CHAPTER 85

SEA-BED CONFIGURATION IN RELATION TO BREAKWATER STABILITY

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ABSTRACT

        Stability tests an the Eurapaart breakwaters, situated on a shallow foreshare, clearly demanstrated the effect af the foreshore canfiguration on the averall stability. The present article gives a descriptionaf the stability experiments and the inter-pretatian leading ta general canclusians regarding fareshare effects in combination with hydraulic canditions such as wave period, water depth and wave height. Bath regular and irregular waves have been used. The experiments, carried aut in commission of the Netherlands Government Department of Public Warks (Rijks-waterstaat) were af an applied nature and were not directed primarily ta the system-atic study of foreshare effects.

1. INTRODUCTION

        In designing and testing breai.water structures, generally main ₃tress is laid upan the hydraulic conditions such as wave heights, currents and tides, with less attentian being paid to the relation between signifi cant wave heights and average wave periods. The wave period is often considered to be a parameter of minor impartance in this respect. Moreover, the geametry of the sea-bed in frant of the breakwater and its indirect effects an the behaviaur of that structure are seldam taken inta consideration. The authors have come across anly a few examples where these fareshare effects have knowingly been taken inta account (Refs. 1 and 2).
        Nevertheless, the geametry af the foreshare in cambination with bath the wave height and wave periad may in particular situatians be af major impartance in determining design conditians.
        The prafile af the foreshare will generally anly be variable to a limited extent, as it depends on the ariginal sea-bed configuration, the type af structure and the erasion and/or accretion ta be expected.
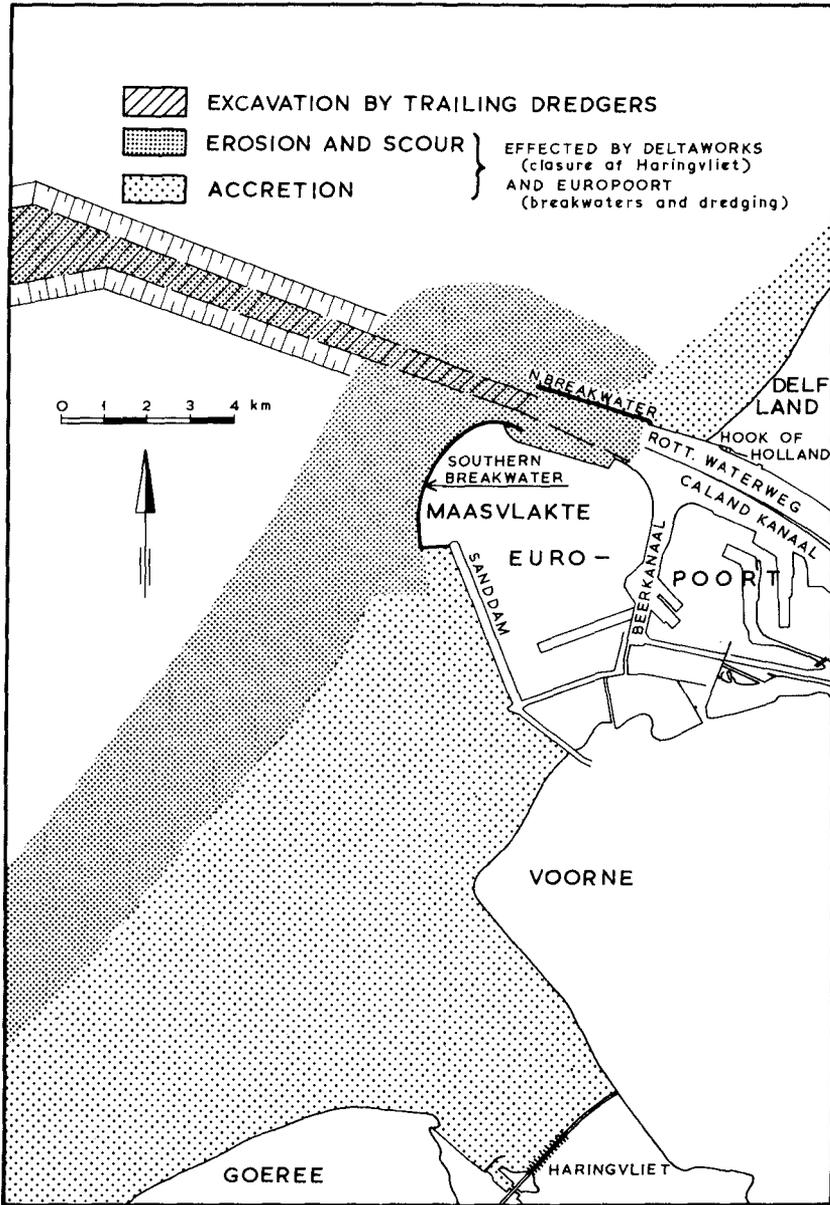
EXCAVATION BY TRAILING DREDGERS

