is in agreement with the earlier findings of Powell and Allsop.

Ahrens presents the following formulae for $K_t$:

for $R_v/H_{m0} \leq 1.0$:

$$K_t = \frac{1.0}{1.0 + \left(\frac{h_c}{h}\right)^{c_1} \left(\frac{A_r}{hL_p}\right)^{c_2} \exp[c_3(R_v/H_{m0})+c_4(A_t^{1.5}/D_{50}L_p)]}$$  \hspace{1cm} (G.3.a)

and for $R_v/H_{m0} > 1.0$:

$$K_t = 1.0 / \left[ 1.0 + P^{0.592} \right]$$  \hspace{1cm} (G.3.b)

with

$c_1 = 1.188$ \hspace{2cm} $[-]$
$c_2 = 0.261$ \hspace{2cm} $[-]$
$c_3 = 0.529$ \hspace{2cm} $[-]$
$c_4 = 0.0051$ \hspace{2cm} $[-]$
$A_t = breakwater \ cross-sectional \ area$ \hspace{2cm} $[m^2]$
$L_p = local \ wave \ length$ \hspace{2cm} $[m]$

Appendix G-6
\[ D_{50} = 50\% \text{ diameter of the sieve curve} \]
\[ P = \text{parameter defined by Hearn (formula G.4)} \]

**Figure G.8** The influence of the peak period, \( T_p \), according to Van der Meer

Hearn (1987) also used the data from Ahrens (1987) as mentioned earlier. She concludes that the parameters which influence the transmission coefficient and hence also the way of computing its value, change when the transmission as a result of overtopping changes into transmission as a result of flow through the breakwater.

Three areas for the relative freeboard, \( R \) (see formula G.5), are distinguished. Firstly, if \( R > 1 \), then the transmission is determined by the flow through the breakwater and not by overtopping. The transmission is then independent of the relative freeboard and only a function of the parameter \( P \), where \( P \) is defined by:

\[ P = \frac{H_s A_t}{L_p(D_{n50})^2} \]  \hspace{1cm} (G.4)

Secondly there is a transition area defined by \( 0 \leq R \leq 1 \) in which the transmission depends on both \( R \) and \( P \). The influence of \( P \) decreases if \( R \) becomes smaller. Thirdly there is the area defined by \( R \leq 0 \) (only negative values for the relative freeboard) where the transmission is only a function of \( R \). The following formulae are valid for the different areas as stated. It has to be kept in mind that the validity range for these formulae consists of \(-4 \leq R \leq 6\).

For \( R \leq 0 \):

\[ K_t = 0.9 - 0.358 \ e^R \]  \hspace{1cm} (G.5.a)
For $0.0 \leq R \leq 1.0$ :

$$K_t = \frac{1.0}{1.845 + (P^a - 0.845)R} \quad (G.5.b)$$

For $R > 1.0$ :

$$K_t = \frac{1.0}{1 + P^a} \quad (G.5.c)$$

with \[R = \frac{R_c}{H_{m0}}\] [-] \[a = 0.5926\] [-]

According to formula G.5.a the transmission coefficient is only a function of the relative freeboard \((R_c/H_{m0})\). This is different from formula G.3.a from Ahrens where the influence of other parameters is also taken into account. Formula G.5.a can hence only be used if a certain amount of scatter is acceptable. Figure G.3 and G.8 clearly show that for instance both the cross-sectional area and the peak period have a certain influence on the amount of transmission which is not included in formula G.5.a.

**Van der Meer (1990b)**

Van der Meer (1990b) used a lot of data from different research to derive a relatively simple overall formula which gives the transmission coefficient as a function of only the relative freeboard. Van der Meer (1990b) used data as collected by Van der Meer (1988) as starting point. In this report the results from tests carried out at Delft Hydraulics in the period 1983-1987 are discussed. Although the main goal of these tests consisted of determining stability criteria, the wave transmission was also measured and recorded. Since these tests were done using breakwaters with crests above, below, as well as at SWL, they formed an ideal basis to compare other research results, because this other research was often only focussed at breakwater heights either above or below SWL. So in this way it was possible to combine different research to come to an overall formula.

