Seminar on Rubble Mound Breakwaters

December 4, 1985

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ON THE RELIABILITY OF RUBBLE MOUND BREAKWATER DESIGN PARAMETERS

1. INTRODUCTION

The recent failures of major rubble mound breakwaters has taken many professional engineers by surprise. Is it really possible that we do not have a reliable design method — after hundred years of breakwater design and construction and also intensive research for the last 20 years? The answer is yes. The state of the art and the design tools are not satisfactory compared to those available in other branches of civil engineering such as for example structural engineering.

I shall try to explain the difficulties we are facing in breakwater engineering, especially for rubble mound breakwaters, by summarizing some of the uncertainties we have to deal with in the design process. A good overview of the uncertainties and the related consequences is of paramount importance to the designer. Without such knowledge it is impossible to evaluate the safety of a structure — a situation that is unacceptable for a professional engineer.

It is important to point out that the damage to a breakwater never depends on one single parameter such as for example the wave height. Moreover, the time history (duration) of the impact is of importance. This means that a discussion of uncertainties in breakwater design really should be based on the joint probability density functions of the involved parameters supplied with statistical information on the related persistance.

The following presentation is not in accordance with this since each parameter is treated separately. This is done for the sake of simplicity and also because it will still serve the main object of the presentation.

2. BASIC NEEDS IN DESIGN

For most civil engineering structures (buildings, bridges etc.) it is possible to design and check the structural performance by means of theory. This is because many years of research and experience have established the prerequisites which are

- *Information on size of all major types of loads*, often stated in standards as characteristic maximum and minimum values, which again are based on information of the statistical properties such as mean, standard deviation and frequency distribution.
- *Information on the structural response to the loads*, implemented in formulae which are in most cases the outcome of theories based on basic physics, but are in some cases more or less empirical.

Both loads and the response to those loads are known quantitatively to such an extent that meaningful safety factors can be specified in the various standards.

Although this is well known to all professional civil engineers, it is deliberately mentioned here as a reference for the following discussion on rubble mound breakwaters, for which the situation is completely different.

3. ENVIRONMENTAL LOADS

3.1 Waves

The ideal situation, depicted in Figure 1, where both short term and long term wave statistics can be established from on-the-site records almost never exists.

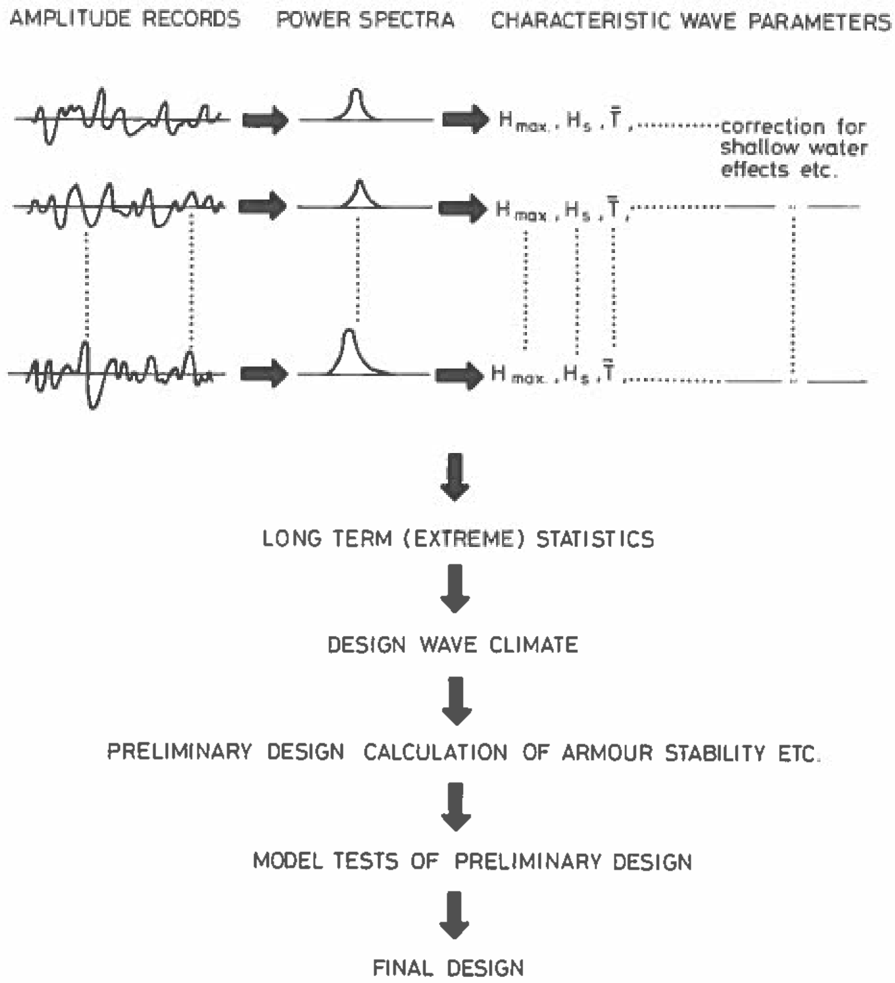


Figure 1. Ideal procedure for the establishment of design wave climate.

Usually one has to establish design wave conditions by hindcasting from meteorological observations and/or some wave records covering relatively short periods. In some areas visual wave observations from ships are available too.

It is clear from this that it is not possible to get reliable statistical values for all the wave parameters of importance. These are wave heights H, periods T, spectral shape, groupiness, direction of propagation and duration of storms.

Let us examine the wave height. This is generally the most important parameter since cover layer stability in terms of block weight is more or less proportional to the wave height in the third power. The uncertainties in the determination of extreme wave heights may result from the following sources:

A. Errors in measurements, visual observations or hindcasting of the wave data on which the extreme statistics are based.

B. Errors related to extrapolation from short samples to events of high return periods, i.e. low probability of occurrence.

C. Lack of knowledge about the underlying distribution for the extreme events.

D. Errors due to plotting positions.

E. Climatological variations.

ad A. Errors in wave data

Le Mehaute et al., 1984 discussed the uncertainties and systematic errors or bias related to the wave data under the assumption of errors being normally distributed. They reported the following "typical" normalized standard deviation σ' defined as the absolute standard variation divided by the expectation ("mean") value of H_s:

Direct wave measurement	$\sigma'_M = 0.05$	bias 0.00
Visual observations from ships	$\sigma'_M = 0.20$	bias 0.05
Hindcast (excluding hurricanes and other tropical storms)	$\sigma'_M = 0.15$	bias 0.05

It should be noted that the two last set of figures are applicable only when the sample populations are ranked statistically. A direct comparison in the time domain, i.e. comparison of individual sea states, generally shows larger discrepancies. Moreover the figures are average figures. For instance it is believed that wave data based on to-day's most advanced hindcast models applied to relatively restricted areas, such as the North Sea, where high quality weather maps are available, will show a smaller uncertainty.

ad B. Errors due to short samples.

Estimates on events of low probabilities are often performed in the following two different ways:

1) The extrapolation of data from frequent measurements or observations. The data are often compiled at intervals $\Delta t = 3$, 6 or 24 hours, which gives a large sample, N events, even in the case of a short time of observation or record length Y in years. The order of magnitude of N is often 1000 - 10,000.

2) The extrapolation of relatively few data sets representing the max significant wave height H_s for a number of storms exceeding a certain level, H'_s. The data are often determined from hindcasts and the sample size N is typically within the range 10 - 30.

Wang et al., 1983, examined the uncertainties related to the first method. They considered the long term distribution of H_s to be of the exponential type which also includes the often used Weibull distribution,

$$P(H_s) \equiv P[H \leqslant H_s] = 1 - \exp\left(-\left(\frac{H_s - A}{B}\right)^{\gamma}\right) \tag{1}$$

where A is signifying the background noise level or lower-bound, B is the scale parameter and γ is the shape parameter. All three characteristic variables are normally determined by best fitting to the observed data.

