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Example application of reliability assessment of coastal structures

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1. Introduction

During the last few decades the believe that absolute security could be provided has been recognized to be mistaken in many branches of engineering. This happens as a consequence of the imperfect predictability of load intensity, structural characteristics and structural behavior, as well as a consequence of human gross errors during the design and construction phases or during the using (exploiting) period.

In the field of coastal engineering this became particularly evident when breakwaters were constructed in deep-water condition, i.e. where water-depth ceases the beneficial effect in limiting the design wave height.

Beyond this special case, it is rather common in coastal engineering practice to dwell with uncertain environmental conditions and sometimes even with uncertain structural behavior. A method which can account for the multiform information available and which can produce a rational risk evaluation is therefore highly advisable.

A rationalization of the risk assessment produce some advantages.

1 – The various uncertainties are rationally incorporated and balanced in the assessment of the reliability of the structure.
2 – A more balanced design of components of the overall system can be made, avoiding the coexistence of overdesigned components blowing up the costs and of underdesigned components causing the risk.
3 – Cost of the structural solution to risk (building a stronger structure) can be assessed and compared to the non structural solution (reducing the uncertainty).
4 – A better insight into how the failure probability is built up from the various uncertainties is obtained: priorities can be established for further research aiming to improve the description of the system and reduce the uncertainty margins.

In the example presented failure and risk are looked at from the client point of view.

1.1. The different approaches to reliability assessment: levels 1, 2 and 3

Given some design of the structure, i.e. a configuration X and a set of parameters $\boldsymbol{x}$, and a set of pertinent safety relations, the safe and unsafe regions in the space of the random parameters $\boldsymbol{q}$ relevant to the problem (load and resistance) may be identified.

Along the failure boundary, i.e. the surface separating the safe and unsafe region, a *design point* $\boldsymbol{q}_d$ can be defined representing the typical failure conditions; the design point is usually defined as the most probable point on the failure boundary (the point where the probability density distribution is highest) or alternatively as some mean point on the surface.
Similarly a *characteristic point* $\boldsymbol{q}_k$ may be defined assigning to each variable some characteristic (average or cautious) value.
Quotients between the design and the characteristic values (for load parameters and the opposite for resistance parameters) represent the partial safety coefficients.

The failure probability P_f of the structure may be evaluated integrating over the unsafe region the probability density distribution of the relevant random parameters.
Such methods of checking the safety of the structure are often denoted as Level 3 reliability assessment methods. They are conceptually exact but some approximations may derive from the employed integration rule.

As the standard numerical integration techniques require unrealistic computing resources as soon as the number of random variables comes over six, the most commonly used methods of this level pertain to the Monte Carlo simulation techniques.
Basically all the random variables are simulated according to their known statistics, the safe-unsafe behavior of the structure is then evaluated in a deterministic manner for each simulated case. Failure frequency within simulated cases is a good approximation of failure probability if the number of cases is great enough.
Some special techniques have been set up in order to improve the efficiency of such computation, reducing the number of experiments necessary to reach an assigned accuracy.

Simplified reliability evaluation methods may be classified into two categories or levels, namely:
– Level 2, at which safety is checked under the assumption of joint normal distribution of the random variables and of some preassigned shape of the failure boundary (rectilinear or circular);
– Level 1, at which the appropriate reliability is provided by a number of partial safety factors or safety margins related to characteristic values of the basic random variables; this type of calculation is most suitable for everyday design practice.
In most cases safety factors are not explicitly related to probability distribution of random variables nor to failure probability but reflect some standard variability and tolerance to risk. Only if the safety factors are explicitly related to the failure probability and to the probability distribution of the variables the method may be used to evaluate the risk.

Level 2 evaluation methods yield naturally the design point and partial safety coefficients.

1.2. Structure lifetime and related admissible failure probability

Failure probability grows up during the working period of the structure in a rather simple manner if failure probability in unit lifetime depends on the current state but not on past history and if the structure characteristics as well as the environmental parameter statistics are stationary.

$$P_f(t) = 1 - \exp(-\mu t) = 1 - (1 - P_f(L))^{t/L} \tag{1}$$

with

$$\mu = -\frac{ln(1 - P_f(L))}{L} \simeq \frac{P_f(L)}{L}$$

This is not the case if some weathering effect occurs or if climatic changes are foreseen.
It is common practice however for the sake of simplicity to refer just to one time, the lifetime L of the structure, and to the related failure probability.

There are no generally accepted value for the lifetime and for the acceptable failure probability. Normally the design life is unique for all the parts of a project and is related to the time the structure is projected to be in service. In ROM 0.2-90 for structure of definitive character the following minimum lifetimes are suggested.

Tab. 1 Minimum Design Lives for Works or Structures of Definitive Character

Type of work	Required security level		
	1	2	3
	Design life, year		
General use infrastructure	25	50	100
Specific industrial infrastructure	15	25	50

Legend:

General use infrastructure: general character works: not associated with the use of an industrial installation or of a deposit.

Specific industrial infrastructure: works in the service of a particular installation or associated with the use of transitory natural deposits of resources (e.g. industry service port, loading platform of a mineral deposit, petroleum extraction platform, etc.).

Level 1: Works and installations of local auxiliary interest. Small risk of loss of human life or environmental damage in case of failure. (Defense and coastal regeneration works, works in minor ports or marinas, local outfalls, pavements, commercial installations, buildings, etc.).

Level 2: Works and installations of general interest. Moderate risk of loss of human life or environmental damage in case of failure. (Works in large ports, outfalls of large cities, etc.).

Level 3: Works and installations for protection against inundations or of international interest. Elevated risk of human loss or environmental damage in case of failure. (Defense of urban or industrial centers, etc.).

The maximum admissible structure failure probability during the service phase, according

to the same source are given in the following table. The values refer to conditions above which the structure itself is damaged and not to the normal operating conditions limit; these last limits are normally fixed for each project separately on economical grounds.

Damages can be divided in two categories:
- economic damages due to loss of use of the structure or need for replacement or repair (tangible damages);
- risk of death or injury to people, concern for public reaction to failure, image of the client and of the contractor (intangible damages).

Tab. 2 Maximum admissible structure failure probability in service phase

Damage initiation

Economic repercussion	Possibility of human loss: Reduced	Expected
Low	0.50	0.30
Average	0.30	0.20
High	0.25	0.15

Total destruction

Economic repercussion	Possibility of human loss: Reduced	Expected
Low	0.20	0.15
Average	0.15	0.10
High	0.10	0.05

The damage initiation or total destruction maximum admissible failure probability shall be adopted according to the deformation characteristics and ease of repair of the structure.
For brittle works without possibility of repair, the total destruction probability shall be adopted.
For flexible, semi-rigid or generally reparable works the damage initiation probability shall be adopted (damage initiation refers to a damage level preset according to the structural type). In these type of works, the total destruction risk shall also be analyzed (presetting according to the structural type the damage level to be considered as total destruction).

Legend

Possibility of human loss:
- Reduced: when human loss is not expected in case of failure or damage;
- Expected: when human loss is foreseeable in case of failure or damage.

Economic repercussion:
- Low: $r \leq 5$
- Average: $5 < r \leq 20$
- High: $20 < r$

where $r = \frac{\text{total cost of direct or indirect losses if work is disabled}}{\text{investment for the work}}$

For risk assessment during the construction phase the *lifetime* should be taken equal to the construction time or 1/50 of the structure lifetime if data regarding the construction phase duration are lacking. The maximum admissible failure probability can be adopted as for the service phase considering the economic repercussion as *mean* and the possibility of human loss as *reduced.*

1.3. Failure, failure modes and failure probabilities

A structure or part of a structure is considered unfit for use when its modification exceeds some particular state, called a limit state, beyond which it infringes one of the criteria governing its performance or use. A failure occur if any limit state of the structure is exceeded.

Among limit states it is usual to distinguish between:
- serviceability limit states, which are related to normal use and durability of the structure;
- ultimate limit states, which correspond to a ruinous collapse or at least to such modifications which make necessary to replace the structure.

Different safety margins or failure probabilities can be accepted for limit states in different classes because their consequences differ considerably.

Normally a failure (a service failure or an ultimate failure) may occur according to different modes, or infringing different limit states. If the event *failure according to the i-th mode* is indicated by E_i and P_{fi} is the related probability then

$$P_f = \text{Prob}[\, E_1 \cup E_2 \cup E_3 \cup \ldots \,]$$

where the union must be performed on all relevant failure modes (modes that produce a relevant increase of the failure probability).
If one is not sure that all the relevant modes have been considered the calculated value is only a lower bound to the actual probability (unsafe evaluation).

For the actual evaluation note that

$$max_i P_{fi} \;\leq\; P_f = \sum_{i=1}^{n} P_{fi} - \text{Prob[failure in two or more modes]} \;\leq\; \sum_{i=1}^{n} P_{fi} \qquad (2)$$

In general the two bounds on the failure probability are too wide for practical applications.
However if all the failure events are stochastically independent from each other

$$P_f = 1 - \prod_{i=1}^{n} (1 - P_{fi}) \qquad (3)$$

If moreover all relevant probabilities of failure are very small, i.e.

$$\sum_{i=1}^{n} P_{fi} \ll 1 \;,$$

as the probability of a combination of modes may be evaluated as the product of the probabilities of each combined mode, the probability of each combination becomes infinitesimal and to good approximation

$$P_f \simeq \sum_{i=1}^{n} P_{fi} \qquad (4)$$

In short, one can say that two errors of opposite sign are introduced when using the above equation. One, when we limit the number of modes to n, causes an underestimation of the structure failure probability. The other, when we neglect the probability of combinations, causes an overestimation. Sometimes these two errors compensate each other and equation (4) gives satisfactory results; sometimes, typically when some failure modes are positively or negatively correlated or failure probabilities are not small, equation 4 is misleading.

