



Five Sisters Breakwater showing accretion behind, Wenthrop Beach, MA

PART III

COASTAL STRUCTURES AND RELATED PROBLEMS

Entrance to Wildwood Harbor, Cleveland, OH



CHAPTER ONE HUNDRED SIXTY FOUR

Safety and Reliability of Breakwaters

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Abstract

This paper discusses the need to incorporate a reliability analysis in the design procedures for rubble mound breakwaters. Such an analysis is defined and a suggested approach is outlined. Failure mechanisms are analysed and categorized in Damage Event Trees. The probability of failure is computed using a level III simulation method to include time and cumulative effects and to account for skewed probability distributions. Typical outputs of the computer program are shown and compared with results according to traditional design approaches. The paper concludes that there is a definite need to include reliability analysis in the design procedures for larger breakwaters and such an analysis must consider the accuracy of design parameters and methods.

1. Introduction

The design of rubble mound breakwaters is traditionally based upon a combination of experience, engineering skill and hydraulic model studies. The criteria for design are based on a design load having a return period in the order of 50 to 100 years. Under such a design load, typically damage may occur to between 2% and 5% of the armour units. When site information is unreliable the design condition may be defined more conservatively by applying an appropriate safety factor (Figure 1). Even so, a certain risk of failure in the lifetime of the breakwater still exists.

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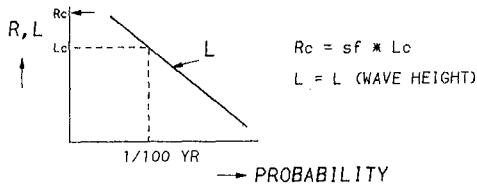


Fig. 1 Traditional Approach

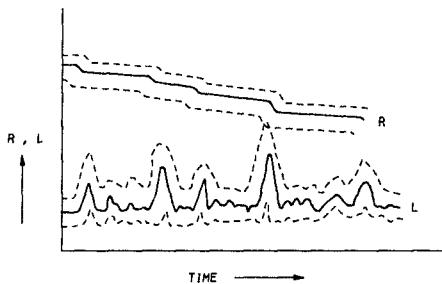


Fig. 2 Load and Strengths versus Time

OBJECTIVE TO ASSESS THE REAL PROBABILITY ON FAILURE

- PROCEDURE - DESCRIBE FAILURE FUNCTION
 $F = F(X_1, X_2, X_3, \dots, X_n)$
- DESCRIBE VARIABLES X_n by STATISTICAL DISTRIBUTION $P(X_n)$
 - DETERMINE STATISTICAL DISTRIBUTION $P(F)$ AND PROBABILITY $F=0$ (DAMAGE EVENT) BY APPROXIMATE METHODS (LEVEL II) OR BY EXACT METHODS (LEVEL III)

Fig. 3 Reliability Analysis Higher Levels

However, the probability and extent of damage is not established solely by the actual wave height exceeding the design wave height. Both the strength of the structure and the imposed loads are influenced by a number of other variables, some of which are a function of time. This results in a typical representation of the breakwater strength ' R ' and the applied load ' L ' as given in Figure 2. Failure principally occurs when the load exceeds the strength: ' $R-L < 0$ '.

For a breakwater, ' R ' and ' L ' are functions of a large number of parameters. ' R ' is described by parameters which include geotechnical properties, armour weight, density and shape, crest height and slope angle. The load ' L ' is a function of parameters which include the offshore wave height, period, direction and refraction. All of these parameters are not precisely defined, but have certain variations due to inaccuracies in studies and measurements, construction constraints, quality of workmanship etc. Furthermore, environmental conditions, including the wave parameters, have significant stochastic characteristics.

A better assessment of the probability of a given degree of damage, then according to the traditional approach outlined above, can be achieved by using more sophisticated probabilistic methods presently available, see Figure 3. The Joint Committee on Structural Safety (Reference 6) has structured the various methods for reliability analysis. In this respect the traditional design approach can be defined as level I. References 1 and 7 describe the application of a level II approach for rubble mound breakwaters. However in the level II method the computations are carried out utilizing a linearisation of the failure function ' $F = R - L$ ' and utilizing assumptions for the shape of the probability functions. Also the method does not take into account accumulation of damage in subsequent storms.

