CHAPTER 77

Oblique wave attack on block revetments

Adam Bezuijen*and Mark Klein Breteler[†]

November 12, 1992

Abstract

The influence of the angle of the incoming wave on the stability of a placed block revetment on a filter layer has been investigated. A computer program is described to simulate the flow in the filter layer for various wave conditions. The flow in the filter layer together with the wave conditions determine the loading on the revetment. Two different boundary conditions have been studied, A schemed wave pressure distribution based on results obtained for wave loading perpendicular to the revetment and regular waves measured in a wave basin. The results show that for regular waves the influence of the angle of the incoming wave on the loading of a placed block revetment is small for a revetment with a leakage length equal or larger than the wave height. Oblique incoming waves can result in a considerable increase in loading when the leakage length is much smaller than the wave height and near transitions in the revetment.

1 Introduction

Model tests on placed block revetments are mostly carried out in a wave flume. In these tests the angle of the incoming waves is perpendicular to the revetment. In reality the incoming waves can make different angles with the slope. In the field the revetment will be loaded with oblique waves. A perpendicular wave loading will be an exception.

Until recently this was not considered to be a problem. The perpendicular wave loading was assumed to be the most severe wave loading on the revetment. However Gadd and Leidersdorf (1990) presented results of field observations which indicate that the loading on the revetment by oblique wave attack can be more severe than the loading by waves coming in perpendicular to the revetment. They found that a block mattress on a permeable core was damaged due to oblique incoming waves, while the same wave loading coming perpendicular to the revetment had not caused any damage.

^{*}Consultant, Delft Geotechnics, P.O. Box 69, 2600 AB Delft, the Netherlands.

 $^{^{\}dagger}$ Head Coastal Structures group, Delft Hydraulics, P.O. Box 152, 8300 AD Emmeloord, the Netherlands.

The influence of oblique wave attack on the loading of a placed block revetment on a filter layer has been studied in a research programme commissioned by the Dutch Ministry of transport and public works (Rijkswaterstaat) and performed by Delft Geotechnics and Delft Hydraulics. Large scale experiments with oblique incoming waves are nearly impossible, because there are hardly facilities available for this type of research. On the other hand small scale experiments on placed block revetments cannot be done without scaling effects, Klein Breteler (1992). Therefore a small scale facility was used to measure only the wave pressures, the wave pressures apart from the wave impact can be measured without scaling effects and the wave impact is not the critical loading for the revetment. The filter flow in the revetment, the pore pressures and the loading on the revetment are simulated with a numerical finite difference program. The results of a comparable program for perpendicular wave attack are verified with the results of large scale model tests in a wave flume, Bezuijen et al. (1987). A comparable procedure, which combined the results of small scale model tests with the results of numerical simulations, is described by Bezuijen et al. (1988).

2 Description of numerical program STEEN3D

The numerical program is based on the STEENZET/1 program (Bezuijen et al., 1987; Bezuijen et al, 1990). The preliminary assumption, on which the STEENZET/1 program is based, is that the cover layer is relatively impermeable compared to the filter layer and that the flow through the base layer can be neglected. For most of the placed block revetments built in the Netherlands this appeared to be the case. In that case the pressure distribution can be described as a semi-confined aquifier. The flow in the filter layer is quasi-static. In the filter layer a mean potential ϕ can be derived in a plane perpendicular to the slope assuming that the flow in the filter layer is in a plane parallel to the slope. Further the flow in the cover layer is assumed to be perpendicular to the slope. With these conditions the differential equation for the flow in the filter layer in two dimensions can be written as:

$$k_x \frac{\partial^2 \phi}{\partial x^2} + k_y \frac{\partial^2 \phi}{\partial y^2} = k' \frac{\phi - \phi'}{bD} \tag{1}$$

where:

ϕ	:	the mean piezometric head in the filter layer	(m)
ϕ'	:	the piezometric head on the cover layer	(m)
x, y	:	co-ordinates, see figure 1	(m)
k_x	:	the permeability of the filter layer in x direction	(m/s)
k_y	:	the permeability of the filter layer in y direction	(m/s)
k'	:	the permeability of the cover layer	(m/s)
b	:	the thickness of the filter layer	(m)
D	:	the thickness of the cover layer	(m)



Figure 1: Finite difference scheme used.