The data of Van der Meer (1988) clearly show that for the same \(R_c/H_{m0}\) value a larger peak period results in a higher transmission. An investigation was carried out whether or not using the value \(R^*\) gave better results than using the value \(R_c/H_{m0}\). This because Powell and Allsop (1985) used this parameter \(R^*\). For values of \(R_c/H_a > 0.5\) this was indeed the case. When considering all values of \(R_c\) however, it appeared that using \(R^*\) did not yield better results than using \(R_c/H_{m0}\). At negative values of \(R_c\) the parameter \(R_c/H_{m0}\) even gave better results with less scatter than the parameter \(R^*\).

Van der Meer (1990b) used the research as earlier mentioned and the results of Seelig (1980). Seelig performed tests on a lot of different cross-sectional areas for both impermeable- as well as permeable breakwaters. Seelig also made the same distinction as Hearn between transmission by overtopping and by flow through the structure. It was also noted that the influence of overtopping became larger if the crest of the breakwater was located

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further below SWL, i.e. if $R_e$ gets a larger negative value (see Figure G.9). Van der Meer (1990b) only uses the data which were obtained by performing tests with random waves, which is in agreement with the advice given by Daemrich et al (1985). From the data from Seelig the conclusion is drawn that small wave heights in relation to the stone diameter may cause a large amount of scatter. This conclusion is supported by the results of Ahrens (1987).

![Figure G.9 The influence of overtopping ($K_{op}$) on the total transmission ($K_T$), and the influence of the incoming wave height ($H_i$), after Seelig](image)

Powell and Allsop (1985) originally used the mean period, $T_m$, instead of the peak period, $T_p$, which was needed for comparing the results. This peak period has however been added to the data later on so that a comparison was possible. Another problem which arose during this comparison was that it was not clear which crest height should be used in the data on the reef breakwaters. This because the crest height changed during the tests. Van der Meer (1990b) uses the crest height as measured after the test series, i.e. the value of $h_c$ in Figure G.6. This all results in a total combination of all available data as shown in Figure G.10. Taking the part which is valid for $R_e \leq 0$ into account, it seems that the data are in good agreement with each other, the only exception being the data from Seelig.

The formulae G.6.a through G.6.c have been derived with the comment that a relative large amount of scatter has to be accepted.

For: $-2.0 < R_e/H_s < -1.13$ \[ K_i = 0.8 \tag{G.6.a} \]

For: $-1.13 < R_e/H_s < 1.2$ \[ K_i = 0.46 - 0.3R_e/H_s \tag{G.6.b} \]

For: $1.2 < R_e/H_s < 2.0$ \[ K_i = 0.1 \tag{G.6.c} \]

The scatter is a direct result of the simplicity of the formulae. Just as with the formulae G.5.a from Hearn the parameters peak period ($T_p$), cross-sectional area ($A_c$), crest width

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(B) and the nominal diameter ($D_{as}$) are not taken into account. Figure G.11 shows the formulae G.6 with an upper- and lower boundary of $K_t \pm 0.15$, which is the 90% confidence level. The standard deviation amounts to $\sigma(K_t) = 0.09$.

The high transmission coefficient values for $R_c/H_o > 1$ are caused by the very low wave heights relative to the used stone diameter. It is clear that in reality for very large negative values of $R_c/H_o$, the value of the transmission coefficient should approach 1. For very large positive values $R_c/H_i$ the value of the transmission coefficient should approach 0 if the structure is completely impermeable. If that is not the case it is feasible that the transmission coefficient remains at a constant small value not equal to 0, since transmission as a result of flow through the structure will remain possible.