This paper discusses the application of a computer program which computes the probability of failure on level III using a time domain simulation approach. Therefore, in contrary to the level II approach, it includes arbitrary statistical distributions of all variables and the response of breakwater structures depending on previous history of damages.

2. Failure Mechanisms

With traditional methods as well as with probabilistic approaches, gross errors must be avoided. Such errors may occur due to the neglect of one or more of the possible failure modes. For design conditions well within the existing field of experience this might not be important, but it can be critical when the design concepts are stretched beyond existing experience. Recent failures of deepwater breakwaters have exhibited evidence of neglect of certain failure modes through breakage of armour units and geotechnical instability of slopes.

<u>EXTERNAL EVENTS</u>	<u>MINOR EVENTS</u>	<u>MAJOR EVENTS</u>	<u>FAILURE EVENTS</u>
WATER LEVEL	SCOUR OF FORESHORE	DAMAGE FRONT ARMOUR	SETTLEMENT SLIDING AND OVERTURNING OF CROWN WALL
WAVES	EROSION OF TOE FILTER	DAMAGE REAR ARMOUR	EROSION OF UNDERLAYERS AND CORE
CURRENTS	VENTING THRU BREAKWATER	SETTLEMENT OF CORE AND SUBSOIL	
TSUNAMIS	INSTABLE FRONT BERM	SLIDING FAILURE OF SLOPES AND SUBSOIL	GRAUDAL DETERIORATION OF MOUND
EARTHQUAKE	WASHING OUT OF FINES	EROSION AND UPLIFT OF BACKFILL	BREAKAGE OF PARAPET WALL

Table 1 Damage Events

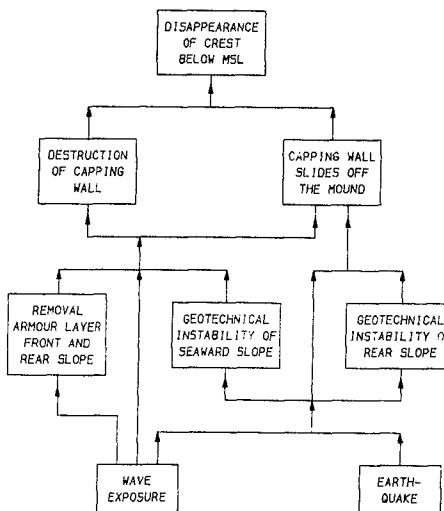


Fig. 4 Main Damage Event Tree

An important step in the probabilistic design method is to list all possible failure modes and to present the interrelation by use of a fault tree consisting of a number of successive or parallel events. Table 1 lists various damage events as they have occurred in failures of breakwaters. The relation between the various damage events is given in 'Damage Event Trees' as shown in Figures 4, 5 and 6. Figure 4 shows the main Damage Event Tree of a breakwater. Based on this, the Damage Event Tree is given for the geotechnical instability of the front slope (Figure 5). Figure 6 concentrates on the hydraulic damage of front and rear slopes.

Significant external loads on a breakwater structure are currents, waves, water level and seismic activities. These loads may cause events such as the hydraulic instability of front and rear armour, geotechnical instability of front and rear slopes, or the destruction or sliding of the capping wall either directly or as a consequence of damage to the slope. For each event the (partial) probability should be defined. Some events are independent whilst others are dependent. However, the total probability of failure of a breakwater is greater than the probability of a single event, for example, the hydraulic instability of the front armour layer only.

Considerations of the probability of failure lead to an important conclusion for the design of the components of a breakwater. If limited finance is available for reducing the overall failure probability, it should be invested in those components which show the largest reduction in overall failure probability for the given amount of money. Generally these comprise the relatively low-cost components to the breakwater which, however, may be vital to the stability of the structure. Examples of such components include filters and berms. The costs of safeguarding these components from failure are minor compared with the costs of achieving a similar increase in overall stability by improving major cost components such as armour or slopes. Hence, from the point of view of both risk analysis and cost, low-cost components may be designed such that the probability of their failure is negligible compared with the probability of failure of high cost items. This also simplifies the Damage Event Trees by virtually eliminating unknown factors and limiting consideration to failure mechanisms which can be quantified.

