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DYNAMICALLY STABLE STRUCTURES AND THEIR NATURAL RESPONSE TO WAVE ATTACK

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ABSTRACT: Wave interaction with dynamically stable structures is studied by means of a verified numerical model. The model can simulate wave motion on and in permeable structures. For dynamically stable structures, including gravel beaches, berm breakwaters and reef-type structures, a procedure is developed to simulate the natural response to wave attack. Also a verification with prototype measurements is performed.

1.0 INTRODUCTION

Dynamically stable structures such as berm breakwaters and reef-type structures are a relatively new type of structure. Their natural response to hydrodynamic loads makes them economically attractive not in the least because smaller rock material can be used than with conventional coastal structures. On the other hand, the dynamic behaviour requires special attention. For berm breakwaters the seaward slope undergoes reshaping until it fits the wave conditions. This dynamic behaviour of the seaward slope is very much depending on the hydrodynamic loads and vice versa. This interactive character of the hydrodynamics and the reshaping process are studied here by means of a new numerical wave load-response model.

A verified wave model has been combined with a procedure to simulate the response of dynamic structures. Procedures for initiation of movement of individual stones and for the reshaping of the seaward slope as a result of moving stones along the slopes determine the response of the structure. Both the wave motion and the response of the structure are simulated in the time-domain which means that a response of the structure immediately effects the computed wave motion.

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2.0 NUMERICAL WAVE MODEL

A numerical model for simulating wave motion on and inside permeable structures is described in Van Gent (1994, 1995-b). The wave dynamics of normally incident wave attack on various types of structures are approximated by the non-linear shallow-water equations with steep wave fronts represented by bores. The model is based on concepts by Hibberd and Peregrine (1979) who developed a numerical model with an explicit dissipative finite-difference scheme (Lax-Wendroff) for impermeable slopes without friction. Using this concept, many practical applications have been obtained, see for instance Kobayashi *et al.* (1987) for wave reflection and run-up on impermeable rough slopes. For the permeable part of a structure the same types of equations can be applied although the friction coefficients for the porous medium need to be assessed, see Van Gent (1995-a). A proper coupling of the external flow to the internal flow is applied in Van Gent (1994, 1995-b).

After several validations, applications and extensions, the model (ODIFLOCS) has now become a user-friendly P.C.-model for both practice-based and research-based engineers. The model is able to deal with waves either regular or irregular which attack various types of structures with arbitrary seaward slopes, smooth or rough, permeable or impermeable, overtopped or not. Although the model uses a one-dimensional description of the flow it can, however, give a useful impression of the flow field in two dimensions, see Figure 1 and Figure 2.

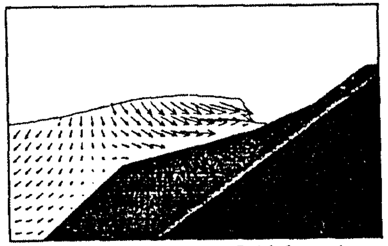

Fig.1 Computed flow-field for a berm breakwater with an impermeable core.

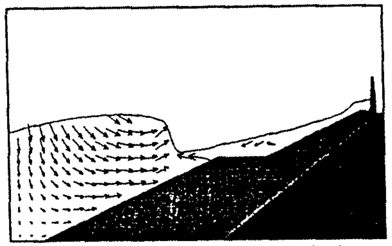

Fig.2 Computed flow-field for a permeable breakwater with crown-wall.

3.0 NUMERICAL RESPONSE MODEL

3.1 APPROACH FOR SIMULATING PROFILE DEVELOPMENT

The stability of the stones is strongly dependent on the hydrodynamic properties. Several expressions for this stability have been developed. Iribarren (1938) and Hudson (1953) derived widely used expressions where the hydrodynamic

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properties are represented by the wave height. Van der Meer (1988) performed many laboratory tests to study the influence of other hydrodynamic properties as well. The results were summarised in empirical relations which also contain hydraulic parameters like the wave period and number of waves. Although these design recommendations are rather accurate for many applications, more generally applicable results can be obtained by simulating the wave motion first and then using flow properties like the velocities and accelerations to predict forces on stones. This can be done numerically.

Results obtained from such a numerical approach might be less hampered by scale-effects than those from physical model tests on a small-scale. Also the sensitivity to parameter variations of the reshaping process, like for instance the permeability of structures, can be studied more easily with a numerical model than with physical model tests. Such a numerical model can also be applied for cases for which no empirical relations exist, like for instance for structures or beaches which contain large immovable components such as gravel beaches fronting seawalls or rubble mound slopes in front of rigid crest elements.

