CHAPTER 102

Ponta Delgada Breakwater Rehabilitation
Risk Assessment with respect to Breakage of Armour Units

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Abstract

The outer portion of the Ponta Delgada Breakwater in the Azores was constructed in the sixties, applying an armour layer of 25 t Tetrapods on a slope of 3 in 4. After two decades of recurring damage and repairs, a redesign and rehabilitation were undertaken.
An armour layer of 40 t Tetrapods on a slope 1 in 2 proved to meet the more severe design wave conditions established for the rehabilitation. This was the most attractive solution from cost and constructability viewpoint. To check the risk of breakage of these units under design conditions, the rocking was analysed both in hydraulic model and by means of an analytical simulation program. This allows to assess the actual impact velocities of rocking units and the percentage breakage under given wave conditions.

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Introduction

Since the major failures of some large rubble mound breakwaters in the period 1978 - 1982 the importance of concrete strength of armour units has become evident. Research was undertaken into this aspect and a joint-industry research project in The Netherlands succeeded in developing algorithms for impact velocities of rocking armour units and the stresses in the concrete. A simulation program was developed to compute the percentage breakage in the armour layer, using these algorithms. (Ligteringen et al, 1990).
The simulation program ROCKING was validated on the basis of several failure cases. One of the first applications in design was for the rehabilitation of the Ponta Delgada breakwater and is presented in this paper.

After a description of the original design and the general approach followed for rehabilitation, the details of the ROCKING analysis are presented.

Breakwater Extension at Ponta Delgada

The port of Ponta Delgada on the island of San Miguel in the Azores is protected by a breakwater of which the outer 1000m is located in over 20 m water depth (Figure 1). This new part was designed and built in the sixties as a rubble mound structure, using an armour layer of 25 t Tetrapods at a slope of 3 in 4 (Figure 2).

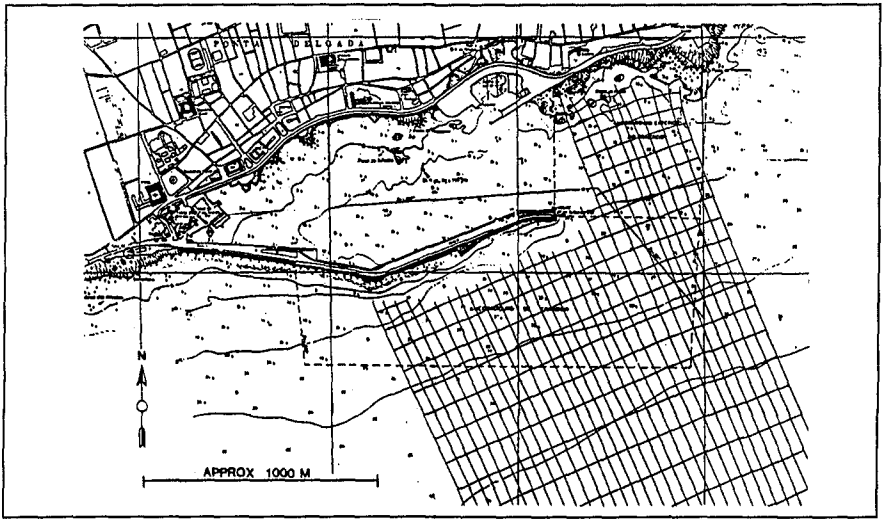

Figure 1. Ponta Delgada Breakwater Lay-out

The design wave height for the project was determined at Hs = 6.5 m, having a return period of 50 year. Using a mass density of 2.5 t/m³ for the concrete, this meant a stability coefficient of 7.5, which is considered to be acceptable for a Tetrapod armour layer.
The head of the breakwater was built as a vertical wall structure, composed of 5 caisson elements.