EROSION AND SCOUR } EFFECTED BY DELTAWORKS
(closure of Haringvliet)

ACCRETION } AND EUROPOORT
(breakwaters and dredging)

O 1 2 3 4 km

N. BREAKWATER

DELF LAND

HOOK OF HOLLAND

SOUTHERN BREAKWATER

ROTT. WATERWEG

MAASVLAKTE

CALAND KANAAL

EURO—

BEERKANAAL

POORT

SANDDAM

VOORNE

GOEREE

HARINGVLIET

Fig. 1 Expected coastal erosion ond accretion neor Hook of Hollond
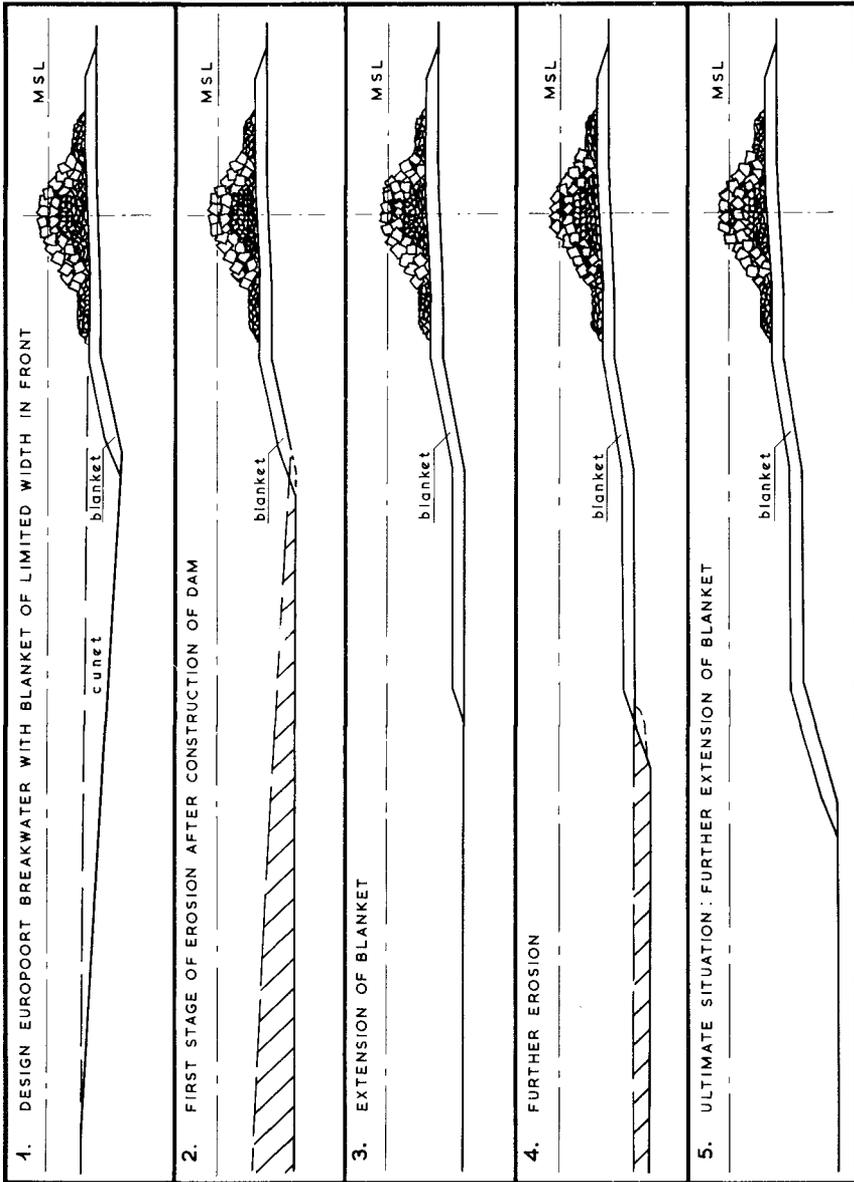
Fig. 2   Program for bottom protection in front of southern Europoort breakwater

