CHAPTER 160

BLOCK REVETMENT DESIGN WITH PHYS. AND NUM. MODELS

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Abstract

A combination of a physical model and numerical models has been used in the design of a block revetment for the Danish North Sea coast. The wave pressure loading on the revetment during design conditions was investigated in a physical scale model. The measured wave pressures were used as a boundary condition for the numerical models. Solutions for the flow equations through the coverlayer, filter layer and subsoil were then obtained in the numerical models, taking into account the influence of turbulence. With these solutions the stability of the coverlayer and subsoil was evaluated. The paper presents a description of the various models and information about the design of the revetment.

1. Introduction

Numerical models were developed within the scope of a research programme on block revetments in order to calculate the loading on a block revetment during wave attack, Hjortnaes-Pedersen et al (1987) and Bezuijen et al (1987). This research programme was commissioned by the Dutch Department of Public works (Rijkswaterstaat) and was performed by Delft Hydraulics in cooperation with Delft Geotechnics. The numerical models were used to calculate the pressure distribution in the filter layer and subsoil below a block revetment when the pressure distribution on the revetment due to wave attack is known. Both the wave pressures and the calculated pressures underneath the revetment determine the uplift pressures on the coverlayer of the revetment. These uplift pressures can be compared with the "strength" of the coverlayer. The wave pressures which were required as inputs for the numerical models were determined by means of physical model tests. Since only wave pressures on the slope had to be measured and the influence of wave impacts on the stability of a block revetment could be neglected, these pressures could be measured in a small scale model.

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The combination of a physical scale model and numerical models was used to evaluate the design of a dune toe protection which is now under construction in parts of the Danish North Sea coast. The design of the block revetment is presented in this paper and the physical model tests described. A brief description of the numerical models is given together with the results of the calculations for this revetment. Finally the modifications in the design, based on the results of the calculations, are discussed.

2. Block revetment used as dune protection

On some parts of the Danish North Sea coast erosion has been very large, on average 3 - 4 m a year. This has caused some dunes to disappear and others to become very weak. As a result the low areas behind the dunes are open to flooding. On these stretches of coast it is necessary to stop the erosion and to re-establish the flood protection. The measures being taken against the erosion are a combination of onshore and offshore beach nourishment and low detached breakwaters parallel and close to the shore line. The flood protection is being re-established by building artificial dunes protected by a concrete block revetment. Concrete blocks are being used because:

a. They are cheap compared with other types of artificial coverlayers.

b. Denmark has no quarries close to the North Sea coast.

c. Concrete blocks look relatively attractive into the sandy coast environment.

d. Constructional procedures are relatively easy and a high quality can be achieved.

e. The concrete is very durable in a marine environment.

A sketch of the concrete block revetment is presented in Figure 1. Water level and wave conditions vary depending on the location. The conditions expected with a return period of 100 years, for the most exposed structure, are shown in Table 1.
Waterlevel: 3.65 m above datum
Waves in a depth of 19 m
\[ H_s = 8.2 \text{ m} \]
\[ T_p = 12 \text{ s} \]
Shore profile: slope
1:20 from -0.30 to -6.00 m below datum
1:100 below -6.00 m below datum
Revetment slope
1:2 from -0.30 to +8.10 m above datum

Table 1: Design conditions

The design crest height at the various locations is taken as the sum of the high water level, the wind set-up and the maximum breaking wave height above still water level. For the conditions given in Table 1 the crest height will be 8.1 m. Since waves will run-up the slope to above the level of the block revetment, the first 5 m of the crest of the structure has been protected with rubble.

The toe of the structure has been designed at such a level that there will be no damage by scouring during the design storm.

The blocks are placed on a filter structure of 40 - 60 mm rubble between two layers of geotextile. The purpose of the upper layer is to prevent the rubble layer from being filled with sand from above. The blocks are 0.75 m high and weigh 3,000 kg. Some of the blocks are 1.0 m high in order to introduce slope roughness.