In all the formulae presented above failure is taken as exceeding any limit state in the original unmodified structure. In actual situations one cannot avoid to consider what is really happening to the structure after the limit state is exceeded.
The above formulae may be correctly applied to *brittle* structures, i.e. structures that, like a chain, fail as soon as one of the elements fails.
In other type of structures, like a rope of ductile wires, as one of the elements fails the load is automatically redistributed between the elements in such a way that the structure can resist even further loads. For this kind of structures, usually denoted as *failsafe* structures, the structure failure should be analyzed following the evolution of its state as loads are increased.

Caisson breakwaters behaves like brittle structures, as their ultimate failure take place as soon as -any caisson slides, or -any caisson is turned over, or -the foundation berm is substantially eroded.

A conventional rubble mound breakwater, within certain limits, and, more typically, a reshaping breakwater behave as failsafe structures. In a traditional breakwater the less stable armor units are removed first, leaving a slightly damaged but stable armor layer. In reshaping breakwaters armor stones are more on removed from the area near the mean water level, where the wave action is stronger, and displaced to form the submerged berm. The resulting mild slope and convex profile is more stable than the original one. In both cases failure should not be considered just the displacement of an armor unit, but a substantial modification of the structure which makes it unfit to resist further: e.g. the removal of so many armor units that the less stable core becomes exposed.

As a partial closure, the relevant step for a reliability analysis of a structure are:
- identification of the type of failure behavior and of all the relevant failure modes;
- identification of a failure function for each failure mode;
- evaluation or estimate of the statistics of all the random variable appearing in the failure functions: environmental parameters and structure characteristics;
- fixing the lifetime and the admissible failure probability for the analyzed case, accounting for the consequences of a failure;
- evaluating the elementary failure probabilities;
- combining the elementary failure probabilities into a structure failure probability;
- comparing the estimated and admissible failure probability in the light of the consequences of a failure and judge the adequacy of the whole.

We shall reanalyze now in the light of risk assessment a project yet brought to realization in order to exemplify the evaluation procedure and the possible benefits of the analysis.

2. Brindisi harbor and its environment

The development of the industrial activity near to the old natural harbor of Brindisi (main ferry link to Greece) and the need for a new coal terminal for 150'000 DWT ships (16 m draft) to supply a thermoelectric power plant have led to the construction of a long outer breakwater built during years 1985-1989.
Connected to the nearshore island of S. Andrea, the breakwater stretches for a total length of 2386 m offshore Punta Riso and now protects a new large external harbor in front of the industrial area (fig. 1).
The breakwater is divided into segments with slightly variable alignment: a conventional rubble-mound structure covers the initial 1310 m reaching a water depth of 24 m, while a caisson type structure follows on for 1076 m up to a water depth of 32 m. The tidal range at the site is negligible. The two final design cross sections are shown on figs 7 and 10.
The breakwater design was supported by various mathematical and physical hydraulic models with the aim to define the wave climate at the site, the refraction effects induced by a shoal fronting the breakwater, the stability of the sections, and allowing a comprehensive evaluation of

the risk of failure. For a complete description of the studies reference is made to the papers of Franco et al. (1984, 1985, 1986) and Chiumarulo et al. (1990).

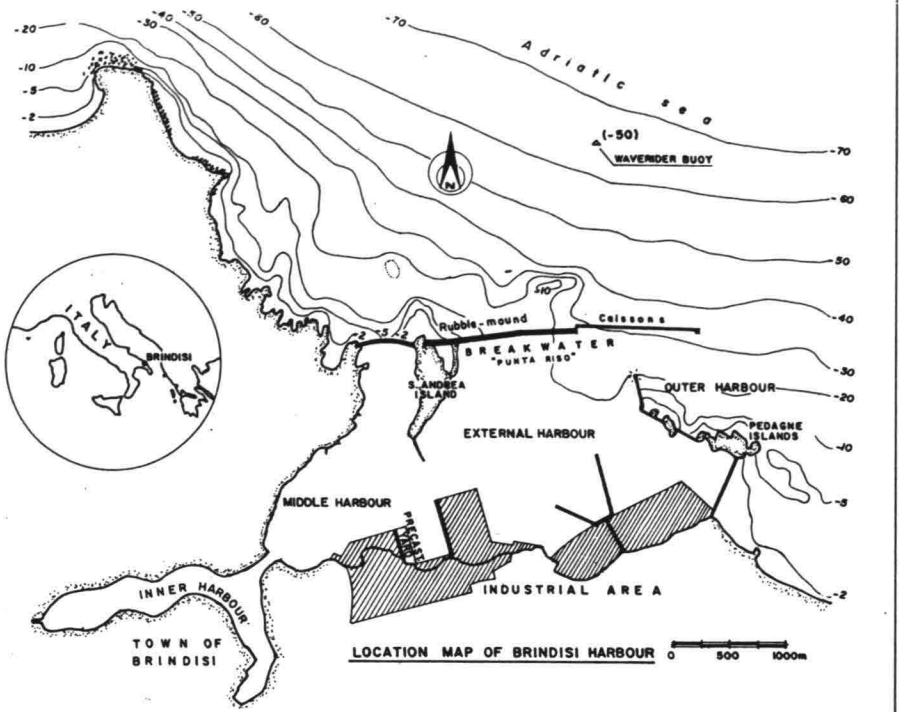


Fig. 1. Map and location of Brindisi harbor

2.1. The offshore wave climate

Due to the lack of instrumental records at the design stage, reference was made to the long series of ship observations and to a series of hindcasted storms.

A set of over 30'000 shipboard visual observations collected by KNMI was available for the period 1961-'80 within the area 40-42°N, 17-19°E. Despite their irregular distribution over time and space the observed deep water wave heights from the main sector (N+NW) were statistically analyzed to give a 25 year return period $H_s = 5.5$ m and a 100 year $H_s = 6.5$ m. The relation between frequency and and return period is based on an event duration of 7.5 hour.

Similar extreme wave statistics were carried out for 30 independent storms with peek significant wave height exceeding 3.5 m (18 with $H_s > 4.0$ m) within the sector 320-020°N hindcasted for the period 1951-'82. The wind field was obtained from weather charts and from wind records at four meteo stations along the Italian coast, while the hindcasting model was based on the SMB method. The results of the extrapolations according to the Fuller probability law are shown on fig. 2; data are plotted according to the Gringorten rule. For 25 year return period $H_s = 5.3$ m, while for 100 year $H_s = 6.0$ m with $T_s = 9.7$ s.

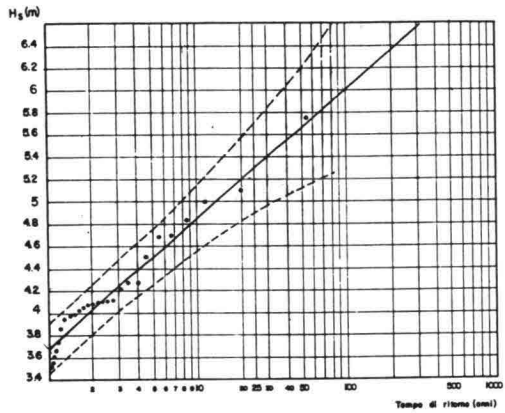


Fig. 2 Extreme wave height statistics: hindcasted values according to SMB method, wind derived from weather map or by space averaging

Similar hindcasting based on data from a single station led to somewhat higher estimates.

In January 1983 a Waverider buoy was installed 2 km offshore from the breakwater in a water depth of 50 m. Wave recordings are available until 1988, although the instrumentation was out of order for long periods, giving an effective total length of the recording period of 2.6 years. The maximum measured significant wave height is $H_s = 4.8$ m in Jan. '83 before the construction started. The severest wave condition experienced by the structure (uncompleted for the caisson segment) occurred on 16 Dec. 1988 with an estimated (not recorded) H_s of 4 m.
Despite the short duration of the series of data, a statistical analysis of peak significant wave height recorded during major storms ($H_s > 2.0$ m). The results are shown on fig. 3. When the threshold value was set at 3.5 m, the extrapolation (solid line) leads to $H_s = 6.08$ m for $T_r = 100$ years, which agrees very well with the value obtained from the hindcasted series (fig. 2) although intermediate wave heights seem more frequent (7 occurrences above 3.5 m instead of the expected 3 ones).

Few kms North of Brindisi a directional wave buoy was installed in July '89; the recording was remarkably regular and covers practically the full installation period. Data are available to me until September '91. The frequency distribution of the observed significant wave height is shown on fig. 6. The climate of the period is less severe than normal: extrapolation leads to $H_s = 4.7$ m for $T_r = 25$ year and the threshold $H_s = 3.5$ m is exceeded with frequency 0.6 event/year.

Wave measurements substantially confirm the climate obtained from hindcasting.

In order to use a longer storm case-history I will consider also the hindcasted extreme wave climate for Bari, a location not too far from Brindisi. 123 storm events were reconstructed using the 1951-'82 wind time series observed on land, SMB method and wind correction derived from the comparison of the intensity frequency distribution observed on ships.
Events were subdivided into sectors and the statistical analysis performed separately for each sector. Tab. 3 shows the results of the statistical analysis. Using the probability composition principle, the marginal distribution over wave height (disregarding direction) was evaluated. The distribution is presented as a ***wave height - return period*** relation in fig. 5. The relevant number of storms determining the extreme wave statistics is assumed to be 58, as only the sectors N and NNW are likely to produce the extreme waves.