The construction of a special foreshore substructure as part of the total structure
may be considered for the design of deep-harbour protection works. Here the
question may arise whether it will economically be more feasible to build the
actual breakwater on a mound of relatively small-sized material and with such a
configuration that the actual wave attack on the superstructure will be limited
by the interaction of the incident wave motion with the substructure. This may lead
to the use of smaller armour units in the superstructure. However, the amounts of
material involved are huge, so that in most cases the sea-bed profile will be com-
posed of the existing sea-bed configuration with possible erosion and/or accretion.

The effect of the foreshore configuration on the overall stability of the
breakwater has been clearly demonstrated by model experiments carried out for the
design of the Europoort breakwaters at the new harbour entrance to Rotterdam in
commission of the Netherlands Government Department of Public Works (Rijkswater-
staat). These breakwaters will partly be constructed in relatively shallow water. After
the new entrance has been completed, the depth in front of the bed protection can
be expected to increase due to the contraction of the tidal currents and large-
scale dredging, as is shown in Figure 1. In addition, design conditions are such
that wave attack increases with depth. The final depth cannot be determined
accurately at all places, partly because of the uncertainties in the development
of ship sizes and partly because of the complexity of the coastal development.
Therefore, when the deepening tends to exceed a certain value, a fixation of the
sea-bed over a sufficiently large area is envisaged rather than a design based on the most
unfavourable conditions that might occur. The different stages of construction are
indicated in Figure 2.

2.   GEOMETRY OF THE BREAKING WAVE

Maximum wave attack can be expected to occur when a structure is sub-
jected to plunging breakers. After the waves have become initially unstable, they
will progressively deform up to a point where part of the wave front becomes ver-
tical, after which the crest will fall as a free-falling jet (i.e., the so-called
plunging breaker). The place where the wave front becomes vertical is called here
the breaking-point. Dependent on the location of the breaking-point and the path
of the plunging wave, the falling jet may hit the front face of the structure at about
the still water level, or may fall into the water of the wave trough preceding the

breaking wave. In the latter case, a substantial amount of wave energy is converted into splash and turbulence, resulting in a largely reduced wave attack.

According to shallow water wave theory, the velocity of wave propagation is equal to:

$$v = \sqrt{g \cdot d}$$

where
- v = velocity of wave propagation
- g = acceleration of gravity
- d = water depth.

On the analogy of this equation, the horizontally-directed velocity of the breaking wave crest is equalized by:

$$v_B = \sqrt{g \cdot y_B}$$

where
- v_B = horizontally-directed velocity of the breaking wave crest at the breaking-point
- g = acceleration of gravity
- y_B = vertical distance from breaking wave crest up to the bottom at the breaking-point.

This latter equation has also been verified experimentally.