The following questions were raised when evaluating the design:
- Can block thickness be reduced?
- Is a 1:3 slope preferable to a slope of 1:2?
- What is the influence of revetment roughness on run-up produced by using blocks of different height?
3. Physical model

Small-scale model tests were carried out in a wave flume to obtain information about the wave pressure distribution on the slope. The wave flume has a depth of 0.8 m and is equipped with a system to compensate for wave reflections. The tests were carried out with irregular waves on slopes of 1:2 and 1:3. Since only the wave pressures on the slope had to be measured in a physical model test (to provide inputs for the numerical models) it was unnecessary to model the revetment itself. Only the geometry of the revetment and the foreshore were of interest in the physical model and these were modelled in concrete. The model layout is shown in Figure 2.

![Model layout diagram](image)

Figure 2: Model layout

Wave heights were measured on the profile with five wave gauges; run-up was measured with a wave run-up meter on the 1:20 slope, see Figure 2. The positions of the pressure gauges on the 1:2 slope are shown in Figure 3. Because wave impacts shorter than 0.2 s duration are not of importance to the stability of the revetment, the pressure signals could be filtered by a 6.25 Hz (in model 25 Hz) low pass filter. The low-pass filter is necessary to prevent interference with high frequencies. Most tests were run at a geometric scale of 1:16. The stability of the rubble on the crest was measured directly in the model tests and therefore this was modelled to the same geometric scale factor. The nearshore incident wave conditions for the model tests were calculated from the deep water conditions with the ENDEC computer program, Stive (1984).
The model tests indicated that the wave runup on the 1:20 slope can be considerable and during the design conditions, Table 1, and with a crest height of 8.2 m in prototype, more than 10% of the waves passed the highest indicator of the wave run-up meter. This implies a wave run-up of over 43 m up the 1:20 slope. These run-up values led to an investigation of the run-up as a function of the crest height. The results are shown in Figure 4, for the design conditions given in Table 1.

On the 1:20 slope a line 37 m from the top of the revetment in prototype was chosen to indicate the extent of run-up, because information was available about the waves crossing this line also for the tests runned at different geometric scales.
More waves passed the 37 m line with the 1:3 slope than with the 1:2 slope, but the difference was small. It appeared that the increased roughness, obtained by using blocks of different height, would lead to a 10% reduction in the number of waves crossing the 37 m line during design conditions. During moderate storm conditions this reduction would be even higher. With a significant wave height of 3 m in deep water the reduction would be up to 25%.

The wave pressures measured in the physical model have been used in the numerical model. This topic is discussed in the following chapter.

4. Numerical models

In order to evaluate the stability of the revetment, it is essential to determine the uplift pressures on the blocks. These pressures are determined by the difference between the pore pressure in the filter layer under the blocks and the wave pressures on the blocks. If the mean uplift pressure on the block is larger than the pressure corresponding to the weight of the block plus the friction forces between adjacent blocks then a block can be lifted out of the revetment. The stability of the sand underneath the revetment is also important since no sliding may occur. These stability criteria cannot be investigated in a small scale model test because of the soil mechanical scale effects which occur in such a test. A large scale investigation is a possibility, but in this project it was decided to use a numerical approach to evaluate the stability criteria. The uplift pressures on the blocks and the stability of the blocks were calculated with the STEENZET/1 program, to calculate the pore pressure in the filter layer and the resulting block movement (see Section 4.1). The pore pressure distribution in the subsoil was calculated with the STEENZET/2 finite element program which can be used to calculate the pressure distribution in both the filter layer and the subsoil, assuming no block movement (see Section 4.2). This pore pressure distribution was used in a stability calculation to evaluate the geotechnical stability against sliding. These numerical models are described briefly in the following sections.

