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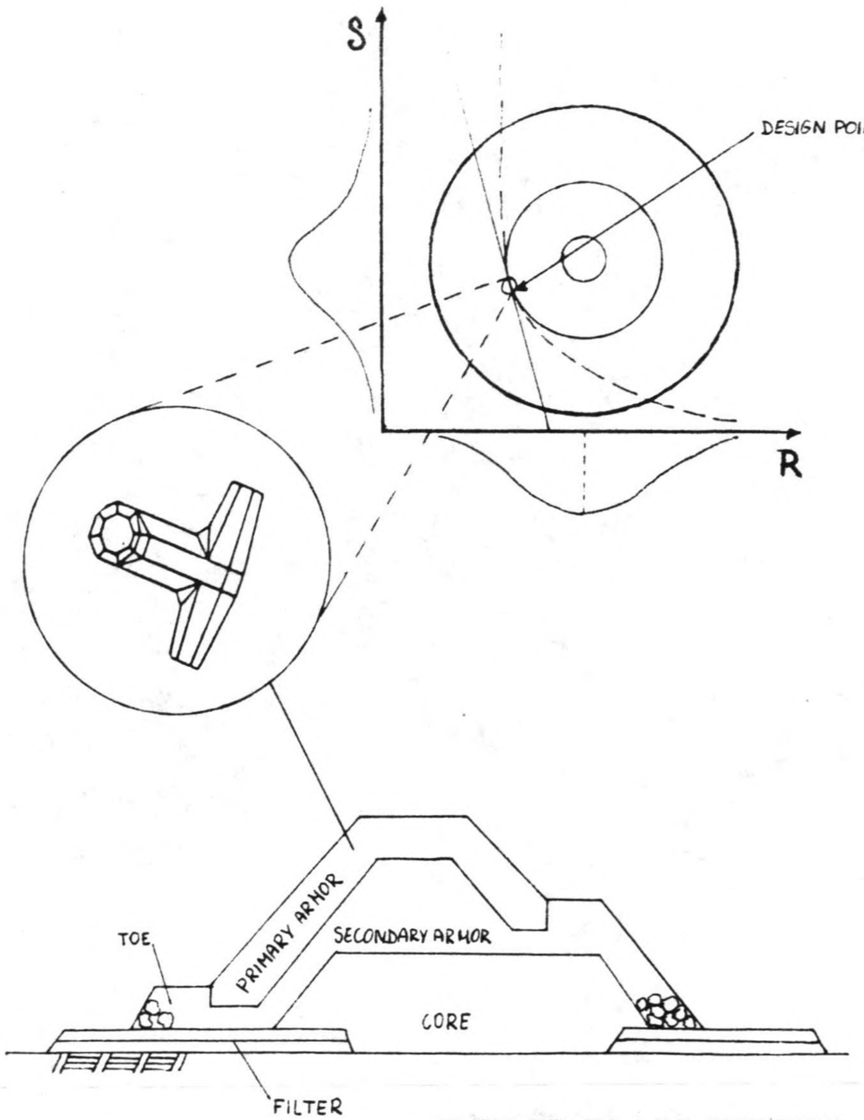

DIRK VAN EPEN
JASMIJN NIEUWENHUIJS

TU Delft
Faculteit der Civiele Techniek
vakgroep Waterbouwkunde

A PROBABILISTIC APPROACH OF THE HYDRAULIC AND STRUCTURAL DAMAGE TO THE DOLOS-LAYER FROM THE SOUTHERN BREAKWATER IN RICHARDS BAY

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authors:
D. van Epen
J. Nieuwenhuijs

graduate committee
Prof. dr. ir E.W. Bijker
Prof. ir. K. D'Angremond
ir. H. Ligteringen
ir. W. Meermans

PREFACE

The Permanent International Association of Navigation Congresses (PIANC) is a worldwide non-political and non-profit technical organization which objective it is to promote the maintenance and operation of navigation by fostering progress in the design, construction, maintenance and operation of waterways and ports. This includes the field of studies on coastal structures like breakwaters.
Following the failure of, or damage to, a number of large breakwaters, PIANC's PTC II (Permanent Technical Committee II) produced a report on the stability of rubble mound breakwaters in deeper water in 1985. This report highlighted areas of uncertainty in the analysis, design and construction of rubble mound breakwaters. In late 1986 a further working group was set up for the analysis of rubble mound breakwaters with the following terms of reference:

> "the immediate need for designers of breakwaters is to achieve a better understanding of the overall safety aspects in the design of rubble mound breakwaters, this is important because the existing design tools are still very incomplete."

The practical approach was:

1. Identify and list the parameters to be considered in the design of breakwater structures. Evaluate the relative importance and the quality of existing available knowledge related to these parameters.
2. Evaluate the safety aspects and propose ways of dealing with the safety problems in practical ways. This should include a check of the proposed safety guide-lines against the behaviour of selected existing breakwater structures.

The working group decided to operate through several sub-groups. It was the task of one subgroup to make a selection of existing breakwater structures for which reliable data on environmental conditions, structural parameters and structural response (damage or no-damage) were already available or could be made available. These cases should provide a basis for 'calibration' of the safety guide-lines, obtained by probabilistic analysis.
We participated in this subgroup. Using the results of work done by the previous subgroups, our contribution has been to summarize and structure the information on the selected cases. This in order to facilitate the more detailed analysis carried out by the other groups. A list of 10 selected cases was made up, with four different types of breakwaters. The selected cases were all at least 10 years in operation, to allow a meaningful performance evaluation. The detailed data collection on each of the selected cases was structured on the basis of a questionnaire. This questionnaire asked for data on geometry, material parameters, environmental parameters and design procedures. In addition we made some preliminary checks on the consistency of the data. The results are presented in our first report (Lit[1]).

SUMMARY

From the ten breakwaters we selected, on the amount of reliable information available, one breakwater in Richards Bay, South Africa. The preliminary check, made in the first phase, showed the design of this breakwater was rather conservative. From surveys done on the breakwater although, it appeared that the armor-layer was damaged in storms with a significant wave height below the design wave height. In the chapters 1 and 3 a description is given of the breakwater in Richards Bay.
The traditional design process of armor units on a breakwater consists of the dimensioning of the units with the Hudson-formula together with model tests. The Hudson formula is described in chapter 2, this formula has several short-comings such as: no influence of wave period, spectrum shape and permeability of the structure. Therefore modeltests are required to complete the design.
This traditional approach in the design process, based on creating a sufficient margin between the load and resistance, takes no notice of uncertainties in the results of wave climate, model testing and construction. Probability methods can be used to account for the distribution of the parameters in the design-formula. The theoretical background of these probabilistic methods is described in chapter 4.
Chapter 5 includes a calculation of the probability of exceedance of a certain damage level to the armor layer. This calculation is done on level II, with data provided by the breakwater in Richards Bay. The parameters giving the highest contribution to the probability of exceedance in this calculation are the significant wave height and the damage coefficient. Although the calculation has several short-comings, it shows how this method can be used in the design process.
With traditional methods as well as with probabilistic methods, errors may occur when a relevant failure mode is not taken into account. Chapter 6 observes the consequences of not considering breakage in the design process of the armor units used on Richards Bay's breakwater.
Although there is little knowledge on the relation between wave action on the breakwater and breakage of armor units, in chapter 8 a relation will be derived from prototype data. Together with this relation a method will be derived to calculate the probability of failure of the breakwater concerning breakage and hydraulic failure. (chapter 9)

1 RICHARDS BAY BREAKWATER

1.1 Introduction

Richards Bay is a harbour on the east coast of South Africa protected by three breakwaters. A breakwater is a protection made of stone, or concrete elements, in order to provide quieter water for ships to navigate and moor. Fig 1.1 shows the layout of the breakwaters and local seabed topography. The seabed is sandy with no rock outcrops.
The data from the Southern Breakwater, the largest breakwater of the three, will be used in a probabilistic calculation. For a complete description of this breakwater, we refer to our previous report, Lit [1].
The breakwater has many different cross-sections but this report will deal only with the cross-section of the round-head, which can be seen in fig 1.2.

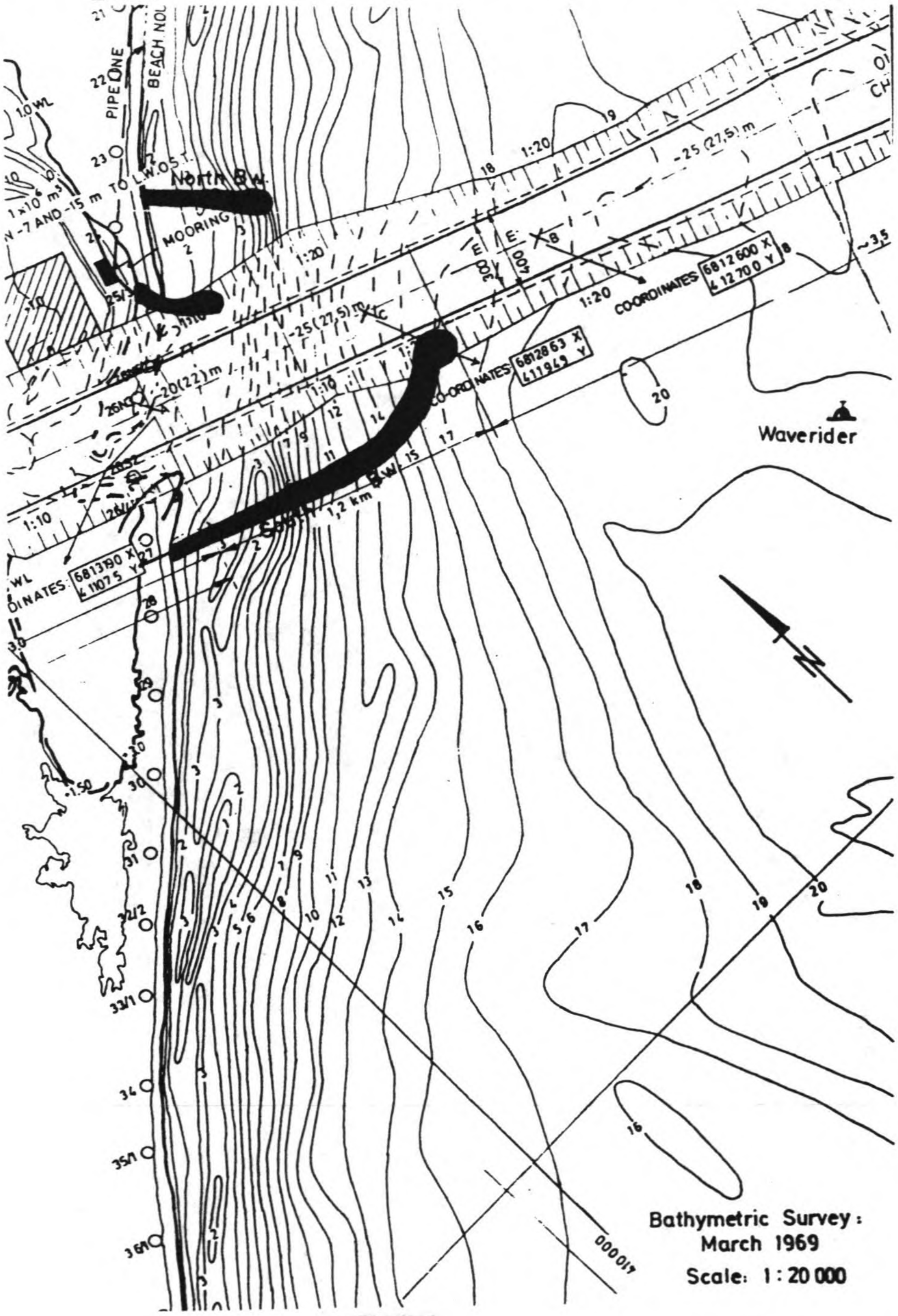
1.2 Dolosse

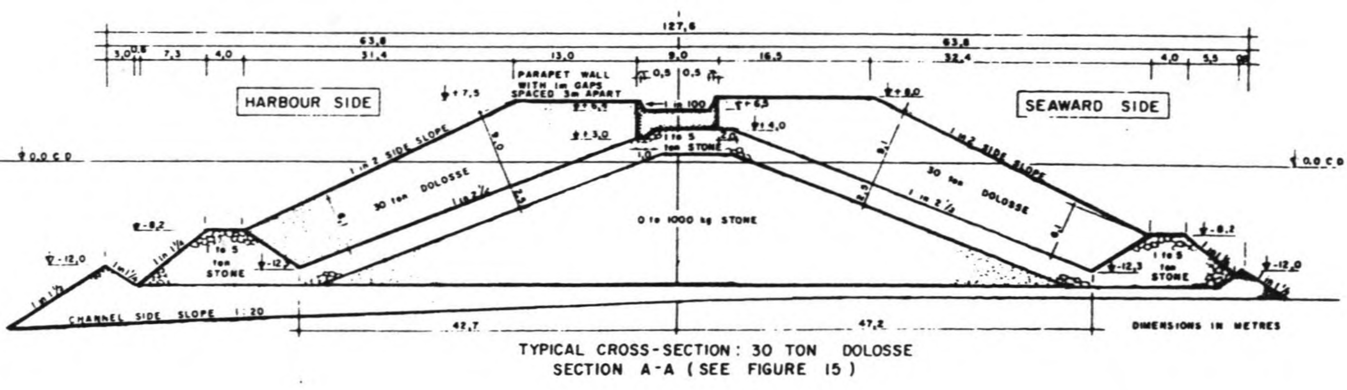
The breakwater is constructed in layers of different stone sizes. As a general rule, each layer of the breakwater must be designed in such a manner, that the adjacent layer of finer material cannot escape by being washed through its voids. The outer layer (primary armor layer), both in final form and during construction, must be designed to withstand the expected wave attack. The stabilizing force, provided by the unit in the primary armor layer, is the collective contribution of:

- weight,
- friction resistance,
- interlock resistance.