In case of laminar flow with constant permeabilities $k = k_x = k_y$ and k', this leads to the more familiar equation:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = \frac{\phi - \phi_t}{\Lambda^2} \tag{2}$$

where:

 Λ : The leakage length (m)

The leakage length or leakage factor is then defined as:

$$\Lambda = \sqrt{\frac{kbD}{k'}} \tag{3}$$

Analytical solutions of this equation, for 1 dimension $(\partial \phi / \partial y = 0)$ with a schemed wave pressure distribution as a boundary condition have been presented by Sellmeijer (1981) and Burger at al. (1990). A numerical approach as STEENZET/1 can use measured wave pressures as a boundary condition and turbulent flow can be included. To obtain a numerical solution equation 1 is written as a set of finite difference equations, using the points presented in figure 1 and the general relation:

$$rac{\partial^2 y}{\partial x^2} pprox rac{y_{i-1} - 2y_i + y_{i+1}}{(\Delta x^2)}$$
(4)

OBLIQUE WAVE ATTACKS

Where Δx is the distance between the points *i* and *i*+1. In figure 1 it is shown that the distance along the X axis between two points in the finite difference scheme is L and it is B along the Y axis. It is also shown for what part of the cover layer the permeability is $k'_{i,j}$ and where in the filter layer the permeability is $k'_{i,j}$. With the equations 1 and 4 a set of equations for the points $\phi_{i,j}$ can be obtained:

$$\phi_{i,j} = \frac{B^2(k_{i-1,j}\phi_{i-1,j} + k_{i,j}\phi_{i+1,j}) + L^2(k_{i,j-1}\phi_{i,j-1} + k_{i,j}\phi_{i,j+1}) + \kappa_{i,j}\phi'_{i,j}}{\kappa_{i,j} + B^2(k_{i-1,j} + k_{i,j}) + L^2(k_{i,j-1} + k_{i,j})}$$
(5)

Where:

$$\kappa_{i,j} = rac{B^2 L^2 k_{i,j}'}{b D}$$

The filter layer is assumed to be impermeable at the boundaries of the revetment. This means that when a point on a boundary has to be calculated with equation 5, the value of ϕ just outside the boundary (for example $\phi_{i,j-1}$ when $\phi_{i,j}$ is the lower boundary) is given the same value as the point just inside the boundary (the point $\phi_{i,j+1}$ at the lower boundary). The situation is a bit different for the upper boundary. Here are two possibilities. It can be the same impermeable boundary as for the other boundaries, or it is the phreatic surface in the filter layer. It is assumed that when there is a phreatic surface it is located at the upper boundary because there is no need to simulate the revetment above the phreatic surface. If the upper boundary is the phreatic surface, then the piezometric head in the filter layer is not calculated with equation 5, but the piezometric head is given the same value as the height of that point. The program developed, which is called STEEN3D, is a static program. Up to now it does not include time effects. For a given wave pressure distribution at a certain moment the pore pressure distribution in the filter layer and the resulting loading on the revetment is calculated. Since it is a static program the phreatic surface is an input parameter, as in the analytic solutions mentioned before. The permeability of both the cover layer and filter layer is described with the Forchheimer relation:

$$i = aq + bq^2 \tag{6}$$

where:

1	: the hydraulic gradient		(-)	1
q	: the specific discharge	(m)	(s))

a : linear Forchheimer coefficient (s/m)

b : turbulent Forchheimer coefficient $(s/m)^2$

The coefficients a and b for the cover layer, as well as the different coefficients for the filter layer are calculated from the width of the joints, the dimensions of the blocks and grain size and porosity of the filter layer as described by Bezuijen and Klein Breteler (1988). The hydraulic gradient in the cover layer is $(\phi'_{i,j} - \phi_{i,j})/D$. The permeability of the filter layer $(k_{i,j})$ is assumed to depend only on the value of the hydraulic gradient in the filter layer (i_f) , not on the direction of it, leading to:

$$i_f = \sqrt{\left(\frac{\phi_{i+1,j} - \phi_{i,j}}{L}\right)^2 + \left(\frac{\phi_{i,j+1} - \phi_{i,j}}{B}\right)^2} \tag{7}$$