Van der Meer (1990b) states that a more in depth study is needed in which also the influence of the other factors as mentioned above ($T_p$, $B$, $A_t$ and $D_{as}$) has to be studied, in order to minimize the scatter and to improve the reliability of the formulae. This study along with a new test series was carried out by Daemen (1991).
Daemen (1991), using all the data which Van der Meer (1990b) used plus the data from the new test series, separates the parameters $R_c$ and $H$. This because of two reasons; firstly because in this way the large scatter at the $R_c = 0$ point will be reduced as a result of the fact that now the influence of the wave height will not be lost as occurs if use is made of the parameter $R_c/H$. Secondly because it is not clear if the results when using $R_c/H_i$ are the same for $R_c$ being held constant and varying the value of $H_i$ and vice versa. To be able to use dimensionless parameters use is made of the nominal diameter of the primary armour, $D_{n50}$. This diameter is a breakwater property which is constant by structure and it determines the characteristics of the breakwater for a large part. This is clearly demonstrated by the stability factor, $H_i/\Delta D_{n50}$, as introduced in Chapter 8, which leads to the conclusion that a direct relation exists between the design wave height and the diameter of the stones.

Daemen (1991) starts with the relation between the transmission coefficient and $R_c/D_{n50}$, which appears to be linear for a constant wave steepness, $s_m$ (Figure G.12). This figure shows again that a smaller value of the wave steepness, which is the same as a larger value of the peak period, results in a higher transmission. Theoretically speaking the increase of the transmission coefficient should start at 0 (or a very small positive value as outlined earlier) and be expressed by a certain (smooth) curve to 1 for large negative values of the freeboard. Within certain limits ($-5 \leq R_c/D_{n50} \leq 5$) it is however possible
to derive a relation for the transmission coefficient which consists of two constant parts for relative large positive and negative values of $R_c/D_{n50}$ and a linear relation between these parts (for $-2 \leq R_c/D_{n50} \leq 2$) between $K_t$ and $R_c/D_{n50}$. The starting point is then given by:

$$K_t = aR_c/D_{n50} + b \quad \text{(G.7)}$$

Next the influence of incoming wave height is studied, see Figure G.13. As long as $R_c/D_{n50} > 0.5$ a higher wave height results in a higher transmission. If however $R_c/D_{n50} < 0.5$ the reverse is true. The wave height influences both the coefficient "a" and "b" of formula G.7.

The coefficient "a" is determined through the process of "curve fitting" which results in formula G.8.

$$a = 0.031 \frac{H}{D_{n50}} - 0.24 \quad \text{(G.8)}$$

---

3 This is caused by the fact that when regarding a submerged breakwater ($R_c < 0$), the higher waves are more influenced by it than the lower ones. These lower waves may pass the breakwater nearly unhindered. In other words, a larger incoming wave height results in a smaller transmission coefficient. If the freeboard is positive however, then the transmission is mainly determined by overtopping as a result of the run up which is larger than the crest height. In that case a larger wave height produces more overtopping and hence a larger transmission.
necessary. This is clearly shown in Figure G.14. The data from Powell and Allsop (1985) are not taken into account because uncertainties existed about the measured wave heights.

The crest width also influences the transmission, although this is only present in the results of tests on conventional breakwaters and not in the results of tests on reef breakwaters. This might be due to the fact that the data from Ahrens did not include large differences in crest width. Daemen points out that the same curve of line as the one for conventional breakwaters could be assumed, but since there is no further test data available this is not discussed any further. So the crest width value, B, is not present in the equation for the determination of the transmission coefficient for the case of a reef breakwater. This might be worth taking into consideration when results on reef breakwaters are examined. Taking all this into account the coefficient "b" was determined for both types individually. This results in the following formulae.

\[ K_t = a \frac{R}{D_{n50}} + b \]  \hspace{1cm} \text{(G.9.a)}

\[ a = 0.031 \frac{H}{D_{n50}} - 0.24 \]  \hspace{1cm} \text{(G.9.b)}

with for a conventional breakwater:

\[ b = -5.24s_{op} + 0.0323H/D_{n50} - 0.0017(B/D_{n50})^{1.84} + 0.51 \]  \hspace{1cm} \text{(G.9.c)}

with for a reef breakwater:

\[ b = -2.6s_{op} - 0.05H/D_{n50} + 0.85 \]  \hspace{1cm} \text{(G.9.d)}

These formulae are presented in Figure G.15. As can be seen in this figure (and in Figure G.16) there are distinct maximum and minimum values of \( K_t \). These are given by Daemen as:

- Conventional breakwater: \( K_{t_{\text{min}}} = 0.075 \), \( K_{t_{\text{max}}} = 0.75 \)
- Reef-type breakwater: \( K_{t_{\text{min}}} = 0.15 \), \( K_{t_{\text{max}}} = 0.60 \)

The formulae G.9 are valid for:

\[ 1 < \frac{H}{D_{n50}} < 6 \] \quad \text{and} \quad \[ 0.01 < s_{op} < 0.05 \]

The standard deviation of formulae G.9 is \( \sigma = 0.05 \). This means that the 90% confidence level can be given as \( K_t \pm 0.08 \). Compared to the formula G.6 of Van der Meer where the standard deviation was \( \sigma = 0.09 \) this is a much better result. Formulae G.9 can be used outside the given boundaries but the reliability will be lower according to Daemen. The upper boundaries are physically bound, because if \( \frac{H}{D_{n50}} > 6 \) then instability of the structure will occur and if \( s_{op} > 0.05 \) wave breaking will occur.
It does seem that the width of the breakwater has an effect on the maximum values of $K_t$, taking the data of Daemrich into account. This is shown in Figure G.16. This is something which might be worth to check out in the future, since this was not done by Daemen.

Figure G.14 Difference between reef breakwaters (data from Ahrens) and conventional breakwaters (rest of data)

Figure G.15 Wave transmission graph according to Daemen

Figure G.16 Minimum and maximum values of the transmission coefficient, $K_t$
APPENDIX H

REEF BREAKWATER WAVE TRANSMISSION COMPARISON
The same procedure as is outlined in Chapter 6 for a conventional submerged breakwater, is followed in this Appendix H for a reef breakwater. The two formulae which were used are the formulae from Daemen (1991) and Hearn (1987). A detailed description of these formulae is given in Appendix G.

Since the formula of Hearn (1987) is one of the used reference formulae, extra computations are necessary. For the incoming wave, $H_i$, the wave at the location where the transmitted wave, $H_t$, will be computed has to be determined for the same topographic situation but then without the breakwater present. So an extra run was made during which the wave heights were computed at the required locations without the presence of a breakwater.

For a description of the different breakwaters as used reference is made to Chapter 6. Only the results regarding the reef breakwater type are mentioned here.

- **Breakwater 1**

In the case of the reef breakwater the comparison of $K_t$ values is shown in Figure H.1. It is immediately clear that all the values computed with the formula of Daemen never differ from the maximum value of 0.6. This is the cause of the rather strange horizontal line as formed by the squares. The values of $K_t$ according to Hearn ($K_{th}$) do change as the $K_t$ UNIBEST changes. In general the values of $K_{th}$ are higher when regarding low values of $K_t$ and lower for the higher values.

![Figure H.1 Comparison of values of the transmission coefficient, $K_t$, according to the formulae of Daemen, Hearn and as computed with UNIBEST-TC](image)

The deviation between $K_t$ values is larger than the deviation displayed in Figure 6.4 for a conventional breakwater. The reason for this is that the formula from Hearn is derived specifically for reef breakwaters with a larger degree of permeability than the conventional breakwaters for which the formulae of Van der Meer and Daemen are meant. A reef breakwater, as mentioned in Chapter 1, consists of a homogeneous pile of stones so no
less permeable second armour layer or core is present. That is why it is logical that the
device from the K, UNIBEST values in Figure H.1 is larger than in Figure 6.4 since
the K, UNIBEST values are the same in both figures. The overall picture is not that bad
but the agreement is less than in the case of a conventional submerged breakwater.