The first two components are related to the submerged weight of the units, and the third component is mainly dependent on the shape of the unit.
The primary armor layer of the head-section of the Southern Breakwater in Richards Bay consists of two to three layers of randomly placed dolosse of 30 ton. The dolosse armor unit is developed in 1963 by E.M. Merrifield, Republic of South Africa. This concrete unit closely resembles a ship anchor or an "H" with one vertical perpendicular to the other. Detailed dimensions of the dolosse in Richards Bay are shown in fig 1.3. Dolosse have a great hydraulic stability under wave attack, this is mainly due to their great interlock resistance. Having a great interlock resistance means the mass of the units can be reduced to get the same amount of hydraulic stability. If this breakwater was designed with the Hudson formula with a primary armor layer made of quarrystone, the mass of the stones used would be approximately 70 ton.

fig 1.1 layout breakwaters in Richards Bay
fig 1.2 cross-section of the Southern Breakwater

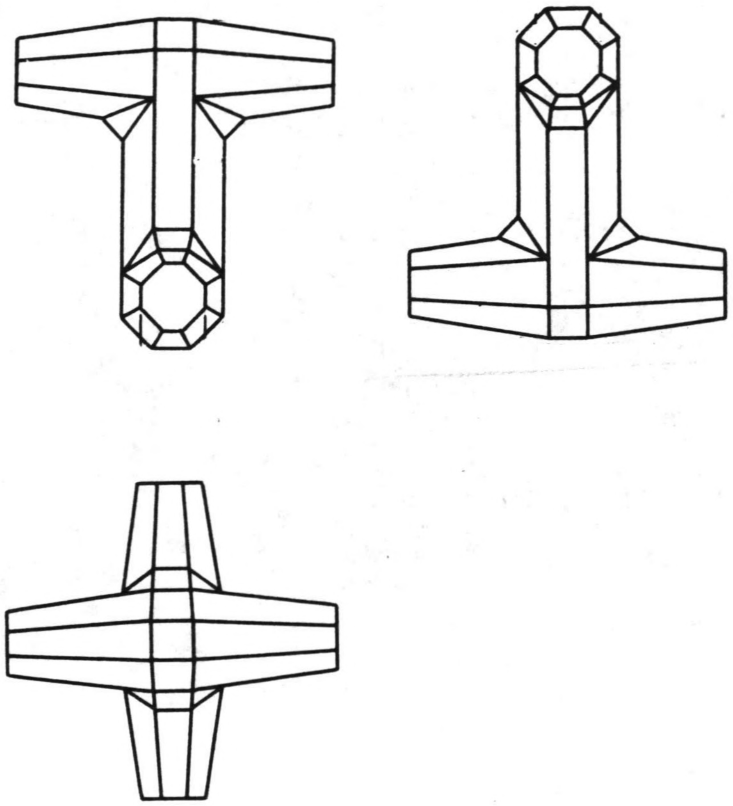




TYPICAL CROSS-SECTION: 30 TON DOLOSSE
SECTION A-A (SEE FIGURE 15)

fig 1.3 30 ton dolosse

dolos height	4.24m
waist ratio	0.36
volume	12.5 m^3
specific weight	2.4 ton/m^3
placement	random



1.3 Design

Based on a wave height which is exceeded only once in 50 years, with a six hours storm duration, a design wave of 7.2m. was selected, under this condition about 2% damage to the armor layer is considered acceptable. Three dimensional stability tests were carried out (Lit[7]) with the dolosse, to check the 2%-limit of the damage. The test results showed a weakness of the original head design which was improved by increasing the dolos layer thickness. The final layout of the breakwater head showed no damage in excess of two percent for the design wave, and the damage for larger wave heights did not increase too drastically.

1.4 Breakwater after completion

The breakwater was finished in May 1976. The wave characteristics occurring after completion of the breakwater were measured by a wave rider buoy anchored in 20 m. waterdepth 1.5 km. seaward of the head of the breakwater. Table 1 shows the significant wave heights (Hs is the average value of the highest one-third of the waves recorded in a storm), correlated with the damage to the armor layer, determined from close-up photographs. The table shows no significant wave height in front of the breakwater above the 5.4m. therefore the expected damage must be below the 2%. The table however shows damage figures up to 4.5%.

Table 1
wave conditions during storms correlated with damage

date	Hs[m.]	damage[%]
0309'78	5.3	
2806'79		2.0
2407'79	4.3	
1704'80	5.3	
1708'81	4.5	
2406'83		4.0 (2.0*)
1702'84	5.3	
03'84		4.3 (0.3*)
0904'84	2.5	
1004'84	4.4	
2704'84	5.4	
3005'84	5.4	
06'85		4.5 (0.2*)

*increase since last survey

The high damage values are probably due to breakage of dolosse. The fact that breakage is a problem is demonstrated by the repair works carried out during Sept/Nov. 1985 by placing 52 new dolosse. At least 20 of them were lost or broken by Dec. 1987, probably due to the lack of interlock of the single layer of new dolosse placed over pieces of broken dolosse.

1.5 Summary

The preliminary check, made in the first phase, showed the design of this breakwater was rather conservative (this is described in chapter 3). From surveys done on the head of the breakwater although, it appeared that the 30 ton dolosse-layer was damaged at a storm with a significant wave height of 5.3 m. This is 1.9 m below the design wave height of 7.2 m.

2 HUDSON THEORETICAL

2.1 Introduction

Until about 1930, design of rubble structures was based only on experience and general knowledge of site conditions. Empirical formulae that were developed are generally expressed in terms of the stone weight required to withstand design wave conditions. Following work by Iribarren (1938) and Nogales Y Olano (1950), comprehensive investigations were made by Hudson (1953). A formula was developed to determine the stability of armor units on rubblemound structures.

2.2 Hudson formula

Hudsons stability formula, based on the results of extensive small scale model testing and some preliminary verification by large scale model testing is:

$$M = \frac{\rho H_s^3}{K_d \Delta^3 \cot(\alpha)}$$

in which:

M =	mass of an armor unit [ton]
H =	wave height [m.]
ρ =	specific density of the stone used [ton/m^3]
cot(α) =	angle of structure slope
Kd =	damage coefficient
Δ =	relative density

The formula, as originally derived by Hudson, is only valid for:

. slopes not steeper than 1: 1.5 and not smoother than 1: 6,
. the front slope of a breakwater,
. breakwaters made of quarrystone,
. breakwaters subjected to non-breaking waves.

2.3 Damage coefficient

The dimensionless Kd-factor accounts for all variables other than structure slope, wave height, and the specific gravity of water at the site. Tests were done with a view to establishing values of Kd for various conditions of some variables, this resulted in the basis for selecting Kd:

1 shape of armor unit
2 number of units comprising the thickness of the armor layer
3 manner of placing of the units (random or not-random)
4 type of wave attack on the structure (breaking or non-breaking waves)
5 part of the structure (head- or trunk-section)

The part of the structure where the armor units are placed, is important because the convex shape of the end of the breakwater can be expected to increase the exposure of the armor units to wave attack. In addition, the convexity can reduce the degree of interlocking o the units. In the Hudson formula this is represented by reducing the value of the damage coefficient Kd.

2.4 Use of the Hudson formula

The Shore Protection Manual (_SPM, Lit [4]) gives values for the Kd for dolosse according to the variables listed above. In the design process, the Kd-value is chosen according to this table. The Kd-values in the SPM are indicated as 'no-damage-criteria', but actually comprise damage up to 5%. The Kd-values must be reduced by 50% according to the SPM (Lit 4) if a damage level of 0-2% is desired. The hydraulic damage is defined as the number of armor units displaced over a distance equivalent to their own height as a percentage of the total number of dolosse present within an area from the middle of the crest to a depth equivalent to the design wave height below SWL.
Hudson's original tests were done using regular waves. The latest version of the SPM (Lit 4), uses therefore $H_{0.1}$ (which is the average value of the highest 10% of the waves, equivalent to about 1.27 times the significant wave height) in the Hudson formula.

2.5 Summary

The Hudson formula has several short-comings such as: no influence of wave period, spectrum shape and permeability of the structure. Because the Hudson formula is the only design-formula used for dolosse it is applied in this report. In practice the formula should be considered to be a guide-line for preliminary design, model testing is required to complete the design of the armor layer.
The dimensionless Kd-factor accounts for a lot of variables, and is chosen according to the SPM (Lit. 4). For the situation in Richards Bay, dolosse, head of the structure, subjected to non-breaking waves, with a slope of 1: 2, and a damage level of 0-2%, a Kd-value of 8 would be needed.

3 DETERMINISTIC DESCRIPTION

3.1 Introduction

Historically the size of concrete armor units has generally been determined using, one or a combination, of the following items:

- simple empirical expressions, principally the Hudson formula, relating armor mass to wave height;
- experience of other similar structures;
- site specific hydraulic model tests.

These methods used in the design process should introduce a sufficient safety margin between load and resistance to prevent severe damage or collapse of the primary armor layer.

3.2 Hudson

The design-values from the breakwater in Richards Bay for the different variables are:

variable	value	
Hs	7.2m.	(return period 50 years)
Δ	1.33	
ρ	2.4 ton/m ^{3}	
Kd	8.0	(0-2% damage)
$\cot(\alpha)$	2.0	

Using the Hudson formula with the values above, results in a mass of the dolosse on the breakwater-head of 24 ton. As mentioned in chapter 1, the mass used for the dolosse is 30 ton, this means the dolosse have 25% more mass then would be necessary according to the Hudson formula.

3.3 Model tests

Tests were done with different wave directions for wave periods of 12 sec. Each direction was tested with increasing wave height:

4, 5, 6, 7.2, 8, 9, 10 meter.

Each wave height was allowed to run for 40 minutes, resembling more or less a storm condition of 6 hours duration. This resulted in the damage figures show in fig 3.1.

Tests were done with waves up to 10 meter in order to check on the increase of damage beyond the design wave height. This increase of damage with wave height showed to be almost linear.

This relation however, between hydraulic damage and the significant wave height, is no exact relation. Due to uncertainties in the parameters, this relation has a certain dispersion. The dispersion can be found with a lot of model tests, and expressed in the Hudson-formula by a variation in the damage-coefficient Kd.

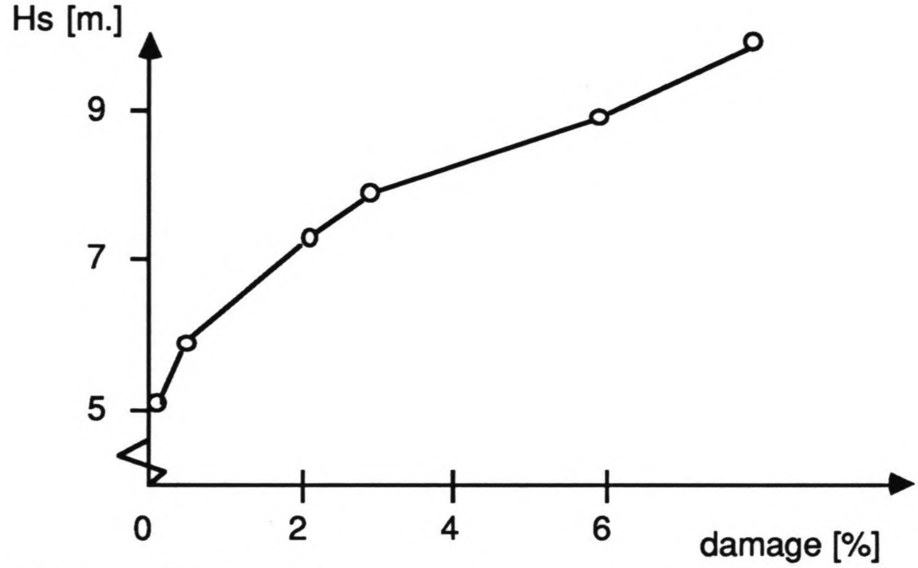


fig 3.1 model test results

3.4 Summary

As stated in chapter 1, according to the Hudson formula and the model tests done on the breakwater, the design showed to be rather conservative. This traditional approach in the design process, based on creating a sufficient margin between the load and resistance, takes no notice of uncertainties in the results of wave climate (Hs), model testing (Kd) and construction. The understanding has developed that this deterministic approach is unsatisfactory from an engineering point of view (safety factors rely on empirism and do not allow large extrapolations beyond the field of experience) and from an economic point (the design is often conservative). Probability methods, described in the next chapter, can be used to account for the distribution of the parameters in the design-formula.