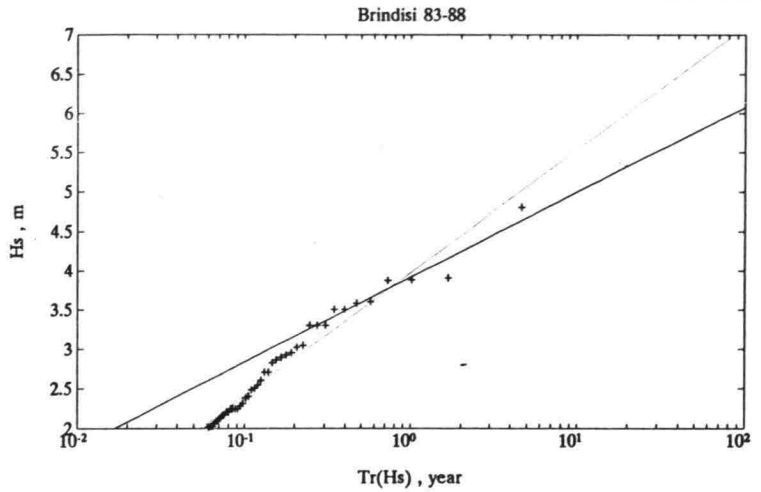


Fig. 3 Extreme distribution of wave height recorded in Brindisi during years 1983-'88

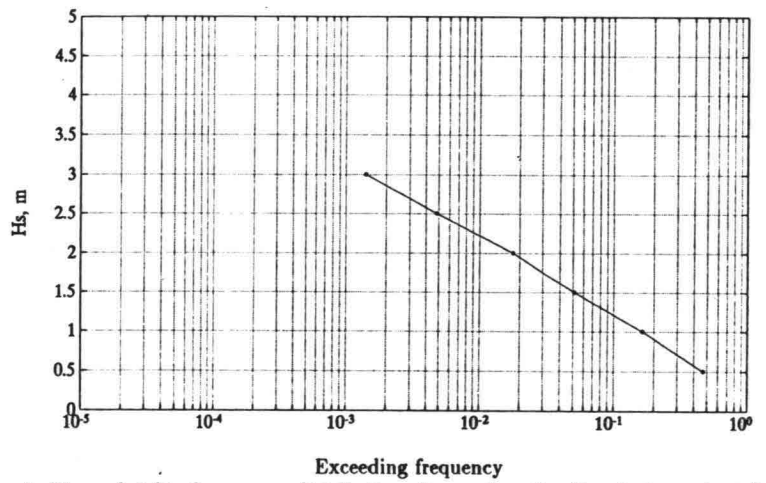


Fig. 4 Wave height frequency distribution from the directional buoy installed in Monopoli, recording period 7/89-9/91

Comparing all the sources they substantially agree about frequent waves giving an average 10 years wave height of 4.80 m and a decimation height of 1.05 m/decade up to 1 year wave height; they differ substantially as for the decimation height above 10 years. The longer records give an average value of 1.60 m/decade.

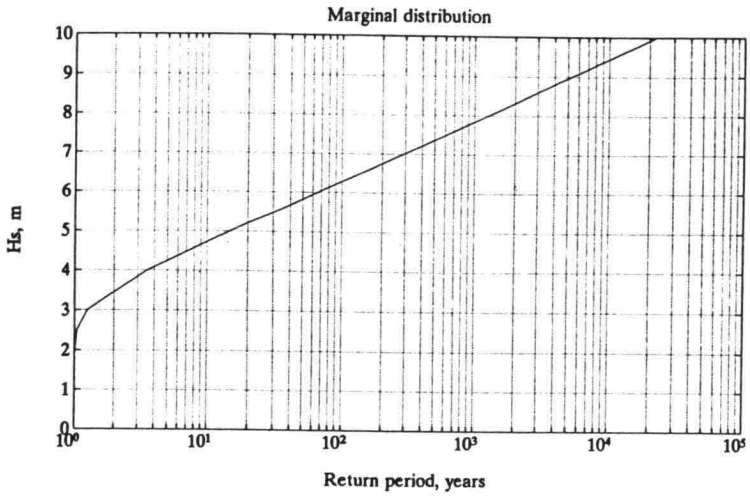


Fig. 5 Extreme wave height distribution derived for Bari from 1951-'82 wind data

Tab. 3. Extreme wave climate derived from storm hindcasting using the SMB method significant wave height in m

Direction °N		Return period years						Events no.
wind	waves	2	5	10	25	50	100	
270	303	1.27	1.48	1.61	1.78	1.91	2.04	4
300	319	2.20	2.64	2.93	3.29	3.56	3.83	11
330	334	2.75	3.58	4.13	4.83	5.35	5.86	26
000	357	3.05	3.75	4.21	4.80	5.23	5.66	32
030	030	2.66	3.27	3.68	4.19	4.57	4.95	19
060	061	2.20	2.42	2.56	2.75	2.88	3.02	4
090	086	1.53	2.01	2.33	2.73	3.02	3.32	2
120	100							0
150	108	0.95	1.28	1.50	1.77	1.97	2.17	18
all		3.35	4.20	4.70	5.30	5.80	6.30	123

From 123 storm events hindcasted for the period 1951-'82

2.2. Refraction and inshore wave climate

Mathematical and physical models were carried out in order to analyze refraction and to obtain the wave condition at the breakwater, which is fronted by a pronounced shoal (fig. 6). Both forward- and reverse-tracking ray techniques were used to produce refraction diagrams for different wave periods and directions. The conventional interpretation of wave ray pattern is not reliable for a quantitative prediction of wave height in front of the breakwater because of the

caustics generated by the shoal.

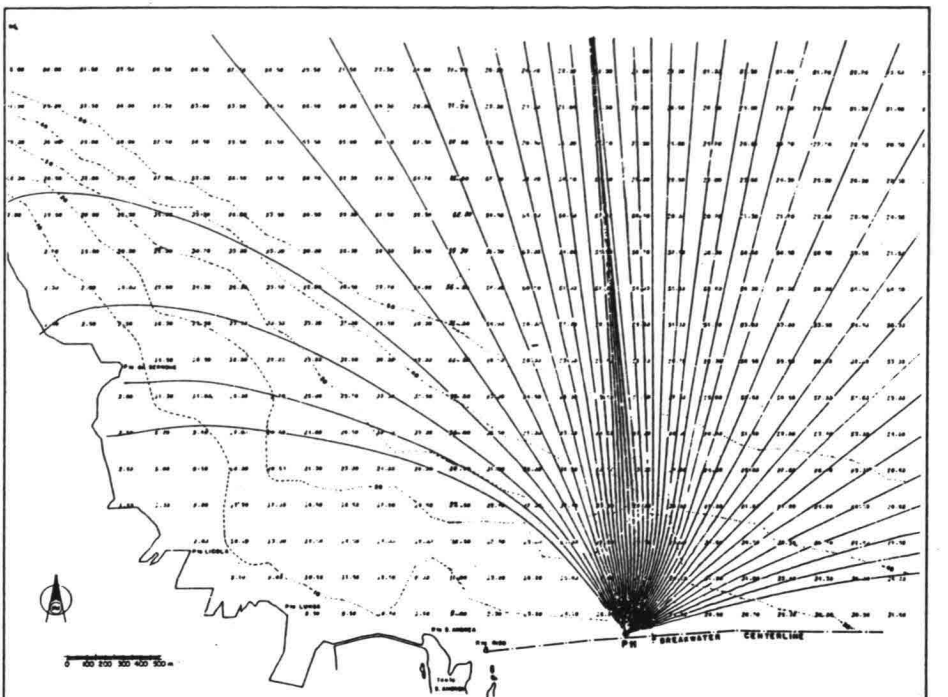


Fig. 6 Example of reverse refraction diagram for T = 10 s

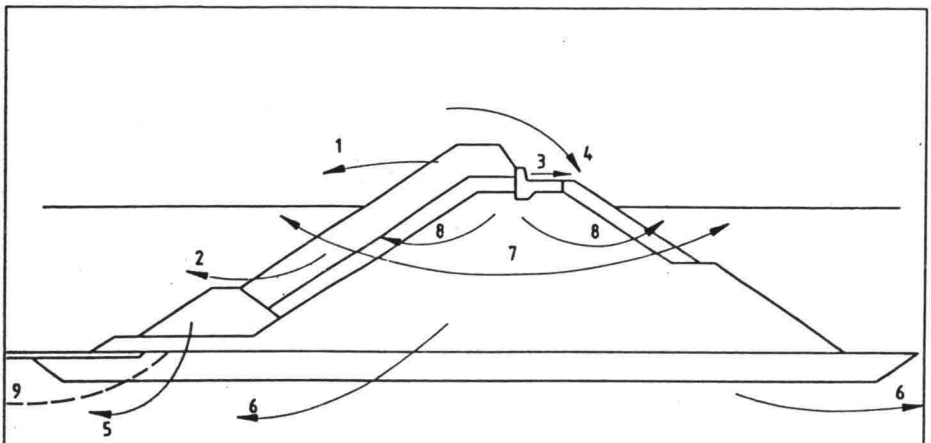
Satisfying results were instead given by the reverse spectral refraction method, as compared with those obtained with hydraulic model tests. The latter can actually incorporate processes such as friction, breaking and diffraction which are neglected in the mathematical model, but this one accounts for the directional spreading of the wave energy (not reproduced in the unidirectional wave trains generated in the physical model basin). As a consequence the correlation between the results of the two models (mathematical and physical) is not so high (0.5); the differences between the wave heights obtained with the two methods are nevertheless of the order or 15% only.

The energy focusing due to the shoal produce an increase of the wave height somewhere (depending on offshore direction) along the rubble mound breakwater segment reaching 20% of the deep-water wave height value.

The transition between the mound and the caisson section was located, as a consequence of these studies, outside of the lee of the shoal: wave focusing is restricted to the rubble mound segment.

3. The rubble mound section

The failure modes for a rubble mound breakwater are shown of fig.7 drawn from British Standards (1991). Neglect of an important mode will cause an overestimation of the safety of the structure as stressed before. Figure 8 shows the actual section of Punta Riso breakwater.



Key

1 Loss of or damage to armour units
2 Movement of armour layer
3 Cap movement
4 Overtopping causing lee scour
5 Toe erosion
6 Foundation failure
7 Loss of core material
8 Slumping due to excess pore pressure
9 Sea bed scour

Fig. 7 Significant causes of failure due to wave action (from BS 6349: Part 7)

3.1. A level 1 design according to formulae

The activity of PIANC's PTC II Working Group 12, set up in order to achieve a better understanding of the overall safety aspects in the design of rubble mound breakwaters (of which I was not a member), has been first presented and documented at the Breakwaters and Coastal Structures Conference in November 1991 (Mettam 1991 and Burcharth 1991).

The WG-12 has decided that the safety guidelines should be based on a *partial coefficient* system, and has given, for the stability of the armor layer (and some other related failure modes), formulae to evaluate the partial safety coefficients from the failure probability and other parameters characterizing the environment and its (lack of) knowledge.