Figure 6 shows a simplified Damage Event Tree which indicates that, basically, only the problem of hydraulic instability of armour, due to waves, remains. Although not fully understood, hydraulic instability can be analyzed with reasonable accuracy using the results of hydraulic model tests in combination with a stability formula, as is shown in Section 4. The geotechnical response of the structure to external loads can be described by using methods as presented in Reference 2 and this paper does not consider this aspect further.

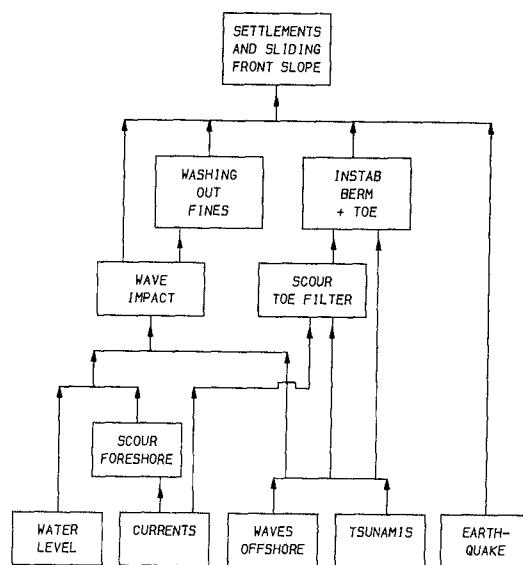


Fig. 5 Damage Event Tree - Geotechnical Instability Front Slope

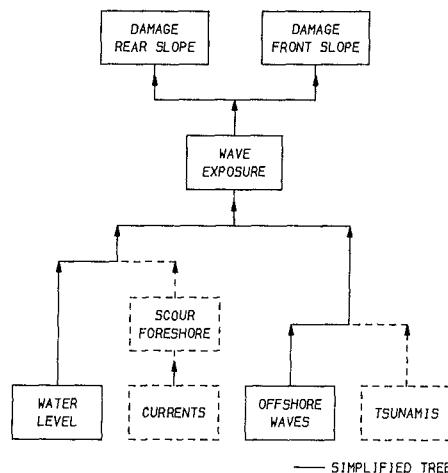


Fig. 6 Hydraulic Damage without Capping Wall

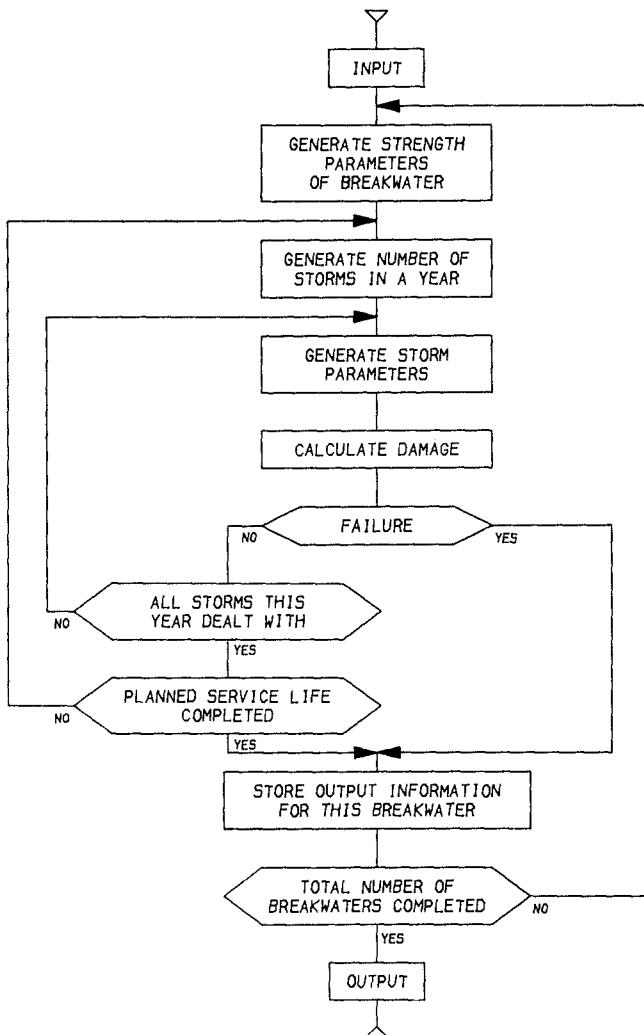


Fig. 7 Flow Diagram - Simulation Model.