4 RISK ANALYSIS

4.1 Introduction

The term 'risk' comprises the probability of an undesirable event and the consequences of the occurrence of that event. In formula this is given as:

risk = probability x consequence

By risk analysis may then be understood the whole set of activities aimed at quantifying this risk. A risk analysis consists of three main elements:

hazard
mechanism
consequence

A mechanism is defined as the way the structure responds to hazards. A combination of mechanisms and hazards leads with a certain probability to failure of the breakwater or one of its components. A structure fails if it can no longer perform one of its principle functions. In the case of a breakwater this function is in general the creation of safety for navigation and port protection. A breakwater consists of several components, such as the armor layer, toe, crest element, core, filter; each of which may be prone to many hazards and mechanisms. Collapse of one component may in turn pose a hazard to another component. A useful aid to establishing an ordered pattern in the many hazards, mechanisms and components is provided by a fault tree, comprising all possible failure modes of various components of the structure each with their own partial probability of failure.

4.2 Probability of failure

To calculate the probability of failure due to a particular mechanism it is necessary to have a computational model of the mechanism. On the basis of that model a so-called reliability function Z is established with regard to the limit state (boundary between failure and non-failure) considered.

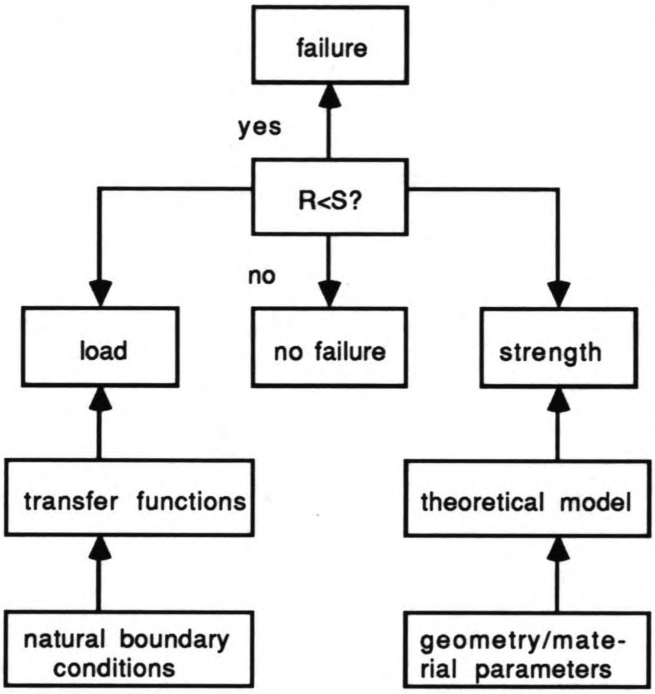


fig 4.1

$Z = R - S$

with:

R= strength of the system (resistance) and

S= load on the system (surcharge)

Failure corresponds to negative values of Z (fig 4.1). The probability of failure can thus be represented as $P\{Z<0\}$ (see fig 4.2). This reliability function is a function of a number of stochastic variables.

There are various techniques available to determine the probability of failure for a given reliability function and given statistical characteristics of the variables. Three methods may be distinguished of determining the probability of failure of a system which are listed in order of detail in analysis and complexity as follows:

Level I: calculations based on characteristic values and safety factors

Level II: comprises a number of approximate methods in which the reliability function is linearized and all probability density functions are locally approximated by normal distributions.

Level III: calculations in which the complete probability densities of the stochastic variables are introduced and the possible non-linear character of the Z-function is fully accounted for.

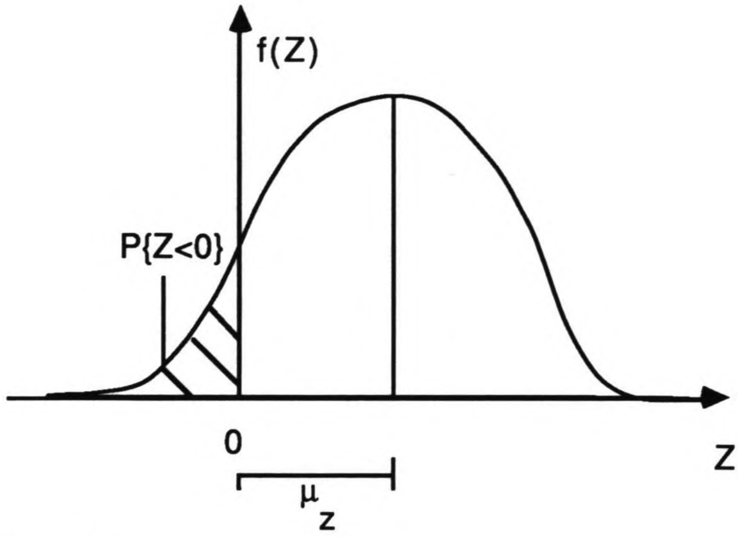


fig 4.2

4.3 Level II

In the case study presented in this report level II methods are applied. The central feature of a level II analysis is the linearization of the function Z (see appendix). Two methods have been developed in this regard.
The Mean Value Approach assumes that all stochastic variables follow the normal distribution characterized by mean value μ and standard deviation σ. This is further improved in an other method, which will be used in this report:

- advanced full distribution approach (AFDA), where the actual distribution is locally approximated by a normal distribution with the same probability and probability density in the design point.

The design point is defined as the point on the failure boundary where the probability density attains a maximum (fig 4.3).
A level II approach has several short-comings:

- all stochastic parameters are assumed to be normally distributed (or are locally approximated by normal distributions)
- all parameters are assumed to be independent

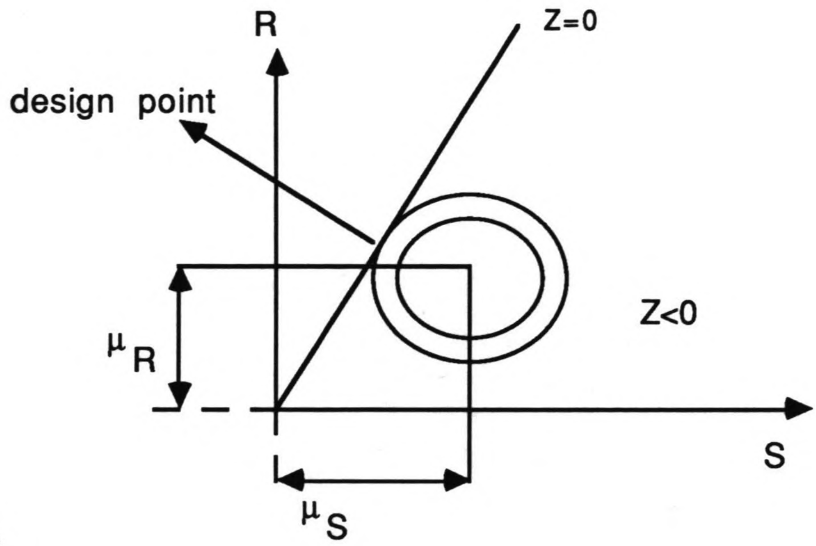


fig 4.3 design point

4.4 Summary

Although the level II approach is an approximation of the probability of failure, as the level III approach is an exact calculation of it, the level II approach has the advantage of providing insight into the contribution of various variables to the overall probability of failure. Apart from quantifying thus the probability of failure, it is shown how the probabilistic approach can serve the designer in achieving a balanced study approach in which most effort can be put into those parameters that have the largest influence on the result. The next chapter includes a level II calculation, with the data provided by the breakwater in Richards Bay.

5 PROBABILITY OF FAILURE

5.1 Introduction

The total probability of failure of a breakwater is composed of the partial probabilities of failure connected with the failure mechanisms. This chapter is limited to the hydraulic instability of the armor units.
If damage to the armor layer due to wave-action is taken as a mechanism, the Hudson formula can be used as a reliability function for describing the displacement of armor units. In order to quantify the probability of failure, first the reliability function Z is derived. Secondly, the statistical distributions of the parameters in the Hudson formula are determined.

hazard: wave action
mechanism: movement dolosse
consequence: displacement dolosse

5.2 The reliability function

Derivation reliability function:
Hudson formula:

$$M = \frac{\rho H_s^3}{K_d \Delta^3 \cot(\alpha)}$$

with the nominal diameter: $D_{n50} = (M/\rho)^{1/3}$
eliminating M yields an expression:

D_{n50}^3 Kd $\cot(\alpha)$ $\Delta^3 = H_s^3$........ D_{n50} Δ (Kd $\cot(\alpha))^{1/3} = H_s$
with:
$Z = R - S$
S = load
R = resistance
the only load parameter is the wave height H_s:
$S = H_s$
$R = D_{n50}\, \Delta\, (Kd \cot(\alpha))^{1/3}$
the reliability function is:

$$Z = D_{n50}\, \Delta\, (Kd \cot(\alpha))^{1/3} - Hs$$

The expected damage to the armor layer is depending on the choice of the Kd-factor (see 2.3 damage coefficient). Therefore the failure criterion in this reliability function is the damage considered when choosing the Kd-value. In other words, when we use this reliability function in a level II calculation P{ Z < 0} gives the probability of exceedance of the damage level related to the chosen Kd-factor.

5.3 Uncertainties

The word uncertainty is used as a general term referring to errors, to randomness of variables and to lack of knowledge of the distribution function. In order to quantify these uncertainties, the parameters in the Hudson formula are supposed to be stochastic variables.
The estimated statistical distribution for structural parameters is normally based on the allowable tolerances in the specifications and on the quality of supervision, specified measurement methods and expected quality of workmanship. Other parameters will depend on the quality and quantity of the studies carried out. For parameters describing environmental conditions, this factor should be superimposed on the natural variation.

5.3.1 Load side

The Hudson formula works directly with Hs characterizing a storm. The design wave height in Richards bay was defined from extrapolation of the wave clinometer data (February 1968 - May 1972). The uncertainty in the wave height is depending on errors related to instrument response, length of measuring period, method of analysis, visual observations and extrapolation.
The long term distribution of the wave height which was chosen in Richards Bay, can be described by an exponential distribution:

$$P\{Hs > H\} = \exp(-(H - 4.9)/0.588)$$

In which 4.9 signifies the one-year wave height, and 0.588 is a scale parameter. The long term distribution is shown in fig 5.1.
Errors due to the choice of distribution as a representative of the unknown true long term distribution, and the choice of the parameters within this distribution are represented by an extra parameter FHs in the reliability function. Arbitrary a normal distribution was chosen for FHs, with an average value zero. The crude estimation of the coefficient of variation of 0.5 is used. Because of the extrapolation to events of low probability of occurrence, the FHs-factor will probably increase with increasing wave height.