-Breakwater 2

The comparison of data points is given in Figure H.2. Again the deviation is greater as
compared to the conventional breakwater case as shown in Figure 6.8. Again the K,
computed with the formula of Daemen has only one, the maximum, value of 0.6. The
data points show less deviation now as a result of the same reasons which hold true for
the conventional breakwater. The agreement is now better as compared to the first case
but the deviation is still rather large.

\[ \text{Figure H.2 Comparison of values of the transmission coefficient, } K_t, \text{ according to the}
\text{formulae of Daemen, Hearn and as computed with UNIBEST-TC} \]

-Breakwater 3

As can be seen in Figure H.3 the scatter at the value zero for the K, UNIBEST in the
case of a reef breakwater resembles the scatter as displayed in Figure 6.12, although the
device is even larger. As expected since the reef breakwater has a larger degree of
permeability than the conventional one, the deviation is also greater when the K, UNI-
BEST becomes smaller, i.e. when this permeability plays a larger role. The conclusion
that for small values of K, when using a reef breakwater, values computed with UNIB-
EST-TC do not give satisfactory results seems to be justified.

Another point of concern is the fact that the values as computed with the formula of
Daemen (Kd) still don't deviate from the maximum value of 0.6. Only two values differ,
namely 0.59 and 0.597 (see Appendix D). So it seems that in the region of small values
of K, UNIBEST the Kd values are too high and are thus given the maximum value of 0.6.
The conclusion that the formula of Hearn is more suited to this situation (cases 1 to 3)
than the one from Daemen seems to be justified.
COMPARISON $K_t$, THIRD DAM
Reef breakwater, with max./min. values

Figure H.3 Comparison of values of $K_t$ according to the formulae of Daemen, Hearn and as computed with UNIBEST-TC

Conclusion

If the type of breakwater used is not a conventional one but a reef breakwater, then only one approach may be used in order to determine how a profile will develop if a conventional breakwater and a nourishment behind it are present. Since in this case the degree of permeability plays a more important role and since this cannot be modelled in the UNIBEST-TC program, very large deviations at especially values of the transmission coefficient smaller than about 0.6 appear. So in this case a new wave climate has to be determined using the formula of Hearn for the waves passing the breakwater and only a part of the profile may be used in the computations as outlined as the second approach for the conventional submerged breakwater in Chapter 6. It is recommended to use the formula from Hearn and not the one from Daemen since the last has a very low maximum value. As a result this value is given nearly all of the time when it is clear that a certain different value should be used. It also seems that the values of the transmission coefficient according to the formula of Daemen tend to be very large and are thus scaled down to the low maximum value of 0.6.
APPENDIX I

ORDER OF MAGNITUDE VALUE OF THE TRANSITION ZONE WIDTH
APPENDIX I: ORDER OF MAGNITUDE VALUE OF THE TRANSITION ZONE WIDTH

I.1: Introduction

In this Appendix I an order of magnitude value of the transition zone width is derived. This is done so that a comparison with computed values with the UNIBEST-TC program is possible. It is a rather theoretical piece of work, mainly based on empirical approaches and on assumptions. If the reader is mainly interested in the results and not so much in the background, it is advised to skip this Appendix I and to only read Section 9.2.4. The formulae from Nairn, Roelvink and Southgate (1990), also mentioned as NRS (1990), which were given in Section 9.2.4 as formulae 9.2, are reprinted here for the sake of convenience.

I.2: NRS (1990) approach

NRS (1990) derive an empirical expression for the width of the transition zone. The width can be determined with the help of the ratio between the depth at the inner limit of the transition zone (d_i) to the depth at breaking point (d_b) which is determined as follows:

For \( \xi_{bb} \geq 0.05 \): \[ \frac{d_i}{d_b} = 0.47 \xi_{bb}^{-0.275} \] (I.1.a)

For \( \xi_{bb} \leq 0.05 \): \[ \frac{d_i}{d_b} = 1 \] (I.1.b)

with \( \xi_{bb} = \frac{m_b}{(H_b/L_b)^{0.5}} \) (I.1.c)

with

\[ \xi_{bb} = \text{surf similarity parameter at breakpoint} \] [-]

\[ m_b = \text{bottom slope in the vicinity of the breakpoint} \] [-]

\[ H_b = \text{wave height at breaking} \] [m]

\[ L_b = \text{wave length at breaking} \] [m]