3. A Computermodel for Level III Reliability Analysis

As stated in References 1 and 7 considerable assumptions were necessary to compute the probability on failure. As a result the outcome is not very accurate, and is very limited.

A much more promising method is an approximate level III approach based on a time domain simulation of the service life of a large number of breakwaters from which the strength parameters are randomly generated from given statistical distributions. Figure 7 shows a flowchart of the computer program developed to handle this approach. Firstly the properties determining the strength of the breakwater are randomly selected from the given distributions, which can be of an arbitrary shape. Subsequently the impact of various generated storms is recorded over the service life of the breakwater with a range of options on accumulation of damage, historical effects and repair works. The program restarts by generating the properties of a new breakwater and subsequently recording the impact of newly generated storms, up to the required number of simulations.

The computational effort to obtain a reliable result strongly depends on the width of the statistical distributions. Checks on the accuracy are based on comparing the output of runs with various numbers of simulated breakwaters. It was found that for the presented case the computational effort was within acceptable limits.

The Software program can be expanded to include modules for geotechnical instability, (due, for example to wave action) and for seismic activity (generated independently according to given distributions); both simultaneous with the modules for hydraulic instability. In the following section a mathematical description is given of the relation between damage and input parameters for the hydraulic instability of the armour layer.

4. Mathematical Description and Quantification of Hydraulic Stability

In order to assess damage as a function of various input parameters, a mathematical description of hydraulic stability is required. This description is complex because it must take account of the load history, as exhibited by settlements of the armour and armour breakage. Although research on this subject continues, it is entering the stage of being used for engineering purposes (References 3, 4 and 9).

The damage function can be assessed using a combination of stability formulae (to include the effects of known parameters), and hydraulic model testing. Model tests help to assess the level of stability of the structure and verify the relationship between parameters, such as wave period, wave groupiness, the time history and the resulting

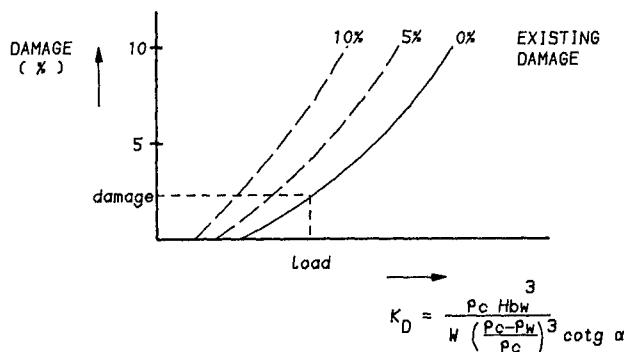


Fig. 8 Shape Factor

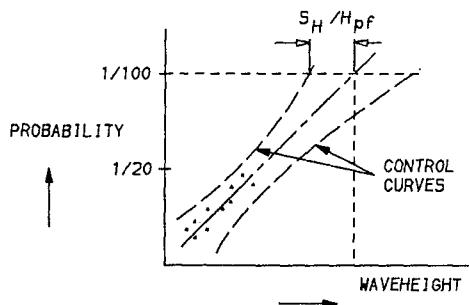


Fig. 9 Scatter Extreme Wave Heights

armour breakage. The graph shown in Figure 8 with damage along the 'y' axis and the Hudson Number 'Kd' along the 'x' axis illustrates. The Hudson Number is computed from the following equation:

$$Kd = \frac{\rho_c H^3}{W (\frac{\rho_c - \rho_w}{\rho_w})^3 \cot \alpha}$$

with H = wave-height at the location
of the breakwater without
wave breaking
 ρ_c = density of concrete
 ρ_w = density of water
 W = armour weight
 α = slope angle