Unlike the older part of the breakwater, which had required relatively little maintenance, this new extension gave several problems, including displacement of Tetrapods, settlements of and cracks in the large superstructure, and settlements of the caissons at the head.
Consecutive repair works were carried out in 1977/'78, 1983/'84 and in the period 1986 till 1988. It was after this rather extensive repair that the Portuguese Government ordered a detailed study into the causes of damage and the necessary improvements. This study was awarded to Consulmar in Portugal, with specialist assistance provided by Frederic R. Harris in the Netherlands.

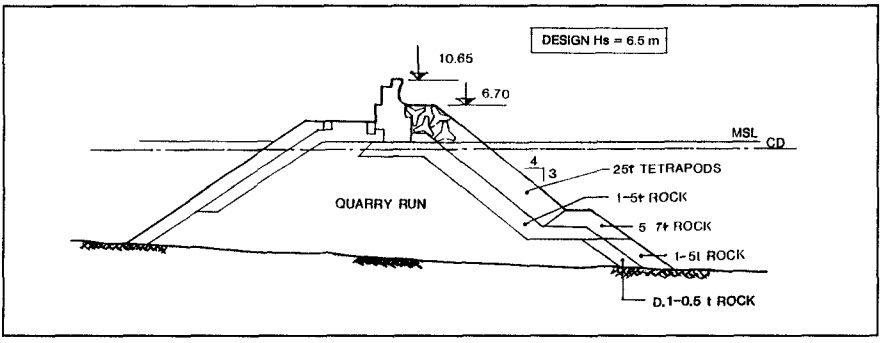

Figure 2. Original Breakwater Design

Rehabilitation Study

The study comprised re-analysis of the wave climate at Ponta Delgada, assessment of the main causes of the damage to the existing structure, generation and evaluation of alternatives for rehabilitation and selection of the concept for detailed design and construction.

The deep-water wave climate at the Azores was established by using hindcast storm data for the area covering a period of 8 years, obtained from the U.K. Meteorologic Office, refraction computations and in-situ measurements by a wave buoy. Because the execution of the measurements was delayed, a two-step approach was followed: a preliminary wave climate was defined for the first phase of the study, while for the detailed design the wave measurements and additional hindcast computations would provide the final design conditions.

The preliminary results indicated the 100-years wave height at the site to be about Hs = 10 m. Spectral periods could range from Tp = 10 to 18s.

Relevant wave directions for Ponta Delgada were in the sector 90 - 270°. The refraction analysis did not show significant concentrations of wave energy. As an example one such computation has been shown on Figure 1.

The great difference with the original design wave height explained most of the damage. Since its construction the structure had seen several storms with wave heights at or above design level. Along the older part of the breakwater the water depth had provided a natural limitation of the wave height to levels at or below the original design value, but along the new part much higher waves could reach the structure. Hydraulic damage to the armour layer at the seaward and leeside slopes (due to overtopping), but also higher forces on the superstructure and on the end- caissons were the result.

The development of alternatives was based on the following criteria:

- small damage was acceptable for the 100-yrs wave conditions (Hs = 10 m, Tp = 10-18s)
- run-up and overtopping should be reduced to avoid ongoing damage to the superstructure.
- minimum cost for the rehabilitation and improvement.

Four basic solutions were developed, as shown on Figure 3:

A. A berm placed directly in front of the existing structure, with its crest at -5.0m CD and a small slope towards -10.0m CD.
B. A submerged breakwater at 150m distance seaward of the existing structure.
C. A strengthening of the seaward face of the existing breakwater, maintaining the slope of 3 in 4.
D. A strengthening of the seaward face at a reduced slope of e.g. 1 in 2.

Preliminary designs were made for each solution, based on test data for comparable structures, e.g. alternatives studied for the Sines and San Ciprian rehabilitations. Also aspects of constructability and future maintenance were taken into account. The main results are indicated on Fig. 3. Subsequently cost estimates were prepared for each solution.