Assuming that the breaking wave crest at the breaking-point is exposed to gravity only, the path of the breaking wave tongue can be computed, starting from the horizontal initial velocity at the wave crest. In illustration: for the Europoort breakwater it may be assumed for the water depth at the breaking-point, d_B = 13.5 m, the breaking wave height, H_B = 10.0 m, and the vertical distance from the wave crest to the bottom, y_B = 21.0 m. Then the horizontal velocity of the wave front at the breaking-point equals v_B = 14.5 m/sec, whilst the plunging distance x_p, i.e., the horizontal distance from the breaking-point up to the point where the falling jet intersects the still water level, amounts to 19.5 m. From Figure 3 it follows that if the wave starts to break at the beginning of the horizontal bottom protection, the plunging jet will fall into the wave trough preceding the breaking wave, whereas if the wave breaks at the end of the horizontal bottom protection the falling jet will hit the front face of the breakwater.
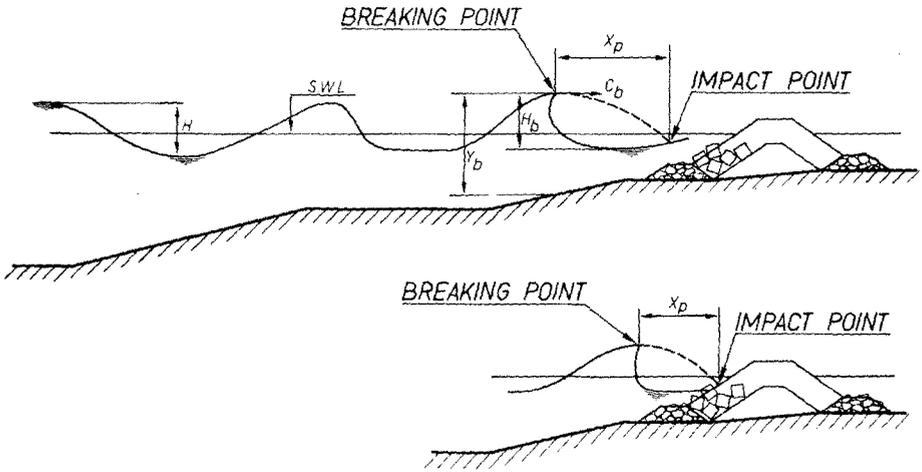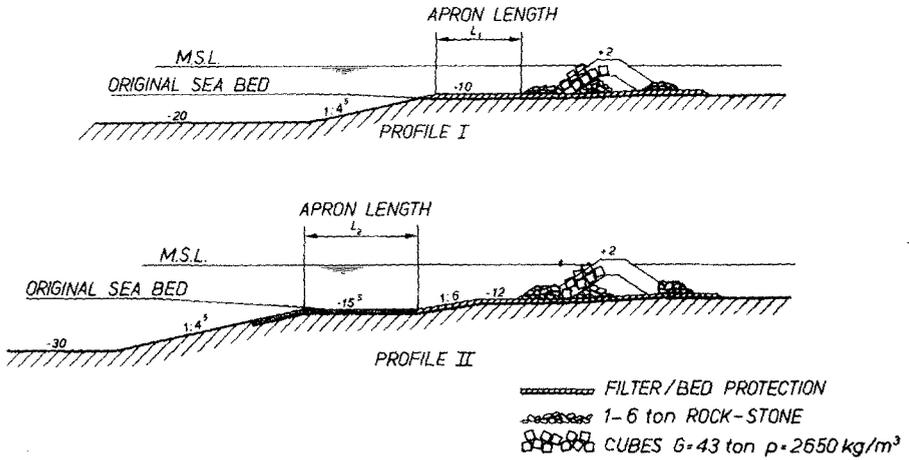
Fig. 3 Breaker geometry

Fig. 4 Europoort breakwater profiles

From the foregoing it will be clear that the location of the breaking-point together with the wave dimensions at the breaking-point will strongly influence the amount of damage to be expected. Exact breaker dimensions as a function of foreshore configuration, water depth and wave periode cannot be predicted from theorerical considerations, nor are sufficient experimental results available, so only general tendencies can be derived from available theoretical and experimental investigations.

Assuming that the ratio between the breaking wave height above the still water level over the total breaking wave height will be nearly constant, the following general tendencies can be predicted with respect to the values of d_B, i.e., the water depth at the breaking-point, and H_B, i.e., the breaking wave height:

-   An extension of the foreshore in a seaward direction, at constant wave period and water depth, can be regarded as a flattening of the slope, which will result in a decrease of the ratio H_B/d_B. This means that if d_B is kept constant, lower wave heights will form plunging breakers at corresponding depths in case of the longer foreshore.

-   An increase of the wave period, at constant water depth and bottom configuration, has the same effect as the extension of the foreshore. Due to the increase of bottom friction, lower wave heights will start to break at corresponding depths when there are longer wave periods.

-   At constant wave period and bottom configuration but increasing water depth, wave heights have to increase equally with water depth to form plunging breakers at corresponding depths.

3. STABILITY EXPERIMENTS

3.1. General

The effect of the foreshore geometry in relation to the wave characteristics will now be discussed on the basis of a selection of experiments carried out on two profiles (viz., Profiles I and II of Figure 4). Profile I is a "shallow" water profile which will be constructed in a previously-dredged trench (cunet). After completion of the breakwater, deepening in front of the breakwater will occur, but is not expected to exceed the level M.S.L. - 20.0 m. Tests with irregular waves were carried out to determine the effect of the apron length L_1 on the stability of the superstructure.

Profile II is a so-called "deep" water profile, which also will be constructed in a previously-dredged trench. Deepening which will occur in front of the break-water is limited by a bed protection, as outlined in Figure 2, and is not expected to exceed the level of M.S.L. - 30.0 m. Experiments with both regular and irregular waves were carried out in order to determine the optimum horizontal apron length L_2 of the bottom protection, so that the same armour units (viz., concrete cubes of 43 tons with a specific density of 2,650 kg/m^3) can be used in all cases. The eroded sea-bed in front of both aprons has been schematized by a fixed slope of 1 : 4.5.

