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 atractive 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
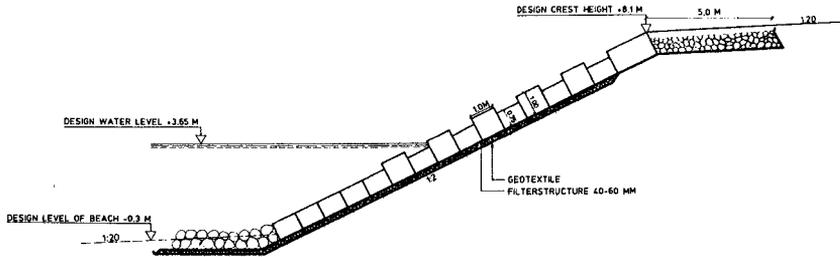of 100 years, for the most exposed structure, are shown in
Table 1.

Figure 1: Original cross section for the design parameters
          given in Table 1

Waterlevel: 3.65 m above datum
Waves in a depth
   of 19 m H_s = 8.2 m
                              T_p = 12   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
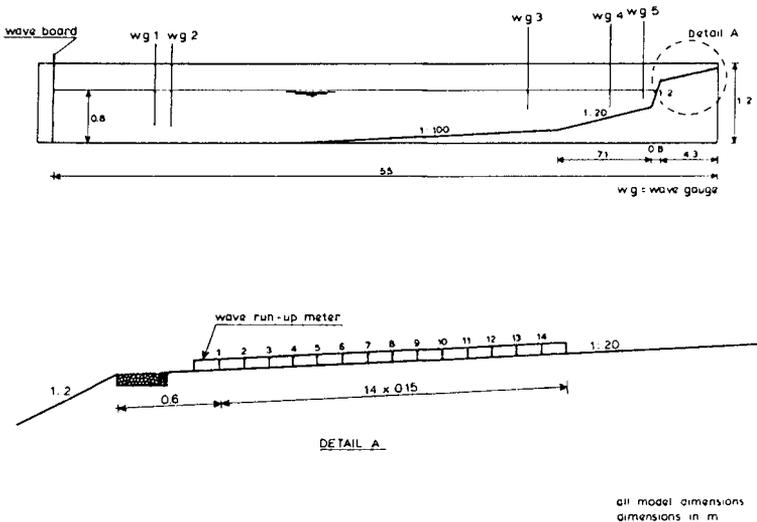in Figure 2.

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
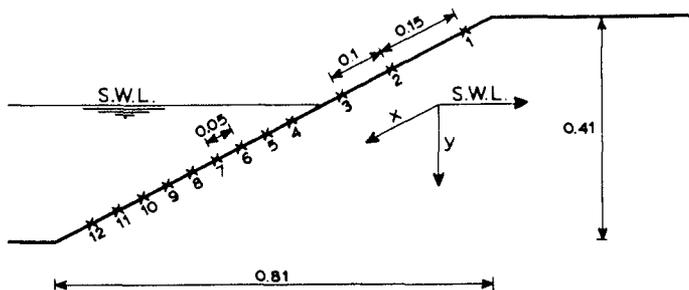from the deep water conditions with the ENDEC computer
program, Stive (1984).

Figure 3: Location of wave pressure gauges on the 1:2 slope

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.
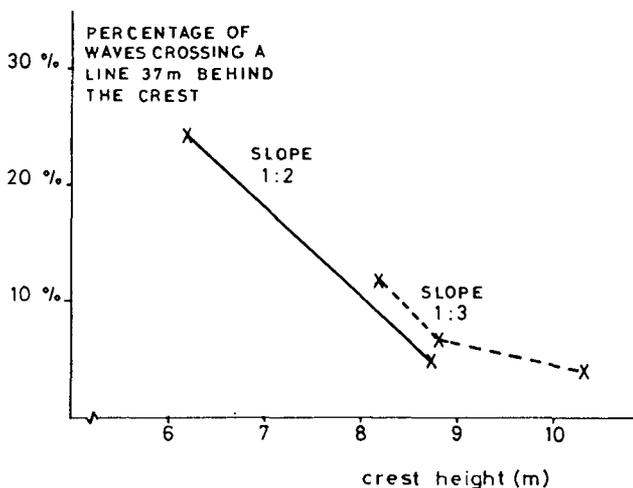

Figure 4: Percentage of waves crossing the 37 m line (see text) as a function of the crest height

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 led 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 calulate 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 calulate 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:

$$\phi_i = \frac{1}{1 + 2 \frac{kbD}{k'l^2}} \left\{ \frac{kbD}{k'l^2} \left(\phi_{i-1} + \phi_{i+1} \right) + \phi_{t,i} \right\} \qquad (1)$$