In the approach towards a numerical wave load-response model several model formulations are required. Firstly, the hydrodynamic flow, both outside and inside the structure, need to be known and modelled numerically. The mentioned one-dimensional model can be used as a first approximation. Secondly, information concerning the magnitude of forces on stones is necessary. Attempts to measure forces on idealised stones have been made by Sigurdsson (1962) and Sandstrøm (1974). Tørum (1992) measured forces on a single stone in the cover layer of a berm breakwater. Thirdly, relations between the forces on stones and the hydrodynamic behaviour are necessary. As mentioned before, the hydrodynamics can be represented by local velocities and local accelerations. As a first approximation, a Morison-type of expression (Morison *et al.*, 1950) can be used, see for instance Kobayashi and Otta (1987) or Tørum (1992). Fourthly, information concerning failure mechanisms and forces causing damage is needed. Often failure mechanisms referred to as *rolling*, *sliding* or *lifting* are distinguished. These mechanisms or other failure mechanisms need to be modelled. Finally, the new positions of unstable stones need to be known if the complete reshaping process is to be simulated. For most breakwaters no severe damage is allowed, so for those cases it is not of primary interest to study the new positions of the stones. However, for berm breakwaters and gravel beaches these new positions are of primary concern.

Norton and Holmes (1992) described a simulation model for the reshaping process of berm breakwaters under normally incident, monochromatic wave attack by modelling individual displacements of stones based on a Morison-type of equation. In the present model, initiation of movement of stones is also based on a Morison-type of equation including drag, inertia and lift forces. However, in contrast to the approach by Norton and Holmes (1992), the present model can also be applied using irregular waves since it simulates the reshaping process in the time-domain. Furthermore, in the model described here the new positions of unstable stones are determined by the hydrodynamics.

3.2 MODELLING OF FORCES ON STONES

The hydrodynamic loadings on a single stone can be modelled using a number of forces representing different phenomena. For the relation between the hydrodynamics and the forces, local velocities and local accelerations are required. The numerical model provides these local properties although averaged over the depth. Differences between these properties at the position of the particles and the depth-averaged velocities naturally cause inaccuracies.

Three forces as a result of the hydrodynamic loadings have been distinguished; the drag force acting parallel to the slope in the direction of the velocity, the inertia force acting parallel to the slope and the lift force acting perpendicular to the slope. For the drag force and the inertia force expressions similar as in the Morison equation can be used. The lift force is the most difficult one to determine. Often, the assumption that the lift force is proportional to the squared velocity and the squared diameter of the stone is used.

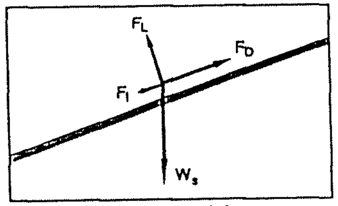

Fig.3 Forces on particle.

$$F_D = \frac{1}{2} \rho \, c_D \, k_2 \, D^2 \, u |u| \tag{1}$$

$$F_I = \rho \, c_M \, k_1 \, D^3 \, \frac{Du}{Dt} \tag{2}$$

$$F_L = \frac{1}{2} \rho \, c_L \, k_2 \, D^2 \, u^2 \tag{3}$$

where the acceleration Du/Dt is approximated with $\partial u/\partial t$; c_D, c_M, c_L are the drag coefficient, the inertia coefficient and the lift coefficient respectively; k_1 and k_2 are the volume shape factor and the area shape factor respectively. With the area shape factor k_2 the actual projected area in the flow direction can be incorporated. Since a cover particle is partially sheltered by other particles, the actual projected area is smaller than for a single particle in a flow. The sheltering effect has not been incorporated separately and therefore affects the values of the coefficients which will be derived through calibration. For spheres, the value for k_2 is $\pi/4$ since the projected area, neglecting the sheltering effect, is $\pi/4 \, D^2$. The volume shape factor k_1 is $\pi/6$ for spheres since its volume is equal to $\pi/6 \, D^3$. For stones slightly higher

values must be used: $k_1=0.66$ and $k_2=0.9$ were used in all computations. The stone diameter is taken constant while the equivalent sphere diameter D_{EQ} is taken as the characteristic stone size ($D_{EQ} \approx 1.24 \cdot D_{n50}$).