As mentioned before, this rational level 1 approach has been completed and published only for the armor layer hydraulic stability, described by the Hudson or van der Meer formulae. The analysis was performed and results are included in the draft of the WG-12 reports also for hydraulic stability of low crested rock breakwaters, hydraulic stability of rock toe berm and run-up on rock armored slopes.

The Hudson formula as normally written

$$M = \frac{\rho_a H^3}{K_D \Delta^3 \, cotg\alpha} \tag{5.1}$$

can, as an example, be reorganized into a response function

$$g = Z \Delta \left(\frac{M K_D \, cotg\alpha}{\rho_a} \right)^{1/3} - H \tag{5.2}$$

where failure occurs if and only if $g \leq 0$, and Z is a random variable, normally with mean value 1, accounting for the imperfection of the formula.

Having decided for simplicity to introduce only two partial coefficients relative to the variable Z and H, γ_Z and γ_H, the response function can be developed into a design equation

$$G = \frac{Z}{\gamma_Z} \Delta \left(\frac{M \; K_D \; cotg\alpha}{\rho_a} \right)^{1/3} - \gamma_H H \geq 0 \tag{5.3}$$

In this equation for the variables $(Z, \Delta, M, \alpha, \rho_a)$ a characteristic value shall be taken equal to the expected or average value $(\hat{Z}, \hat{\Delta}, \hat{M}, \hat{\alpha}, \hat{\rho}_a)$, while for H the central estimate of the significant wave height likely to be exceeded once in the lifetime of the structure $(\hat{H}_s^L)$ is assumed.
K_D is a deterministic variable referring to the damage level accepted (when the significant wave height is assumed as wave height parameter).

In a similar way every stability formula can be reorganized into a response function including a variable representing its imperfection, and be developed into a design formula including the same two partial safety coefficients: γ_H accounting for all the uncertainties affecting the most important load variable H and γ_Z accounting for the uncertainty of the formula and of all the other random parameters appearing in it.

The formulae proposed for the partial coefficients are

$$\gamma_H = \frac{\hat{H}_s^{L_{Pf}}}{\hat{H}_s^L} + \sigma_{FH_s} \left[1 + \left(\frac{\hat{H}_s^{3L}}{\hat{H}_s^L} - 1 \right) k_\beta P_f \right] + \frac{k_s}{\sqrt{P_f N_e}} \tag{6.1}$$

$$\gamma_Z = 1 - k_\alpha \, lnP_f \tag{6.2}$$

where

$\hat{H}_s^T$ is the central estimate of the T-year return period of H_s; γ_H is applied to the lifetime return period value $\hat{H}_s^L$;

L_{Pf} is the return period of an event having probability P_f to be exceeded during the structural lifetime L; it is calculated from the encounter formula as $L_{Pf} = [1 - (1 - P_f)^{1/L}]^{-1}$;

σ_{FH_s} is the coefficient of variation of the measured or estimated significant wave height used for establishing the extreme wave distribution;

N_e is the number of independent data used for fitting the extreme wave distribution;

k_α, k_β, k_s are optimized parameters; $k_s \simeq 0.05$ for all failure modes; k_α and k_β are given in the WG-12 report for each formula.

The three terms appearing in eq. (6.1) may be easily interpreted: the first term gives the safety margin deriving from the intrinsic uncertainty of the future provided no statistical or measurement error were present in the data; the second accounts for measurement (or hindcasting) errors; the third term signifies the statistical uncertainty of the estimated extreme distribution.

In our case the objective of the example design could be: determine the mass (or the nominal diameter) of the tetrapods in the armor layer and the average mass of the quarry rock in the toe berm according to the following conditions;

1 – structure destruction with a probability $P_f \leq 0.10$ within the structure lifetime $L = 50$ years, according to the suggestion given in tables 1 and 2;

2 – damage initiation with probability $P_f \leq 0.30$ within approximately 1/5 of the structure lifetime (some maintenance is foreseen).

Let us use van der Meer (1988) formula for tetrapods on the usual slope 2/3, which in the form of design equation may be written:

$$\frac{1}{\gamma_Z} \left(3.75 \frac{N_D^{0.5}}{N_w^{0.25}} + 0.85 \right) S_m^{-0.2} \Delta D_{n50} \geq \gamma_H \hat{H}_s^L \tag{7}$$

where N_D is the damage level measured from the number of displaced units and for which the WG-12 suggests to evaluate the partial coefficients according to formulae (6.1) and (6.2) with $k_\alpha = 0.026$ and $k_\beta = 38$.

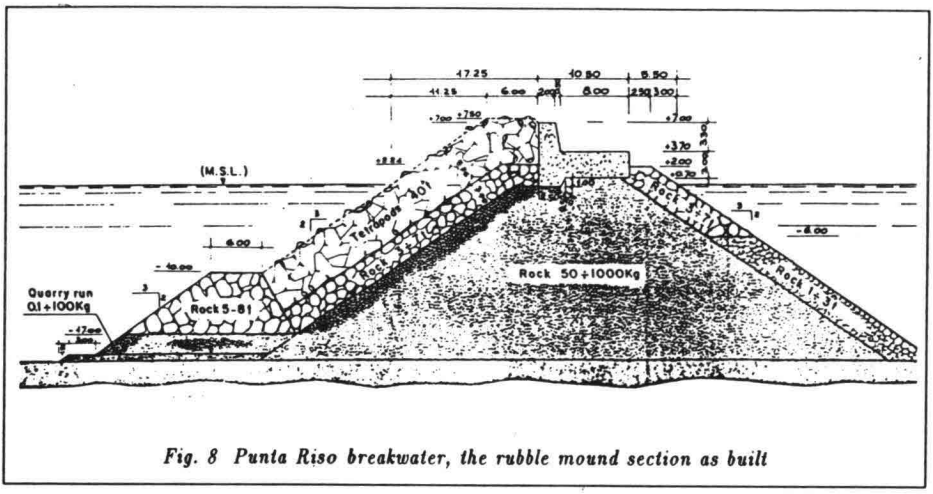


Fig. 8 Punta Riso breakwater, the rubble mound section as built

Let us assume the extreme offshore wave condition be given by

$$H_s(T_r) = 3.1 + 1.6\ log_{10} T_r \quad (H_s \text{ in m and } T_r \text{ in year}) \tag{8}$$

based on 58 significant hindcasted events, and $S_m = 0.038$.
Let us assume that no systematic reduction of the wave height occurs due to refraction, friction and breaking, but in some random location related to exact offshore wave direction some focusing of waves may occur causing a local increment of wave amplitude about 15%.
Due to the combination of hindcasting and refraction analysis let us assume the inshore wave estimation error $\sigma_{FH_s} = 0.15$.
Let us assume $\rho_a = 2400$ kg/m^3, $\rho_w = 1025$ kg/m^3, $N_D = 0$ for the initiation of damage and 1.0 for severe damage (the author suggests 1.5 for destruction, but in the light of what is said later the lower value is preferred).

The evaluation of the necessary armor units mass is straightforward. The following table 4 gives the relevant results.
It is clear that the maintenance condition $-2-$ is more severe than the no destruction condition $-1-$, the first being only almost satisfied in a space average sense, while the second is almost everywhere satisfied.

Had we used Hudson formula, we could have obtained similar results by a proper choice of the damage parameter K_D. The following table shows the result of computations carried out with $K_D = 8.0$ for the structure destruction and $K_D = 6.0$ for damage initiation; the value are the extreme of the range of values suggested by British Standards (1991) to be used in conjunction with significant wave height as wave intensity parameter. It is evident that the range of wave conditions assumed to be included between start of damage and destruction is much wider according to van der Meer suggestion (a factor 1.6 on wave height) than the range corresponding to K_D value suggested by British Standards (a factor 1.1 on wave height).
Actually the factor 1.6 is not far to the value 1.5 observed by Jackson (1968) and quoted by CERC (1984) corresponding for tetrapods to the damage range 5-50%.
Jensen (1984) for tetrapods and for slopes of 2:3 or 1:2 quotes K_D values in the range $3.0 - 8.0$, where the lower value correspond to the initiation of damage and very long waves, while the higher value correspond to a 5% damage after exposure to the given wave condition for durations

from 3 to 5 hours. It is evident that the value of 6 is inadequate for damage initiation, and that, trusting on the value 3 quoted by Jensen, the mass shown on the last rows of table 5 should be doubled.

Tab. 4 Necessary mass of tetrapods according to van der Meer formula and the partial safety coefficient design method

T year	$\hat{H}_s^T$ m	P_f	γ_{H1}	γ_{H2}	γ_{H3}	γ_H	γ_Z	M_{50} t	D_{n50} m	
50	5.80	.1	1.266	.059	.021	1.346	1.060	29.5	2.32	
50	5.80	.1	1.266	.059	.021	1.548	1.060	44.9	2.67	*
50	5.80	.2	1.178	.023	.015	1.215	1.042	20.7	2.06	
50	5.80	.2	1.178	.023	.015	1.398	1.042	31.4	2.37	*
10	4.70	.3	1.153	.005	.012	1.170	1.031	40.7	2.59	
10	4.70	.3	1.153	.005	.012	1.345	1.031	61.8	2.97	*
10	4.70	.5	1.058	.000	.009	1.068	1.018	29.8	2.33	
10	4.70	.5	1.058	.000	.009	1.228	1.018	45.3	2.68	*
8	4.54	.3	1.159	.004	.012	1.175	1.031	37.3	2.51	
8	4.54	.3	1.159	.004	.012	1.351	1.031	56.8	2.89	*
8	4.54	.5	1.062	.000	.009	1.071	1.018	27.2	2.26	
8	4.54	.5	1.062	.000	.009	1.232	1.018	41.4	2.60	*

* accounts for a 15% wave height increment due to refraction

I wish just point out the fact that, as all van der Meer formulae are more sensitive to wave conditions (including a period effect) than Hudson formula, they require a greater γ_H which cancels completely, at least in our conditions, the lower γ_Z value due to the lower error of the formulae. Maybe the effect as it comes out from the WG-12 partial coefficients formulae is overestimated, but in principle it may be true: a more sensitive formula requires better data and could be useless if these are only poor or uncertain, as estimates for the future.