Where: ϕ_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)
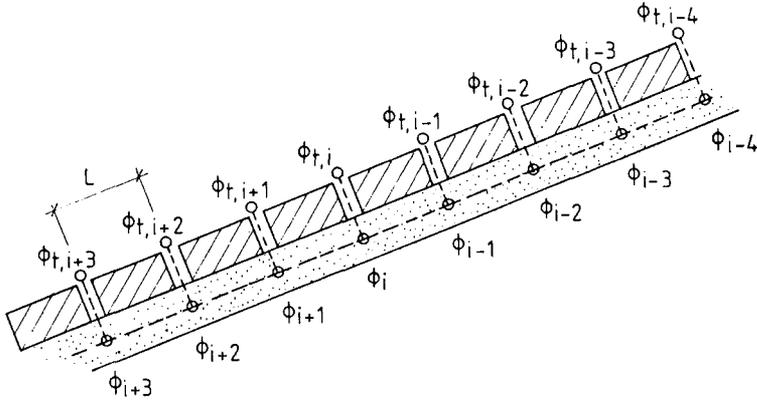   k' : the permeability of the cover layer   (m/s)

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 + bv², with i the hydraulic gradient and v
the filter velocity) as a function of the porosity and d_{15}
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 linearisation 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 $\sin(\alpha)$, with α the slope angle.

The piezometric head on the revetment $(\phi_{t,i})$ 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 ϕ_i values were calculated for the various time steps using different values of $\phi_{t,i}$. 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:

$$F_n = M_b \cdot a$$

Where: M_b : the mass of the block   (kg)
        a : the acceleration of the block   (m/s²)

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 sub layer 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
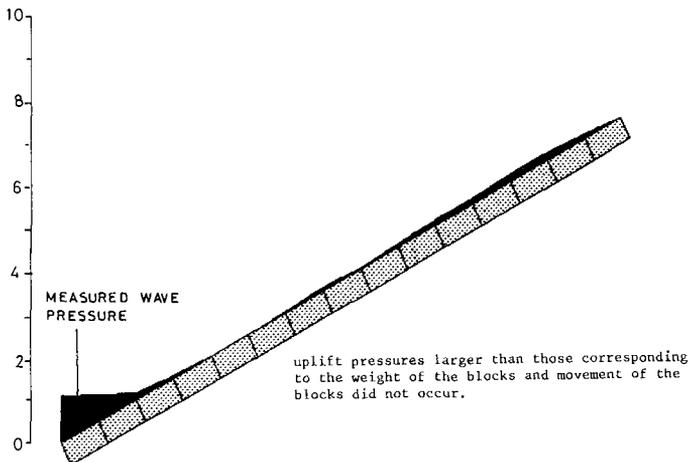pressures are very small.

Figure 6: Wave pressure measured during wave run down.
          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
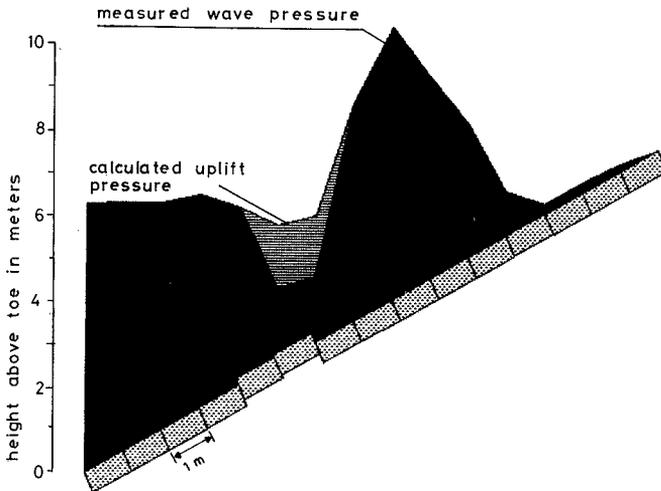area with small wave pressures.