The submerged weight is often taken as the counter-acting force although occasionally other counteracting forces have been proposed, see for instance Brandtzaeg and Tørum (1966). The submerged weight acts vertically and can be written as (ρ_s represents the density of the stone material):

$$W_s = (\rho_s - \rho) \, g \, k_1 \, D^3 \tag{4}$$

Several concepts can be used for initiation of movement. Stability criteria for the phenomena referred to as *lifting* and *sliding* can respectively be expressed by:

$$F_L \le W_s \cos\phi \tag{5}$$

$$|F_D + F_I - W_s \sin\phi| \le \tan\mu \, (W_s \cos\phi - F_L) \tag{6}$$

where μ denotes the angle of internal friction and ϕ the local slope angle. Here, the phenomenon referred to as *rolling* is assumed to occur if both stability conditions are not satisfied.

An additional force is implemented at the intersection of the free surface with the slope (wave front). The first particle near the wave front is assumed not to be submerged ($W=\rho_s g k_1 D^3$). If velocities are in the direction of the particle, the pressure at the wet side of the particle is expressed by the pressure thrust approximated by $0.5 \cdot \rho g h^2 D + \rho u^2 h D$. If this force, acting parallel to the slope, exceeds the counteracting component of the weight of the particle, the particle is regarded as unstable. For unstable particles, the direction in which they will possibly move, has to be determined.

3.3 MODELLING OF STONE DISPLACEMENTS

In this section a method is discussed to simulate stone displacements on the seaward slope of structures and gravel beaches. Initiation of movement is calculated as described in the previous sub-section.

In general the flow pattern around particles is very complex. Therefore, the forces as a result of the pressure gradients around the particles are not easy to determine. In the model for the initiation of movement the drag, inertia and lift forces are the result of these pressure gradients. For particles moving along the slope other forces might be of importance. It is assumed that for particles moving along the slope the pressure gradient directly depending on the slope of the free surface (hydrostatic pressures) is of more importance than for stable stones in the cover layer. Therefore, such a force (Froude-Krylov force) is taken into account to determine the direction in which the unstable particles will move. This force is assumed to act parallel to the slope and acting on the volume of the particle. Calibration, however, will show that this force is of minor importance compared to the magnitude of the other forces.

$$F_P = \rho \; c_P \; g \; k_1 \; D^3 \; \frac{\partial \eta}{\partial x} \tag{7}$$

where η is the free surface elevation and c_P is a coefficient to be determined through calibration. This force as well as the drag and inertia forces and the weight of the stone determine in which direction an unstable particle will move after one of the stability criteria is (*Eq.5* and/or *Eq.6*) not satisfied:

$$|F_D + F_I - F_P - W_s \sin\phi| > 0 \;\rightarrow\; \text{UPWARD} \tag{8}$$

$$|F_D + F_I - F_P - W_s \sin\phi| < 0 \;\rightarrow\; \text{DOWNWARD} \tag{9}$$

The inertia force and the drag force act probably differently on non-moving stones in the cover layer than on stones moving along the slope. However, as a first approximation the same formulations and the same values for the drag and inertia coefficients are applied for initiation of stones as well as for moving stones; the formulations for the drag and inertia forces and the coefficients c_D and c_M in Equations 1 and 2 are also used in Equations 8 and 9.

After determining the direction in which an unstable particle may move, the local hydrodynamic properties at a position one space-increment (Δx) away from the original position, will be regarded. It is verified whether the particle would be stable or unstable in that neighbouring position. If the particle is stable at that position, the particle will stay at its original position. If the particle is also unstable at the neighbouring position the particle will be moved to this position. This is done without any time-delay which means that the particle is moved over a space-increment Δx within a period of Δt. The choice of Δt depends on the space-increment Δx and the wave celerity which means that the velocity of the stones is in fact related to the (average) wave celerity (see Section 4.3.2).

This response/morphological model for cross-structure transport is interactive with the hydraulic model. At each time-step (Δt) the hydraulic properties are determined at all positions. At each position and each time-step it is verified whether the particles are stable at their present position or not and whether they need to be displaced. The profile changes due to the movement of the particles while the new profile is immediately incorporated in the hydraulic model.

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Some numerical problems remain. Particles are moved from over one space increment within one time-step. The space-increment Δz by which the profile is adapted in the vertical direction must still be determined. The space-increment Δx is generally not equal to the size of the particles. For instance for small material several particles are positioned within one space-increment Δx. For the space-increment Δz a value is taken such that an area in the cross-section equal to $D \cdot \Delta x$ is replaced with in a period $\Delta x / u_r$. For the velocity u_r a representative velocity of $\sqrt{(g \cdot H_{rms})}$ is used although in principle a time and space dependent velocity of the particles can be applied here.