Equation 5 needs the linearized permeability (k = q/i), which can be derived from equation 6:

$$k = \frac{-a + \sqrt{a^2 + 4bi}}{2bi} \tag{8}$$

By using equation 8 to calculate the permeabilities non-linearity is introduced in the set of equations 5. Therefore an iterative solution method is used. The starting values of $\phi_{i,j}$ is the hydrostatic pressure for long leakage lengths (Λ) and the pore pressure distribution on the slope for small leakage lengths. With these starting values and the equations 5 new values of $\phi_{i,j}$ are calculated and new permeabilities. This procedure is repeated until the maximum difference between two solutions of $\phi_{i,j}$ is less than 10^{-4} m. Using the Forchheimer relation, as given in equation 6, The leakage length (equation 3) depends on the hydraulic gradient. To characterize a revetment the leakage length is calculated for a gradient 1 over the cover layer and a gradient equal to the sinus of the slope angle in the filter layer.

3 Boundary conditions

3.1 Schemed boundary condition

A schemed boundary condition has been developed to investigate the influence of the angle of the incoming wave on the flow in the filter layer. It is based on a pressure distribution proposed by Burger et al. (1990) for regular waves propagating perpendicular to the revetment. It was found that the moment of maximum loading on the revetment is present just before the wave impact. The pressure distribution for that situation can be schemed as is shown in figure 2. In case of oblique wave attack it is assumed that the maximum loading on the revetment is the same as in case of perpendicular wave attack, but on one side of the revetment the angle β (see figure 2) becomes less, on the other side the angle β remains the same and d_b and ϕ_b decrease, as is shown in figure 2. The obliqueness of the incoming wave is represented with an angle γ which represents the deviation from perpendicular incoming waves.

3.2 Measured boundary condition

The wave pressures have been measured in a 3 dimensional wave basin (the Vinjé basin). This wave basin is capable in generating regular and irregular waves with a wave height up to 0.2 m. The irregular waves can be long and short crested. In this study only regular waves were used. In the basin a



Figure 2: Schemed pressure distribution proposed by Burger et al. (a) and 3-dimensional proposal for oblique wave attack (b).

model was constructed with a slope 1:4. Tests have been performed with a wave steepness of 2 and 4%. The angle of incidence of the incoming waves varied between 0 and 50 degrees, where again 0 degrees means perpendicular incoming waves. The wave height was measured with 5 wave gauges. The wave height and direction were measured with 3 multi-directional wave gauges (These are combined wave gauges and 2-axis current meters). The wave pressures were measured at 10 positions on the slope on a line with a constant X position along the slope with varying depth. A top view of the model in the basin is presented in figure 3. The position of the pore pressure gauges is shown in figure 4. The pore pressure gauges were sampled with a sampling rate of 25 Hz and the results were stored in a computer. Since the velocity of the waves propagating along the slope is constant the wave pressure measured at one line can be transformed in a pressure field over the slope. When the origin of the X-axis (see figure 3) is chosen at the location of the wave pressure gauges, then the wave pressures measured at X=0 at a certain moment t_0 can be transferred to the wave pressures that can be measured at a different position at a different moment according to the formula:

$$\phi'(x,y,t+T\frac{x\sin\gamma}{L_g}) = \phi'(0,y,t) \tag{9}$$



Figure 3: Top view of model in basin



Figure 4: Pore pressure gauges on slope



Figure 5: Wave pressure distribution under oblique wave attack on a slope 1:4 with wave height of 1.2 m and a wave period of 4.4 s. The angle of incidence of the incoming waves was 40 degrees (scaled up from a measured wave height of 0.12 m).

Where:

 L_g : the wave length in front of the slope (m) T : the wave period (s)

Figure 5 shows an example of a measured wave pressure distribution.

The plot shows the result of the fact that pore pressure gauges were present only below the still water level. The pressures higher on the slope were not measured. If a pressure was measured on the highest wave pressure gauge it is assumed that this pressure was present higher on the slope in a way that the piezometric head was constant. This leads to an interpolation error. However this error is only present higher on the slope and the maximum loading on the revetment is always well below the still water line.