4.1 STEENZET/1

The pressure distribution in the filter layer is determined by the flow through this layer and the flow through the joints around the blocks. The flow in the subsoil itself has no influence because of the low permeability of the sand compared to the permeability of the filter layer. Assuming a flow parallel to the slope in the filter layer, a flow perpendicular to the slope in the coverlayer and a coverlayer permeability which is concentrated in the "horizontal" joints, see Figure 5, the following formula can be derived for the potential in each joint:
\[
\phi_i = \frac{1}{1 + 2 \frac{kbD}{k'1^2}} \left[ \frac{kbD}{k'} \left( \phi_{i-1} + \phi_{i+1} \right) \right] + \phi_{t,i} \quad (1)
\]

Where: 
- \( \phi_i \): the piezometric head in the filter layer (m) near joint \( i \)
- \( \phi_{t,i} \): the piezometric head on the revetment (m) near joint \( i \)
- \( b \): the thickness of the granular sublayer (m)
- \( D \): the thickness of the blocks (m)
- \( l \): the length of the blocks (m)
- \( k \): the permeability of the filter layer (m/s)
- \( k' \): the permeability of the cover layer (m/s)

Figure 5: The STEENZET/1 finite difference scheme

At the phreatic surface the piezometric head in the filter layer is given by the position of the phreatic surface. At the lower end of the revetment the condition \( \phi_{i+1} = \phi_{i-1} \) is assumed. If there is no phreatic surface, \( \phi_{i-1} = \phi_{i+1} \) at the top of the revetment. The potential distribution in the filter layer can be solved numerically with Equation (1) for all joints if the different values of the piezometric head on the revetment \( (\phi_{t,i}) \) and the position of the phreatic surface are known. In the STEENZET/1 computer program this solution is obtained using an iterative scheme. Results of den Adel (1987) can be used to calculate the permeability of the filter layer. On the basis of permeability tests den Adel described the coefficients \( a \) and \( b \) in the Forchheimer relation \( i = av + bv^2 \), with \( i \) the hydraulic gradient and \( v \) the filter velocity) as a function of the porosity and of the granular material. The permeability of the cover-layer is determined by various flow resistances in and near the joints as described by Klein Breteler and Bezuijen (1988). Turbulence in the flow through the joints is included by adapting the permeability of the coverlayer to the calculated gradient during each iteration cycle. A linear flow condition is used in the filter layer.
The turbulent flow has therefore to be linearized. This linearization is performed in such a way that the flow velocity calculated with the linear flow relation is the same as the flow velocity calculated using a turbulent permeability relation at a gradient equal to \( \sin(a) \), with \( a \) the slope angle.

The piezometric head on the revetment \( (\theta_{t,i}) \) is determined from measured wave pressures by linear interpolation of the results of the pressure gauges. The wave pressures measured in the small scale model can be transferred to prototype values by using Froude's law. The measured pressures can be scaled up using the geometric scale and the time scales using the square root of that scale. This means that the 50 Hz sampling frequency in the model tests, at a scale 1:16, corresponds to a 12.5 Hz sampling frequency in prototype.

The \( \theta_i \) values were calculated for the various time steps using different values of \( \theta_{t,i} \). The position of the phreatic surface is calculated for each time step by taking the still water level as a starting position and adapting the phreatic level to the nett flow of water in the filter layer. In this way the potential distribution can be calculated for each sampling of the wave pressures over a period of several waves. The results of this program have been compared with the results of large scale model tests and show good agreement (Bezuijen et al (1987)).

If the mean calculated uplift pressure on one block exceeds the pressure corresponding to the weight of the block and the friction forces, then a block will start to move. This movement can also be calculated in the program using a simple routine. The uplift pressure, multiplied by the block area, determines the uplift force. Subtracting the weight of the block and the friction force gives the nett force \( F_n \), which causes block movement. The acceleration of the block can then be calculated using the well known relationship:

\[
F = M_b \cdot a
\]

Where: \( M_b \) : the mass of the block (kg)
\( a \) : the acceleration of the block (m/s\(^2\))

Double integration of the acceleration giving the block movement.

The movement calculated in this way however is too large. In reality the block movement is less, because the moving block itself causes a pressure decrease in the filter layer. A routine that includes the influence of the moving block on the pressure distribution has been developed. This routine was not used in this project because, due to lack of experimental evidence, it was not certain that the results would always be on the safe side.
4.2 Results of STEENZET/1 calculations.