Assuming the data asymptotically normally distributed about the underlying probability distribution function, eq (1), the authors obtained for large N the normalized standard deviation,

$$\sigma_s' = \frac{1}{\gamma \ln(R\,\nu)} \left(\frac{R}{Y}\right)^{0.5} \tag{2}$$

where R is the return period in years, ν is the number of observations per year compiled at interval Δt and Y is the number of years of observations. Formula (2) is valid only for low probability levels and only for large samples $N = \nu Y$ of uncorrelated data. The latter implies that Δt should exceed approximately 24 hours, but because of little sensitivity on the confidence bands for H_s smaller values, as for example $\Delta t = 6$ hours, are often used.

Example.

Taking R = 50 years, Y = 5 years, ν = 365 observations per year and $\gamma = 1.2$ gives $\sigma_s' = 0.27$

Changing R and Y to 100 years and 3 years respectively gives $\sigma_s' = 0.46$

The second method mentioned above is relevant to situations where data have to be obtained from hindcasting, which, due to the costs involved, restricts the number of data.

Rosbjerg, 1981, considered this case, where only maximum values η of H_s for independent storms exceeding a chosen level H_s' are taken into consideration, cf. figure 2.

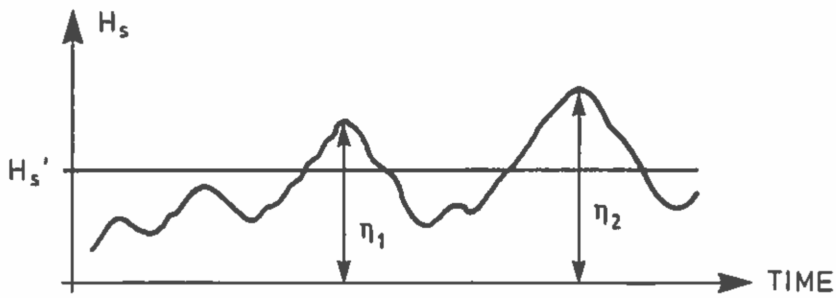


Figure 2. Data reduction by application of exceedence level, H_s'.

Rosbjerg assumed all the exceedences $\eta - H_s'$ to follow the exponential probability distribution,

$$P(H_s) \equiv P[\eta \leqslant H_s] = 1 - \exp\left(-\frac{H_s - H_s'}{\alpha}\right) \tag{3}$$

which is of the same type as the Weibull distribution, eq (1), with $\gamma = 1$.

The author also assumed the events η to occur at times corresponding to a Poisson-process with time dependent intensity. He arrived at the following expression for the R-year event defined as the value of η, which in average is exceeded once every R years,

$$H_s = H_s' + \alpha \ln \nu R \tag{4}$$

The corresponding absolute standard variation is

$$\sigma_s = \frac{\alpha}{(\nu\, Y)^{0.5}} \, (1 + (\ln \nu\, R)^2\,)^{0.5} \tag{5}$$

and the normalized standard deviation consequently

$$\sigma_s' = \frac{\sigma_s}{H_s} = \frac{\frac{\alpha}{(\nu\, Y)^{0.5}} \, (1 + (\ln \nu\, R)^2\,)^{0.5}}{H_s' + \alpha \ln \nu\, R} \tag{6}$$

The maximum likelihood estimate for α is

$$\hat{\alpha} = \bar{\eta} - H_s' \tag{7}$$

where $\bar{\eta}$ means average of η.

Nielsen et al., 1985, extended the analyses to include the Weibull distribution

$$P(H_s) = P[\eta \leqslant H_s] = 1 - \exp(-\,(\frac{H_s - H_s'}{\alpha})^{\gamma}) \tag{8}$$

and found the following

$$H_s = H_s' + \alpha\,(\ln \nu\, R)^{1/\gamma} \tag{9}$$

$$\sigma_s = (\ln \nu\, R)^{1/\gamma - 1} \left[\frac{\alpha^2}{\gamma^2\, \nu\, Y} + (\ln \nu\, R)^2 \, \frac{\alpha^2}{\nu\, Y} \, \left(\frac{\Gamma(1 + \frac{2}{\gamma})}{\Gamma^2(1 + \frac{1}{\gamma})} - 1\right) + \frac{\alpha^2}{\gamma^4} \, (\ln \nu\, R) \cdot \ln(\ln \nu\, R))^2 \, \mathrm{Var}[\hat{\gamma}] \right]^{0.5} \tag{10}$$

ν is the average number of data per year and Γ the Gamma function.

The variance of $\hat{\gamma}$, $\mathrm{Var}[\hat{\gamma}]$, cannot easily be estimated, but by means of numerical simulation it is found that the term in (10) containing this quantity is highly dependent on the method for estimating the parameters in the Weibull distribution.

Petrauskas and Aagaard, 1971, found, by using a least square method, that the last term in (10) is insignificant. In this case the normalized standard deviation is

$$\sigma_s' = \frac{\sigma_s}{H_s} \cong \frac{(\ln \nu\, R)^{\frac{1}{\gamma} - 1} \left[\frac{\alpha^2}{\gamma^2\, \nu\, Y} + (\ln \nu\, R)^2 \, \frac{\alpha^2}{\nu\, Y} \, \left(\frac{\Gamma(1 + \frac{2}{\gamma})}{\Gamma^2(1 + \frac{1}{\gamma})} - 1\right) \right]}{H_s' + \alpha\,(\ln \nu\, R)^{1/\gamma}} \tag{11}$$

Nielsen et al., 1985, fitted the Weibull parameters by the method of moments, i.e. equating the first three moments of the distribution to those of the data, and found that the last term in (10) was of significance, namely in the order of 1/3 of the total standard deviation. The estimates on the parameter by the applied method of moments are given by

$$\frac{\Gamma(1 + \frac{3}{\hat{\gamma}}) - 3\Gamma(1 + \frac{2}{\hat{\gamma}})\,\Gamma\,(1 + \frac{1}{\hat{\gamma}}) + 2\Gamma^3(1 + \frac{1}{\hat{\gamma}})}{(\Gamma(1 + \frac{2}{\hat{\gamma}}) - \Gamma^2(1 + \frac{1}{\hat{\gamma}}))^{3/2}} = \frac{\overline{\eta^3} - 3\,\overline{\eta^2}\,\bar{\eta} + 2(\bar{\eta})^3}{(\overline{\eta^2} - (\bar{\eta})^2\,)^{3/2}} \tag{12}$$

$$\hat{\alpha}^2 = \frac{\overline{\eta^2} - (\bar{\eta})^2}{\Gamma(1 + \frac{2}{\hat{\gamma}}) - \Gamma^2(1 + \frac{1}{\hat{\gamma}})} \tag{13}$$

$$\hat{H}'_s = \bar{\eta} - \hat{\alpha}\,\Gamma\,(1 - \frac{1}{\hat{\gamma}}) \tag{14}$$

$\overline{\eta^2}$ and $\overline{\eta^3}$ mean the average of sample values of η^2 and η^3, respectively, which are unbiased estimates of $E[\eta^2]$ and $E[\eta^3]$.

It should be noticed that the R-year event given by eqs (4) and (9) has a probability E of being equalled or exceeded in the specific lifetime L of the structure. For instance, if L is set equal to the return period R, this "encounter probability" E is as large as 63%. The relationship between R, L and E is given by

$$E = 1 - (1 - \frac{1}{R})^L \quad \text{or in case of R large} \quad R = -\frac{L}{\ln(1 - E)} \tag{15}$$

For design purpose R in eqs (4) and (9) should be evaluated with respect to E and L through eq (15). For example in a 50 years lifetime there is a 10% probability that the structure is hit by the 500 years' return period storm.