From this plot it appeared that the schematisation used in the schemed boundary condition see figure 2 is reasonable for the area before wave impact. In that area there is clearly a steepening of the next incoming wave. The wave impact causes a pressure distribution very different from the schemed boundary

Parameter	Case 1	Case 2
top of revetment	2 m	1.5 m
toe of revetment	-1 m	-6.1 m
slope angle	18.43° and 14.03°	18.43° and 14.03°
block length and width	0.5 m	1.2 m
block thickness	0.2 m	0.23 m
width of joints	0.002 m	0.06 m
thickness filter layer	0.2 m	2 m
d ₁₅ filter layer	0.02 m	0.001 m
porosity filter layer	0.4	0.35
leakage factor	1.1 m	0.92 m
still water level	1 m	0.0 m
wave height	1.2 m	3.3 m
wave angle γ	0 - 60	0 - 70
Wave steepness (measured)	0.04	0.04
ϕ_b in fig. 4 (schemed)	1.0 m	$4.2 \mathrm{m}$
d_b in fig. 4 (schemed)	0.8 m	2.78 m
β in fig. 4 (schemed)	30°	46.5°

Table 1: Parameters used in the calculations

condition. Since the schemed boundary condition is reasonable in the area of maximum loading it still can be used for situations where no wave pressure recordings are available. However it should be taken into account that the angle of the wave on the revetment is less that the angle of the incoming wave due to refraction of the waves on the slope. For the wave shown in figure 5 with an angle of incidence of 40° for the incoming waves it was found that on the slope at a depth of 1.6 m below the still water level the angle of incidence of the wave pressures on the slope was approximately 25° , at 1 m below the still water line it was only 11° .

4 Results

The calculations were run with the parameters presented in table 1. Case 1 are the parameters of a revetment as can be expected in the Dutch estuaries. With the results of the field tests (see Stoutjesdijk et al., 1992) now available it must be stated that these are the parameters for a new revetment with no fine material in the filter layer and between the joints. The second case is an attempt to simulate the revetment described by Gadd and Leidersdorf (1990) at Northstar Island in the Beaufort Sea.

An example of a calculation with the schemed boundary condition presented in figure 2 is shown in figure 6, which showed the distribution of the difference in piezometric head over the blocks. A positive difference in this figure means



Figure 6: Contour plot of the calculated uplift pressure, schemed boundary condition $\gamma = 30^{\circ}$.

Figure 7: Contour plot of the calculated permeability in the filter layer.

that there is an uplift pressure over the blocks. If this uplift pressure is higher than the weight of the blocks and possible clamping forces blocks can be lifted from the revetment. In this calculation the parameters of case 1 in table 1 are used. Figure 7 showed the corresponding permeability in the filter layer. Due to turbulency the linearized permeability, see equation 8 is high in areas with a small hydraulic gradient and lower in area's with a higher gradient. It appeared from the calculations that using turbulency in the calculation leads to a slightly higher maximum uplift pressure than would result from a calculation with laminar flow. Comparing the calculated maximum uplift pressure with the boundary condition it appeared that the maximum uplift pressure can be found just before the impact of the next incoming wave as could be expected.

An example of the simulation results using a measured boundary condition for the revetment with the parameters of case 1 is shown in figure 8. It shows the difference in piezometric head as it is distributed over a placed block revetment at a certain time when loaded with oblique wave attack. The high values (the 'mountains' in the plot) correspond with a high uplift pressure. At these locations the blocks can be lifted out of the revetment. The difference in piezometric head is at maximum just in front of the areas of wave impact at X is approximately 9 and 50 m (the wave is propagating from high to low X values). Although the figure presents the result for an incoming wave angle of 40° , there is for each wave over a large part of the revetment only one Z value (one height on the revetment) with the largest difference in piezometric head and thus the



Figure 8: Example of calculated difference in piezometric head over the blocks. Calculation for a slope 1:4 by a wave attack of 1.2 m wave height, a period of 4.4 s and an incoming wave angle of 40° , the leakage length is 1.1 m.



ence in piezometric head as a function ence in piezometric head as a function of the angle of the incoming wave, case of the angle of the incoming wave, case 1.

Figure 9: Calculated maximum differ- Figure 10: Calculated maximum differ-2.

maximum loading.