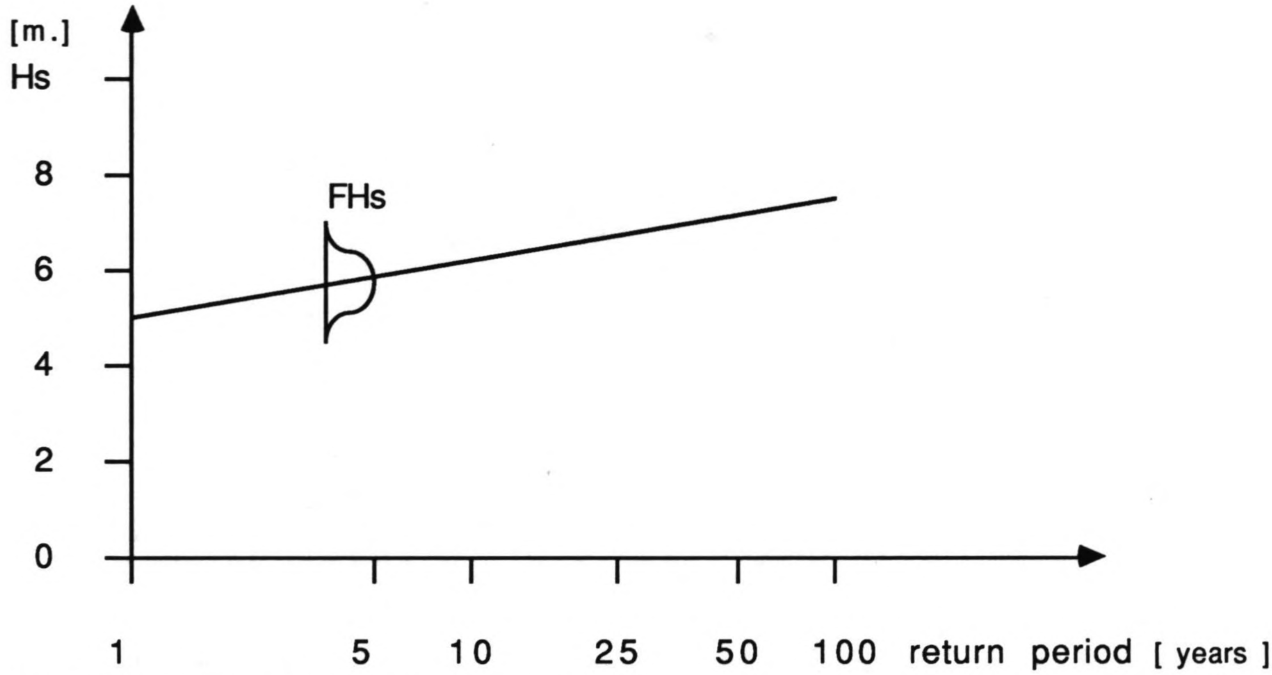


fig 5.1 long term distribution of wave heights

5.3.2 Strength side

Due to lack of information in the report on Richards Bay the distribution of the density of the concrete, the slope, and the diameter of the dolosse are supposed to be normal.

A review done by PIANC on 160 existing breakwaters, showed Kd-values used for dolosse ranging from 2 to 28. There is not yet experimental data available for dolosse to determine a proper standard deviation.

Van der Meer (Lit. 3) used a coefficient of variation for the Kd-value of quarrystone of 0.18. Because Kd-values for dolosse are depending on more variables (for instance the interlock) then these for quarrystone, the standard deviation used here is somewhat higher, namely 0.2.

5.4 Effect of uncertainties

Using Hudson in a probabilistic calculation in the case of the breakwater in Richards Bay the reliability function is:

$Z = D_{n50}\,\Delta\,(Kd \cot(\alpha))^{1/3} - (Hs + FHs)$

the following values were used:

variable	mean value	standard deviation	distribution
diameter	2.32m.	0.05m.	Normal
Kd	8.00	1.60	Normal
cot(α)	2.00	0.25	Normal
FHs	0.0	0.50	Normal

This method requires a computer. Using AFDA (Lit 14) with these values, and the Hs with the long term distribution shown in fig 5.1, the contribution of each parameter to the overall variance σZ^2 expression:

diameter	1.0%
Kd	13.7%
cot(α)	4.3%
Hs	70.3%
FHs	10.7%

The probability of failure: <u>1.7E-2</u>

Design point values:

diameter	2.30923
Kd	6.74224
cot(α)	1.89036
Hs	6.82753
FHs	0.34657

This is just an example to show the possibilities of such a calculation. This calculation has several short-comings, such as:

-the diameter has to be replaced by the basic variable, volume and ρ
-the delta is used deterministic (almost no contribution to the overall probability of failure), using the delta as a stochast it has to be replaced by its basic variables (ρ-stone and ρ-water)

5.5 Summary

In this chapter the probability of exceedance P{ Z < 0} of the 0-2% damage level is calculated on level II. Failure is thus defined as the exceedance of the 0-2% damage-level. The calculation resulted in a probability of failure of 0.017.

The variance of the parameters having the largest influence on the result are the significant wave height (70.3%), the damage coefficient (13.7%) and the FHS-parameter (10.7%).

According to this calculation the damage level will be exceeded when a storm with a significant wave height of 6.8 meter occurs.

The probability of failure of the 0-2% hydraulic damage of the dolosse, during the lifetime L of the breakwater is:

$$1 - (1 - P\{Z<0\})^{L} \qquad L \text{ in years}$$

6 TOTAL DAMAGE

6.1 Introduction

In table 1 in 1.4 are listed the storm data from the period 1978-1985, correlated with the recorded damage from close-up photographs for the head section of the breakwater. Looking at a period of four years, between the survey on 29-06-1979 and 24-06-1983 the recorded damage increased with 2%, to a total value of 4%. During this period three storms were recorded:

date	Hs [m.]
24-07-1979	4.3
17-04-1980	5.3
17-08-1981	4.5

No storms with design conditions occurred.

6.2 Modeltests

From the results of the modeltests (Lit 7), it is possible to determine the hydraulic damage which occurred in the model with the storm-data provided by the wave rider buoy. This resulted for the storms in:

Hs	damage%
4.3	0
5.3	0.3
4.5	0.15

The damage recorded from surveys is more then according to the design would have been expected.
The calculation done in the previous chapter resulted in a probability of exceedance of the 0-2% damage level of 0.017.
From photographs of sections of the armor layer can be concluded that the damage figures provided from the surveys consist beside the hydraulic damage also of broken dolosse. This could explain the high damage figures, which were found in the surveys.

6.3 Breakage

Overlooking a certain failure mode, like in this case breakage, could result in unexpected damage figures.
Due to breakage the interlock of the layer of dolosse will change, resulting in a different Kd, and the weight of a part of the dolosse will decrease. Another effect of the breakage is that the broken dolosse will be thrown about by the waves because of their decreased weight, the so-called 'armoured waves', which will cause an extra load on the dolos-layer.

6.4 Summary

If we want to design the dolosse with probabilistic methods a way must be found to account for the breakage. It would be possible to change the Kd-factor in the Hudson-formula, but in order to quantify this, a lot of model tests will be needed, with a scaled strength of the dolosse.
Another possibility would be to change the mass of the dolosse in the Hudson formula. This method takes no account of the decreased interlock of the layer, and the effect of the so-called 'armoured waves'.
If the damage-figures from the surveys done on the breakwater in Richards Bay were specified in breakage and hydraulic damage more could have been said about the use of probabilistic methods in the design process of dolosse.

7 BREAKAGE OF ARMOR UNITS

7.1 Introduction

Breakwaters have been built in increasing waterdepths using larger and more slender concrete elements as armor units. Because of absence of other sources the design of these units has been commonly based on extrapolated experience, while limits on the structural strength (breakage of units) have been overlooked. Scale effects in induced moments and loads as well as in structural strength attribute to the relative weakness of larger armor units; induced moments and loads are proportional respectively to the fourth and third power of the characteristic length while resisting moment and area are respectively proportional to the third and second power of the length, so stresses are proportional to the length. Thermal stresses also increase with the size of the unit thus making larger units relatively weaker. Each type of concrete armor unit has its characteristic dimension, beyond which structural strength dominates as failure cause. Analysis of failure of breakwaters has revealed that breakage of multileg slender concrete blocks is one of the major reasons that several breakwaters, not at all exposed to there design conditions, suffered great damage. The breaking-mechanism of concrete armor units in breakwaters is complex. Several aspects are yet unknown, such as:

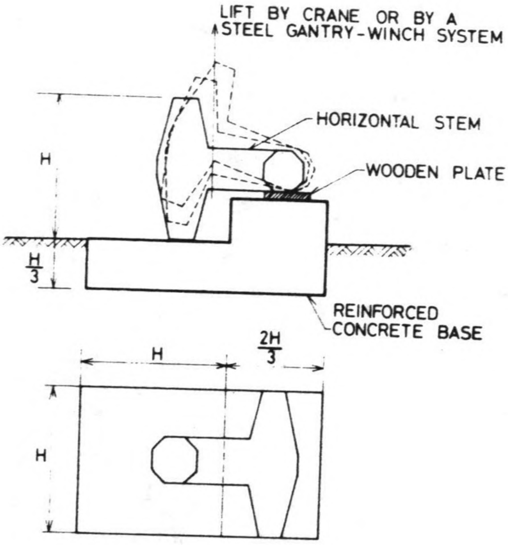
- relation between breakage and direct loads from waves,
- influence of load history,
- progressive failure from breakage.

One of the results from research on these mechanism is that breakage in large multileg blocks is mainly caused by rocking motion induced by wave action. Hydraulic damage will increase because of the decreasing mass of the broken units and a lack of interlock of these units. Progressive breakage can result if broken parts are thrown about by waves ("armored waves").

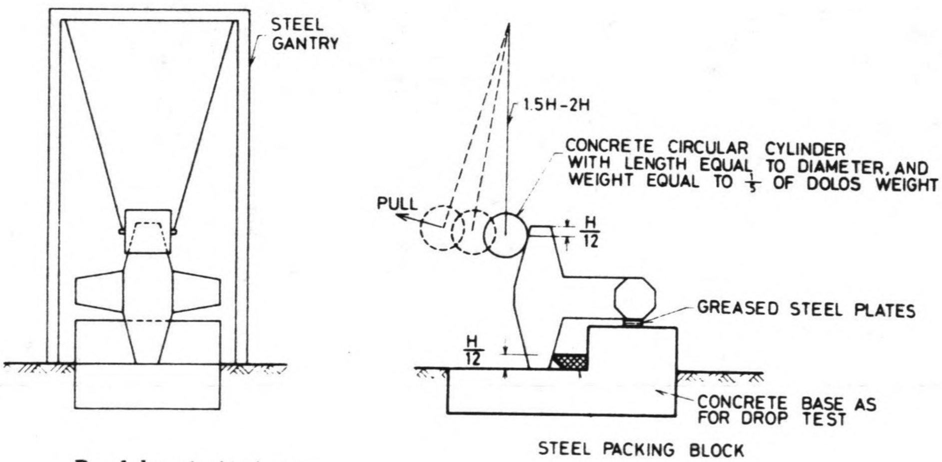
7.2 Destructive tests on dolosse

To investigate the breakage of armor units, Burcharth (1981) carried out dynamic loading tests on dolosse. These tests considered two different loading configurations: a drop test which simulated the wave-induced rocking of the dolosse, and a pendulum test which simulated the impact from pieces of broken units that are thrown about by the waves. (fig 7.1)
In the drop test, one end of a unit is lifted up at a predetermined height and then dropped on a thick concrete base. In the pendulum test, a pendulum made out of a concrete mass of one-fifth of a unit is drawn back a certain distance and released to hit the dolos-unit. The loading history of the test was chosen in such a way that failure occurred after six to eight impacts. In both types of tests on unreinforced dolosse the fracture of the units occurred

FIG. 7.1



Drop test set up.



Pendulum test set up.

TABLE 7.1

Burcharth's test results

	Series No.						
	1	3	4	5	6	7	8
Mass of unit, M (kg)	1500	1594	5400	5400	5400	9740	19,790
Height of unit, H (m)	1.65	1.65	2.32	2.32	2.32	3.00	3.80
Width of unit in trunk section, a (m)	0.500	0.500	0.813	0.813	0.813	0.950	1.20
Compressive strength of concrete (MPa)	29	88	46	46	39	41	41
Drop height for centre of gravity in drop tests, h_1 (m)	0.153	0.171	0.117	0.115	0.138	—	—
Mass of pendulum, m (kg)	294	294	990	990	990	2060	3930
Lifted height of pendulum in pendulum tests, h_2 (m)	0.0465	0.0458	0.0405	0.0399	0.0399	0.0232	0.0232
Angle of rotation in drop tests, α (°)	13°8	15°5	7°5	7°3	8°9	—	—
D (J/m²)	10,900	13,000	11,300	11,100	13,400	—	—
P (J/m²)	650	640	720	710	710	630	750

in the trunk section close to the fluke. The input energy of the drop test was:

$$M\,g\,h$$

where:

M = mass of the dolos,
g = gravitational acceleration,
h = drop height (measured vertically from the centre of gravity of the dolos).

Test results are the following: table 7.1

The mass of the dolosse used in the tests ranged from 1.55 to 5.4 ton for the drop tests and 1.5 to 20 ton used in the pendulum tests. For these tests however, the test conditions were idealized and direct correspondence of the results to a dolos in a breakwater is not possible, because a dolos in a breakwater is subjected to both static- and dynamic stresses. The response of the dolosse will also be dependent on the interaction with the other units.

7.3 Timco

To investigate this complex behaviour,Timco (1983) considered the armor layer as a intertwined mat of dolosse which will fail if the ratio of input energy to the area of fracture of the dolos (Ω) is higher than some characteristic constant value (Ω-critic). Timco used the test results of Burcharth in deriving a strength criterion for a dolos breakwater:

$$\Omega = \frac{\text{input energy}}{\text{fracture area}}$$

The fracture is considered to occur in the trunk section of the dolos, the fracture area is:

$$A = \frac{1}{4}\pi\,\beta\,a^2$$

Where:

A = fracture area,
β = the ratio of the area of an octagonal trunk section of width "a" to the area of a circle of diameter "a", $\beta = 1.05$,
a = the width of a unit in the trunk section.