If one assumes that the slope to be used in the surf similarity parameter at breakpoint, \( \xi_{bb} \), is equal to the slope of the front face of the breakwater (since this is the slope causing the waves to break and thus probably determining the transition zone width -this is treated in more detail further on-), a comparatively large value for this parameter will be found. This results in a small value of \( \frac{d_i}{d_b} \). Since the difference between \( d_i \) and \( d_b \) is then divided by the large value of the bottom slope, a comparatively small width is the result. If the same wave characteristics would be used but the slope would be much less steep causing the waves to break in a different manner, then although the ratio \( \frac{d_i}{d_b} \) is larger, the final width is larger due to the fact that the difference between \( d_i \) and \( d_b \) is now divided by the much smaller value of the bottom slope. So the result is that spilling breakers (the second case in which the bottom slope and thus the surf similarity parameter is smaller) will have a larger transition zone width than plunging ones\(^3\). Basco and

\(^3\)Spilling breakers are specified if the crest of the wave spills or rolls down the front face of the wave. Plunging breakers are specified if the wave becomes vertical, rotates forward and plunges into the base of the wave.

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Yamashita (1986) also show this by giving examples of both spilling and plunging breakers.

I.3: Breaker type

Since the front face of the breakwater is so steep, all the waves from the wave classes as used in this study (Table 3.1) will fall into the category of plunging breakers as defined by Battjes (1974) as 0.5 < \( \xi_0 \) < 3.0. \( \xi_0 \) is defined by \( m/(H_d/L_0)^{1/2} \) where the subscript 0 denotes deep water values and \( m \) equals the beach slope. Smith and Kraus (1991) show however that if the slope of the front face of a submerged breakwater (or bar for that matter) is used as \( m \) for \( \xi_0 \), the transition values between the different breaker types will be significantly lower. In other words, some waves that will break by spilling on a plane slope will plunge when a bar is present. This is one of their results of an extensive study on differences between wave properties breaking on regular beach profiles and on plane profiles. Smith and Kraus (1991) define the transition values for barred profiles as:

\[
\begin{align*}
\xi_0 &> 1.2 & : \text{surging or collapsing} \\
0.4 &< \xi_0 &< 1.2 & : \text{plunging} \\
\xi_0 &< 0.4 & : \text{spilling}
\end{align*}
\]

Because the transition values have now decreased, some of the waves used even belong to the collapsing category. Since the surf similarity parameter at breakpoint for a certain wave is the same regardless of the type of breaking, the outcome of formulae I.1 will not be influenced. Qualitatively these new boundaries have influence though. If a surf similarity parameter for a certain wave is equal to 0.45, one would assume it to be a spilling breaker according to Battjes (1974). According to the new transition values it would belong to the plunging class and hence a smaller transition zone width has to be expected.

All the waves as used belong to either the plunging or the collapsing class as given by Smith and Kraus (1991) when using the slope of the front face of the breakwater in \( \xi_0 \). If this value of the slope is also used to determine the transition zone width after computing the value of \( d/d_n \), the first problem of the three problems mentioned below, arises.

I.4: Problems when using the NRS (1990) approach

Three problems arise; the first is that the testing range of formulae I.1 lies between bottom slope values of 0.005 and 0.05. So the slope of the breakwater (\( m_s = 0.3 - 0.5 \)) is not within this range. Second, twice abrupt changes in the bottom slope appear, the first one at the point where the seaward slope of the breakwater reaches crest height, and the second one where the crest changes into the shoreward slope. Third, the depth is not monotonically decreasing but an increase of the depth just behind the breakwater is found, before a decrease appears again.

I.5: Possible solutions for the problems

Since this value of the slope is not within the validity range, two approaches may be used, either using the minimum measured value of about 0.4 (NRS [1990]), or using formulae I.1 and examining the outcome critically. Since the formulae tends to over...
estimate values of the transition zone width occurring at high values of the surf similarity parameter at breaking, $\xi_{bb}$ (NRS [1990]), the first approach is probably the better one.

Now that a certain value of $d/d_b$ can be computed and hence the theoretical value of the depth at the inner limit of the transition zone, $d$, is known, the third problem arises. If one would assume the computed value of $d$ to be right, the width would probably be far too great since the bottom slope first decreases behind the breakwater and then slowly increases again. So before this certain absolute value of $d$ is reached, a large horizontal distance has to be crossed. As a result this value of $d$ should not be seen as the absolute shoreward boundary of the transition zone, but should only be used in the dimensionless width $d/d_b$ to determine the absolute value of the width using the bottom slope.