Calculations were made for slope angles of 1:2 and 1:3 and various coverlayer and sublayer permeabilities. The influence of the slope on the maximum uplift pressures appeared to be small. This means that a 1:2 slope is in fact the most economical. The results of a typical calculation are shown in the Figures 6 and 7. The black area shows the measured wave pressures and the grey area the uplift pressure, but only when larger than corresponding to the weight of the blocks. The vertical height of the area represents the pressure, in metres of water, at the location were the pressure was measured. Figure 6 shows the position of the wave front at maximum wave run down; there is hardly any water on the revetment and as a consequence the wave pressures are very small.

![Figure 6: Wave pressure measured during wave run down.](image)

**Figure 6:** Wave pressure measured during wave run down. Results of STEENZET/1 calculations

Figure 7 shows the pressure distribution just after wave impact. From these figures it is clear that, for the revetment being studied the highest uplift pressures can be expected just after wave impact, when two areas of high wave pressure occur on the revetment separated of an area of low pressures. The high wave pressures are transmitted through the filter layer, leading to high uplift pressures in the area with small wave pressures.
Figure 7: Wave pressure measured after wave impact. Uplift pressure when larger than corresponding to the weight of the blocks and calculated movement of the blocks. Results of STEENZET/1 calculations.

Although during wave run down the filter layer is almost completely filled with water, the maximum wave run down does not appear to be a critical situation. The uplift pressure is never larger than the pressure that corresponds to the weight of the blocks. Due to the hydraulic gradient in the filter layer the pressure distribution in that layer is by no means hydrostatic, leading to small pore pressures and uplift pressures.

The permeabilities of coverlayer and filter layer had a distinct influence on the uplift pressures. The lowest uplift pressures were found for a minimum filter layer permeability and a maximum coverlayer permeability. The permeability of the coverlayer is determined by the permeability of the joints and is reduced due to the geotextile between the blocks and the filter layer. The design was therefore adapted and the geotextile removed. Without the geotextile some sand will be transported into the filter layer. However this sand will only reduce the filter layer permeability. Dutch experience has shown that this sand never causes trouble.
A design criterion had to be chosen for determining the block thickness. The criterion, no block shall ever move during design conditions, leads in fact to a very large block thickness and is too strict to be of practical use. The clamping forces between the blocks are completely neglected; however, due to these clamping forces, the strength of the revetment is increased significantly. Pulling tests have shown that sometimes forces more than 10 times the weight of a block are necessary to extract it from the revetment (Burger 1985). However the reliability of this extra strength decreases if calculations show that blocks immediately above and below can move at the same time. The following design criterion was therefore used: the revetment is considered to be stable if calculations with STEENZET/l show that only one block moves and that the movement calculated is much smaller than the thickness of the block. In reality this will be a stable situation due to the clamping forces. Apply this criterion a block thickness of 0.5 m was found, assuming a filter material with d15 of 20 mm and an average joint width of 5 mm.

An unexpected result was the influence of the toe permeability on the calculated uplift pressures. This was simulated in the calculations by changing the permeability of the lowest joint in the revetment; Figure 8 shows this influence. The increased permeability leads to higher piezometric head in the lower end of the filter layer and, as a consequence higher uplift pressures.

Figure 8: Influence of toe permeability on uplift pressures
4.3 STEENZET/2

The results of the STEENZET/1 calculations showed that a 1:2 slope would be more economical, because less concrete is needed. The calculation however omits the effects of sand body sliding. This kind of failure can occur locally when the grain stress in the sand is reduced by seepage. The pore pressure distribution in the subsoil due to wave attack was therefore calculated with the STEENZET/2 program.