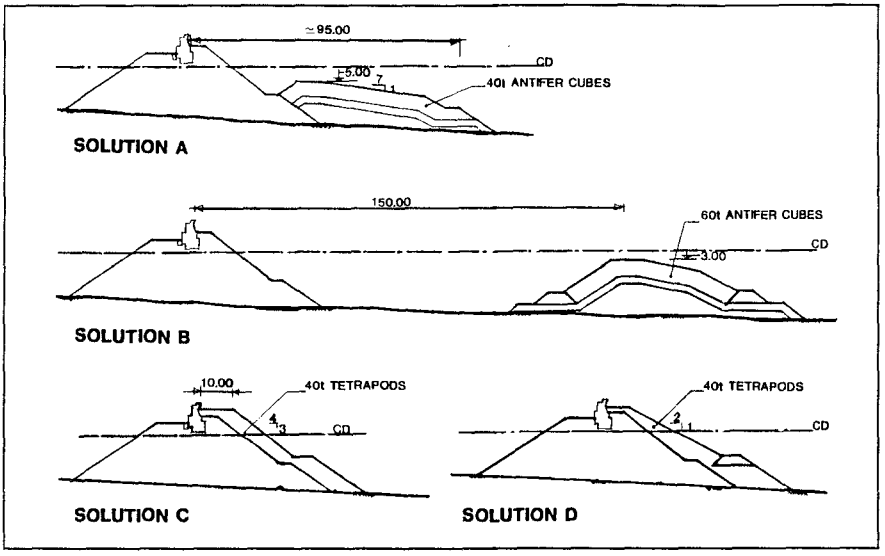

Figure 3. Basic Solutions

Comparison of the four solutions showed that A and B would be far more expensive than solution D, which in turn was slightly more expensive than solution C. Notwithstanding the latter price difference, the overall evaluation led to selection of solution D, for several reasons:

(i) in both solutions the application of large size Tetrapods should be checked during detailed design on possible breakage of these units. In this respect Solution D gave some "reserve" strength, which Solution C did not have.

(ii) the overtopping of Solution D was estimated to be lower, due to the larger volume of the seaward body.

(iii) the new layer of Tetrapods will underwater be entirely placed on a new secondary armour and hence more reliable. Above water the preparation of the existing Tetrapod layer and placing of the new units could be achieved accurately.

A more detailed cross-section of the selected concept is given on Figure 4. The proposed solution for the head was to place a rubble mound structure around the caissons. For the primary armour of this round head 70 to 80 ton Antifer Cubes were selected, because Tetrapods of this size were considered to be too vulnerable with respect to breakage.

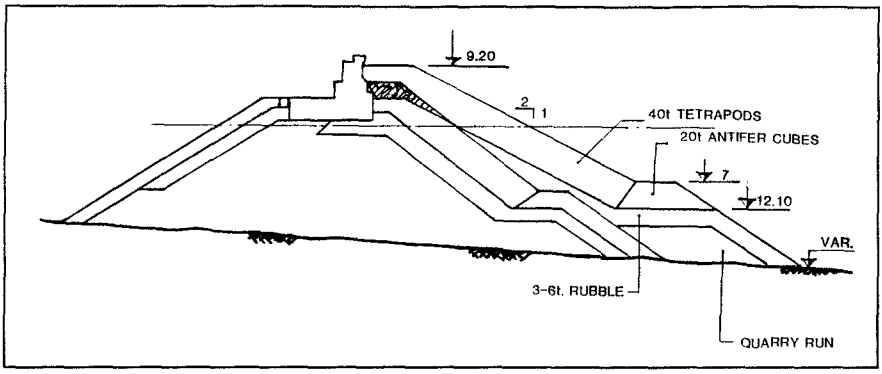

Figure 4. Selected Concept

Detailed Design

Based on the selected concepts, the tasks for detailed design were to check and optimize both trunk and roundhead in modeltests, and to analyse the rocking and potential breakage of the 40 t Tetrapods on the trunk. The procedure followed during this phase of the project is given on Figure 5.