Eqs (6) and (11) make it possible to determine the necessary sample length when a prediction for a given return period with a prescribed accuracy and confidence is required. Following the normal distribution the products of σ'_s with 0.84, 1.28, 1.65 and 2.32 define the upper bound of spread corresponding to a confidence level of 80%, 90%, 95%, and 99%, respectively. For instance, the prediction of an event with 90% confidence and an uncertainty of no more than 0.20 imply that $1.28\ \sigma'_s \leqslant 0.20$. Inserting this in eqs (6) or (11) gives the corresponding number of years of observation Y for given ν and R.

Example.
The accuracy of estimates based on a restricted number of hindcasted data sets might be illustrated by the following example. The Delft Hydraulics Laboratory did a hindcast study for a specific deep water location in the Mediterranean Sea and found for a 20 years period the following 17 most severe storms, Table 1:

Table 1. Example of hindcasted storm wave data for a 20 years' period.

Rank i	Max H_s (= η) meters	Peak period T_p seconds	Average wave direction degrees
1	9.32	14.0	143
2	8.11	14.1	139
3	7.19	13.4	123
4	7.06	10.8	123
5	6.37	11.9	143
6	6.15	11.1	185
7	6.03	12.3	135
8	5.72	10.5	176
9	4.92	10.7	150
10	4.90	10.6	129
11	4.78	11.8	161
12	4.67	9.9	120
13	4.64	9.2	122
14	4.19	10.5	137
15	3.06	11.1	154
16	2.73	8.2	153
17	2.33	8.3	126

If we choose $H_s' = 4.0$ m we find $N = 14$ storms exceeding this level over a period $Y = 20$ years, which gives $\nu = 14/20$. According to eq (7) α can be estimated to $\hat{\alpha} = 2.00$ m. It can now be tested if the data follow the assessed distribution, for example the exponential type given by eq (3). In this case a straight line with slope 1:1 should be obtained by plotting $\eta_i - H_s'$ against $-\hat{\alpha}\ln(1 - P(\hat{\eta}_i))$, where $P(\hat{\eta}_i) = 1 - \frac{i}{N+1}$, (Gumbel plotting positions). Figure 3 shows that the fit is reasonable.

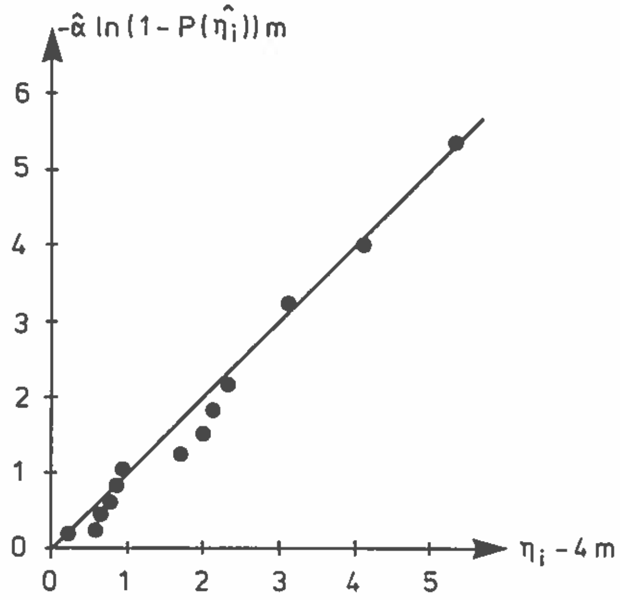


Figure 3. Test on exponential distribution of wave height exceedences.

Formulae (4) - (6) are then valid and the expectation values and the standard deviations can be calculated for various return periods, for instance

Return period R years	H_s meters	σ_s meters	σ_s'
50	11.11	1.97	0.18
100	12.50	2.33	0.19

Note that a change in H_s' for example to 3.50 m, which still gives $N = 14$, will change H_s and σ_s! This important problem is not discussed further here.

It is obvious that the 14 data points also fit a Weibull distribution.

If all the 17 data points given in Table 1 are considered, it corresponds to a exceedence level of $H_s' \cong 2.25$ m because the lowest value in the data set is $H_s = 2.33$ m. It turns out that in this case the data do not fit neither the exponential distribution, eq (13), nor the Weibull distribution, eq (8). However, if the exceedence level is not interpreted as the physically true cut-off level, but is regarded a fitting coefficient only, like α and γ, then the 17 data points follow the Weibull distribution very closely, as demonstrated in Figure 4. The coefficients are in this case H'_s = 0.73 m, α = 5.27 m and γ = 2.80, all estimated by the method of moments.

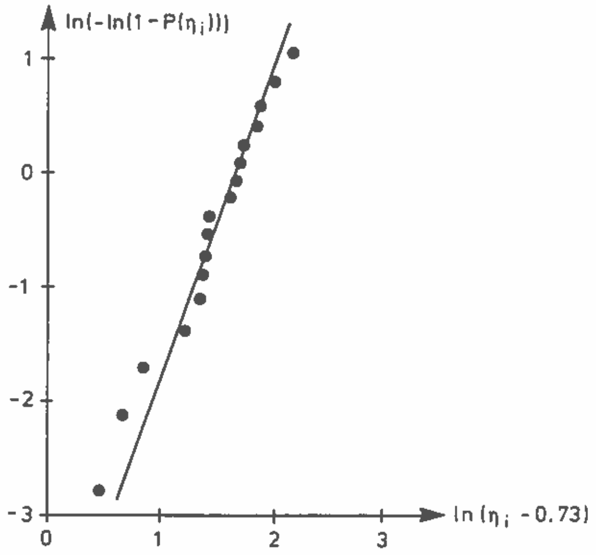


Figure 4. Data fit to the Weibull distribution. Gumbel plotting positions.

From eqs (9) - (11) we obtain the following corresponding values

Return period R years	H_s meters	σ_s meters	σ'_s
50	9.19	0.88	0.10
100	9.71	0.97	0.10

The Weibull distribution shown by the straight line in Figure 4 is a result of the chosen method of fitting. A least square fit or a visual fit will produce different lines and different estimates on the extreme events.

ad C. and D. Errors due to the lack of knowledge on the true long term distribution and due to plotting positions.

Several probability distributions are used to describe extreme wave height statistics. These include for example the log-normal distribution, the extremal type I or Gumbel or Fisher-Tippett I distribution, the extremal type II or Fretchet or Fisher-Tippett II distribution, the Ward-Borgman distribution and the extremal type III or Weibull distribution. Although each of these distributions has a theoretical base, they cannot be evaluated and related to the extreme waves on a physical base. As a consequence they are only fit to the available data. Most often the scales used for the plotting are such that the chosen distribution lies on a straight line, simply because of the more convenient visualization of the extrapolation. However, when extrapolating, one should always be aware of possible physical processes, such as for example wave breaking, which might interrupt the probability distribution at some probability level.

It follows from these comments that due to unknown extreme distribution errors can only be estimated by a sensitivity analysis in which various distributions are fitted. Table 2 shows such an analysis by the Delft Hydraulics Laboratory performed on the wave data given in Table 1.

Table 2. Example of influence of choise of extremal distribution and plotting position on low - probability wave heights. Data by Delft Hydraulics Laboratory.

Extremal distribution	Plotting position	Correlation coefficient	Return period H_s 50 year	100 year
Type I Gumbel	Gumbel	0.9875	11.0 m	12.2 m
	Gringorten	0.9852	10.3 m	11.3 m
Ward/Borgman	Gumbel	0.9872	9.8 m	10.5 m
	Gringorten	0.9920	9.4 m	10.1 m
Type III Weibull	Gumbel	0.9877	9.6 m	10.2 m
	Gringorten	0.9877	9.3 m	9.9 m

Although no accurate figures can be given it seems reasonable from this table and the above given example based on the distribution, eq (3), that due to unknown extreme distribution a normalized standard deviation σ'_D might be in the order of

$$\sigma'_D \cong 0.05 - 0.10.$$

In order to plot the data a position formula must be adopted. Many different plotting positions, all based on some statistical considerations, exist, but it is not easy or possible to select a specific one as the most correct. For this reason it is reasonable to estimate the error due to plotting positions by sensitivity analyses involving a number of reasonable plotting rules.