3.2. Models and Testing Facilities

Tests with regular waves have been carried out in the wave basin of the Laboratory De Vaarst, in which waves are generated by a piston-type wave generator. The distance between wave generator and model was about 19 m. During each test, wave period and water level were kept constant, whilst the wave height was increased in steps, each step lasting about 8 hours prototype. At the end of each step the damage was recorded and classified according to so-called damage functions, as shown in Figure 5. During the tests visual observations of the breaker geometry, and especially the location of the impact point, were made.

Tests with irregular waves have been carried out in the wind-wave flumes of the Delft Laboratory (Ref. 3), in which irregular waves with any arbitrary wave spectrum can be generated by a programmed wave generator. The testing procedure was similar to that for the regular waves.

3.3. Results

The experiments with regular waves on the "deep" water Profile II were carried out with wave periods of 12, 14 and 15.5 secs, water levels of M.S.L. + 0.50 m and + 1.50 m, and apron lengths, L_2, of 30 m and 50 m.

Damage functions are given on Figures 5, 6 and 7. The wave period and the apron length as well as the water level appear to have a more or less significant effect on the stability of the structure.
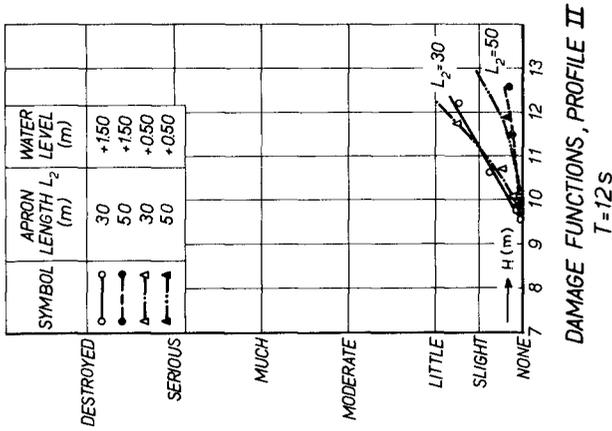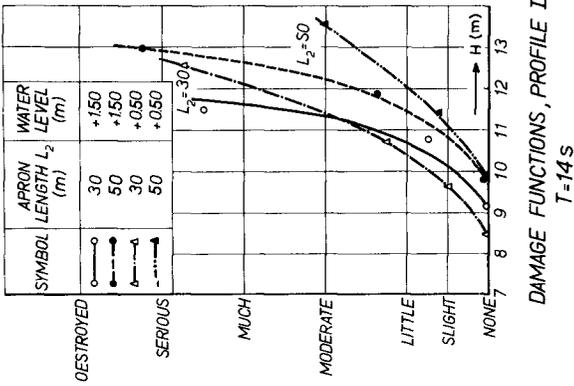
DAMAGE FUNCTIONS, PROFILE II
T=15,5 s