Now the problem arises that the bottom slope is not constant but changes rapidly twice. Furthermore the question arises what the slope of the crest of the breakwater should be. If the assumption is made that the crest will be more or less horizontal, then the slope will be very small and thus the transition width very large. The latter is the case if the slope of the crest is used to determine the transition zone width since this is the slope just shoreward of the point of wave breaking. Since the breakwater armour layer consists of rather large rocks (Chapter 8) it will be difficult to determine a value for the slope, $m_b$. If however the $m_b$ of the slope on which the waves break is used, i.e. in this case the slope of the front face of the breakwater, the slope value is rather large and thus the width will be much smaller. For example when considering a wave of $H_{m0} = 2.25$ m and $T_{m01} = 6$ s, it follows from computations that $\xi_{bb} = 1.67$. According to formulae I.1.a $d/d_b = 0.41$ which is about the same as the minimum measured value of 0.4. Assuming this 0.41 to be correct the transition zone width would amount to 3.6 m if the front face slope is used and to 60 m if a crest slope of 0.02 is taken. This is a very large difference. It is not clear which approach of the two is right. Further investigation and model tests will be necessary to determine that.

I.6: Qualitative assumption

For now only a qualitative assumption can be made. Figure 1.1 has been printed to illustrate the following line of thought.

Since the slope of the front face of the breakwater is the cause of the type of wave breaking (plunging breakers, pictures I.1.1/2/3), it also determines the speed and the amount of water which generates the new secondary surface disturbance (pictures I.1.5/-6/7/8) as a result of the formed vortex. This vortex is the rotating fluid mass system also displayed in Figure 1.1. To be more precise actually two vortices are formed. The first is the plunger vortex which starts to emerge at picture I.1.4. The second is the subsequently generated surface roller vortex which starts at picture I.1.5 and travels with the surface roller. The transition point occurs just after picture I.1.8 because at that point a new wave with different kinematics is formed in a more or less equilibrium state. This new wave is the bore-like wave as mentioned by Svendsen (1984).

One could argue that thus the slope which determines the kind of wave breaking (in this case the front face slope) should also be used when computing the width of the transition zone using the ratio $d/d_b$. The influence of the slope just shoreward of the breakpoint will probably not be negligible though, because if it is less steep the transition process will

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travel over a larger distance in the same time compared to when the slope is steeper, i.e. the width will be larger. However, it does seem likely that the slope of the front face plays a much bigger role in the process than the slope of the crest of the breakwater. This means that the transition zone width is probably rather small in this case of the submerged breakwater as outlined in the computed example before.

The exact width will probably be larger than when only the high slope value of the front face of the breakwater is used (3.6 m), but definitely smaller then when only the slope of the crest is used (about 60 m, dependent on what value is taken for the slope of the crest). Basco (1985) proposes values of the transition zone width of 10 till 30% of the wave length at breaking, which in the example amounts to 5.6 till 16.8 m and thus would fit between the two earlier computed values of 3.6 and 60 m.
These percentages are derived for plane, monotonically depth decreasing beaches. Smith and Kraus (1991) point out that the plunge distance on a barred profile is 60-70% shorter than on plane slopes for the same value of the surf similarity parameter. This plunge distance is defined as the distance from the breakpoint to the touch down point of the crest and thus forms an important part of the transition zone. In other words, the percentage range as given by Basco (1985) probably over estimates the transition zone width and so the value for this width will probably be even less than the ones stated above (5.6-16.8 m).

I.7: Conclusion

All in all the above stated supports the assumption that the slope of the front face of the breakwater should play a more important role than the slope of the crest when determining the transition zone width using the NRS (1990) approach. The value for this width should thus indeed be rather small, in the order of 10 m. Again the point is stressed that this is only a qualitative assumption and further investigation as well as model tests are necessary to confirm this assumption.