Another numerical problem occurs in the case the concept is used in combination with a one-dimensional hydraulic model. For relatively large particles compared to the wave height and compared to the numerical space-increment Δx, the variation in the vertical direction Δz may disturb the hydraulic model to an unacceptable degree. The applied numerical model is a one-dimensional hydrostatic model without solving a non-hydrostatic momentum-equation in the vertical direction and is therefore relatively sensitive (causing inaccuracies) for abrupt changes of the profile. This means that for relatively large particles, the space-increment Δz must be decreased. This leads to a slower response of the (numerical) structure to a certain wave climate. This can be partially overcome by increasing the total simulation-time but if the relation between a smaller space-increment Δz and the profile adjustment-time is not linear, the development in time is not correct and therefore less suitable for studying the development in time of structures with relatively large particles. Several comparisons indicated that this relation was close to linear and therefore this does not effect the accuracy of the results seriously.

4.0 CALIBRATION OF THE WAVE LOAD-RESPONSE MODEL

Dynamically stable profiles can in first instance be classified using the parameter $H_s / \Delta D_{n50}$. For dynamically stable profiles this value varies roughly between *3* and *500*. This parameter varies between *4* and *6* for berm breakwaters. For gravel beaches this value is higher. Since the procedure described in the previous sections can in principle be applied to processes where suspension transport can be neglected, also gravel beaches can be dealt with. For calibration of the model a gravel beach was taken instead of a berm breakwater slope since for gravel beaches much more displacements occur and material is often transported both upward and downward.

Tests performed by Van der Meer (1988) are used for calibration and validation of the described morphological model. The coefficients that need to be determined through calibration are the coefficients c_D, c_L, c_M and c_P. The combination of the coefficients derived from the calibration test are used in other computations where several parameters vary. Van der Meer (1988) derived expressions for the prediction of reshaped profiles from his test-results. Conditions that these expressions have been derived from are used for comparison. Since for these tests the differences between the measured profiles and the profiles prescribed by the expressions are relatively small, the expressions have been used for convenience.

The computations for both calibration and validation have been done with a TMA-spectrum although physical model tests have been performed with different spectra. However, neither Van der Meer (1988) nor Kao and Hall (1990) observed a clear influence of the spectral shape. Therefore, these dissimilar spectra are supposed not to contribute to possible deviations between the data from the measurements and computational results. The spectra are represented by the significant wave height H_s and the mean wave period T_m. The material is characterised by the D_{n50}. The computations have been performed for approximately *500* waves. For the friction coefficients in the porous medium and for the added mass coefficient, the expressions given in Van Gent (1995-a) have been used.

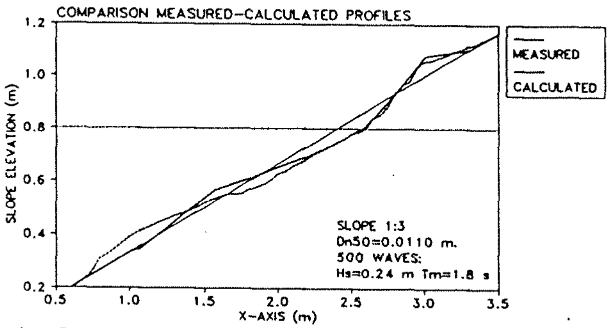

Fig.4 *Comparison of profiles.*

For calibration, a test is used where in the dynamically stable situation accretion occurs both above the still water level and below the still water level. The test concerns a uniform *1:3* slope with material with a diameter of *0.0110 m* (D_{n50}). The wave height H_s, the wave period T_m and the still water level were *0.24 m*, *1.8 s* and *0.80 m* respectively ($H_s/\Delta D_{n50}=13.2$). The friction factor f was set at *0.10*. Figure 4 shows the reshaped profile after *500* waves. For the coefficients the following values were found: $c_D=0.018$, $c_L=0.075$, $c_M=0.08$ and $c_P=0.01$ (with an angle of internal friction of $\mu=50°$).

5.0 VALIDATION OF THE WAVE LOAD-RESPONSE MODEL

In Van Gent (1993), *20* comparisons between the profiles derived from the expressions and the simulated profiles were presented. Tests with the stone sizes

$D_{n50}=0.0041$ *m*, *0.0062 m*, *0.0110 m* and *0.0257 m* were used. The initial slopes were *1:5*, *1:3* and *1:1.5*. Wave spectra represented by *500* waves with combinations of $H_s=0.14$ *m*, *0.18 m*, *0.24 m* and $T_m=1.3$ *s*, *1.8 s*, *2.5 s* and *3.0 s* were simulated. For all computations the friction coefficient f was set at *0.10*.