STEENZET/2 is a 2-dimensional finite element program specially developed to calculate the pore pressure response under revetments due to wave attack. The model is based on what is referred to as the storage equation:

\[ Vq = C \frac{d\theta}{dt} \]  

(3)

Where: 
- \(\theta\): the piezometric head (m)
- \(q\): the specific discharge (m/s)
- \(C\): \(\rho g n \beta\) (1/m)
- \(\rho\): the mass density of the fluid (kg/m³)
- \(g\): acceleration due to gravity (m/s²)
- \(n\): porosity
- \(\beta\): compressibility (m²/KN)
- \(V\): \(d/dx, d/dy\)

With solutions of this equation it is possible to investigate the influence of the air content of the pore water on the results. The program can handle turbulent flow on the base of the Forchheimer relation and various materials can be considered. Measured wave pressures can be used as boundary conditions in the same way as in the STEENZET/1 program. A more extended description of this program is given by Hjortnaes-Pedersen et al (1987). The pore pressure distributions calculated with STEENZET/2, were used in a stability analysis as described by Bishop. This method is well known and is described in, for instance Terzaghi and Peck (1967).

4.4 Results of STEENZET/2 Calculations

Calculations were performed for the 1:2 slope and the block thickness indicated as stable by the STEENZET/1 calculations (0.5m). The element mesh used is shown in Figure 9. The model tests showed that a large amount of overtopping can be expected during design conditions and it was therefore assumed that, during these conditions, the sand body directly behind the revetment would be completely filled with water. Outward directed hydraulic gradients perpendicular to the slope, are most dangerous for revetment stability. The wave period which led to the highest uplift pressures in the STEENZET/1 calculations was used as a (time varying) boundary condition. Very little is known about the air content of the pore water and therefore calculations were run with different values of air content. Results are shown in Figures 10 and 11. In these figures lines of equal piezometric head are shown at the moment of maximum outward directed hydraulic gradient; the wave boundary condition is also shown at this particular moment.
Figure 9: Finite element mesh used in STEENZET/2 calculations

Figure 10: Measured wave pressures and calculated lines with equipotential. Results of STEENZET/2 calculations - assumed air content in the pore water: 1%

Figure 11: Measured wave pressures and calculated lines with equipotential. Results of STEENZET/2 calculations - assumed air content in the pore water: 5%
This wave condition was measured just after a period of rapid run down. Since Equation (3) is a transient equation, the boundary condition prior to this particular moment is also of importance. The figures clearly show the influence of the air content. The higher the air content, the higher the outward directed gradient. The stability calculations showed however that even with an assumed air content of 5%, the safety of the 1:2 slope against sliding is more than 1.5 with a friction angle of 35° for the sand, 40° for the gravel and schematizing the concrete blocks to a friction angle of 25°. From these results it was concluded that the 1:2 slope will be stable against sliding during design conditions.

\[ \text{DESIGN CREST HEIGHT: 1.5m} \]

\[ \text{DESIGN WATER LEVEL -1.5m} \]

\[ \text{DESIGN WAVE LEVEL: 1.0m} \]

\[ \text{DESIGN BRUSH: 0.5m} \]

\[ \text{DESIGN GRAVEL: 0.3m} \]

\[ \text{DESIGN NET: 0.1m} \]

\[ \text{DESIGN TENT: 0.2m} \]

\[ \text{DESIGN COARSE: 0.3m} \]

\[ \text{DESIGN FINE: 0.2m} \]

Figure 12: Adapted design based on model studies

5. Conclusions

The combination of physical and numerical models can be a valuable tool in the design of a placed block revetment. As a result of this study the following adaptations were made to the design:
- a heavier rubble is used for the crest protection.
- the geotextile between the blocks and the filter layer is removed.
- a filter layer of relatively small gravel (16-32 mm) is used.
- During the manufacturing process the surface of the blocks is made irregular so that the joints will be about 5 mm wide.
- The geotextile between the blocks and the filter layer is replaced by a filter of relatively small gravel in order to reduce the uplift pressures. This makes it possible to reduce the block thickness from 0.75 to 0.5m.

The model tests showed that increasing the roughness of the slope by varying the thickness of the blocks reduces the wave run-up, especially during moderate storm conditions. According to the stability calculations a structure with a slope 1:2 is stable during design conditions. Since the difference in calculated maximum uplift pressure and measured run-up with a 1:2 slope and 1:3 slope is only small, a 1:2 slope is preferred because less concrete has to be used. Figure 12 shows the adapted design.
A total of 11,420 m of this revetment had been built by September 1988.

References