Fig. 7 Damage functions, profile II

DAMAGE FUNCTIONS, PROFILE II
T=14 s

Fig. 6 Damage functions, profile II
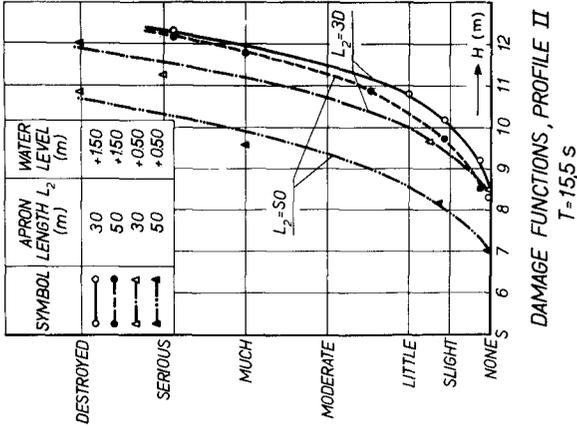

DAMAGE FUNCTIONS, PROFILE II
T=12 s

Fig. 5 Damage functions, profile II

The results on Figures 5, 6 ond 7 have been summarized on Figure 8 by charac-
terizing each damoge curve by the wave height which couses a "little" damage
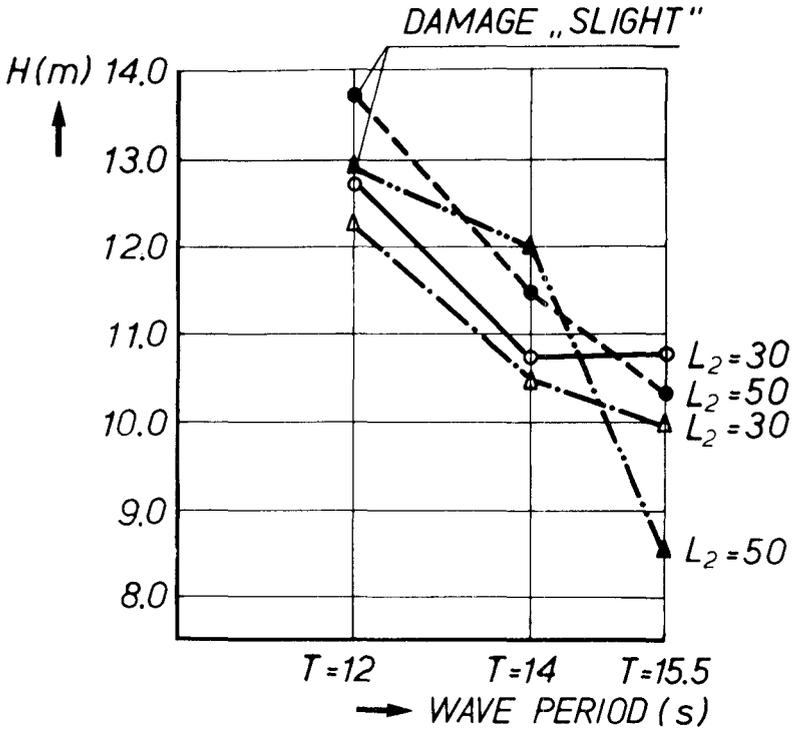to the breakwater.

On analysing these results it appeared that for the most dangerous combi-
nation, viz. $T = 15.5$ sec, $L_2 = 50$ m, and water level M.S.L. + 0.50 m,
critical breaking conditions, as described in Chapter 2, occurred at a wave height
of 8.5 - 9.5 m. At that time waves were just breaking at the toe of the structure,
ond the front face of the breakwater was heavily attacked by the plunging jet,
whilst the stability of the blocks wos olready reduced by the downrush and outflow
of the water.

At a higher water level and/or shorter apron length the incident wave
height hod to be increased beyond 10 m to have the impact point at a similar
pasition an the breakwater face. Lower wave heights were abserved to merely
surge over the crest of the breakwater.

With decreasing wave periods and constant wave heights the rate of
deformation of the wave decreoses as a result of the diminishing effect of the bottom
friction. For $T = 14$ secs and $L_2 = 50$ m plunging breakers which start to break
at a critical distance in front of the breakwater only occur for incident wave
heights which are considerably higher than those at $T = 15.5$ secs. The absolute
difference in rate of deformation between the wove periods $T = 15.5$ secs and
$T = 14$ secs is less for the apron length $L_2 = 30$ m than for $L_2 = 50$ m, which
may explain the small difference in the wave heights, causing "little" damage,
between $T = 15.5$ secs and $T = 14$ secs at this shorter apron length.

With a further decrease of the wave period to 12 secs, incident waves
were observed either to surge over the breakwater crest or to fall into the water
in front of it when the plunging jet did not reach the front face of the break-
water when breaking. On increasing the wave heights, the breaking-point moved
more offshore, resulting in largely reduced wave attack. In two such situotions the
damage described as "little" wos not even reached.

In general, it may be concluded that the foreshore acts as an obstacle
on which the waves become unstable anc' subsequently may break at a certain dis-
tance from the breakwater. The initiation of damage, however, depends entirely
on a very subtle combination of water depth, sea-bed configuration, wave period
and wave height.