The above mentioned combination of the four coefficients has been used in *16* of the *20* simulations. For the simulations with the larger material, $D_{n50}=0.0257$ *m*, the lifting process appeared to be underestimated. An adaption to the calibrated lift coefficient was made; for material larger than $D_{n50}=0.0110$ *m*, a linear relation is used as a first approximation: $c_L=7.85 \cdot D_{n50}$ with a maximum value for c_L of *0.38*. This relation appeared to give rather good results but it is in fact a procedure without a proper physical background.

As expected, differences occurred between the measured and calculated profiles. However, in most cases the trends were the same; in most cases accretion and erosion took place in roughly the same sections as observed in the measurements. For the computations with *1:5* slopes, often both the accretion and the erosion were underestimated; for the computations with the *1:3* slopes, the section above the still-water level was rather good but the accretion below the still was positioned too much downward; the computations with the *1:1.5* slopes showed both above and below the still-water level a rather good comparison. In general, it seemed as if accretion was underestimated in cases where it occurred above the still-water level whereas it was overestimated where accretion occurred further down the slope. This conclusion was, however, not valid for all simulations.

For the calculation of forces, the model uses depth-averaged velocities rather than velocities near the bottom (slope). During up-rush, it is expected that a depth-averaged velocity is a rather good characteristic velocity in the run-up area. More downward along the slope, the layer of water above the slope becomes thicker. Here, the depth-averaged velocity may differ much more from the velocity near the bottom. This may be an explanation for the relatively weaker correspondence with the measurements for the section below the still water level in comparison with the slightly better results above the still water level.

Besides the validation with *20* cases with initially uniform slopes, some additional validations will be discussed in the subsequent sections. In Van Gent (1995-b) a sensitivity analysis has been described. It appeared that the model is relatively sensitive to the values of the model parameters c_D (drag-coefficient) and c_L (lift-coefficient) as well as to the value of the bottom friction factor f. To obtain a significant influence of a variation of the inertia coefficient c_M and the porosity n, these parameters must be varied over a large range. The sensitivity to the coefficients c_P, the angle of internal friction, the implementation of the phenomenon *added mass* in the wave model and the implementation of the flow-dependency of the porous-flow friction coefficients in the wave model appeared to be very small.

6.0 APPLICATIONS WITH THE WAVE LOAD-RESPONSE MODEL

6.1 PROFILE DEVELOPMENT OF GRAVEL BEACHES

Most of the computations for model calibration and validation as described in the previous section concern gravel beaches. In addition to those comparisons and the sensitivity analysis, in this section a qualitative verification is given. It is verified whether the influence of variation of parameters corresponds to those observed in physical model tests. The parameters wave height, wave period, stone diameter and the initial slope have been varied, while the other parameters were kept constant.

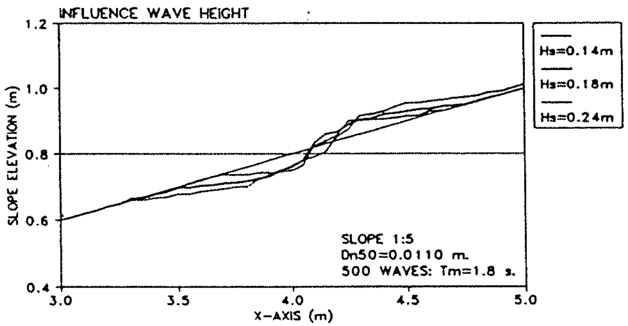

Fig.5 Influence of the wave height on calculated profiles.

In Van Gent (1993, 1995-b) several figures with the results of this parametric study are given. Figure 5 shows an example of the influence of variations in the wave height. It is clearly shown that an increased wave height leads to longer reshaped profiles. This was also observed in physical model tests described by Van der Meer (1988). The same trend occurs for longer wave periods. This trend occurs also in the numerical simulations. Figure 6 shows an example of reshaped profiles with variations of the wave periods. The stone diameter was also varied. Figure 7 shows reshaped profiles after *500* waves with $H_s = 0.24\ m$ and $T_m = 1.8\ s$ and an initial slope of *1:3*. The figure shows that smaller material results in a more affected slope. This was also observed in the physical model tests. The simulations show that smaller material leads to more accretion below the still water level. For two stone sizes, the initial slope has been varied as well. Initial slopes of *1:5*, *1:3* and *1:1.5* were used. Figure 8 shows an example of such a comparison. The figure shows that reshaped profile near the still water 'shoreline' is hardly influenced by the initial slope. This was also observed in physical model tests. Further upward or downward, the reshaped profiles evolve more towards the initial slope.