Finally we should remark that the mass of the tetrapod units (40 t) is greater than the cautious maximum value suggested by British Standard (1991) (30 t) in order to avoid the risk of unit breakage. Problems may arise for rocking or displaced units and attention should be paid to units resistance.

Referring to toe berm stability, WG-12 quotes the design relation

$$\frac{1}{\gamma_Z} 8.7 \left(\frac{h_t}{h_s}\right)^{1.43} \Delta D_{n50} \geq \gamma_H H_s^L \tag{9}$$

where h_t = depth of the toe-berm,

h_s = depth where the structure is based on,

to be used in connection with formulae 6.1, 6.2 and $k_\alpha = 0.087$ and $k_\beta = 100$.
Results of computation are shown on table 6.

Tab. 5 Necessary mass of tetrapods according to Hudson formula and the partial safety coefficient design method

T year	$\dot{H}_s^T$ m	P_f	γ_{H1}	γ_{H2}	γ_{H3}	γ_H	γ_Z	M_{50} t	D_{n50} m	
50	5.80	.1	1.266	.004	.021	1.291	1.083	48.4	2.74	
50	5.80	.1	1.266	.004	.021	1.484	1.083	73.6	3.15	*
50	5.80	.2	1.178	.000	.015	1.192	1.058	35.6	2.47	
50	5.80	.2	1.178	.000	.015	1.371	1.058	54.1	2.84	*
8	4.54	.3	1.159	.000	.012	1.171	1.043	20.7	2.07	
8	4.54	.3	1.159	.000	.012	1.347	1.043	31.5	2.38	*
8	4.54	.5	1.062	.000	.009	1.071	1.025	15.0	1.86	
8	4.54	.5	1.062	.000	.009	1.232	1.025	22.9	2.13	*

* accounts for a 15% wave height increment due to refraction

Tab. 6 Necessary mass of toe-berm rock units according to a partial safety coefficient design method

T year	$\dot{H}_s^T$ m	P_f	γ_{H1}	γ_{H2}	γ_{H3}	γ_H	γ_Z	M_{50} t	D_{n50} m	
50	5.80	.1	1.266	.013	.021	1.300	1.200	9.3	1.52	
50	5.80	.1	1.266	.013	.021	1.495	1.200	14.2	1.75	*
50	5.80	.2	1.178	.001	.015	1.193	1.140	6.2	1.33	
50	5.80	.2	1.178	.001	.015	1.372	1.140	9.4	1.53	*
50	5.80	.3	1.122	.000	.012	1.134	1.105	4.8	1.22	
50	5.80	.3	1.122	.000	.012	1.304	1.105	7.4	1.41	*

* accounts for a 15% wave height increment due to refraction

In the actual design the choice fell on:
- tetrapod mass of 40 t as main armor units,
- toe-berm rock units with mass 5-8 t.

Based on the formulae only, the destruction probability in the lifetime, would have been:
- for the main armor failure $\ll 0.10$ on the average
  $\simeq 0.13$ where refraction focuses wave energy;
- for the toe-berm failure $\simeq 0.18$ on the average
  $\simeq 0.30$ where refraction focuses wave energy.

Assuming that no other failure mode produces any relevant failure probability, and that the two mentioned modes are independent (i.e. actually that what is meant as a failure of the toe berm doesn't force the subsidence or the armor layer), the structure failure probability, evaluated according to equation (3), is < 0.26 on the average and $\simeq 0.39$ where the refraction focuses wave energy. The assumption that the two modes are independent is actually not relevant: if we assume that a toe berm failure forces the failure of the armor layer the two probabilities would be 0.26 and 0.40 respectively.

The use of the formulae compared to the case of almost perfect knowledge of the

phenomena requires an overdesign in order to maintain the failure probability. The overdesign is measured by γ_Z. We may expect, referring to a better design tool as a physical model test would be and failure probability remaining constant, a 20% decrease in the required tetrapod mass and a 70% decrease for the toe berm armor units mass. Does this pay for the model test, the model is worth to be carried out.

3.2. Model tests and the actual design

In our case model tests were conducted by Delft Hydraulics Laboratory (1983-a and -b) at a scale 1:51 either in a flume and in a basin. The permeability of the core and of the sublayer were correctly scaled according to Cohen Lara formula.
Both flume and basin tests were carried out to verify both the rubble mound and the caisson sections subjected to increasing wave height (H_s = 5-8 m , $T_s = 0.8H_s + 4.5$ s). Results referring to the rubble mound section are summarized here.
Following measurements were made in flume tests:
– overtopping frequency and discharge,
– wave disturbance behind the breakwater due to overtopping,
– frequency of rocking and displacement of armor units,
– displacement frequency of the toe berm stones.
The test in the basin were basically used to check the complete final design of the breakwater and to verify refraction effects.
The flume test results are essentially: the 16 m^3 (40 t) tetrapod armor with 0.12/m^2 placement density was stable for waves up to $H_s = 8$ m. The toe berm (1-7 t in the original design) was flattened out for $H_s > 5$-6 m, but this did not affected the armor stability. The preliminary designed concrete superstructure (without the seaward heel) was stable against sliding for $H_s \leq 7$ m and not stable for $H_s = 8$ m, assuming a friction coefficient of 0.7. The overtopping frequency reached 12%.
During the initial test series in the basin noticeable damage occurred to the armor due to the low placement density and to the weak rubble toe berm. The following tests were conducted with the final characteristics: 0.143/m^2 placement density and 5-8 t rock toe berm. No significant damage to the breakwater was observed in the model up to H_s = 8.0 m.

Rocking of tetrapods started for $H_s > 4.0$ m while displacement for $H_s > 6.0$ m. The observed number of rocking and displaced units were regularized (Franco et al. 1986) obtaining the relations, tested up to $H_s/(\Delta D_n) = 2.4$:

$$D_{\text{rocking}} = 1.78 \cdot 10^{-2}\left(\frac{H_s}{\Delta D_n}\sqrt{\frac{lnN_w}{2}} - 1.79\right)^2 \tag{10.1}$$

$$D_{\text{displaced}} = 0.46 \cdot 10^{-2}\left(\frac{H_s}{\Delta D_n}\sqrt{\frac{lnN_w}{2}} - 2.66\right)^2 \tag{10.2}$$

Trusting completely on these relations, trusting on the assumption that after 7 rocking events an average unit will break up, on the assumption only partially confirmed by observations that the frequency of rocking of a unit is equal to the frequency of wave height exceeding some critical value specific to that unit (equation 10.1 correspond rocking frequency equal to once in N_w waves, being $H_s\sqrt{lnN_w/2}$ the median highest wave height, and describes the frequency distribution of the threshold wave height between the different units) and on the extreme wave climate of fig. 2 Franco et al. (1986) have evaluated a total expected damage over 100 year lifetime of 4% due to rocking and of 0.7% due to displacement.
Damage $D_{...}$ is measured as the ratio of the number of rocking or displaced tetrapods to the total number of units present on the analyzed breakwater segment.

A reálistic risk assessment should account also for the random character of the environmental parameters and of some of the structure parameters, e.g. the concrete density and strength or the tetrapod placement density, and of the actual response functions for all the

relevant failure modes.
The previous analysis has shown however that unit breakdown due to rocking is probably the critical (most dangerous) failure mode. The experimental tests proved moreover that rocking frequency is strongly dependent on placement density (more than hydraulic damage), but all but one test series were carried out with the final suggested density; data are not sufficient to quantify the dependence.
Moreover I am not informed of any reliable model useful to estimate accurately the breakdown process of tetrapods. We might perhaps guess the distribution of the (random variable) number of rocking events leading a unit to destruction, just to be able to evaluate a subjective failure probability. The simulation would follow the lines sketched in paragraph 3.2; rocking frequency could be estimated as in Franco et al. (1986) combining the Rayleigh distribution for wave height with the assumption of equal frequency of rocking and of exceeding the of a critical wave height threshold specific to each unit.
As a consequence of the strongly subjective character of the assumed distributions and consequently of the results, these are not presented even if nothing extraordinary comes out from them.

4. The caisson section

The caisson section was tested in the physicals model mentioned before.
Vertical and horizontal forces and overturning moments due to wave action were measured in the wave flume for the original design, the final design and a few different alternatives. Overtopping frequency and mean discharge, as well as wave generated by overtopping on the harbor side were also measured. The final design was tested in the wave basin.
The main results of the tests may be summarized:
– wave overtopping started for $H_s = 4.7$ m, while for the highest tested waves ($H_s = 8$ m) overtopping frequency reached 30%, overtopping mean discharge was 0.4 $m^3/s/m$ and the significant wave height in the harbor was 1.5 m;
– the rubble berm and the perforated toe slabs in front of the caissons showed to be stable even with the severest conditions;
– the original design caisson (concrete density 2.4 t/m^3, sand fill density 1.9 t/m^3) slid by 2 m when H_s was equal to 7.3 m, the corresponding friction coefficient was 0.7;
– the final design caisson showed to be stable up to the severest tested conditions.
The final design section (figure 9) does not differ significantly from the one tested in the wave basin.

3.1. Failure modes and mechanisms

The failure modes identified for a vertical breakwater in British Standard (1991) are reproduced on figure 10. The failure mechanisms considered in this example are the classical ones: sliding and overturning. Wave overtopping is not critical for the structure itself.

The choice of the two considered failure modes reflects the uncertainty on the stability of caissons, persisting notwithstanding the physical tests due to scale effects for caisson-foundation friction and for foundation bearing capacity; on the contrary the probability of a failure of the seaward foundation appears irrelevant after the tests, where hydraulic stability was correctly reproduced.
Wave forces and structure weights are correctly and reliably reproduced in the model according to Froude scaling law, the same is not sure for the bearing capacity of the mound foundation and for friction between the caisson and the mound as the strength of materials is not reproduced. Actually friction coefficients measured in small scale model tests ($f = 0.7$-0.9) are significantly greater than the coefficients suggested by experience in prototype situations (ranging between 0.5

and 0.7) where some crashing take place in the contact layer. Cohesion effects are similar.