Table 2 gives an example where only two plotting rules are used, namely

Gumbel/Weibull
$$P(\eta_i) = 1 - \frac{i}{N+1} \tag{16}$$

and

Gringorten
$$P(\eta_i) = 1 - \frac{i - 0.44}{N + 0.12} \tag{17}$$

It is seen that significant deviations in the estimated extreme wave height occur due to the plotting rules. It is believed that a realistic normalized standard deviation σ'_p on extreme events will be in the order of

$$\sigma'_p \cong 0.05$$

ad E. Errors due to climatological variations.

An additional source of uncertainty is the natural variation of the wave climate. Le Mehaute et al., 1984, considered this difficult problem under the assumption of the natural climatology being ergotic and stationary and governed by the statistical law of Weibull distribution. By setting Y = R in eq (2) they found that the normalized standard deviation of climatological variations in R years at a particular location is given by

$$\sigma'_C = \frac{1}{\gamma \ln(R\nu)} \tag{18}$$

If we for instance estimate $\gamma \cong 1.2$ as proposed by the authors we find for $\nu = 365$ and R = 50 or 100 years $\sigma'_C \cong 0.08$.

Combined errors.

The above mentioned sources of uncertainty can be assumed mutual independent except for an unknown but probably weak correlation between the climatological variation and the data samples.

The total normalized standard deviation might then be estimated by

$$\sigma' \cong \left(\sigma_M'^2 + \sigma_s'^2 + \sigma_D'^2 + \sigma_p'^2 + \sigma_C'^2\right)^{0.5} \tag{19}$$

With reference to the foregoing discussion one can establish the following two examples:

Examples.

Direct wave height measurement. ν = 365 observations per year. Y = 5 years. R = 50 years.

$$\sigma' \cong (0.05^2 + 0.27^2 + 0.07^2 + 0.05^2 + 0.08^2)^{0.5} = 0.30$$

Hindcasted wave heights. 14 data sets over Y = 20 years. R = 50 years.

$$\sigma' \cong (0.15^2 + 0.18^2 + 0.07^2 + 0.05^2 + 0.08^2)^{0.5} = 0.26$$

From this it is seen that, even with what is generally regarded reasonable lengths of data sample and observation period, the uncertainty related to the 50 year event is significant and in the order of $\sigma' \cong 0.25 - 0.30$. If we assume normally distributed random variables it means a 16% probability of the wave height being bigger than 1.25 - 1.30 times the estimated height.

The uncertainty increases significantly when the lengths of data sample and the period of observation are reduced to figures below those given above.

The difficulties in obtaining reliable estimates on design wave heights might also be illustrated by the following example from the Norwegian Ekofisk North Sea offshore field given by Professor Tørum of Norway.

In 1970 the 100 year design wave height was estimated to be 19.6 m. In 1972 it was 23.6 m; in 1977 28.0 m; in 1981 up to 34 m. And finally in 1984 the estimate was 28 m with an uncertainty of approximately ± 15%! This big uncertainty exists despite the large resources spent on wave recordings, wave statistics etc. in this prospective offshore area. These resources are much larger than those available for the design of breakwaters.

It is not only the wave height that is of importance but also

- the wave period
- the spectral shape
- the horizontal, directional spread of the wave energy / short crestedness of the waves
- the groupiness
- the direction of the propagation
- the duration / time history of the storms

Therefore the uncertainty related to the estimation of these parameters should also be evaluated. It takes a lot of work and research to perform such an analysis, also because generally it is the reliability of the "joint parameters" which are of interest. This problem is not discussed further here. However, it is obvious that it all adds to the uncertainty on design wave climate estimations.

If the breakwater is in "shallow-water" and the wave data are from an offshore location then we have to include the uncertainty related to shallow water effects such as:

- Refraction, i.e. change of wave direction and wave height due to oblique wave approach.
- Shoaling, i.e. change of wave height and wave length due to water depth variations perpendicular to the coast.
- Wave breaking, due to instability by decreasing water depth.
- Wave set-up, i.e. change of the mean water level due to changes of the wave radiation stress.

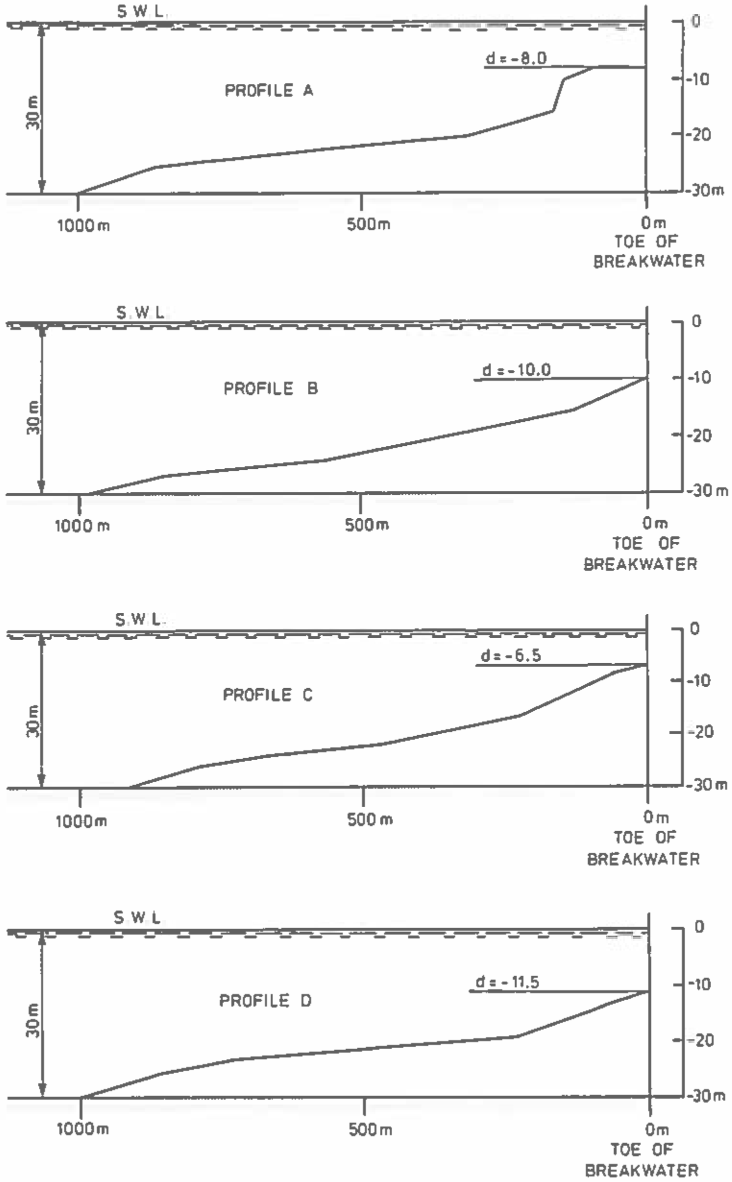
Besides these effects we also have:

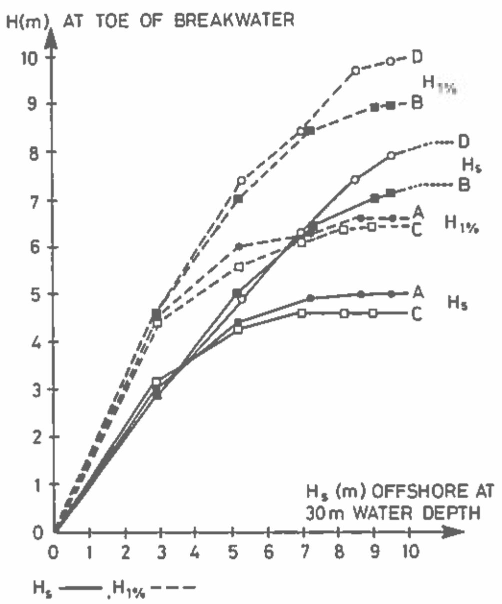
- Tidal water level variations.
- Wind set-up, i.e. wind induced change of the mean water level.
- Currents.