The values of the strength criterion

$$\Omega = \frac{M\,g\,h}{A}$$

were:

Ω = 11,900 (+/- 1,200) J/m^2 for the drop test (value D in table 7.1)

and

Ω = 690 (+/- 50) J/m^2.for the pendulum test. (value P in table 7.1).

In a breakwater environment, the incident waves provide the dynamic input energy. The complex nature of the wave field has not been taken into account; the incident wave energy is simply chosen to be defined in terms of the deep water wave energy as:

$$Einc = E\,n\,c_o T$$

where:

Einc = input energy for one period per metre wave front,
E = wave energy,
n = 0.5, (deep water),
c_o = wave celerity,
T = wave period.

Together with:

$$E = \frac{1}{8}\rho\,g\,Hs^2$$

and

$$c_o = \frac{g\,T}{2\,\pi}$$

gives:

$$Einc = \frac{\rho\,g^2 Hs^2 Tp^2}{32\,\pi}$$

where:

Einc = input energy for one period per metre wave front,
Hs = significant wave height at the structure,
T = wave period,
Tp = peak wave period,
ρ = density of water, [kg/m^3]
g = gravitational acceleration.

Thus, for the breakwater environment:

$$\Omega = \frac{E\,inc}{A} = \frac{\dfrac{\rho\, g^2 Hs^2 Tp^2}{32\,\pi}}{\dfrac{1}{4}\beta\,\pi\, a^2}$$

$$\Omega = \frac{\rho\, g^2 Hs^2 Tp^2}{8\,\pi^2 \beta\, a^2}$$

The input information was obtained from various sources which described dolos armored breakwaters that has suffered breakage. These data are shown in the next table: (7.2). Trunk = tr, head = hd.

Breakwater	Hs [m]	Tp [s]	Mass [ton]	$\Omega\ 10^6$ [J/m^2/m]	damage
Sines	9.5-10	18-20	42	13-18	extensive
Sines	7	14	42	4.3	some
Gansbaai	6.1	11-17	17.1	3.7-9	extensive
Baie Comeau	4-4.6	8	5 (tr)	1.9-2.5	some
Baie Comeau	5.6-6.4	8	8 (hd)	2.8-3.6	some
Riviere-au-Renard	5.1	12-14	4.5 (tr)	7.5-10	extensive
Riviere-au-Renard	6.3	12-14	12.7 (hd)	5.5-8	extensive
Cleveland	2.5	6.9	2	1	little (< 2 %)
Cap-aux-Meules	4.3	9	3.6	3.5	little
Hirtshals	3.2	9	8.6	1.1	little
Gioia Tauro	7.25	13.4	15	8.5	extensive (50 %)
Oranjemund	5.5	12.9	10.7	5.5	moderate (10 %)
San Ciprian	5.6	17.4	50	3.8	moderate (5 - 10 %)

The Ω values of the various breakwaters are shown with the amount of damage for each breakwater. An examination of the table reveals that regardless of the mass of the units, the amount of breakage sustained by the breakwater increased with increasing Ω-values such that for Ω => $6.10^6 J/m^2/m$ there is extensive damage in each case. (Timco 1983)

7.4 Critical comment on Timco.

Timco has based his assumption of a constant ratio of incoming energy and fracture area, regardless of the mass, on the dynamic loading tests of Burcharth. The appearance of scale effects in the relation between incoming energy (transmitted to the dolosse as moments and loads) and fracture area is explained in the introduction of this chapter (7.1).
Secondly the stresses in armor units are not only dynamic. Burcharth distinguishes three kinds of loads (1981):

Static:

- Weight of units, prestressing due to wedge effect and to arching caused by movements under dynamic loads and by settlements of underlayers.

Dynamic:

- (impact)- Rocking and rolling of units under wave action, missiles of broken units thrown around by waves, placing of units during construction.
- (pulsating)- Gradually varying wave forces, earthquakes.

Thermal:

- Thermal stresses due to temperature differences during the hardening process.

Roughly it can be said that the stresses due to static loads are proportional to the characteristic length while stresses due to impact loads are proportional to the square root of the characteristic length. Thermal stresses increase also with the size of a unit. Also the material characteristics do not enter the ratio incoming energy versus fracture area, still the constant ratio is derived from elements with different compressive strength (table 7.1). As stated before, there is not yet a correct relation between wave-power and breakage of units. Although Timco's relation has its short-comings, we have used it in a probabilistic approach as reliability function for the breaking-mechanism.

7.5 Timco in a probabilistic approach

In a deterministic approach there is one breakage level which will be reached if certain wave conditions occur to the breakwater and will not be reached if wave conditions are less severe. Since the uncertainties in the Ω and the variations in the other parameters like Hs and Tp are large, a probabilistic approach is more convenient. The relation derived by Timco must be written as a reliability function Z:

$$\Omega = \frac{\rho\, g^2 Hs^2 Tp^2}{8\pi^2 \beta\, a^2}$$

$$Z = \Omega - \frac{\rho\, g^2 Hs^2 Tp^2}{8\pi^2 \beta\, a^2}$$

where:

- ρ = specific density water [kg/m^3],
- g = gravitational acceleration,
- Hs = significant wave height,
- Tp = spectral peak period,
- β = ratio of the area of an ortogonal trunk section of width "a" to the area of a circle of diametre "a",
- a = dolos width (trunk).

This relation can be converted to the same variables used in the Hudsons equation by:
Zwamborn's expression (lit[6]) for dolosse:

$$V = 0.675\, r^{1.285} l^3$$

where:

- V = volume of unit,
- l = height of a dolos,
- r = waist ratio (a/l).

with

$$r = \frac{a}{l}$$

and

$$V = D_{n50}^{\;3}$$

this results in:

$$Z = \Omega - \frac{\rho\, g^2 Hs^2 Tp^2}{8\,\pi^2\, \beta\, 1.30\, r^{1.14} D_{n50}^2}$$

Now it is possible to calculate the probability of exceedance of a certain percentage breakage. This percentage breakage depends on the Ω value used in the reliability function. To obtain a direct relation between Ω and percentage of breakage, we collected the data which Timco had gathered (see table 7.2) in a graph.
This resulted in the choice of five states of damage:

$\Omega\ 10^6$ [J/m²/m]	breakage	%
0-1	none	0
1-3	little	2
3-4	some	5
4-6	moderate	10
>6	extensive	> 50

and the graph:

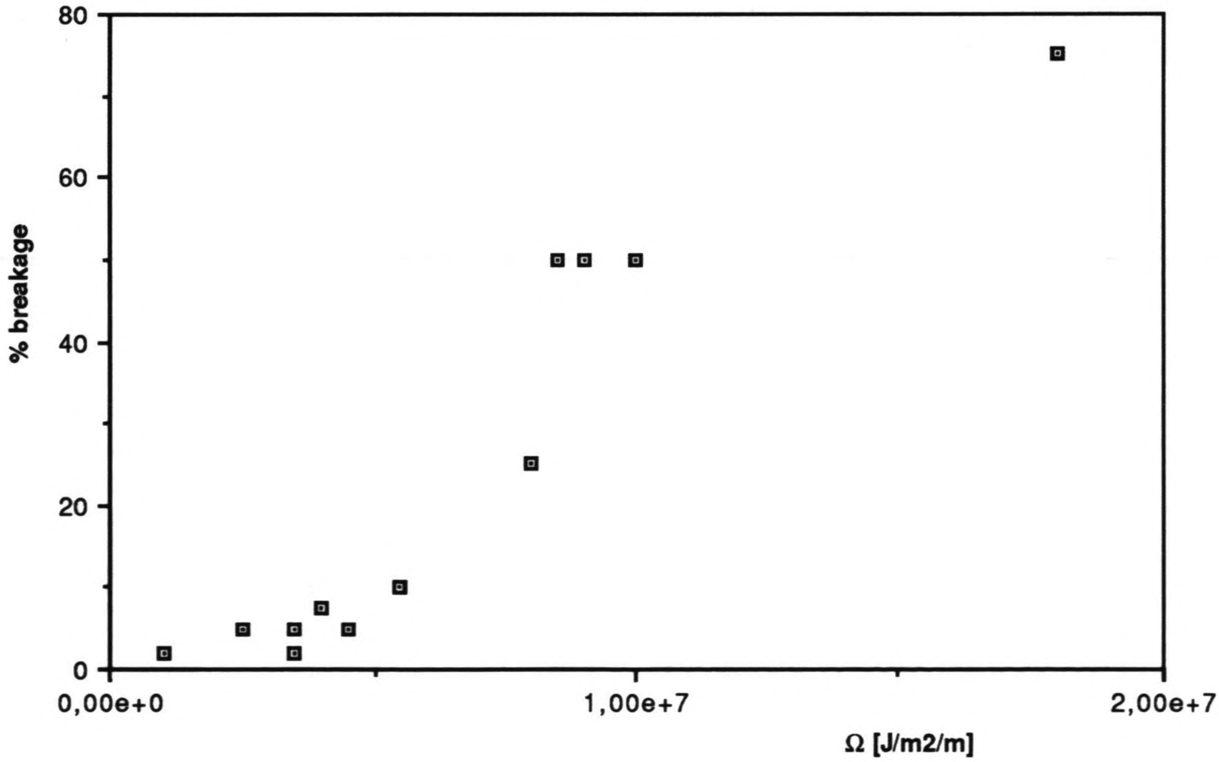


Fig 7.2. Representation of percent breakage as function of Ω.

Rough curve-fitting (pensil) provides the relation that we can use in a probabilistic approach.
As an example probabilities of exceedance for several breakage levels will be calculated using the reliability function of Timco in an advanced level II approach (chapter 5.3) with the design conditions of the breakwater of Richards Bay.
The reliability function of Timco is: (chapter 7.5)

$$Z = \Omega - \frac{\rho\, g^2 Hs^2 Tp^2}{8\,\pi^2\,\beta\, 1.30\, r^{1.14} D_{n50}^{\;2}}$$

Design conditions Richards Bay:

	μ	σ
Hs	7.2	0.1
Tp	11.5	0.2
r	0.36	0.01
Dn50	2.32	0.05
ρ	1000	1

And from figure 7.2 (the curve has been drawn by hand):

$\Omega * 10^6$	% breakage
1	1.5
2	3
3	5
4	8
5	12
6	17.5
7	25
8	30
9	40
10	50

All variables are normally distributed and the variation coefficient (σ/μ) of Ω is chosen at 0.2, also from the graph.

Calculations have been made for breaking percentages of 0 to 25 % (Ω = 0 to 7 10^6). The results are the probabilities of exceedance of a certain breakage percentage:

$\Omega * 10^6$	% breakage	P { breakage > % }
0	0	1.0
1	1.5	1.0
2	3	1.0
3	5	$6.4 * 10^{-1}$
4	8	$1.8 * 10^{-1}$
5	12	$4.2 * 10^{-2}$
6	17.5	$1.2 * 10^{-2}$
7	25	$3.9 * 10^{-3}$

In a graph:

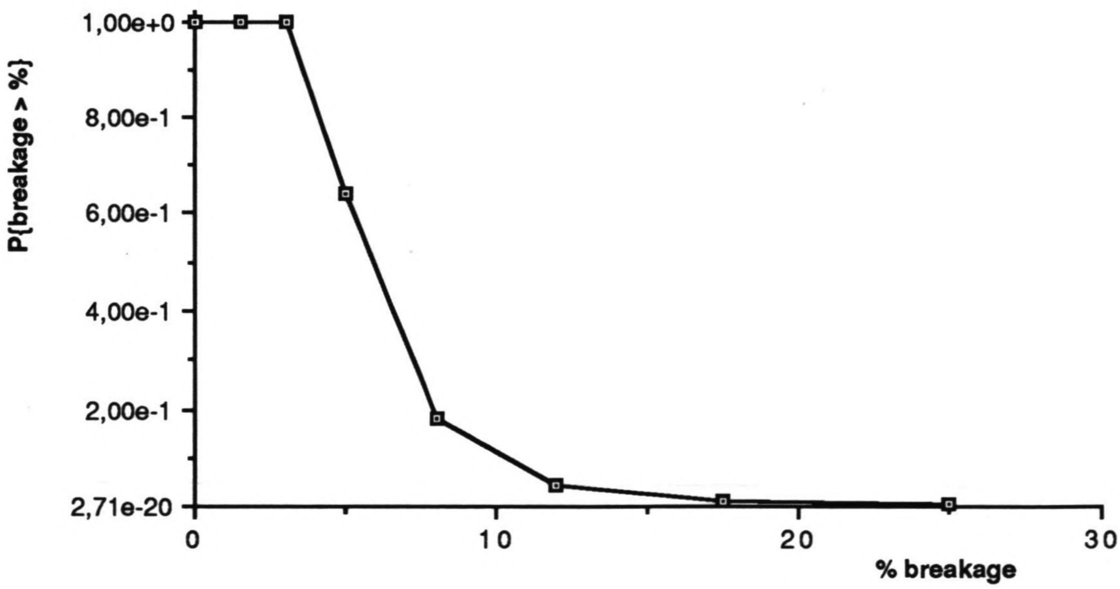


Fig 7.3. Distribution function breakage Richards Bay.