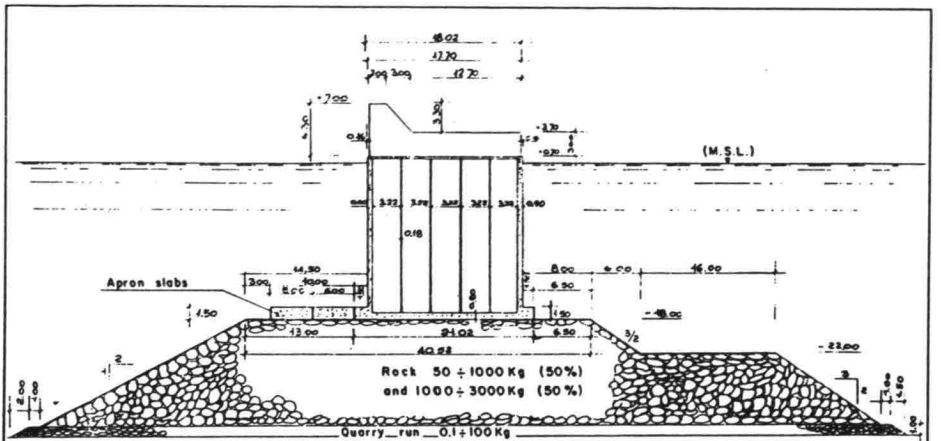


Fig. 9 Punta Riso breakwater composite section

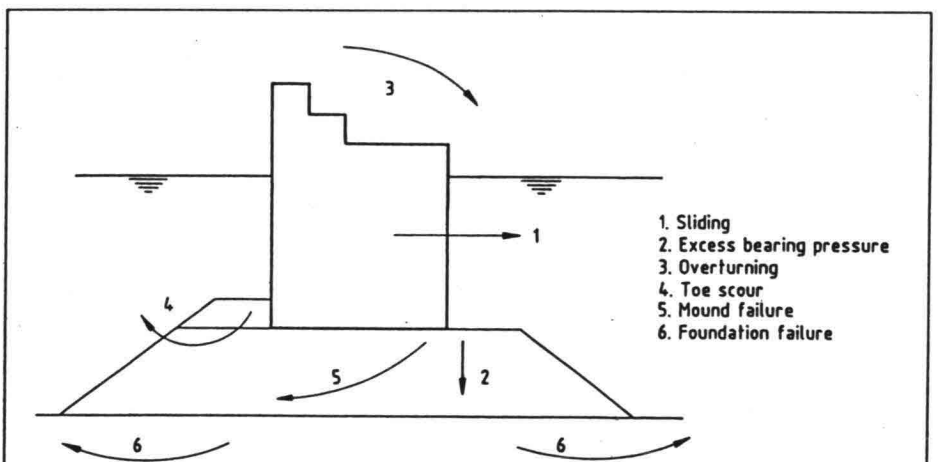


Fig. 10 Failure modes for a composite breakwater structure (from BS 6349 Part 7)

The equilibrium relations (12) for the caisson unit and the relevant bounds (13) (the signs in each equation are assumed in such a way that every force or moment is actually not-negative) are:

$$G - P_B + P_V - R_V = 0 \tag{12.1}$$
$$P_H - \mathrm{R}_H = 0 \tag{12.2}$$
$$\mathbf{G} - \mathbf{P}_B + \mathbf{P}_V - \mathbf{P}_H - \mathbf{R}_H = 0 \tag{12.3}$$

$$R_H \leq cA_{\text{eff}} + f R_V \tag{13.1}$$
$$R_V \leq R_{\text{ult}} \tag{13.2}$$

where

G is the caisson weight,
P_B is the underpressure lift force (buoyancy),
P_V is the vertical component of the wave-force,
P_H is the horizontal component of the wave force,
R_V is the vertical component of the foundation reaction,
R_H is the horizontal component of the foundation reaction,
$\mathbf{G}$ is the moment of the caisson weight around the heel,
$\mathbf{P}_B$ is the moment of the underpressure lift force around the heel,
$\mathbf{P}_V$ is the moment of the vertical component of wave force around the heel,
$\mathbf{P}_H$ is the moment of the horizontal component of wave force around the heel,
$\mathbf{R}_H$ is the moment of the vertical reaction of the foundation, its positivity ensure the usual no-overturning condition,
f is the caisson-foundation friction coefficient,
c is foundation cohesion,
A_{eff} is the effective contact area between caisson and foundation,
R_{ult} is the bearing capacity of the foundation, accounting for the eccentric and oblique load.

Weight and pressure forces should be evaluated first, then the foundation reaction components from the equilibrium equation and eventually the bounds should be verified.

For the evaluation of the pressure forces we should choose some appropriate formula: the Saintflou theory pressure formulae or, alternatively, Goda formulae. Since from the model tests direct measurements of the pressure forces are available the two formulae were controlled. The comparison was restricted to the most important component P_H; from the comparison with measured forces an estimate of the reliability of the two approaches was also derived.
Saintflou theory was observed to overpredict systematically wave pressure forces, whilst figures given by Goda procedure were in good average agreement with measures of the specific physical model, Delft Hydraulics 1983-a, the difference remaining in every controlled case below 6%.
Goda (1990), reexamining the safety against sliding of prototype concrete caissons over rubble mound foundation for 21 slided breakwaters and 13 not-slided, obtained the following statistics for the safety factor (average $\pm$ standard deviation).

Slided cases	0.81 ± 0.15
Not-slided cases	1.01 ± 0.11

The design procedure suggested by Goda is essentially a level 1 procedure: characteristic values for the wave load (pressure formulae) and for the geotechnical parameters are suggested in combination with global safety factors for sliding and overturning, which should not be lower than 1.20. These limits are mandatory in Japan.
The underlying risk is not clear. Actually a safety level of the procedure giving the design wave for sure (or almost sure as is the case for past events) may be estimated comparing the assumed safety coefficient with the statistic of slided and not-slided cases. The probability of a sliding if the test is passed and wave characteristics are correctly estimated should be $< 5\%$, since the standardized variate corresponding to s.f. $= 1.2$ is about 2 ($1.20 \simeq 1.01 + 1.7\text{x}0.11 \simeq 0.81 + 2.6\text{x}0.15$), i.e. normally significantly smaller than the exceeding probability of the assumed design wave height.

4.2. A level 1 risk assessment

Given the brittle nature of a caisson breakwater, the structure should be tested against

complete destruction. Lifetime L is assumed to be 50 years and failure (encounter) probability $P_f = 0.10$; the corresponding offshore design wave-height and -period according to wave climate (8) are:

$$H_{s0} = \gamma_{H1}\, \hat{H}_s^L = 1.27 \text{ x } 5.80 = 7.35 \text{ m}, \qquad T_s = 10.4 \text{ s}.$$

The other environmental parameter are assumed as in the example of the following paragraph. Friction coefficient is assumed 0.6 as suggested. Concrete and tout-venant filling density are assumed 2.4 and 2.1 t/m^3. Water level is assumed +0.50 m a.m.s.l.
The resulting parameter of the pressure distribution are.

$H_{1/3} = 6.76$ m	$H_{max} = 12.13$ m	$\eta^* = 18.19$ m
$p_1 = 9.25$ tf/m^2	$p_3 = 6.47$ tf/m^2	$p_u = 5.80$ tf/m^2

Forces and moments acting on the caisson (the reaction of the foundation is excluded) are:

		Forces tf/m	Moments tf
Weight of the	crownwall	155	1823
	caisson	742	7786
Static buoyancy		344	3613
Wave forces	horizontal	195	2485
	vertical	64	810
Resultant	vertical force	487	5047
	horizontal force	195	2583

The safety factor of the actual design against sliding and overturning are:

Against sliding:	s.f. = 1.50
Against overturning:	s.f. = 1.95

The structure failure probability should be hence probably smaller than 0.10, the exceeding probability of the assumed design wave height.

4.3. A level 3 risk assessment

The evaluation is performed according to a plain Monte Carlo method.
The number N of randomly generated cases should be great enough to give a satisfying convergence of frequency to probability. As, for the plain Monte Carlo method

$$\sigma_f^2 = \frac{p(1-p)}{N}$$

we shall assume $N = 10000$, in order to have a relative error of 1% for $p \simeq 0.5$ and a 10% error for $p = 0.01$. Obviously the computation must be automatized.

The "new" Terzaghi formula can be used to evaluate rapidly the bearing capacity of a ribbon shaped foundation per unit length:

$$R_{ult} = B(\, c\,\xi_c N_c + q(1 + \xi_q N_q) + \tfrac{1}{2}\gamma B \xi_\gamma N_\gamma) \qquad (11.1)$$

where

B is the foundation width,
c is the foundation cohesion,
q is the vertical load acting on the foundation plane aside the caisson, 0 in our case,
γ is the unit weight of foundation material,
N_c, N_q, N_γ are shape coefficients depending on internal friction angle of the foundation mound given on textbook or manuals,
ξ_c, ξ_q, ξ_γ are shape coefficients depending on eccentricity and inclination of the load, given according to different methods on textbook or manuals.
In order to account for eccentric and inclined loads the method described in Meyerhof (1953 and 1955) has been used.

Since the bearing capacity decrease to zero when the resultant of the active forces reaches the heel point, condition 13.2 is more stringent than the usual no-overturning condition. That's the reason why I won't consider any more the true overturning failure mode, and what is meant as overturning is failure mode 2 of figure 10.

Due to its brittle character, the structure behavior should be examined only for the maximum wave encountered in its life and a cautious attitude should be preferred.
The distribution of the extreme wave height (wave height of relevant storms) is supposed to be coherent with the set of data supporting eq. (8). Selecting the storm events coming from the sector NW-NE with significant wave height greater than $H_0 = 2.50$ m, the distribution fits well (fig. 11) an exponential law with parameter H_1 estimated as the mean excess over the threshold from a set of $N_e = 63$ events in $T = 32$ year, giving $H_1 = 0.79$ m.