The uncertainties related to all these parameters or phenomena are in general not well established except for tidal water level variations. Consequently a quantitive discussion on uncertainties is not possible. However, in the next paragraph we shall evaluate the importance of reliable data by a sensitivity analysis of the structural response to some of the parameters.

It has often been pointed out that estimates on design waves are much more reliable in shallow water than in deep water due to the depth limited wave heights. This is true but it should be mentioned that no wave theory exists which can predict with good accuracy the absolute wave height distribution and maximum wave heights in shallow water with breaking waves. Moreover, it should not be overlooked that the water level is also a very important parameter when breakwaters are designed for a certain amount of overtopping.

Another point which should be stressed is the sensitivity of shoaling/wave breaking to variations in the sea bed profile. This is illustrated in Figure 5, where the wave heights of the incoming waves at the toe of a breakwater are determined for four different foreshore bottom profiles. The breaker index $\gamma_{H_s^{max}}$, defined as the ratio of the max significant wave height, H_s^{max} and the water depth, d at the toe, is also given in the figure together with the breaker index $\gamma_{H_{1\%}^{max}}$ related to the maximum value of wave heights, exceeded by 1% of the waves.





PROFILE	BREAKER INDEX $\gamma = \frac{H^{max}}{d}$ AT TOE OF BREAKWATER	
	$\gamma_{H_s^{max}}$	$\gamma_{H_{1\%}^{max}}$
A	0.63	0.83
B	0.73	0.90
C	0.71	0.98
D	0.70	0.87

Figure 5. Example of sensitivity of depth limited wave heights to differences in foreshore bottom profiles. Delft Hydraulics Laboratory.

The wave heights was determined by DHL in wind-wave flume model tests without the breakwater.

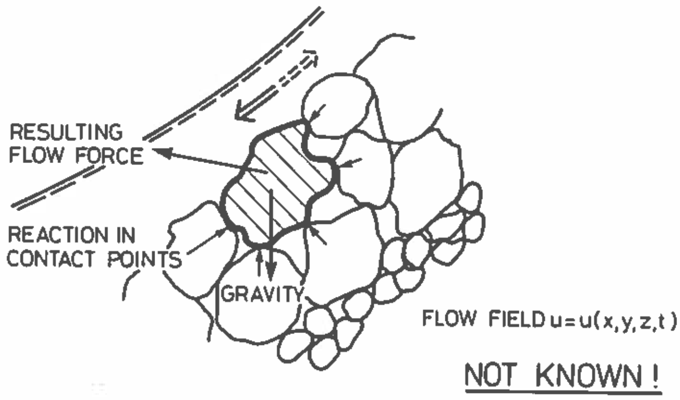
It is seen that a good estimate on the wave height in front of a breakwater in shallow water must be based on model tests with a correct reproduction of the foreshore topography. This means that in case of significantly varying bottom topography along the breakwater it is necessary either to test many sections or preferably to test the hole structure in a three-dimensional model.

4. SENSITIVITY IN STRUCTURAL RESPONSE TO THE ENVIRONMENTAL LOADS

The following is not intended to be a complete discussion as only a few, but important, problems will be discussed.

4.1 Hydraulic stability of the armour layer

The difficulties related to a purely theoretical stability analysis might be illustrated by considering the forces on an armour unit, see Figure 6.



GRAVITY: $F_G \approx g\rho_w\left(\frac{\rho_s}{\rho_w}-1\right)d^3$

FORM DRAG: $F_{D,F} \approx C_F\ \rho_w\ d^2|u|u$

SURFACE DRAG: $F_{D,S} \approx C_S\ \rho_w\ d^2|u|u$

LIFT: $F_L \approx C_L\ \rho_w\ d^2u^2$

INERTIA, FROUDE-KRYLOV: $F_I \approx C_I\ \rho_w\ d^3u'$ (pressure grad. undisturb. flow)

INERTIA, ADD. HYDRODYN. MASS: $F_H \approx C_M\ \rho_w\ d^3\ u'$ (change of flow field by the body)

COEFFICIENTS C are functions of Keulegan-Carpenter No. and Re No. and will vary considerably in time.

Figure 6. Forces on armour unit.

As a consequence stability formulae are semiempirical and formulated as an equality between a characteristic drag flow force and the stabilizing gravity force multiplied by unknown functions to take care of slope angle, friction, interlocking, wave period etc.

The various formulae show that the stability in terms of required mass, M of the armour unit is more or less proportional to the wave height in the third power. This very strong dependence put emphasis on the need for precise estimates on wave heights. This is depicted in Figure 7, where the relative variation of M in the range H ± σ (H) is shown. σ (H) is taken as 0.3 $\hat{H}$, cf. paragraph 3.

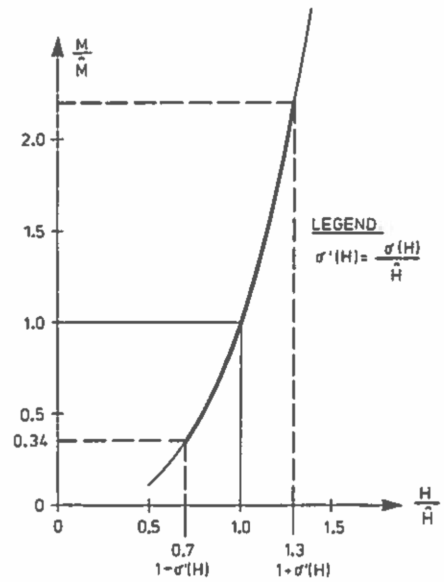


Figure 7.
Influence of wave height, H on required mass, M of armour unit.

The armour layer stability is also affected by the wave period T, but the variation with T is generally found to be much weaker than the variation with H. However, Gravesen et al. 1979 found a strong influence and proposed that the wave period is taken into consideration by using H_s^2 L_p in the stability formulae instead of H_s^3. L_p is here the wave length corresponding to the spectral peak period T_p. As L_p is more or less proportional to T_p^2 this implies a dependence of M on T_p as schematized in Figure 8. As a characteristic standard deviation is chosen $\sigma(T_p) = 0.15\ \hat{T}_p$.

Gravesen et al.'s findings are related to an armour layer of cubes with slope 1:2 but surmounted by a vertical wave wall, which affects the stability in the case of larger wave.

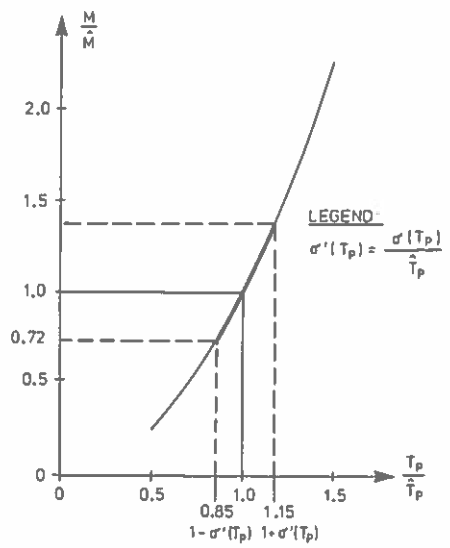


Figure 8.
Influence of peak period on required mass, M of armour unit as proposed by Gravesen et al., 1979.