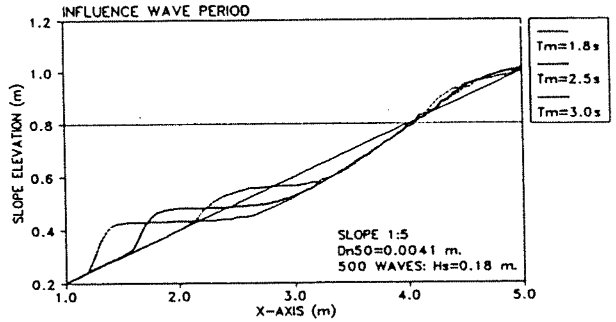

Fig.6 Influence of the wave period on calculated profiles.

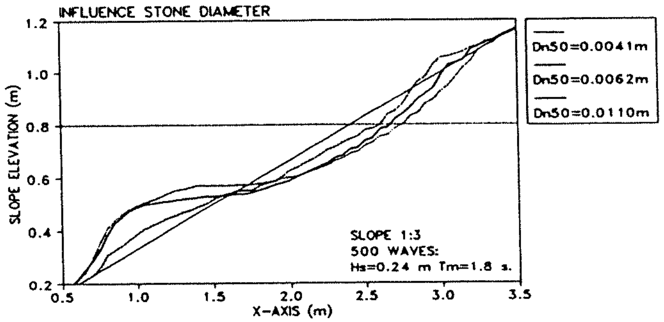

Fig.7 Influence of stone diameter on calculated profiles.

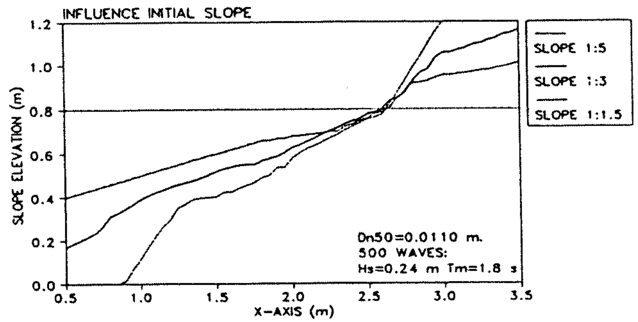

Fig.8 Influence of initial slope on calculated profiles.

It can be concluded that the simulations described in this section show that the variation of the parameters wave height, wave period, stone diameter and initial slope show the same trends as observed in physical model tests with gravel beaches.

6.2 PROFILE DEVELOPMENT OF BERM BREAKWATERS

Most of the computations described in the previous sections were performed with relatively high values of $H/\Delta D_{n50}$ which represent gravel beaches. Although some reshaped profiles from calculations are similar to those of berm breakwaters, no validation was described for berm breakwaters where the initial slope contained a horizontal berm. Two additional verifications with berm breakwaters will be discussed, one for the small-scale model and one for a prototype breakwater.

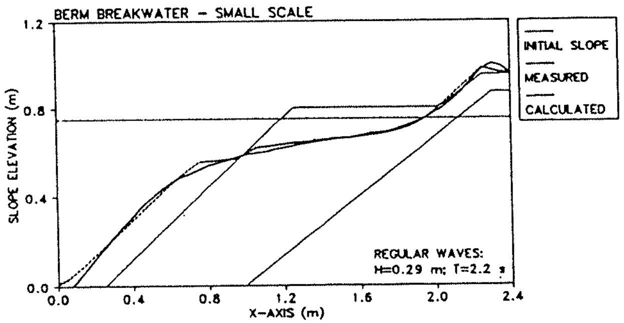

Fig.9 *Comparison of measured and calculated profiles.*

Firstly, the berm breakwater from small-scale physical model tests, described in Van Gent (1993), is treated. Regular waves were generated unlike in all other computations. In the physical model tests, the reshaped profile was formed after four series of regular waves where the effect on the reshaped seaward slope increased for each subsequent series. In the computation, the last wave series which determined the final reshaped profile, has been used; $H=0.29\ m$ and $T=2.2\ s$. The numerical model cannot deal with two layers with different properties of the porous media. Therefore, the structure must be regarded as homogeneous or as a structure with an impermeable core. Here, the berm breakwater was modelled as homogeneous since the permeability of the core material is rather close to the permeability of the material in the cover layer.