DAMAGE „SLIGHT"

H(m) 14.0

13.0

12.0

11.0 ○ $L_2 = 30$

● $L_2 = 50$

△ $L_2 = 30$

10.0

9.0

▲ $L_2 = 50$

8.0

T=12   T=14   T=15.5

→ WAVE PERIOD (s)

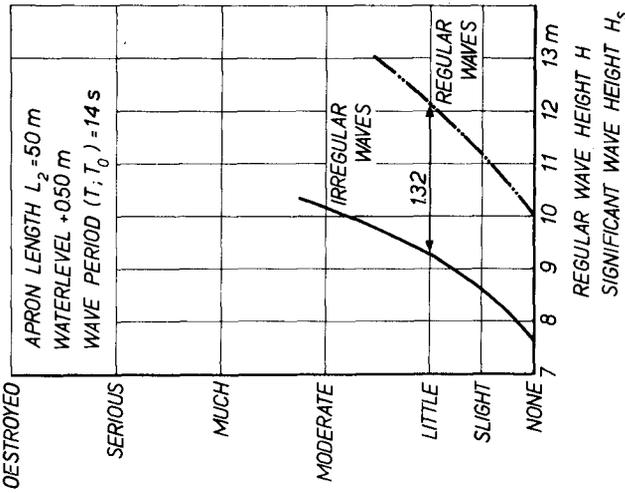|	APRON LENGTH L_2	WATER LEVEL
○——○	30	+1.50
●----●	50	+1.50
△—·—△	30	+0.50
▲—··—▲	50	+0.50

WAVE HEIGHT FOR DAMAGE „LITTLE"
VERSUS WAVE PERIOD.   PROFILE II

Fig. 8   Relation wave height for damage "little" versus wave period
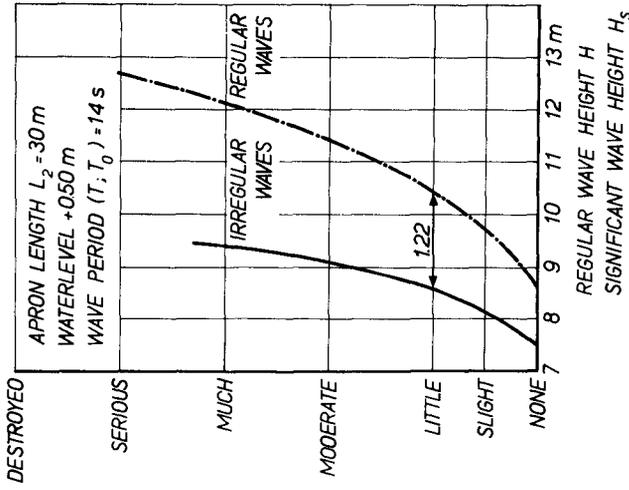
COMPARISON DAMAGE FUNCTIONS FOR REGULAR
AND IRREGULAR WAVES (PROFILE II)

Fig. 10   Comparison damage functions

COMPARISON DAMAGE FUNCTIONS FOR REGULAR
AND IRREGULAR WAVES (PROFILE II)

Fig. 9   Comparion damage functions

In addition to the experiments with regular waves, tests with irregular waves have been carried out. In this case, the significant wave height was increased in steps, every step lasting about 8 hours prototype, whilst the top period, i.e., the period in the wave energy spectrum with maximum energy density, was equal to $T_o = 12$ secs and $T_o = 14$ secs. Some of the results, summarized in damage functions, are shown in Figures 9 and 10. A total of six different combinations of water level, apron length and wave period were tested.

Comparing the damage functions for irregular and regular wave action, both for a "little" damage, it was found that the ratio of the significant wave height H_s, and the regular wave height, H_r to cause similar damage was equal to:

$$\frac{H_r}{H_s} = 1.15 \text{ to } 1.33.$$

So on the average equal damage will occur when

$H_r \cong 1.25 \times H_s$, or $H_r \cong H_4$ %, where H_4 % is the wave height which is exceeded by 4 % of all waves, provided that the regular wave period is equal to the top period of the wave spectrum and that the duration of the tests is also equal.

It is often assumed, partly on the basis of experiments with particular breakwater profiles, that H_r should be equal to H_s in order to produce equal damage (Ref. 4). Although this relation may be correct for particular breakwater cross-sections and a rather high extent of damage, it seems to be purely accidental and it is certainly not generally applicable, as already indicated. In fact, there is no physical basis whatsoever for this equality, as the significant wave height H_s is a more or less arbitrarily chosen characteristic height of an irregular train of waves. In this respect reference is also made to Ref. 5, where from experiments on rip-rap revetments of various slopes and compositions it was concluded that $H_r \cong H_1$ %.