Figure 7: Wave pressure measured after wave impact. Uplift
          pressure when lagrger than corresponding to the
          weight of the bloks 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/1
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 d_{15} 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,
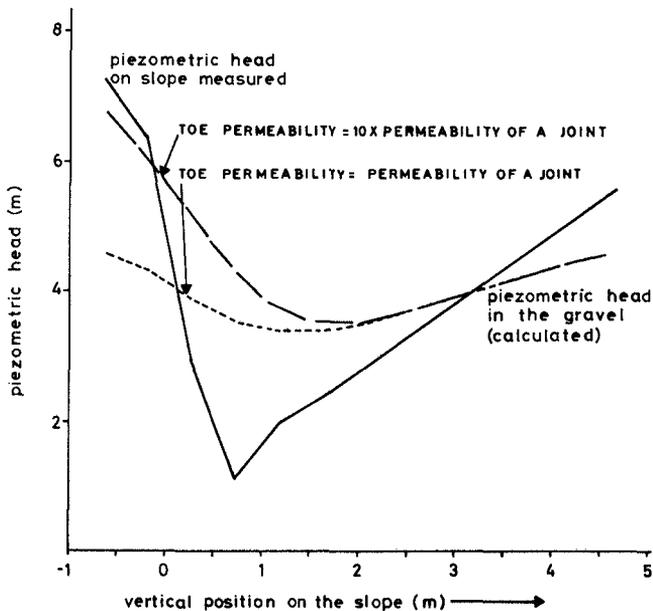as a consequence higher uplift pressures.

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 the 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:

$$\nabla q = C \, d\phi/dt \tag{3}$$

Where: ϕ : the piezometric head   (m)
       q : the specific discharge   (m/s)
       C = $\rho gn\beta$ (1/m)
       ρ : the mass density of the fluid   (kg/m³)
       g : acceleration due to gravity   (m/s²)
       n : porosity   ( )
       β : compressibility   (m²/KN)
       ∇ : 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
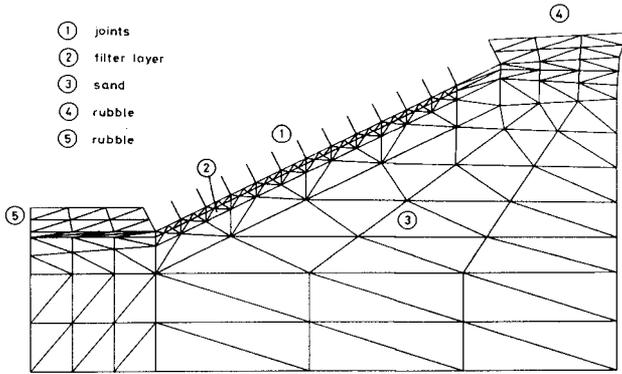also shown at this particular moment.

Figure 9: Finite element mesh used in STEENZET/2
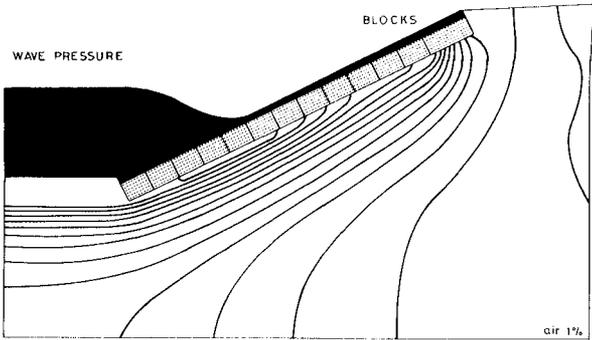   calculations

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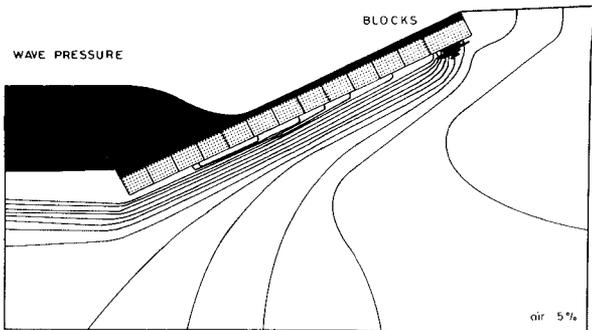

Figure 11: Measured wave pressures and calculated lines with
         epuipotential. Results of STEENZET/2 calculations
         -assumed air content in the pore water: 5%

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
material with a friction angle of 25°. From these results it
was concluded that the 1:2 slope will be stable against
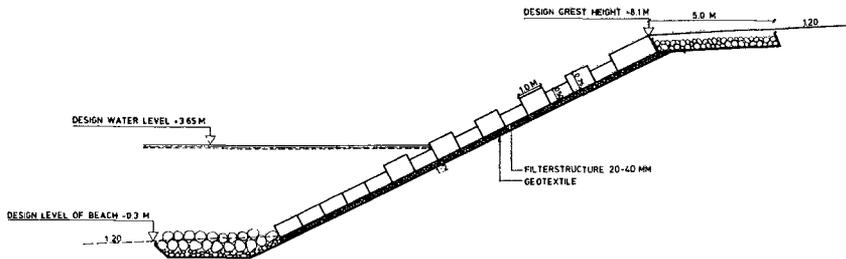sliding during design conditions.

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.

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