And by differentiation the density function is obtained:

$$\frac{P\{\%_i\} - P\{\%_j\}}{\Delta \%}$$

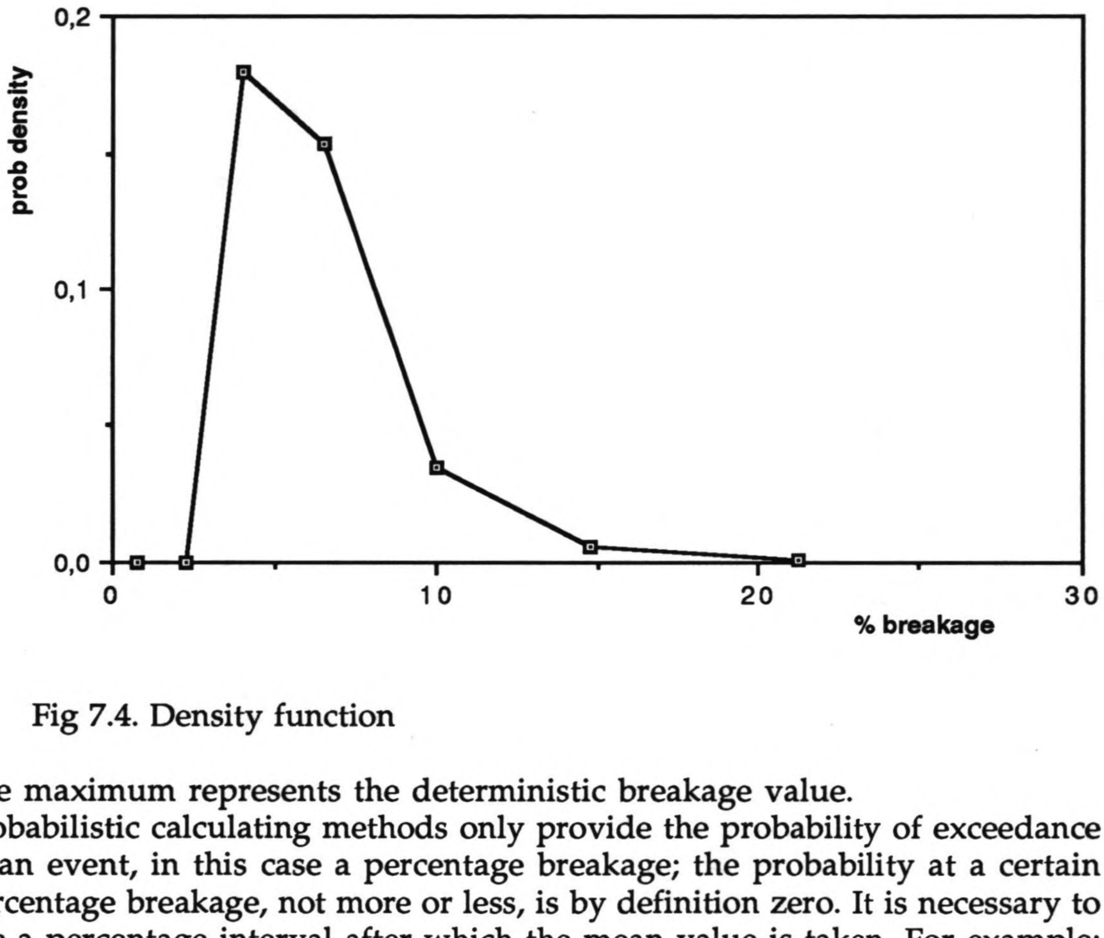


Fig 7.4. Density function

The maximum represents the deterministic breakage value.
Probabilistic calculating methods only provide the probability of exceedance of an event, in this case a percentage breakage; the probability at a certain percentage breakage, not more or less, is by definition zero. It is necessary to use a percentage interval after which the mean value is taken. For example: (fig 7.2)

P { 10 % breakage } = P { br > 5 % } - P { br > 15 % } = $6.4 * 10^{-1} - 2.56 * 10^{-2} = 6.1 * 10^{-1}$

or:

P { 10 % breakage } = P { br > 7.5 % } - P { br > 12.5 % } = $2.6 * 10^{-1} - 3.9 * 10^{-2} = 2.2 * 10^{-1}$

The probability depends on the size of the interval so this only makes sense if the sum of the intervals covers the full 100% breakage. This will be showed in the next chapter.

8 PROBABILITY OF FAILURE OF THE BREAKWATER

8.1 Introduction

In this chapter the possibilities will be examined of calculating the probability of failure of a breakwater that is built with breakable units like dolosse. For a start it is necessary to define the mechanism that will be considered for it is not the aim of this study to examine the entire fault tree with all its successive mechanisms. The mechanism considered here is the next:
Wave action on the breakwater causes breakage of armor units; the diminishing weight of the broken units influences the hydraulic stability and the probability a unit washes away becomes larger.
The definition of failure (top event) is chosen to be the disappearance or movement over a certain distance of.armor units, according to the definition of hydraulic failure. So a breakwater has not failed if all units are broken but none is washed away.
This mechanism can be separated into two sections: the breaking- and hydraulic part. Breaking will be described with Timco and the hydraulic part with Hudson (see chapter 7 respectively 2).

8.2 Breakage part

Although the uncertainty in calculating breakage of armor units is large (the derivation of Timco is doubtful, quantities like the concrete strength do not enter the equation, prototype data are limited and show a large variation, etc.), the calculation itself is possible both deterministic and probabilistic.

8.3 Hydraulic part

Since the definition of failure and the fact that breakage and hydraulic damage are two different failure mechanisms the solution always has to contain the influence of breakage on hydraulic stability. The result of a breakage calculation is a percentage breakage: a certain percentage of the armor units has a different weight and (if the Hudson equation is used) a different Kd factor because of the different characteristics of a broken unit. The best manner to handle this is to carry out tests with different percentages of broken units and determine Kd factors for these situations. Still it is very hard to take into account progressive breakage because of the difficulty in scaling strength.

8.4 Model tests with broken dolosse

An example of such model tests with broken units are the tests carried out by Markle and Davidson (1983) on the influence of broken dolosse on armor layer stability. The purpose of this study was to determine the quantity and distribution of broken dolosse that can exist and not cause a reduction in the stability of the armor layer when subjected to its "no damage" conditions. (= 0 to 5 %). This in order to develop a maintenance strategy for the dolos breakwaters that are already built.

The following dolos breakage conditions were tested:

- Various percentages of broken dolosse uniformly distributed throughout the top layer, bottom layer and both layers.

- Various size clusters of broken dolosse located above and below the still water level. Cluster breakage means all the dolosse in a given cluster are broken and the breakage extends through both layers of dolosse.

On the uniform breakage in the top layer, the entire test section was initially built with unbroken units. Once construction was completed, whole units were removed from the top layer and replaced with broken dolosse. The broken dolos pieces were placed with the same orientation as the whole dolosse had prior their removal. During removal of the whole dolosse and placement of broken dolosse, care was taken not to desturb surrounding armor units.
Placement of breakage in the bottom layer could not be carried out in the same manner. The breakage was uniformly built into the bottom layer during the initial construction of the primary armor under-layers.

On each conditions three tests were carried out with waves that differ in wave steepness: A, B and C. See table 8.1. The wave steepness (ξ) according to Iribarren is:

$$\xi = \frac{\tan(\alpha)}{\sqrt{\frac{H}{L_o}}}$$

where:

ξ = wave steepness,
α = slope angel,
H = wave height,
L_o = deep water wave length.

Wave cond.	steepness (H / Lo)	ξ
A	.016	11.9
B	.031	8.5
C	.065	5.9

$\xi > 5$; all conditions are surging waves. (Battjes 1974)

Table 8.2.

Percent Damage Measurements for Nonbreaking Wave Tests

Test Conditions*	Breaking Condition	Percent Damage First Test	Percent Damage Second Test
	Uniform Breakage Top Layer		
A	15.0**	2.6	1.3
B	15.0**	1.9	1.5
C	15.0**	4.5	3.0
A	25.0**	0.4	1.9
B	25.0**	4.7	5.5
C	25.0**	1.3	2.4
A	35.0**	1.1	4.6
B	35.0**	7.3	8.9
C	35.0**	2.6	6.3
	Uniform Breakage Bottom Layer		
A	15.0**	1.2	3.4
B	15.0**	6.8	6.3
C	15.0**	1.8	4.6
A	25.0**	†	†
B	25.0**	>10.0††	>10.0††
	Uniform Breakage Both Layers		
B	7.5**	1.3	3.1
C	7.5**	2.4	2.4
B	10.0**	9.2	4.8
B	15.0**	9.4	6.5
	Cluster Breakage		
B	5.0‡	2.3	3.0
C	5.0‡	0.5	0.3
B	10.0‡	12.0	7.8

* Refer to Table 1.
** Percentage of broken dolosse per specified layer(s).
† Percent damage measurement not taken.
†† Estimated.
‡ Number of broken dolosse in a cluster.

Table 8.1

Nonovertopping Test Conditions for Stability Tests of Randomly Placed Dolos Armor Units Containing Various Amounts and Distributions of Armor Unit Breakage: W_a = 0.276 lb; γ_a = 142.2 pcf; k_Δ = 0.94 ; P = 56 percent; $W_1 = W_a/5$; Cot α = 1.5

Test* Conditions	d, ft	d/L	H/L	T, sec	$H_{D=0}$, ft	N_s
			Nonbreaking Waves			
A	2.0	0.10	0.031	2.65	0.57	3.57
B	2.0	0.15	0.044	1.89	0.57	3.57
C	2.0	0.25	0.075	1.31	0.57	3.57

Damage is defined as:

$$\text{percent damage} = \frac{A_1}{A_2} 100$$

where:

A_1 = area before testing,

A_2 = area from which armor units have been displaced.

The same definition has been used in the S.P.M.

Results: see table 8.2 and graph 8.1:

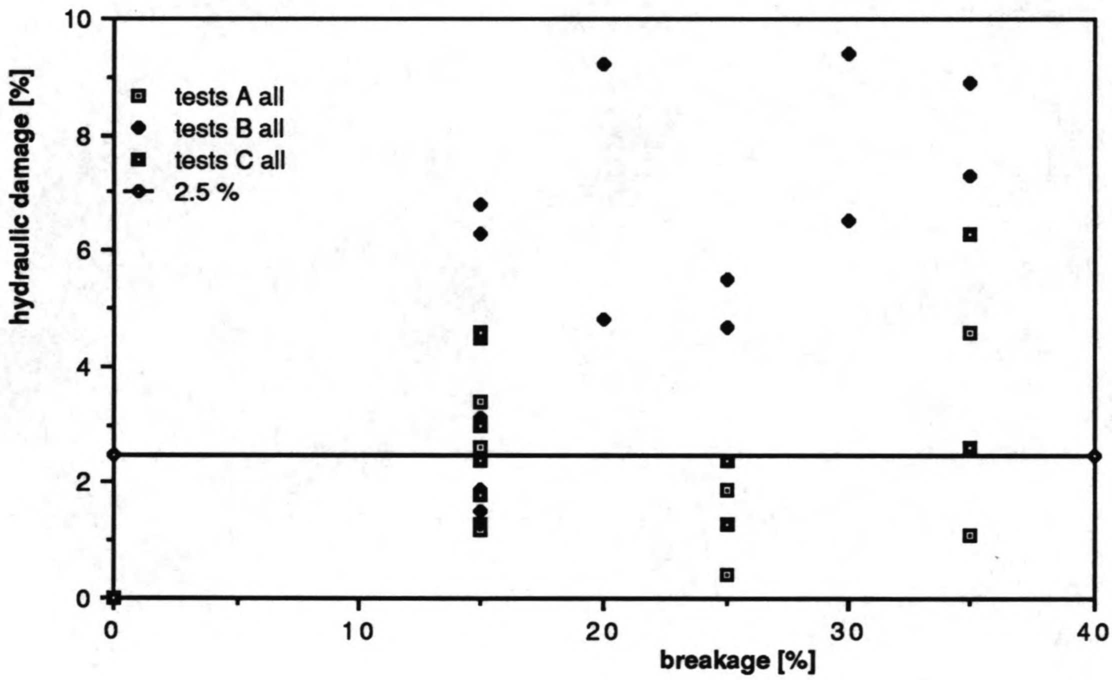


Fig 8.1. Percent breakage versus hydraulic damage.