A somewhat weaker but still significant dependence was found by Burcharth, 1979 in stability tests with Dolosse armour exposed to regular waves, Figure 9. The same reference also shows that run-up increases significantly with the wave period.

Figure 10 shows a replot of stability tests in regular waves with uniform stones, Dai & Kamel, 1969 and rip-rap, Ahrens, 1975. It is seen that the stability sensitivity to wave period is small in the case of uniform stones and large, but with opposite trends, for rip-rap.

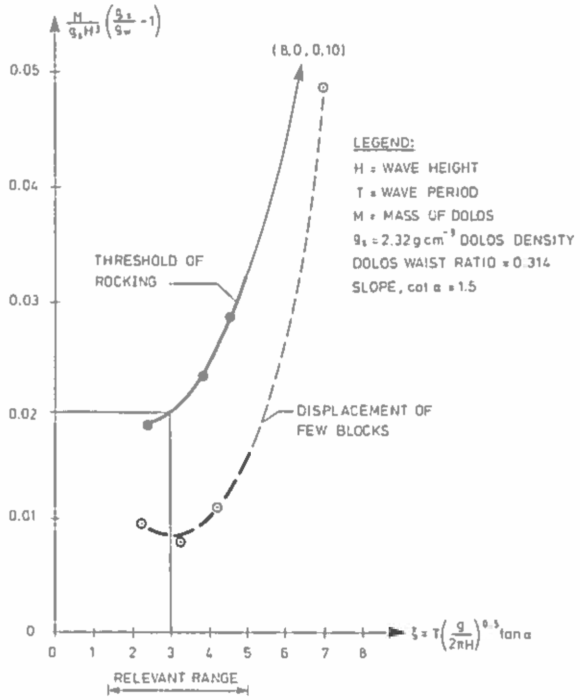


Figure 9.
Example of influence of wave periode on required mass of Dolosse armour units. Tests in regular waves. Burcharth, 1979.

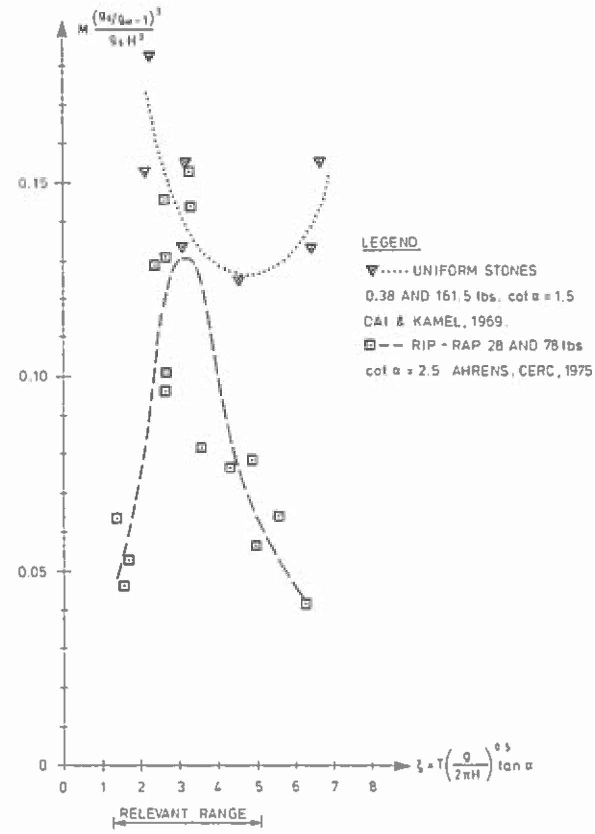


Figure 10.
Examples of influence of wave period on required mass of uniform armour stones and rip-rap. Replot by Dai & Kamel, 1969, and Ahrens, 1975.

In Figure 11 the data are normalized with respect to $\xi = 3$ for easy mutual comparison of the wave period sensitivity. $\xi = 3$ is a characteristic average value for rubble mound breakwater design wave situations. It is seen that an uncertainty on T (for example $\sigma'(T) = 0.15$) around the value $T_{\xi=3}$ only gives relatively small variations on the required mass M.

This is somewhat contradictory to Figure 8 but might be explained by the influence of the wave wall as explained above.

Figure 11 also shows that the larger the porosity of the armour layer the more vulnerable the armour is to large wave periods (Dolos armour has the largest porosity and rip-rap the smallest). This is due to the "reservoir effect" of the pores as explained in Burcharth et al., 1983. A stability minimum seems only present for the relative impermeable rip-rap.

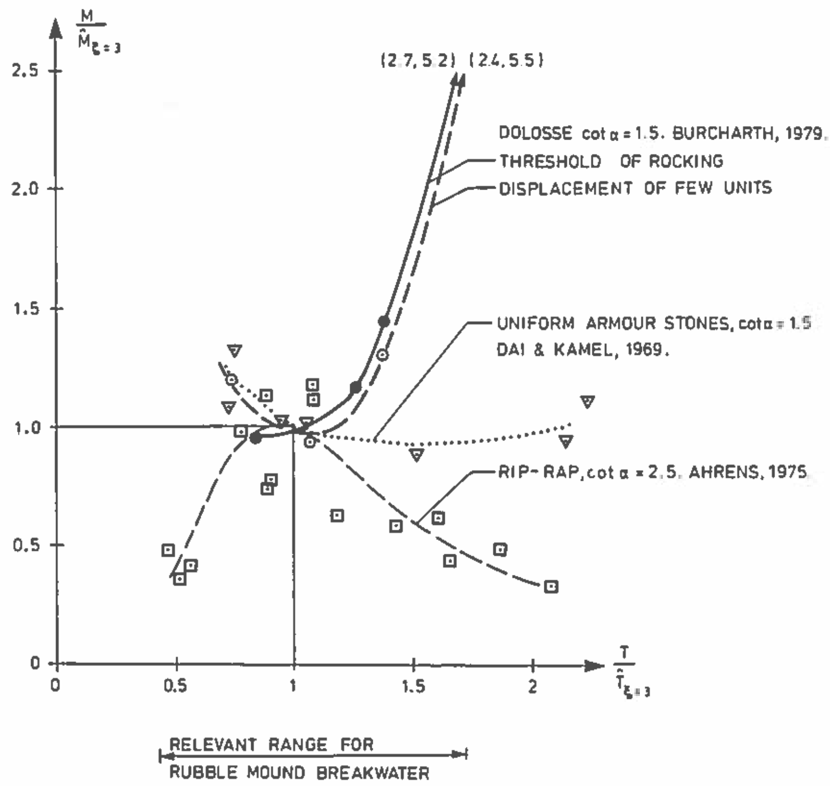


Figure 11. Example of influence of wave period on required mass of armour units and rip-rap. Regular waves. Data normalized with respect to the estimated values $\hat{T}_{\xi=3}$ and $\hat{M}_{\xi=3}$ corresponding to $\xi = T\left(\frac{g}{2\pi H}\right)^{0.5}\tan\alpha = 3$.

The examples show that the effect of the wave period on armour stability is not clarified in general.

4.2 OVERTOPPING

The design conditions are often related to overtopping of the breakwater. This is the case where roads, reclaimed areas, berths, installations etc. are located behind and close to the breakwater.

Overtopping is very sensitive to variations in wave height and mean water level. Besides this also variations in wave period, wave direction and wave shortcrestedness affect the overtopping.

The sensitivity to the wave height can be illustrated by the example given in Figure 12, which shows some scale model test results from a rubble mound breakwater with a wave wall.