DETAILED DESIGN PROCEDURE

WAVE ANALYSES	- 1 YEAR WAVE BUOY - HINDCAST MAJOR STORMS - COMPARISON BUOY/HINDCAST
MODEL TEST	- 2D AND 3D TESTS - HYDRAULIC STABILITY, ROCKING AND OVERTOPPING
TETRAPOD STRENGTH ANALYSIS	- ROCKING SIMULATION - COMPARISON MODEL/COMPUTER - BREAKAGE ASSESSMENT

Figure 5

As mentioned before the results of 1 year of buoy-recordings came only available during the detailed design. The final determination of the design

wave climate included the following analyses:

- from the measurement a total of 35 storm records were selected of which the highest reached a peak wave height of Hs = 9.0 (December 1989).
- execution of additional hindcast computations by UKMO of the selected storm periods using the same model applied for the earlier hindcast analysis.
- comparison of measured and computed wave conditions, to evaluate the accuracy of the hindcast results and to correct for a possible bias in the computed wave climate.

It was fortunate for this analysis that some severe storms occurred during the year of measurements. The buoy kept functioning and an exceptional good basis for validation of the hindcast-model was obtained. The comparison between measured and computed wave heights for the most severe storm is shown on Figure 6, other records gave a similar good comparison. It could be concluded that the hindcast model gave an accuracy of the peak wave heights within 10%.