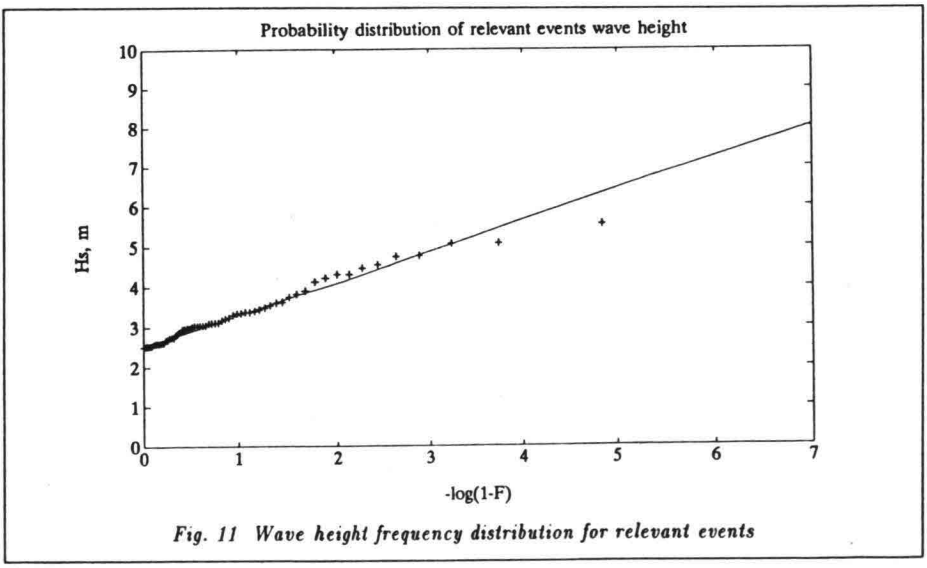


Fig. 11 Wave height frequency distribution for relevant events

$$F_{H_s}(H) = 1 - exp(-\frac{H - H_0}{H_1}) \qquad \text{for } H > H_0 \tag{12.1}$$

The distribution of the maximum significant wave height in the life of the structure is given by

$$F_{H_s^L}(H) = \left(1 - exp(-\frac{H - H_0}{H_1})\right)^{N_L} \qquad \text{for } H > H_0 \tag{12.2}$$

where N_L is the number of storms in the lifetime L. It may be estimated as $L N_e / T$ but it is obviously not certain. Both the observed number of cases and the probable number of storms

may be interpreted as independent realizations of the same Poisson event process. As both the observed and expected number of events are far greater than 10, the normal asymptotic distribution of the number of events in both time intervals can be used, once for the estimate of the mean event frequency λ, solving for λ the following equation, where u is a standardized normal variate

$$\frac{N_e - \lambda T}{\sqrt{\lambda T}} = u \quad \Leftrightarrow \quad \lambda T = N_e - u\sqrt{N_e + u^2/4} + u^2/2$$

and the other for the simulation of the number of storms in the lifetime, i.e. $N_L \sim N(\lambda L, \sqrt{\lambda L})$. The significant wave period was estimated assuming a frequency distribution of significant wave steepness. Similarly a frequency distribution of the mean water level elevation (z_w) was assumed. No wave enhancement due to refraction is supposed to take place in front of the caisson segment of the breakwater.
As will be clear from the results the climate assumed is somewhat more severe than the one given by eq. (8).

The structure resistance parameters, i.e. the density of concrete and caisson filling material, the friction coefficient of the caisson-foundation contact, friction angle and cohesion of the foundation rubble mound and the random factor multiplying pressure given by Goda's formulae have been assumed random with frequency distribution specified in the following table. Geometric dimensions are assumed as deterministic.
The underpressure is assumed linearly distributed along the caisson base if bearing capacity is not exceeded; when bearing capacity at the heel is exceeded, caisson rotates a little; the limit state assumed corresponds to a significant rotation bringing the whole foundation from the turning point to the heel to bearing capacity and letting p_u act from the offshore face to the turning point. A sketch of the pressure and stress distribution on the foundation is given in figure 12.

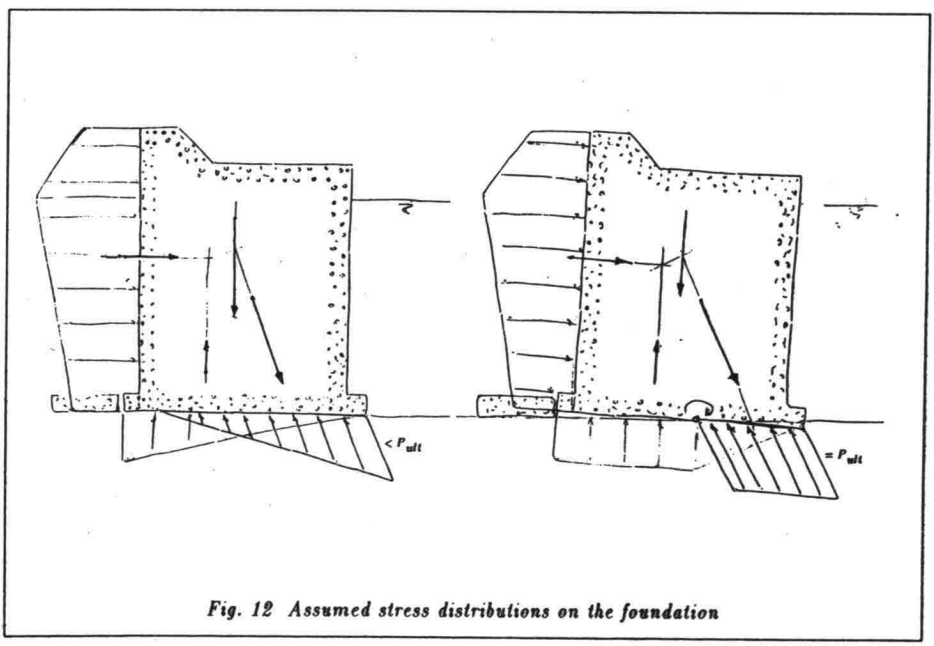


Fig. 12 Assumed stress distributions on the foundation

Once the random load and structure parameters are determined simulating the corresponding frequency distribution, the structure stability is evaluated according to the verification procedure sketched above.

Variable	Distribution	Parameters	
H_1	normal	mean = 0.79 m,	s.d. = $0.79/\sqrt{63}$ m
s_m	lognormal	median = 0.038,	c.v. = 0.05
z_w	lognormal	median = 0.50 m,	c.v. = 0.20
Z	normal	mean = 1.0,	s.d. = 0.05
ρ_c	lognormal	median = 2.4 t/m^3,	c.v. = 0.01
ρ_f	lognormal	median = 2.1 t/m^3,	c.v. = 0.03
f	lognormal	median = 0.63,	c.v. = 0.05
ϕ	lognormal	median = 37°,	c.v. = 0.05
c	lognormal	median = 20 kN/m^2,	c.v. = 0.20
γ	lognormal	median = 1.18 t/mc,	c.v. = 0.04

The Monte Carlo simulation gives the following results

Sliding	probability	5.0 %
Overturning	probability	13.8 %
Structure failure	probability	13.9 %

The number of storms in the lifetime is 99 ± 16; the maximum significant wave height is 6.59 ± 1.15 m. I remind that overturning correspond here to a foundation failure.
These figures give a different image of the risk inherent to the structure. The main causes are:
- considering the local foundation failure near the keel,
- extreme wave height result higher due to accounting for the random character of the (observed and) forecasted number of storms and of the estimated parameters of wave height distribution, and due to a different probabilistic model,
- the random variation of the other variables: wave period, densities etc..

As for the first cause we should remark that different foundation bearing capacity criteria result in highly different risk values. The original Terzaghi approach neglecting friction between the structure and the foundation gives failure probability 5.0, 27.3, 27.4 %, doubling the structure failure probability estimate.

Wave height uncertainty is the other real cause of risk: had we fixed to their central estimates all the variables but wave height, the structure failure probability would have decreased only to 12.9 %. The critical significant wave height causing the failure in this case is 7.87 m.
In our case the mean number of storms in the lifetime ($63 \cdot 50/32 = 98.4$) is high enough to make the random character of the number of storms in the lifetime not really important.

The often quoted "Fuller" relation which for storm significant wave height may be written

$$H_s^T = H_0^* + H_1 \ln \lambda T \qquad \text{with } \lambda \text{ treated as a deterministic variable} \qquad (13.1)$$

could be in its simplicity misleading.
The real point for the correct interpretation of the analysis is to realize that, in the case of a parent exponential distribution and as described in WG12 Report of subgroup B appendix A, eq. 13.1 may be interpreted as the shortcut version of different relations, in most of which p is treated as a constant parameter and hidden. One is the relation giving the design wave height for a given risk index p (which is only approximately equal to the non exceeding probability)

$$H_{s,p}^T = H_0 - H_1 \ln\left(1 - p^{1/\lambda T}\right) \simeq H_0 + \hat{H}_1 \ln\left(\frac{\lambda T}{-\ln p}\right) \qquad (13.2)$$

where the second approximate equality derives from the approximation of the function $\ln(..)$ valid

for $\lambda T \gg 1$ and from the substitution of the unknown true value of the parameter H_1 with its estimate giving actually the estimate of the design wave height $\hat{H}^T_{s,p}$. The other is the relation giving the central value of this estimate

$$E[\hat{H}^T_{s,p}] \simeq H_0 + H_1 ln\left(\frac{\lambda T}{-ln\, p}\right) \tag{13.3}$$

The most frequent use of eq. 13.1 is for estimating the parameter H_1. During the estimation procedure the curve is usually fitted passing through the points, i.e. taking $p \simeq 1/2$ (0.57 is the exact p value corresponding to the mean). In this case there is a difference between H_0, the threshold value used for data censoring, and H_0^*, the first parameter of eq. 13.1, or H_0^p for a generic p value, as

$$H_0^p = H_0 - H_1 ln(-ln\, p) \tag{13.4}$$

Neglecting the random character of $\hat{H}_1$, we might desire to know the distribution of the highest significant wave height in the life of the structure. Under the mentioned hypothesis $H^T_{s,p}$ is distributed as a Gumbel variable with scale parameter equal to H_1 and location parameter given by $H_0 + H_1 ln\lambda T$. In our case the two parameters may be estimated as 0.79 and 6.12 m.
From the well known relations between scale and location parameters and mean and standard deviation for a Gumbel variable, we would have expected that the highest wave in the lifetime as simulated had the following statistics 6.58 ± 1.01 m.