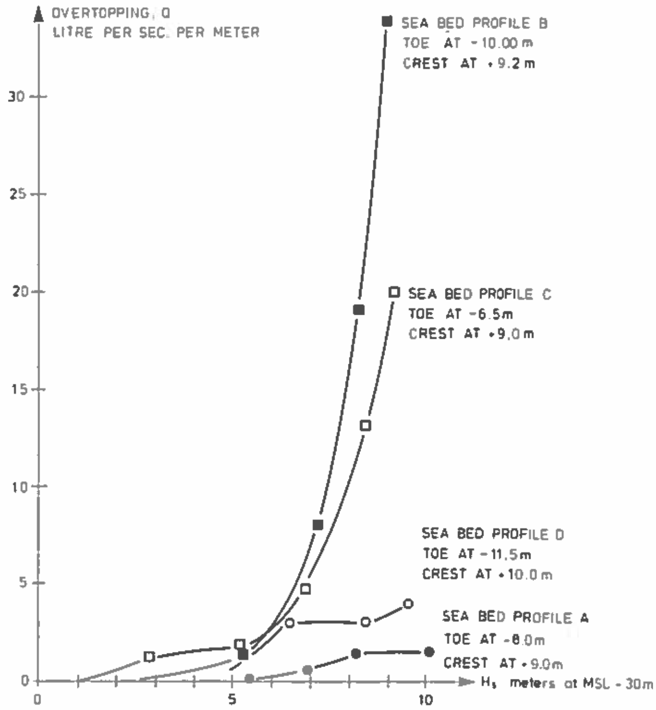


Figure 12.
Example of sensitivity of overtopping to wave height. Delft Hydraulics Laboratory. Sea bed profiles refer to Figure 5.

It is seen that the overtopping, Q increases exponentially when the wave height exceeds a certain value. A 10% increase in H_s can easily cause doubling of Q. The exponential growth of Q with H_s usually makes log Q a linear function of H_s.

Based on different scale model experiments Jensen et al., 1979, presented a more general description by means of the parameters QT_2/B^{*2} and $H_s/\Delta h$. T_2 is mean zero crossing wave period, B^* is a representative horizontal dimension and Δh is the vertical distance from still water level to the top of the crest or wave wall. By introducing Δh also the influence of water level is taken into account. Figure 13 shows an example given by Jensen et al.

Figure 13 clearly shows that even small variations in the still water level might cause significant variations in overtopping.

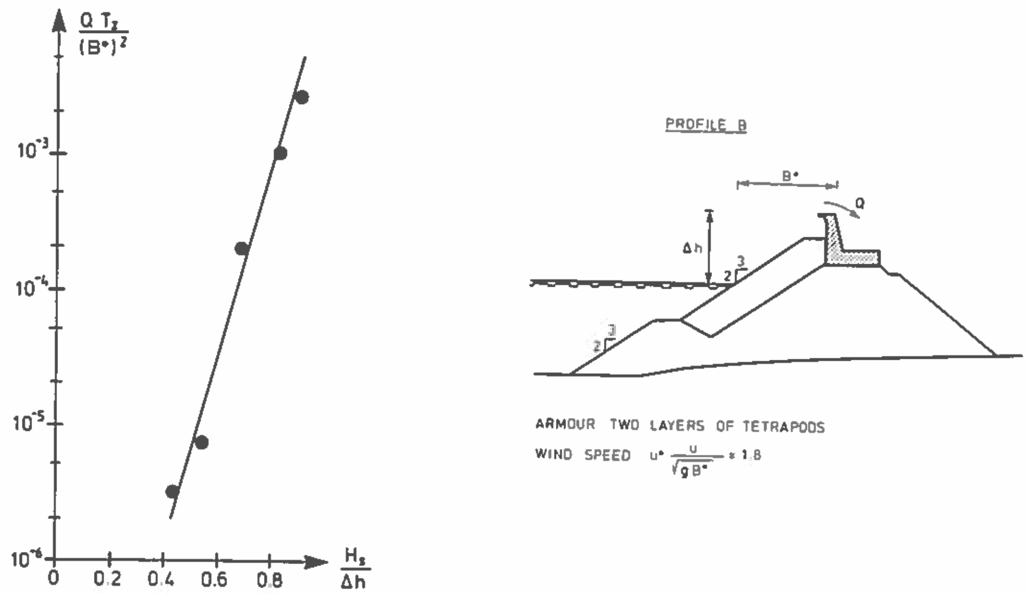


Figure 13. Example of sensitivity of overtopping to wave height and still water level. Jensen et al., 1979.

4.3 Directionality of the waves

Until to day nearly all breakwater model tests have been performed with uni-directional (2-D, long crested) waves. However, in nature the waves are directional (3-D, short crested) with horizontal spread of energy.

It is generally believed that 2-D waves is a good approximation to natural waves in shallow water due to the refraction which tends to make the waves long crested. However, Thunbo et al., 1984, found from a scale model experiment with a stone armoured breakwater with slope 1:2 in shallow water that 2-D waves caused 30-50% more damage than 3-D waves. This compares approximately to the necessity of a 40% increase in armour stone weight when going from 3-D waves to 2-D waves at the same damage level. Figure 14 shows some of the results.

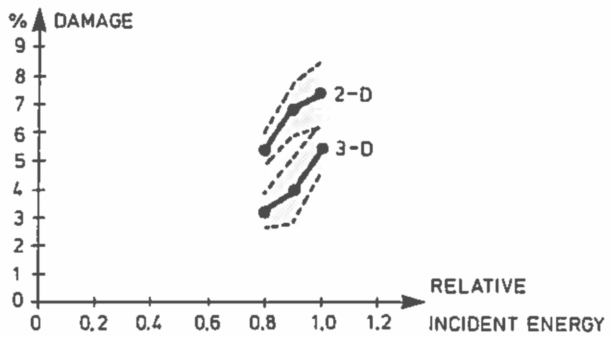


Figure 14. Example of comparison of 2-D and 3-D wave effects on stone rubble mound breakwater. Thunbo et al., 1984.

Shutler of HR, Wallingford reported from similar tests that no significant difference in 2-D and 3-D waves were found (scatter in the test results blurred possible differences).

It is concluded that there is still great uncertainty about the effect of wave directionality.

5. MODEL TESTS

Model tests are still necessary for practically all breakwater designs that depart from the very simple ideal design often tested in basic model studies of armour stability.

The reliability of model tests is therefore a question of great importance.

5.1 Reproduction of waves and data processing

The first point to discuss is the uncertainty related to the generation and analysis of laboratory waves. This problem was investigated by an IAHR Working Group, which was chaired by Joe Ploeg of Canada. The group consisted of representatives from some of the large hydraulic laboratories. Each laboratory performed the same experiment on a breakwater with the crest at MWL and exposed to some pre-specified waves. The wave climate in front of the breakwater and the water level variations behind it were recorded and analyzed. The results from the various laboratories deviated significantly and it was only after a great deal of thought that the reasons for these variations were explained. It turned out that the discrepancies to a great extent were due to differencies in the processing of the recorded data.

5.2 *Scatter in test data*

Another problem in model testing is the scatter in the test data signifying the response to the waves. This can be illustrated by some stability tests performed at the University of Aalborg with a Dolosse armour layer having a slope of 1 in 1.5 and exposed to irregular waves. For each of five different significant wave heights, H_s, 15 tests with identical wave trains were run with the object of studying the movements in terms of rocking and displacement of Dolosse. Very careful visual observations were made simultaneously by four people each covering a small area. A mirror system was used to obtain reliable observations in the splash and underwater zones. Each test was run for 20 minutes corresponding to approximately 1200 waves.

Some test results are shown in Figure 15, which illustrates the observed scatter related to the number of rocking and displaced blocks. These two modes of movement are relevant to the mechanical integrity of the blocks and the hydraulic stability of the armour layer.

Although direct recording of stresses in and/or recording of speed/acceleration of the blocks are much better than visual observations, the diagrams clearly illustrate the fact that reliable estimates of stability can be obtained only when tests are repeated several times. This is a fact which should not be overlooked.