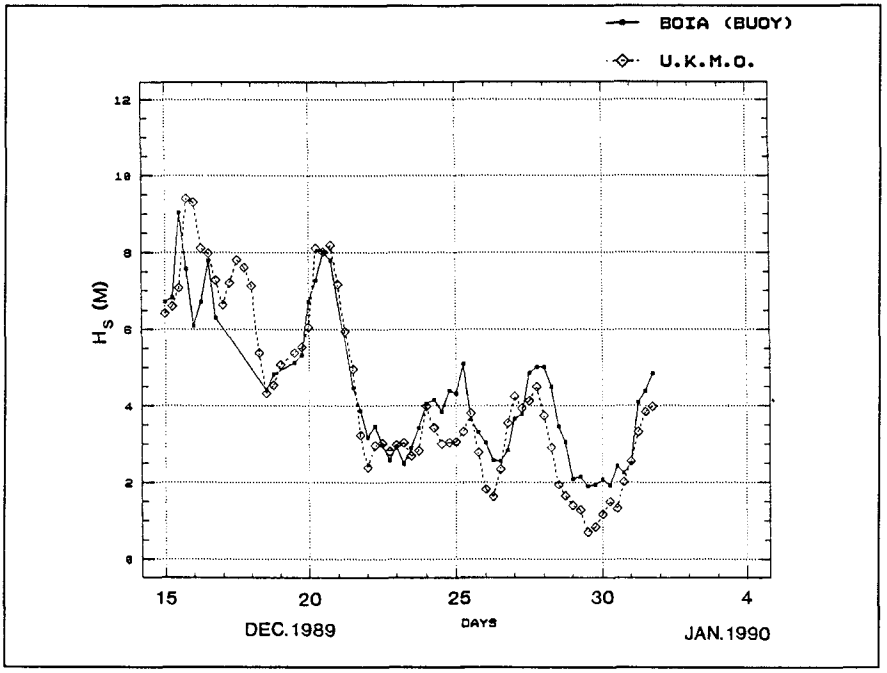

Figure 6. Comparison Measurements with Hindcast

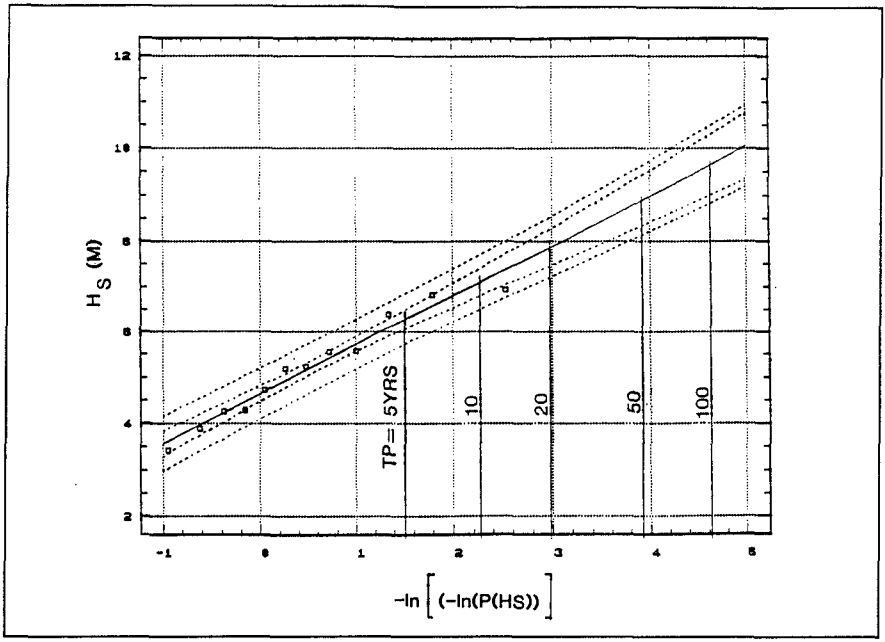

Figure 7. Gumbel distribution of extreme wave heights

The extreme values of wave heights at the breakwater location (original series and additional storms, including refraction effects) were analysed, using several distributions. The best fit was obtained from a Gumbel distribution. As shown on Figure 7, a 100-years wave height Hs = 9.6 m is obtained. Taking into account the above mentioned uncertainty of the hindcast results the value Hs = 10.0 m was maintained as the design wave height.

As the next step in the design process model tests were carried out at the Laboratorio Nacional d'Engenharia Civil (LNEC) in Portugal. Two-and three dimensional tests were carried out on a combined model of trunk and head, scale 1 to 58. The model was subjected to storm sequences, comprising runs with Hs = 4, 6, 8, 10 and 12 m. Other parameters varied are the wave period: Tp = 10, 13 and 18s; and the waterlevel: Lowest Low Water Level (0 m CD) and a high level due to tide and storm set-up (+2m CD). Measurements included recording of wave conditions, hydraulic damage, overtopping and the rocking of armour layers and berms. The latter measurements were made by observations during the tests and by black-and- white photograph overlays after each test run.

The results of the tests on the trunk confirmed the preliminary design to be adequate, but for the level of the toeberm at the seaward face. This had

to be lowered to -10.0mCD.

Of importance for the evaluation of the design with respect to rocking and breakage of Tetrapods was the comparison of hydraulic model with the corresponding results of the simulation model, as presented below.

Rocking Analysis

A schematic representation of the simulation program ROCKING is given on Figure 8. Input parameters for the simulation are:

(i) loads: Hs, Tp and number of waves N.

(ii) structural parameters: relative mass density Δ and nominal diameter of the armour unit, Dn.

(iii) concrete quality: characteristic tensile strength f.