The simulation gave 6.59 ± 1.15 m. The highest significant wave distribution is actually more dispersed than the Gumbel distribution (13.3) as a consequence of the random character of the estimated scale parameter $\hat{H}_1$ and of the number of storms.
The actual distribution of the highest significant wave estimate results from the randomization of the basic Gumbel distribution expressed by eq 13.2 accounting for the distribution of $\hat{H}_1$

$$\hat{H}_1 \sim Normal(H_1,\, H_1/\sqrt{N_e})$$

The standard deviation of the estimated design wave height due to the imperfect knowledge of H_1 is

$$\sigma_{T,p} \simeq \frac{H_1}{\sqrt{N_e}} ln\left(\frac{\lambda T}{-ln\, p}\right) \tag{13.5}$$

In our case equations 13.3 and 13.5 for $p = 0.5$ and 0.9 give $\hat{H}^L_{s,p} \simeq 6.41 \pm 0.43$ and 7.90 ± 0.68 m. The same computation repeated for $p = (1 - 0.129)$ gives result 7.69 ± 0.65 which should be compared to the critical wave height obtained by simulation having the same frequency: 7.87 m.

The differences between the extreme wave statistics used here and in the paragraphs before are essentially due to the different distribution assumed. In the case before the distribution assumed was Type I extremal (Gumbel) for each sector, combined with composition of probability to determine the marginal wave height distribution. In this case wave height of the selected events is assumed to be exponential. The underlying probabilistic model, when we extrapolate from the 32 year record the almost 500 year return period design event, cause an effect which is not negligible (7.35 m compared to 7.90).

5. Other cases

In almost all the cases examined before the failure dynamics is represented by reliable equation (or response functions) associated with an uncertain load: the maximum significant wave height to be encountered in the lifetime, when this time is long relative to wave recording period or the number of independent events which the extreme wave statistics is based on is low. In these cases the random character of the load is the most relevant feature causing a risk of failure, the associated partial security coefficient is the most important, and a great attention should be

paid during the design phase to define accurately the extreme wave climate.

A similar situation occurs considering the risk pertaining to flood defense systems, where the principal environmental parameters are tide (including storm surge) elevation and incident wave height, quite often correlated, and the most relevant failure mechanism is usually overtopping, see for example CUR-TAW (1989-b) or Ronold (1990).

This situation is not general. When dealing with beaches or, more generally, with sediment transport, the dynamics is often more uncertain than the wave climate. Often the reliability of a transport formula is measured by the frequency of cases where the computed and measured values are within a factor 2. This factor is actually the order of magnitude of a single method estimate; more accurate values may be obtained from the comparison of different independent estimates, from redundancy of independent estimates and balance relations, etc. . Anyway sediment transport estimates with a likely error below 30% are not frequent.

On the other side works concerning sediment transport are often maintenance works, having a short lifetime and a low economic repercussion in the sense of table 2: in case of failure the maintenance work should be just anticipated before the scheduled time.
Wave observations are available over periods often longer than the maintenance interval. Nowadays also directional wave recording covering some years are becoming available.

This situation reverse the source of risk but does not affect the opportunity to carry out a reliability assessment. Actually the number of failures of this kind of works is certainly higher than for breakwaters and the total cost of failures is probably of the same order of magnitude.

I will just sketch some possible failure modes for a protective beach and give some references.

4.1. Failure modes for a protective beach

The usual aim of a protective beach artificially maintained by (re)nourishment is to preserve in a naturally eroding situation a beach wide enough to protect the hinterland from direct wave action or to be used for the recreation of a certain number of peoples. The artificially maintained beach or nourishment scheme may fail:

1 – functionally:
   a) – if, as a consequence of some temporary combination of beach profile (or volume), water level and wave conditions, wave run-up reaches the line dividing the beach from the backshore to be protected;
   b) – if beach surface is reduced below what is needed for recreation;
2 – economically: if the combination of nourishment frequency and nourishment volume produce costs higher than the benefit deriving from the defense action or anyway unendurable for the client.

The functional failure modes are characterized by the position of a line, the wave run up line in the first case and the normal high water in the second. The mentioned position shows short-term variation and long-term trend. If the beach in the time interval between nourishments may safely retreat much more than the characteristic fluctuations the first failure mode may be analyzed as suggested in CUR et al. (1987) and CUR, TAW (1989-b):
– the actual beach profile is transformed into an erosion profile with the aid of a beach erosion model and for a properly chosen combination of storm surge level and wave height (see figure 13);
– a characteristic point in the erosion profile is identified according to the failure mode examined: the toe of dune erosion or position reached by waves for mode a) ore the shore line position for mode b);
– the time history of the position of the characteristic point is transformed into trend and fluctuations;
– the statistics of fluctuations gives the safety margin instantaneously needed;
– the erosive trend caused by the deficient sediment balance defines when the nourishment work

is necessary.

The economical failure depends essentially on the beach long term sediment balance, which should be accurately analyzed and compared with the past evolution of the beach.

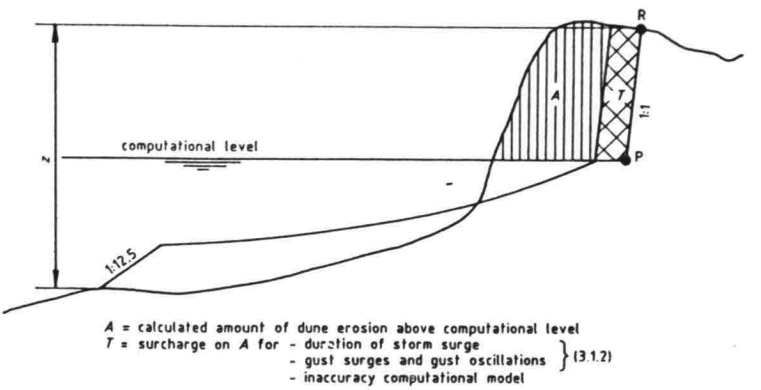


Fig. 13 Definition sketch (from CUR-TAW 1989-a)

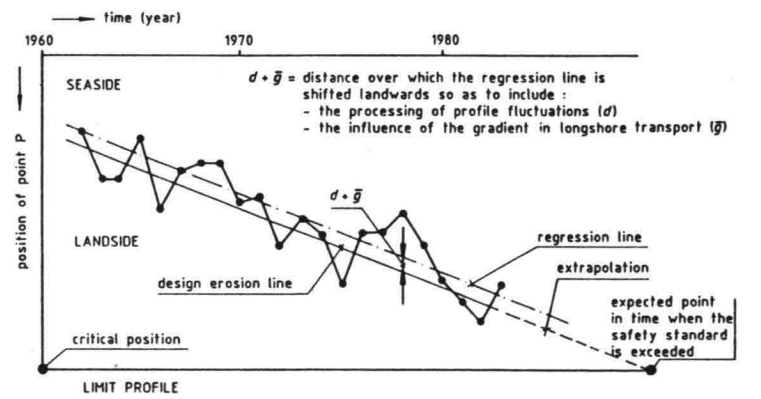


Fig. 14 Principle of the test method for the safety assessment (from CUR-TAW 1989-a)

Longshore transport divergence, offshore transport, subsidence, fluvial and wind transport, biological production should be accurately evaluated as well as the characteristics of sediments used for the nourishment.

5. Conclusions

We have examined some application of level 1 and level 3 risk assessment.

A complete and objective risk assessment is not always possible due to the lack of reliable models for some of the relevant failure modes.

A model is suitable for this kind of use, if its reliability has been tested and quantified (the statistics of the model related variable Z should be known).
The statistics of all the relevant parameters should be known at least in a simplified form (a central value and one or two fractiles characterizing the distribution on the cautious side).

A good pile of scientific and technical work must still be done in order to translate the actual knowledge of the basic phenomena into a rationally based level 1 risk assessment procedure, suitable for the everyday design praxis.

Nevertheless even a not completely objective risk assessment can be useful to answer in a conclusive manner such questions as:

- How deep the preliminary environment characterization should be brought? Does risk come only from the climate vagary or also from our imperfect knowledge of the past and of the present? Does it come from our poor knowledge of some relevant phenomenon?
- What are the relevant failure modes? Could we substantially reduce the risk by controlling carefully the construction?
- Is a physical model or any other more reliable and expensive evaluation tool useful or necessary? Should an overdesign be preferred to a strictly controlled construction or a more deep analysis?

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List of symbols

D_{n50} : median nominal diameter
H : generic wave height or height of regular waves
H_{max} : maximum wave height near the breakwater
H_s^T : highest significant wave in a time period T years long,
$H_{s,p}^T$: significant design wave height referring to lifetime T and risk p
$H_{1/3}$: significant wave height near the breakwater, Goda notation
K_D : damage index in Hudson formula
L : structure lifetime
M : armor unit mass
M_{50} : median armor unit mass
N_D : damage index in Van der Meer formula
N_w : number of waves in the wave attack
N_e : number of elements in the climate statistics sample
N_L : number of events in the structure lifetime
P_f : structure failure probability
p_u : wave pressure under the caisson at the sea side
p_1 : wave pressure at mean water level
p_3 : wave pressure at the caisson sea side toe
S_m : mean significant wave steepness : $2\pi H_s/gT_m^2$
T_m : mean wave period: m_0/m_1
T_s : significant wave period

T_r : return period
Z : response function correction variable

α : mound slope
Δ : relative immersed density of armor units: $(\rho_a - \rho_w)/\rho_w$
γ_H : partial safety coefficient for wave height
γ_Z : partial safety coefficient for
ρ_a : armor units density
ρ_w : water density