A direct relationship between breaker distance and wave attack can be shown from the experiments with Profile I. These experiments were carried out with irregular waves only and with $T_o = 14$ secs and 4 different apron lengths.
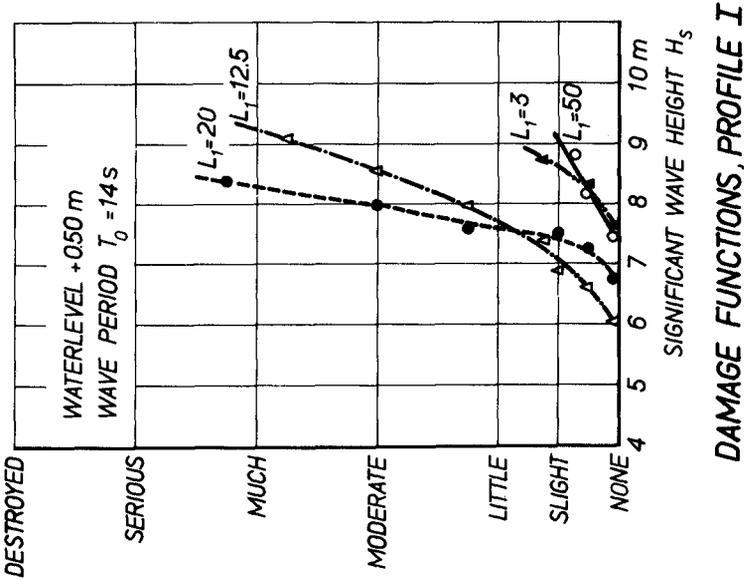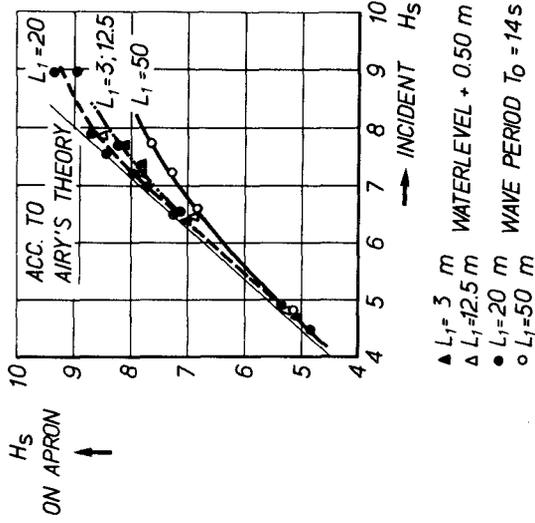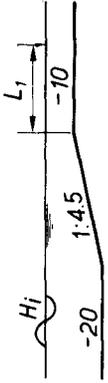
Fig. 12   Variation of wave height on apron

Fig. 11   Damage functions, profile I

Damage functions are presented on Figure 11. Serious damage occurred under design conditions (i.e., H_s = 8.5 m) with the originally proposed apron length L_1 of 20 m. Moreover, the damage increased rapidly with a small increase of wave height. The damage was considerably less for apron length of L_1 = 3 m and L_1 = 50 m. For L_1 = 3 m waves were observed not to break in front of the breakwater, but on the breakwater itself, which causes the wave to surge over the crest, whereas for L_1 = 50 m the impact point of the higher waves is located so far seaward of the breakwater that the wave energy is reduced substantially.

In this respect reference is made to Figure 12, which indicates the relation between the incident wave height and the wave height on the apron in the absence of a structure.

4. CONCLUSIONS

- The provision to extend the bottom protection seaward, entailing the construction of a relatively high foreshore in front of the Europoort breakwater if the depth in front tends to exceed a certain value, proved to be a helpful means of limiting wave forces which would otherwise become prohibitive. However, the design of this foreshore profile should be tested with a large variaty of preferably irregular wave conditions, as particular combinations of wave characteristics and foreshore profiles may give rise to extra large damage.

- The occurrence of maximum wave attack on the Europoort breakwater can be related to the distance between the front face of the structure and the breaking-point. The exact location of the breaking-point and the conditions on which this breaking will occur are very difficult to predict, as they depend on a rather subtle combination of water depth, foreshore configuration, wave period and wave height.

- Experiments with regular waves only give qualitative information, as it is not clear beforehand which wave height within the statistical variety of natural wave heights should be selected as the representative one. The frequently used assumption that $H_{regular}$ has to be equal to the significant wave height H_s may be correct in particular situations, but is in general purely accidental.

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