Taking into account the fact that the "no damage" criterion of the Hudson equation is 0 to 5 percent (2.5 %), the only tests that show a significant enlargement in hydraulic failure are the B tests. Since the tests only differ in wave steepness the Hudson equation covers them all. It seems to be rather inaccurate if not impossible to determine a relation between Kd and percent breakage from these tests.

8.5 Theoretical approach of breakage vs hydraulic damage

Another possibility, if model tests are hard to carry out due to scale effects, is to adjust the Hudson equation:

$$M = \frac{\rho_a H^3}{K_d \Delta^3 \cot(\alpha)}$$

Or in the variables used in the previous chapters:

$$\frac{H}{\Delta D_n} = \left(K_d \cot(\alpha) \right)^{\frac{1}{3}}$$

where:

M = mass of unit,
ρ_a = specific density armor,
H = wave height,
Kd = damage coefficient,
Δ = relative density, (ρ_a / ρ_w - 1),
α = slope angle,
D_n = nominal diameter.

At first it is possible, given a certain percentage breakage (the dolos is assumed to break in two pieces of the same weight), to calculate a new mean diameter of:

$$D = \frac{(100 - \gamma)\, D_1 + 2\gamma D_2}{100 + \gamma}$$

where:

D = new mean diameter,
γ = percentage breakage,
D_1 = nominal diameter, $(M/\rho_a)^{1/3}$
D_2 = nom. diam. of broken unit, $(0.5\, M/\rho_a)^{1/3}$
M = mass of unit,
ρ_a = specific density of armor.

A second adjustment is to use only D_2 and to calculate a larger percentage hydraulic damage (a different Kd, see chapter 2) giving the wanted percentage for the entire armor layer (including the non broken units). The new percentage is:

$$\delta_{(D_2)} = 100 \frac{\gamma}{\delta}$$

where:

δ = total hydraulic damage,

γ = percentage breakage,

δ (D2) = perc. hydr. damage using only D_2.

An obvious shortcoming is that the percentage hydraulic damage can never exceed percentage breakage.
An extension of both methods is to change the Kd factor because of the different characteristics of a broken unit.
Because both methods have little to do with reality they have not been worked out.

At last a different Kd can be determined from the breakage percentage using the S.P.M. and interpolation- and extrapolation techniques. Kd stands for a certain amount hydraulic damage under certain wave conditions. If the possibility exists that units break, the probability of exceedance of the same percentage hydraulic damage will be larger under the same wave conditions. By lowering the Kd factor this can be taken into account.

8.6 Mechanism breakage-hydraulic damage

In the previous chapters two results have been derived: it is possible to calculate the probability at a certain percentage breakage and, given this percentage, it is possible to calculate the probability of exceedance of a certain hydraulic damage if a manner is found to modify the Hudson equation to this certain percentage breakage. Due to the definition of failure for the breakwater in this study (only hydraulic damage, see chapter 2.2). The definition of hydraulic damage refers to the original amount of unbroken dolosse. E.g. if one half of a broken dolos washes away from a layer of originally 100 unbroken dolosse the hydraulic damage (δ) is 0.5 %.

The probability of failure is:

$$P\{\text{ hydraulic damage} > \delta\ \%\ \},$$

independent of the percentage breakage which occurs.This means:

$$
\begin{aligned}
P\{\text{ hydraulic damage} > \delta\ \%\ \} = \\
& P\{\text{ hydraulic damage} > \delta\ \% \cap \text{breakage} = 10\ \%\ \} \\
&+ P\{\text{ hydraulic damage} > \delta\ \% \cap \text{breakage} = 30\ \%\ \} \\
&+ P\{\text{ hydraulic damage} > \delta\ \% \cap \text{breakage} = 50\ \%\ \} \\
&+ P\{\text{ hydraulic damage} > \delta\ \% \cap \text{breakage} = 70\ \%\ \} \\
&+ P\{\text{ hydraulic damage} > \delta\ \% \cap \text{breakage} = 90\ \%\ \}.
\end{aligned}
$$

$$= \sum_{\gamma=10}^{90} P\{\text{ hydraulic damage} > \delta\ \% \cap \text{breakage} = \gamma\ \%\}$$

(For $P\{\text{ breakage} = \gamma\ \%\ \}$ see chapter 7.5).
The probability $P\{\ Z_1 < 0 \text{ and } Z_2 < 0\ \}$ is easily to be determined if Z_1 and Z_2 are independent:

$$P\{\ Z_1 < 0 \text{ and } Z_2 < 0\ \} = P\{\ Z_1 < 0\ )\ P(\ Z_2 < 0\ \}.$$

The reliability functions Z_1 and Z_2 are those of Hudson respectively Timco. In these reliability functions the same wave height, D_{n50} and water density are used, making these probabilities dependent. A solution for this problem is given by Ditlevsen by his upper- and lower boundary approximation. Ditlevsen presupposes Z_1 and Z_2 to be normally distributed and makes use of the correlation coefficient ρ (while $-1 < \rho < 1$ and $\rho = 0$ corresponds to independence).
The formula for determining ρ in accordance with a level II approach is:

$$\rho(\ Z_1, Z_2) = \sum_{i=1}^{n} \alpha_i^{(1)}\ \alpha_i^{(2)}$$

where:

$$\alpha_i = \frac{\sigma(\ X_i\ )}{\sigma(\ Z\ )} \frac{\delta Z}{\delta X_i}$$

and X_i the design point. (chapter 5)

The boundaries are:

$$P\{ Z_1 < 0 \text{ and } Z_2 < 0 \} > \max \{\Phi_N(-\beta_1)\, \Phi_N(-\beta^*_2), \Phi_N(-\beta^*_1)\, \Phi N(-\beta_2)\}$$

$$P\{ Z_1 < 0 \text{ and } Z_2 < 0 \} < \{\Phi_N(-\beta_1)\, \Phi_N(-\beta^*_2) + \Phi_N(-\beta^*_1)\, \Phi N(-\beta_2)\}$$

and:

$$\beta_i^* = \frac{\left(\beta_i - \rho\, \beta_j\right)}{\sqrt{1 - \rho^2}}$$

$\Phi_N(-\beta)$ is the cumulative distribution function for the standard normal distribution and is given in tables in the literature on statistics. (E. g. lit[9]). The probability of hydraulic failure taking into account breakage can be calculated in this way.

8.7 Probability of failure without modification.

In the method described above to calculate the probability of failure of a breakwater with breakable units there is a need for a modification in the Hudson formula. The best solution for this problem is to perform model tests using strength scalled dolosse. The second best solution consists of model tests with broken units put into the armor layer before testing. The third solution is a theoretical modification.
Model tests with strength scalled dolosse are not yet possible due to the lack of knowledge on scale effects in structural strength. The results of model tests with units which are broken before testing seem to be unreliable (see chapter 8.4) and at last model tests should be used (which are not yet available) to verify the theoretical changes in the formula.
To avoid the necessity of a modification in the Hudson formula for breakage the following assumption will be made: if an unit breaks it will be washed away. Damage at the breakwater now is the sum of breakage and hydraulic damage. The probability of failure is the probability of exceedance of the remaining hydraulic damage. This will be determined for different proportions of breakage and hydraulic damage. The results of these calculations will be compared with damage figures actually measured at Richards Bay in order to try to draw a conclusion on the most probable proportion breakage / hydraulic-damage.
First of all the Hudson formula must be adapted to different hydraulic damage levels. This can be based upon the test data of Richards Bay (ref. 7)

in which wave heights larger than H-design have been used. According to the Hudson equation there is a Kd*:

$$Kd^* = \left(\frac{H}{H\,\text{design}}\right)^3 Kd$$

where:

Kd = damage coefficient,
H = wave height,
Kd* = damage coefficient for different percentages.

Kd*'s are showed in the next graph and table (8.3) in which the tests data have been used: (ref. 7):

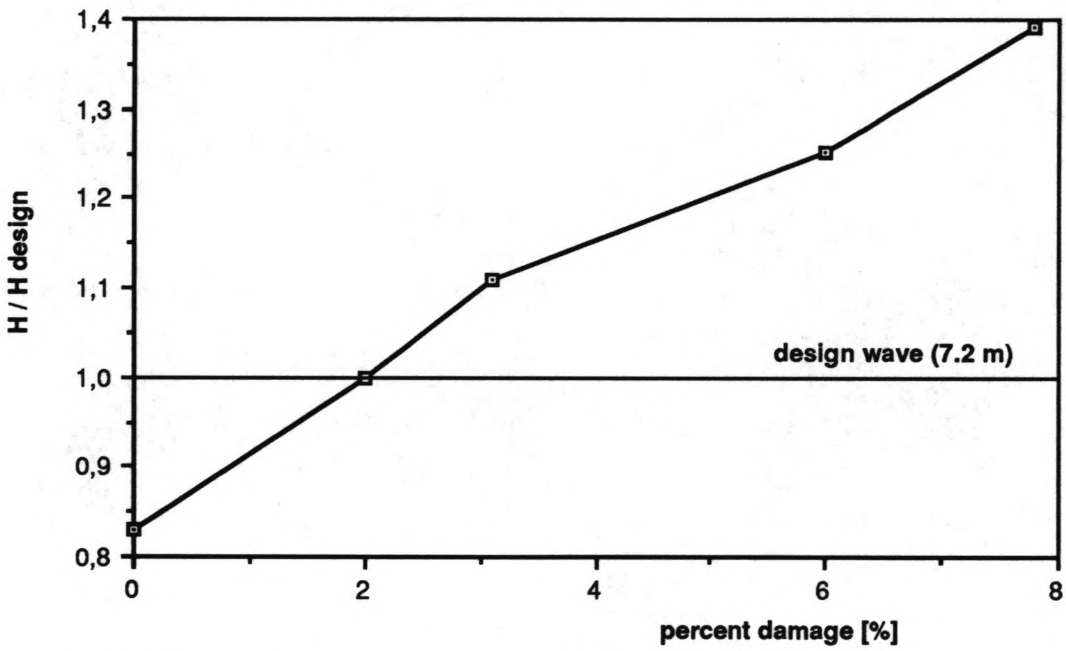


Fig.(8.2): hydraulic damage vs H/H design.

Table (8.3):

H/H design	% damage	Kd*
0.83	0.0	3.6
1.00	2.0	6.35
1.11	3.1	8.7
1.25	6.0	12.4
1.39	7.8	17.1

The Kd* was best fit to the polynom:

$$Kd^* = 3.7592 + 1.2202\,(\%) + 0.0568(\%)^2$$

where:

Kd* = damage coefficient for different hydraulic damage levels,
% = percentage hydraulic damage.

Several level II calculations have been made with AFDA (chapter 4):

- ( ): The semi probabililistic design values of waves vs hydraulic damage. The only uncertainty which has been taken into account is the Weibull distribution of the significant wave height. The waves have been translated to return period according the long term distribution for a six hours storm duration (fig 8.3).
- ( ): The design values taking into account all uncertainties. Still there is supposed to be no breakage.
- ( ): Damage is supposed to contain 50% breakage.
- ( ): Damage is supposed to contain 75% breakage.
- ( ): Damage is supposed to consist of 100% breakage.