In the physical model tests as well as in the computation, the reshaped profile became dynamically stable after a limited number of waves. In the computations this was after approximately *70* waves. Figure 9 shows the comparison of the measured

profile and the simulated profile where for the friction coefficient f the value *0.3* was taken above SWL and *0.1* below SWL. The increase in crest height as observed in the measurements is underestimated. This increase is larger if for the friction coefficient $f=0.1$ would have been taken also above SWL but then the computational results show a slightly less accurate fit in the region of the berm. Although the value of the friction coefficient affects the numerically reshaped profile, the reshaping process is represented rather well; the comparison shows good agreement.

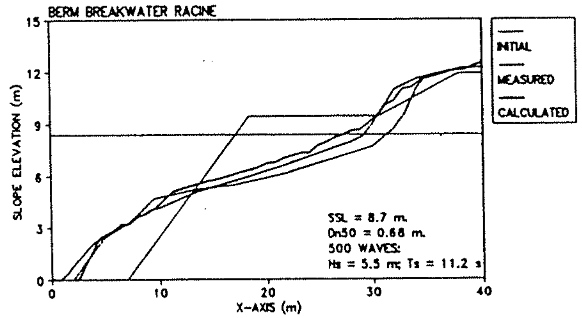

Fig.10 *Comparison of measured and calculated reshaped seaward slopes for the berm breakwater at Racine (USA).*

Measured reshaped profiles for the berm breakwater at Racine, presented by Montgomery *et al.* (1988), were used for comparison. After a storm characterised by $H_s=5.5\ m$; $T_s=11.2\ s$ and SSL$=8.7\ m$ (determined with a hindcast method using measured storm wind data) the two dashed lines shown in Figure 10 were measured. The structure, with stones with a D_{n50} of *0.68 m*, was modelled as homogeneous. For the friction coefficient f, the value *0.4* was used. Figure 10 which shows both the two measured profiles and the calculated profile shows fair agreement.

6.3 PROFILE DEVELOPMENT OF REEF-TYPE STRUCTURES

Reef-type structures can be described as a pile of stones which undergoes reshaping due to wave action. Its initial crest is above the still water level but severe wave action causes reshaping of the structure which mostly leads to lowering of the crest height to a level which is permanently below the still water level.

Wave transmission over such low-crested or submerged structures has been studied by means of the one-dimensional hydraulic model (see Van Gent, 1995-b). Also the reshaping of this type of structures can be modelled numerically by applying the described approach. Some experiments with small-scale models by

Ahrens (1987) have been used to validate the model for this type of structures. Irregular waves where generated on a water-depth of about *0.50 m*, travelling over a *1:15* slope before reaching the structure positioned on a horizontal bottom in a water-depth of about *0.25 m*. The structures reshaped to a new dynamically stable profile of which the new crest height was measured as well as the damaged area of the structure. Also the transmission coefficients were determined.

Four experiments with a relatively large deformation of the structure have been used for comparison with numerical model results. Figure 11 shows the initial cross-section of the structures and the numerically reshaped cross-section after reaching a dynamically stable profile. Based on measured wave parameters irregular wave trains were generated representing a TMA-spectrum. In the computation these waves were generated at about *1.0 m* in front of the structure while an open boundary was applied at *0.5 m* behind the structure. In the physical model tests the structure was positioned at about *30 m* from the wave generator. This difference causes that wave set-up is not modelled equally and causes some additional differences between the wave field in the physical model tests and numerical model.

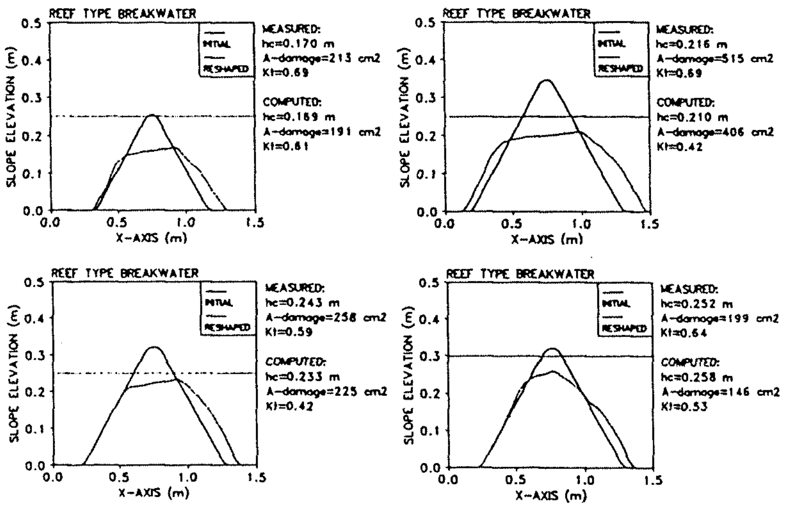

Fig.11 Computations with reef-type structures, initial profiles and reshaped profiles (stable) after approximately 300 waves.