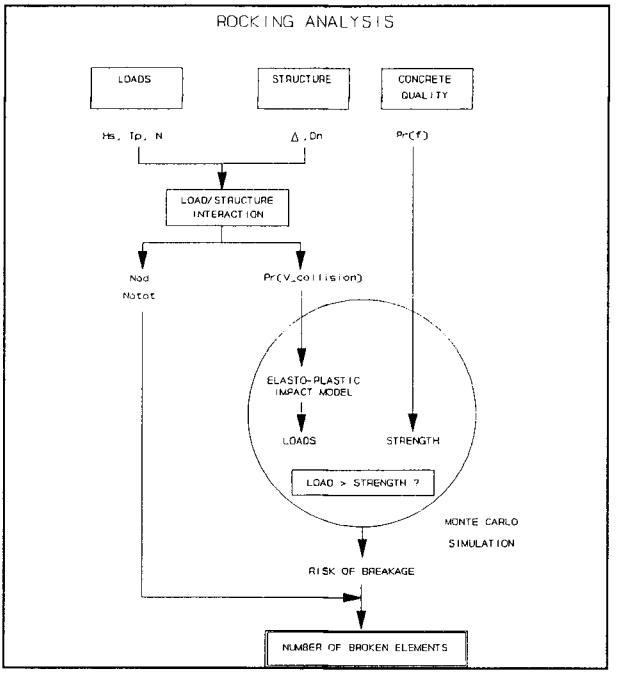

Figure 8. Schematic representation of ROCKING Analysis

For the determination of wave-structure interaction analytical formulae are used, which calculate the number of displaced units in the primary armour layer (Nod) for a given set of input parameters. Based on extensive tests on the correlation between hydraulic damage, rocking intensity and acceleration (Van der Meer and Heydra, 1990), the model generates the following results:

(i) the sum of displaced and rocking units (Notot), from which the number of rocking units can be deduced.

(ii) Impact velocity at collision of two adjacent units, given as a distribution function.

The latter distribution function is combined with the tensile strength of units (also given as a distribution function) in a stochastic determination of the risk of breakage for the applied input parameters. This is done by means of Monte Carlo simulation. For a large number of collisions the sampled value of impact velocity is translated into a peak-load exerted by the colliding units, applying a non-linear elasto-plastic impact model. (Van Mier and Lenos, 1991). Loads are converted into stresses, assuming at random one of a number of typical orientations for each of the two units. The calculated stresses are compared with the tensile strength, sampled from the distribution, and if the stress exceeds the actual tensile strength the unit is assumed to be broken.
The Monte Carlo simulation gives the percentages of rocking units, that will break in the pertaining wave conditions, or the risk of breakage. By multiplication with the number of rocking units the number of broken units, is obtained.

The actual simulation for the 40 t Tetrapods was carried out for the design wave conditions only, as shown on Figure 9. The concrete characteristics were taken in accordance with Portuguese standards, but for the relation between compression and tensile strength different expressions were applied, to test the sensitivity of the results for this value.

On Figure 10 a first comparison of measured results and computed displacement and rocking is presented. This gives a check on the validity of the empirical models for hydraulic displacements and rocking used in the simulation program.

The observed values show the range obtained from the model tests for the different wave periods. The percentage displaced units is very well predicted by the formula for Tetrapods, while the percentage rocking units is slightly overpredicted by the formula.

ROCKING ANALYSIS

BREAKAGE OF 40 T TETRAPODS

* Hs = 10 m
* Tp = 10, 13, 18 s

* COMPRESSION STRENGTH:

$f' = 33.9$ MPa
$\sigma_{f'} = 5.4$ MPa

* TENSILE STRENGTH:

$f = 1.0 + 0.05 \cdot f' = 2.70$ MPa
$\sigma_{f'} = 0.1\ f' = 0.27$ MPa

* SENSITIVITY ANALYSIS

$f = 1.5 + 0.05\ f' = 3.20$ MPa; $\sigma_{f'} = 0.1\ f$ MPa
$f = 0.5 + 0.05\ f' = 2.20$ MPa; $\sigma_{f'} = 0.1\ f$ MPa
$f = -2.4 + 0.15\ f' = 2.70$ MPa; $\sigma_{f'} = 0.1\ f$ MPa

Figure 9. Boundary Conditions Rocking Analysis

The final results of the simulation are presented on Figure 11, depicting percentage of broken Tetrapods for Hs = 10 m and the total damage, including the hydraulic damage given on Figure 10. The main conclusion is that the maximum percentage of total damage (3.5%) for the design wave condition is acceptable. Further it can be observed that the long wave periods are more critical for the hydraulic damage as well as for breakage. Finally the sensitivity of the results for the applied variations of the tensile strength is very limited (this does not mean that tensile strength is not important).