The input data are:

	μ	σ	Distribution
Hs	7.2	0.2	Weibull
$\cot\alpha$	2	0.25	normal
D_{n50}	2.32	0.05	normal
Δ	1.33	0.02	normal
Kd*	$\sigma/\mu = 0.2$		normal

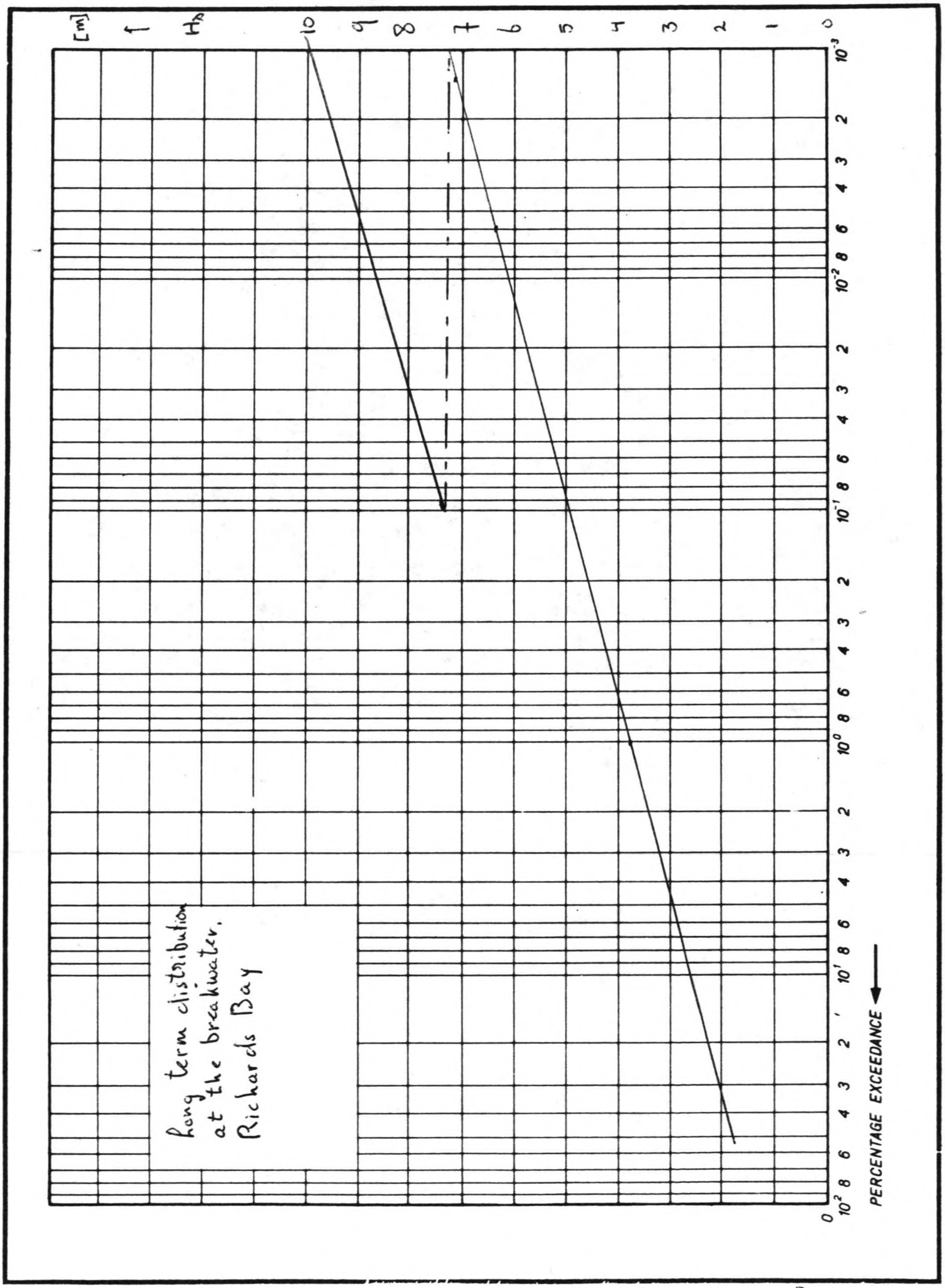

Long term distribution
at the breakwater,
Richards Bay
Hs [m]
PERCENTAGE EXCEEDANCE

Fig. 8.3

The results are shown in the next table and graph (8.4):

Table (8.4): Percentage (!) exceedance of hydraulic damage per year:

Dam. [%]	semi prob.	prob.	50%	75%	100% breakage.
	◆	■	▣	■	◈
7.8	1.0e-6	1.9e-5	4.0e-4	2.2e-3	1.9e-2
6.0	3.5e-5	1.0e-4	9.5e-4	4.8e-3	1.9e-2
3.1	2.0e-4	5.6e-4	2.6e-3	5.8e-3	1.9e-2
2.0	1.0e-3	2.2e-3	5.7e-3	8.0e-3	1.9e-2
0.0	1.0e-2	1.9e-2	1.9e-2	1.9e-2	1.9e-2

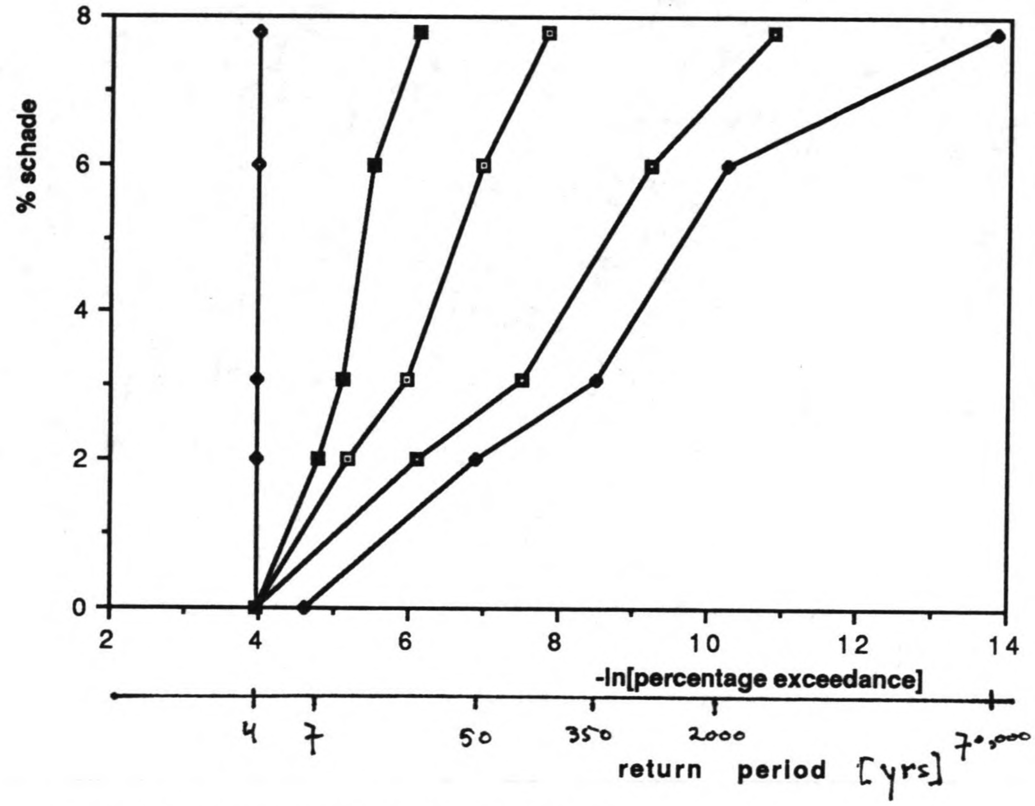


Fig (8.4) Return period vs hydraulic damage.

The damage occurred at Richards Bay is reported for a period of 10 years. The figures above are the probabilities of exceedance for a one year period. The probability a certain damage level is exceeded is P. The probability this certain damage level is not exceeded is 1-P, the probability this level is not exceeded in 10 years is: $(1-P)^{10*1460}$ in which 1460 is the number of possible storms (6 hrs) per year. At last the probability a certain damage level is exceeded in 10 years is: $1-(1-P)^{14600}$. Since we are dealing with exceedance frequencies we are looking for the probability of exceedance of 0.5. This represents the maximum in the probability density function or the value of expectation. (This is only valid if all variables are normally distributed).

Table (8.4) can be transformed:
Table (8.5): Probability of exceedance for hydraulic damage levels for a ten years period:

Dam. [%]	semi prob.	prob.	50%	75%	100% breakage.
7.8	1.5e-4	2.8e-3	5.7e-2	0.27	0.94
6.0	5.0e-3	1.4e-2	0.13	0.50	0.94
3.1	2.9e-2	7.9e-2	0.32	0.57	0.94
2.0	0.14	0.27	0.56	0.69	0.94
0.0	0.77	0.94	0.94	0.94	0.94

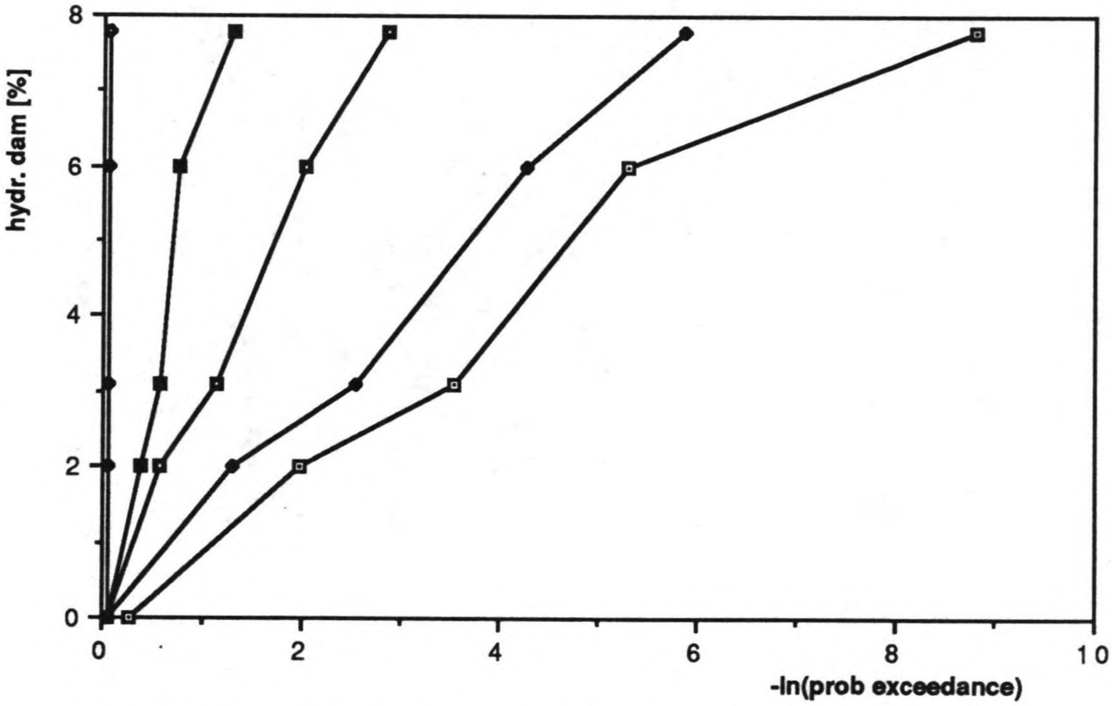


Fig (8.5) Probability of exceedance for ten years.

The probability of exceedance on a logaritmic scale is: -ln(0.5) = 0.69. In this ten years period an actually damage of 4.5% was measured (ref.7.). From graph 8.5 it can be concluded that the most probable combination of damage, if the input data are correct, is nearby 80% breakage and 20% hydraulic damage.

8.8 Summary.

To find the probability of failure of a breakwater with breakable units, the damage function should be determined by model tests. In this way it is possible to find a relation between waves and total (breakage + hydraulic) damage. Due to the lack of knowledge on scale effects this is not yet possible.

A possible solution is to separate breakage and hydraulic damage: breakage occurs; there is a change in the armor layer; because of this change hydraulic damage will probably be larger than under the same wave conditions without breakage. The probability of failure is the sum of the probabilities for the different possible combinations breakage and hydraulic damage (chapter 8.6).
There is little known on the probability of breakage of armor units but based on assumptions of Timco a damage function is derived which can be used in a probabilistic approach. The modification of the hydraulic relation has been less succesful. Results of model tests with prebroken dolosse seem te be unreliable and theoretical modifications still have to be verified.
Under the assumption that once a unit is broken it will be washed away it is possible to avoid this modification. The probability of failure is the probability at the remaining percentage hydraulic damage. Because the probability distribution of breakage is ignored several ratio's of breakage and hydraulic damage must be calculated. By comparing these results to actual measurements at the breakater the most probable combination can be found.

CONCLUSIONS

The three-dimensional tests done on the breakwater-head in Richards Bay were unable to describe the behaviour of the dolosse in the prototype, due to scale-effects in structural strength.

The unexpected damage-figures from surveys done on the dolosse-layer are due to breakage of dolosse.

The parameters giving the highest contribution to the overall variance in a probabilistic calculation, with the Hudson-formula as the reliability function, are the wave height and the damage-coefficient.

Calculating the probability of failure of the dolosse-layer in Richards Bay with the Hudson formula as the reliability function can only be done taking account of the breakage.

If the damage-figures from the surveys done on the breakwater in Richards Bay were specified in breakage and hydraulic damage more could have been said about the use of probabilistic method designing dolosse with the Hudson formula.

Model tests with dolosse that are broken before testing are still unreliable because of the difficulties in building a test section and these tests do not account for progressive breakage.

If broken dolosse are assumed to be washed away the most probable combination between breakage and hydraulic damage can be determined. The most probable combination we have found for the breakwater of Richards Bay is 80% breakage and 20% hydraulic damage.

LIST OF SYMBOLS

A	area	m^2
c_o	wave celerity	m/s
D_{n50}	nominale diameter	m
E	energy	J
Einc	input energy	Jm
Hd	design wave height	m
Hs	significant wave height	m
Kd	damage coefficient	
L_o	deep water wave length	m
l	height of dolos	
M	mass	ton
r	waist ratio	
T	wave period	s
Tp	peak wave period	s
V	coefficient of variation	
W	weight	ton
α	slope angle	degrees
γ	breaking percentage	
δ	percentage hydraulic damage	
Δ	relative density to water	
ξ	wave steepness Iribarren	
σ	standard deviation	
ρ	specific density	ton/m^3
μ	mean value	
Ω	input energy / fracture area	$J/m^2/m$

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