The relationship in Figure 8 is shown as a function of the starting damage. Damage is expressed as the proportion of broken or displaced armour units related to the total number of units in the structure. Depending upon the sensitivity for wave parameters, tests should include the effect of spectral shape (peak period) and groupiness. Hence, extensive testing is required to provide the necessary input for the reliability analysis. The effects of small variations in input parameters, such as the weight and density of the armour unit and the slope angle, will produce damage of the order predicted by the stability formula. The wave-height excludes the effects of wave breakage (model testing will account for this phenomenon) but includes refraction, modelled numerically or physically.

For input into the model, the statistical distributions of the various parameters must be quantified. Table 2 gives a listing of the various parameters, the interrelation between the parameters, and the assumed modelling for the present study.

The estimated statistical distribution for parameters such as the armour weight is 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 for inaccuracy should be superimposed on the natural variation.

In the present study the joint probability distribution of wave-height, period, direction, groupiness and storm surge is equated to relations between the wave-height and other parameters. These other parameters are defined by a scatter parameter and the probability distribution of the wave height.

Particular emphasis should be laid on the scatter of the wave height distribution. Errors may occur due to:

- scatter and inaccuracies of the original data
- selection of probability function
- a considerable extrapolation outside the available period of observations

Table 2
Listing of Parameters

<u>Parameter</u>	<u>Symbol</u>	<u>Relation</u>	<u>Modelling</u>
concrete density	ρ_c	independent	independent
density of water	ρ_w	"	"
slope angle	α	"	"
armour weight	W	"	"
wave height offshore	H_{so}		independent
storm surge	s	joint probability distribution of H_{so} , T , s and GF	$s = F(H_{so})$
wave period	T		$T = F(H_{so})$
wave direction	ϕ		independent
wave groupiness	GF		in model test
tide	t	independent	independent
refraction factor	K_r	depending on ϕ , T	$K_r = K_r(\phi, T)$
wave breaking	K_b	depending on H, L, T, s, t	in model test
stability coefficient - damage relation	K_d	depending on T, GF, s, Kr	$K_d = F(T, GF, s, t)$