The influence of the angle of wave attack was tested by performing calculations for different angles of incoming waves. This was done for the schemed as well as the measured boundary conditions. For case 1 the results of the calculations are shown in figure 9 and for case 2 in figure 10. For case 1 it is clear that the influence of the angle of the incoming wave on the maximum calculated uplift pressure is small. The result shows that for angles of the incoming waves between 0 and 60° the maximum loading remains more or less the same. As is also shown in figure 9, differences found in the maximum uplift pressures for measured waves appear to correspond closely with differences in the amplitude of the piezometric head, measured with the lowest wave pressure gauge. These differences in amplitude correspond with small differences in wave height in front of the structure, which cannot be avoided in a wave basin. For case 2 there is an influence. The maximum calculated uplift pressure increases for measured as well as schemed boundary conditions. For schemed boundary conditions this increase goes on with an increasing angle of the incoming waves, for the measured boundary condition there seems to be a maximum for an angle of 40° for the incoming waves. In case 2 the ratio of the leakage length divided by the wave height of the incoming waves (Λ/H) is much smaller than in case 1, see table 1. The results of the calculations show that in case $\Lambda/H \approx 1$, there is hardly any influence and this influence is larger for $\Lambda/H = 0.33$. It appears that for $\Lambda/H = 0.33$ the loading is 50% higher for oblique incoming



Figure 11: Maximum loading near an impermeable transition in the revetment, case 1, wave conditions as shown in figure 5. In the plot is a drawing of an example of an impermeable transition.

waves $(\gamma = 40^{\circ})$ than for perpendicular incoming waves $(\gamma = 10^{\circ})$. Further the influence of the transition in a revetment have been investigated. The locations where one type of revetment changes to an other type. In the calculations the situation is simulated that at such a transition the filter layer is impermeable. Various calculations have been run to simulate the situation that the moment of maximum loading on a revetment reaches such a transition. This will always happen when a transition is present, because in case of oblique wave attack it looks as if the waves are travelling along the revetment. The results of the calculations are shown in figure 11. The results show that near a transition the uplift pressures on the revetment are higher than far from the transition. This is caused by changes in the flow in the filter layer near the transition.

Normally the strength of the revetment is less when there is transition in the revetment. The discontinuity present at the transition can cause a reduction of the clamping forces between the blocks and difficulties in densification of the subsoil during construction of the revetment, leading to uneven settlement later. This decrease in strength in combination with the calculated increased loading means that during oblique wave attack the edges of the revetment are especially vulnerable.

5 Conclusions

The results show that for oblique wave attack with regular waves the angle of the incoming waves have little influence on the maximum loading on the revetment, as long as the leakage length is larger or equal than the wave height. When the leakage length of the revetment is much shorter, then oblique wave attack with can result in up to 50 % higher loading on the revetment. Further near transitions in the revetment the loading on the revetment is increased in case of oblique wave attack.

6 Acknowledgement

The authors wish to thank P.E. Gadd and C.B. Leidersdorf for stimulating discussion and supplying detailed information about the revetment on Northstar island, where they have observed failure from oblique wave attack.

7 References

Bezuijen A., Klein Breteler M., Bakker K.J., 1987. Design criteria for placed block revetments and granular filters. Proc. 2nd. COPEDEC, Beijing.

Bezuijen A., Laustrup C. and Wouters J., 1988. Design of block revetments with physical and numerical models. Proc. 21st Int. Conf. Coastal Engineering, Malaga. Bezuijen A., Klein Breteler M., and Burger A.M., 1990. Placed block revetments. Chapter in Coastal Protection, Pilarczyk (ed.), Balkema Rotterdam. ISBN 9061911273.

Burger A.M., Klein Breteler M., Banach L., Bezuijen A., Pilarczyk K.W. (1990) An analytical design method for relatively closed block revetments. J. of Waterway, Port Coastal and Ocean Engineering, vol. 116, No 5. ASCE. Gadd P.E., Leidersdorf G.B., 1990. Recent performance of linked mat armor under wave and ice impact. Proc. 22th int. Conf. Coastal Engineering, Delft.

Klein Breteler M., 1992. Scaling rules for placed block revetments (in Dutch). Report XX in the series Placed block revetments, Delft Hydraulics. Sellmeijer J.B. 1981. Uplift pressures by waves in placed block revetments (in Dutch). Delft Geotechnics report CO-255780

Stoutjesdijk Th., Rigter B.P. and Bezuijen A., (1992) Field experiments on placed block revetments. Proc. 23rd. Int. Conf. Coastal Engineering, Venice.