Computations with two types of rubble mound material were performed. The two upper graphs in Figure 11 had material with a D_{n50} of *0.018 m* and a porosity of $n=0.45$. In the computations a bottom friction factor of $f=0.20$ was applied for these two computations. The wave trains were characterised by $H_s=0.153$ *m* and $T_p=3.0$ *s* on a depth of SWL=*0.25 m*. Two computations were performed with slightly larger material, $D_{n50}=0.030$ *m*, and a porosity of $n=0.44$. The friction factor was set at $f=0.25$ for these two computations. The results are shown in the two lower graphs of Figure 11. The bottom left computation had waves with $H_s=0.158$ *m* and $T_p=3.6$ *s* on a depth of SWL=*0.25 m*; the bottom right computation had a wave train with $H_s=0.176$ *m* and $T_p=3.3$ *s* on a depth of SWL=*0.30 m*. For all four computations the angle of internal friction was $\mu=50°$, the grid size was $\Delta x=0.02$ *m* and the time-step $\Delta t=0.005$ *s* while approximately *300* waves were computed.

Figure 11 shows that in all four computations a relatively large lowering of the crest occurred (from $h'_c=0.257$ *m* to $h_c=0.169$ *m*; from $h'_c=0.350$ *m* to $h_c=0.210$ *m*; from $h'_c=0.314$ *m* to $h_c=0.233$ *m* and from $h'_c=0.316$ *m* to $h_c=0.258$ *m* respectively). The difference between measured and computed crest heights after reshaping is smaller than *5%*. The damage to the initial cross-section was in all computations lower than those in the measurements (in average about *20%* too low). The wave transmission was in all four computations significantly lower than those obtained from the measurements (in average about *25%* too low). Although the wave transmission is underestimated, the reshaping of the structures seems to be represented rather accurately.

7.0 CONCLUDING REMARKS

A numerical model for simulating both individual waves and the time-dependent response of dynamic structures is developed. Although the formulations for simulating waves and the wave loads as well as the formulations for simulating the response of the structure are all rather simple, the computations indicate that these simplifications do not make such an approach unrealistic. A qualitative validation of the integrated model showed that the influence of variations of physical parameters are reproduced properly. A quantitative validation showed that for highly dynamic slopes as for instance gravel beaches (large $H/\Delta D$), differences occur that might be expected for such a relatively simple model. Comparisons with measured properties of less dynamic slopes as for instance those of berm breakwaters and reef-type structures show fair agreement. Although the presented model can be improved, it can already be applied as a complementary design-tool, especially for conditions for which no empirical relations for describing the reshaping process exist.

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DYNAMIC DESIGN OF BREAKWATERS

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ABSTRACT

As coastal engineering has been grown in the last decades, new designing techniques have been devoloped. A harbour breakwater is generally designed according to its static stability. This kind of designing, however, generally have given rise to very large stone dimensions which lead to great costs. In order to minimize armour dimensions and costs of breakwaters, new designing techniques are being developed in the last years. Dynamic designing is one of the most important techniques employed in breakwaters.

In this study, dynamic behaviour and designing of breakwaters are experimentally studied in a physical model. The model studies were conducted in Hydraulic Laboratory of Karadeniz Technical University Civil Engineering Department. A wave generating system was established in a flume of which dimensions were 30*2.40*1.40m. Considering dimensions of section of an average breakwater and the flume, undistorted model scale was chosen 1/50. The bed slope near the breakwater was 1/40. Armour stones with 2-4, 4-6, 6-8, and 8-10 ton weights were used. Wave heights were 4, 6, 8 and 10m and wave period was 9.33 sec. The stones were freely put instead of regulary placing. The damages to the breakwater and the slopes of the structure were observed and measured after the each experiment.

In order to study the possibility of using of concrete blocks instead of stones, concrete blocks with dimensions 1.5*1.5*1.5m were also tested; both the damages and the slopes were measured.

At the end of the experiments, it was observed that, a certain slope of the structure nearly equal to 1/5 was occured regardless the wave height, stone and block weights. Thus, it was concluded that, this slope was proper in designing of breakwaters. Finally, the sections were constructed with this slope and it was observed that, no important demages took place.