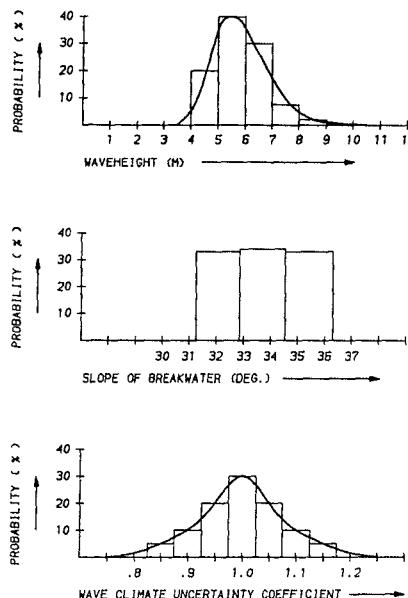


Fig. 10 Probability Density Functions

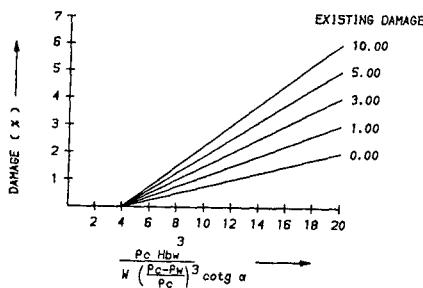


Fig. 11 Response of Structure to Wave Load

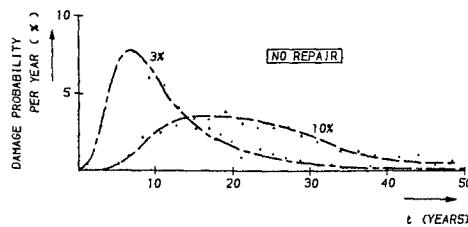


Fig. 12 Probability of Damage versus Time

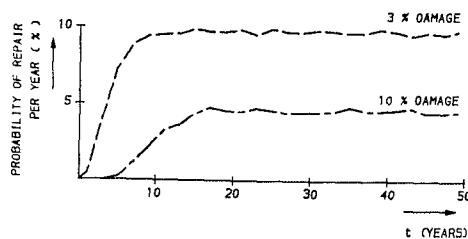


Fig. 13 Probability of Repair versus Time

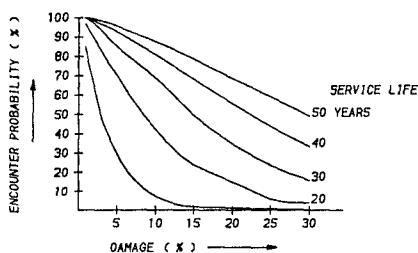


Fig. 14 Encounter Probability of Damage as Function of Service Life

This results in a large confidence band around the extrapolated function and gives standard deviations in the order of 10% to 15% of the average value, as shown in Figure 9 (References 5 and 8).

5. Application

An example of the application of the method as described above is shown in this section. It comprises a breakwater with a tetrapod armour layer of 25 tons. The wave climate is characterised by a 100 year return period wave-height of $H_s = 8.5 \text{ m}$.

Some examples of applied statistical distribution (wave-height, wave-height inaccuracy and slope distribution) are shown in Figure 10. The relationship between stability parameters and damage is assumed to be as in Figure 11. The concrete density is fixed on 2.43 T/m^3 .

Using the traditional level I approach, the design criterium was set at 3% damage in a 100 year event. Applying a 20% safety factor on the wave-height to include uncertainties, such as breakage of armour, the design wave-height is 10.2 m and, according to Figure 8, the criteria are satisfied.

The output of the level III simulation model is shown in Figures 12, 13 and 14. Figure 12 shows the number of breakwaters per year exceeding the 3% or 10% damage in a certain year of the lifetime. From this it can be seen that, although the traditional design approach suggests that the design criteria are satisfied, in closer reality many breakwaters (85%) will suffer more than 10% damage during a period of 50 years.

Figure 13 shows the number of breakwaters that would need to be repaired based on a 3% damage criteria after a given storm period. This shows that the probability of repair per year is 10%.

Figure 14 shows the encounter probability of damage in a certain service lifetime for a given level of damage. The design criterium based on a certain damage in an event with a 100 year return period results in an encounter probability of 39%. As shown in Figure 14 the encounter probability is much larger (99%). The damage belonging to the required encounter probability of 30% can be defined as severe.

In summary, it is essential that not only the wave-height but all parameters should be treated as statistical variables and that these variables must include the cumulative effects of damage.

6. Conclusions

The inaccuracy of the traditional design method is shown in establishing the real probability of damage. It is shown that a more reliable result can be achieved with the proposed level III approach, based on time domain simulation by utilizing a specially developed computer program.

The level selected for design and assessment of the failure probability is only reliable when all possible failure modes are included in the analysis. The neglect of an important mode will lead to an inadequate design. To avoid such errors the designer should be aware of causes and mechanisms of failures such as have occurred recently.

The proposed approach requires the designer to consider the accuracy of the various components of the study. This offers significant advantages over the traditional approach where the question of accuracy is not raised automatically. The proposed approach will further highlight those components where a large study effort is essential to minimize the risk of failure. Examples of these components are wave climate, wave refraction and structure response studies.

The reliability analysis also may lead to conclusions on the relative strengths of components of the breakwater. Components involving a relatively small financial investment can be designed with relatively high factors of safety and should not therefore have a significant contribution to the overall probability of failure.

In addition to variables in the design and in environmental conditions, further variables occur in the construction. Site conditions may impose variations between the design and the actual as-constructed structure. Also inconsistency in quality of the available material and inaccuracy of placement through available construction plant will inevitably play a certain role in the built-in safety of the final product. To what degree the variations will affect the overall quality is often difficult to conclude, in particular when such variation is sensitively related to the design assumptions. The proposed approach described herein will therefore prove to provide a very useful tool also for construction purposes.

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