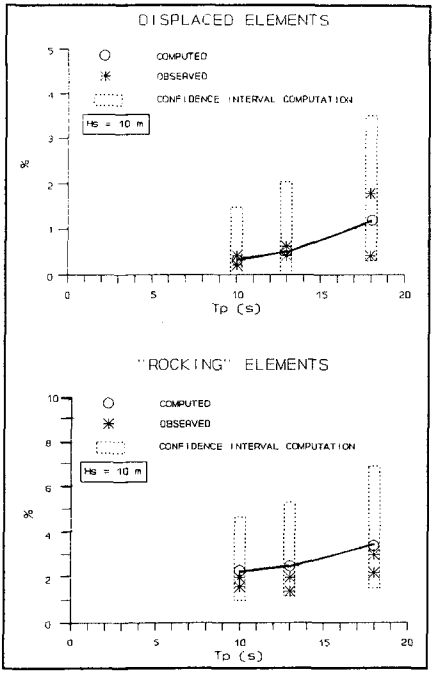

Figure 10. Comparison ROCKING results with test results

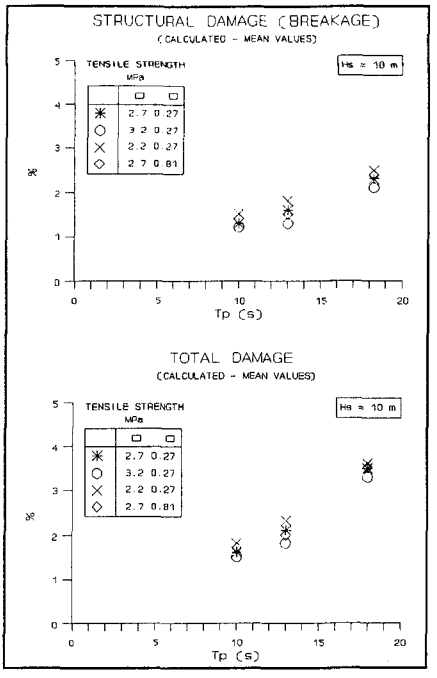

Figure 11. Results ROCKING analysis: risk of breakage.

Discussion of Results

The simulation program ROCKING allows to assess the additional damage to an armour layer due to breakage of the concrete units caused by rocking. The model proves to be a valuable design tool in cases such as presented in this paper, namely when the breakwater is protected by armour units of a size, which makes them susceptible to breakage.

The present version of ROCKING still has a number of limitations:

(i) it can only handle primary armour layers comprising Cubes and Tetrapods.

(ii) the empirical formulae describing wave-structure interaction are based on limited test data. Further testing is needed on influence of slope angle.

(iii) the model assumes broken units to be eliminated from the slope, while in actual fact broken units may either cause further damage or

wedge themselves and contribute to the strength.

Further research will be needed to improve the range of application and the accuracy of the prediction.

Finally it is stressed that the model only describes breakage due to rocking. Other breakage may occur during placing or due to the static load on units, in particular those in the lower part of the armour layer.

Acknowledgement

The program ROCKING was developed as the result of a joint-industry research project in The Netherlands, in which participated Frederic R. Harris B.V., F.C. de Weger International B.V., Royal Volker Stevin, Hollandsche Beton Groep N.V., Ballast Nedam Engineering B.V., TNO-Bouw, Delft University of Technology, Delft Hydraulics and Rijkswaterstaat.

References

Ligteringen, H., Altink, H., and Van Oorschot, J.H., 1990. Strength of Concrete Armour Units; Stresses in Concrete Armour Units, ASCE, New York (1990).

Van der Meer, J.W. and Heydra, G. 1990. Impact Velocities of Rocking Armour Units; Stresses in Concrete Armour Units, ASCE, New York (1990).

Van Mier, J.G.M. and Lenos, S., 1991. Experimental Analysis of the Load-time Histories of Concrete to Concrete Impact. Jnl. Coastal Engineering, Vol. 15 (1991)