It means that it might be necessary to apply a large safety factor if only a few tests are carried out, or to spend a lot more money performing many more tests than is normally the case at the moment. This is especially true for the complex, fragile types of armour units since it is seen from the Figure that the normalized standard deviation σ/μ for the numbers of displaced units is very large for small degrees of movements or damage corresponding to the design criteria for such units.

For large degrees of damage, i.e. failure situations, the scatter is reduced.

It should be mentioned that separation of rocking and of displacement in the "two" diagrams is not entirely meaningful and should be avoided in design diagrams. It is also important to remember that the scatter (e.g. in terms of the standard deviation) is dependent on the size of the test section.

5.3 *Scale effects*

The reliability of breakwater scale models has often been and still is seriously questioned and in most cases exclusively with reference to scale effects (thus forgetting the afore mentioned points). All scale models involve improper representation of some forces simply because only two types of forces at a time can be represented to scale. Therefore the question is "how much" is the model biased.

The two dominating forces in wave action models are gravity and inertia forces. Considering only these two types of forces the Froudian model scale law used for breakwater models ensures dynamic and kinematic similarity of the scale model and the prototype. Consequently viscous forces and surface tension are not reproduced to scale.

Viscous effects

For a wave exposed breakwater the flow is extremely unsteady. In some parts of the porous structure the flow will be turbulent or laminar all the time but in some part intermittent between the two flow-regimes, as discussed by Burcharth 1983.

The turbulent dragforces will scale like the inertia forces, because the viscous contribution is insignificant.

The flow-regime in granular structures is usually characterized by a Reynolds' number defined as

$$\mathbb{R} = \frac{V\,d}{\nu} \quad , \tag{20}$$

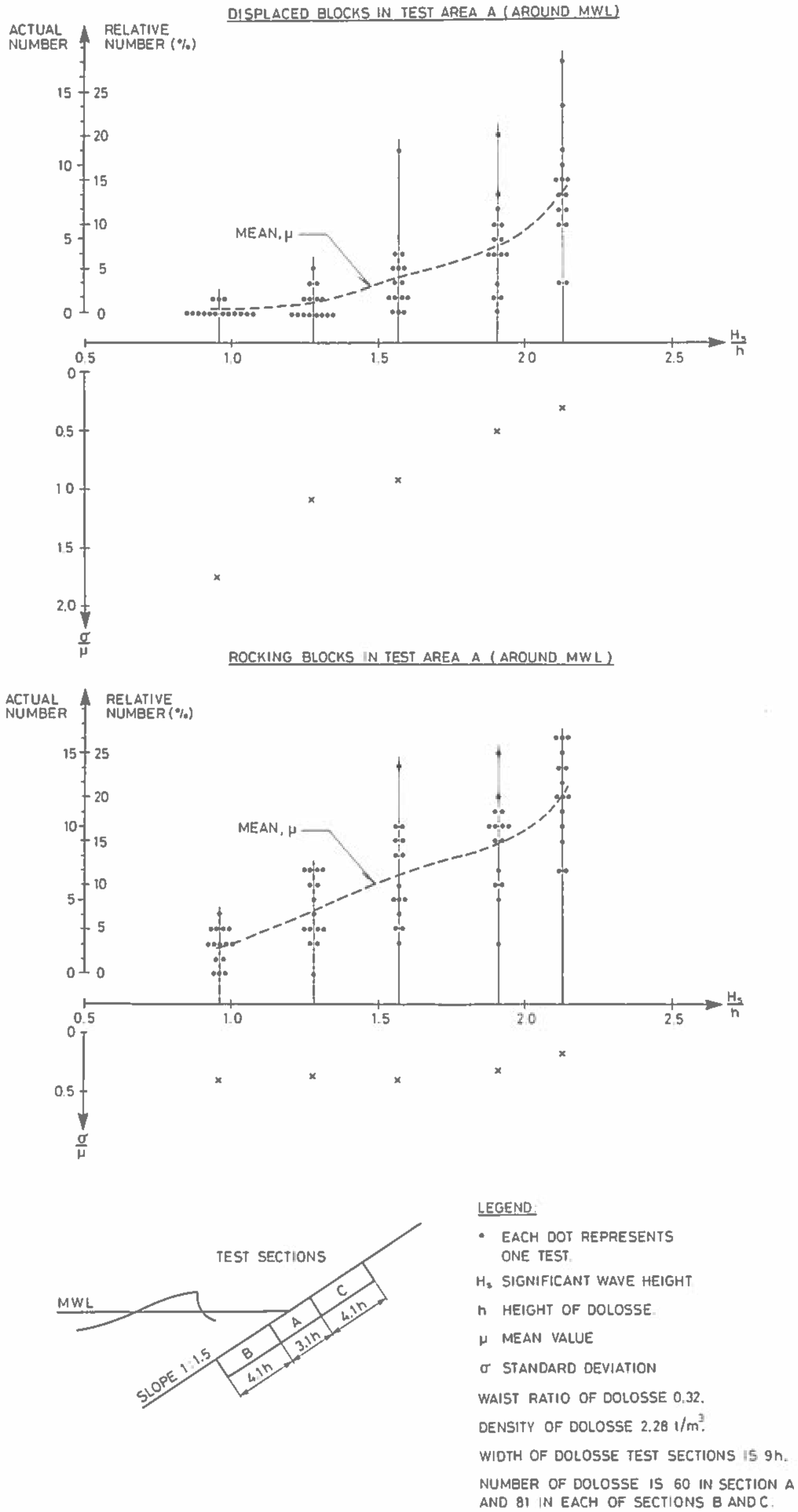


Figure 15. Example of scatter in armour stability tests.

where V is a characteristic flow velocity, d is a characteristic length and ν the kinematic viscosity of the liquid. When evaluating the unsteady flow in breakwaters it has become a tradition to use a constant figure for V which, more or less, is the maximum particle velocity of the incoming wave, i.e. $V \equiv (g\,H)^{0.5}$, where g is the gravitational constant and H is the wave height. d is usually taken as a typical diameter of the armour units/filter layer stones/core material, thus characterizing the width of the flow channels.

The primitiveness of this approach is obvious, but it is difficult to come up with an alternative which is both meaningful and simple.

Many researchers have studied viscous scale effects in breakwater models and the state of the art might be summarized as follows:

- No "significant" scale effect is observed in the "hydraulic stability" of the armour layer if $\mathbb{R} \geqslant 1 - 3 \cdot 10^4$ (d being a characteristic diameter of the armour units) and if the filter stones and the core material are geometrically to scale.

  However, it is important to notice that this statement is conclusive only in relation to mechanically strong armour units such as for example natural stones and concrete cubes. For the more fragile, complex types such as Dolosse and Tetrapods a scale effect which is not identified from visual observations of armour unit movements in the model might, when transferred to prototype, cause a very different amount of breakage. Timco et al., 1984, investigated this in some tests with Dolosse units with correctly scaled mechanical properties. They found that the influence of core permeability on the breakage of the Dolosse was very dependent on the geometrical scale.
- Run-up and overtopping are affected also by the porosity of the filter layer and the core. It has not been properly investigated how much changes in the size of the stones in order to obey the Reynolds' criteria stated above will bias run-up and overtopping.
- The reflection of waves from a breakwater scale model is practically independent of the permeability of the core, Timco et al. 1984.
- There is evidence that ultimate failures of rubble mound structures armoured with strong units can be studied with great accuracy in scale models. This statement is mainly based on a comparative study by DHI, Jensen et al., 1985, of the failure of the Thorshavn breakwater in the Faroe islands. This study is significant because of the availability of the prototype records of the waves in front of the breakwater throughout the damaging storm. The Reynolds' numbers in the model were about $4 \cdot 10^4$ for the armour stones and about $5 \cdot 10^3$ for the quarry run which eventually was exposed to the waves.
- Very little is known about scale effects related to the flow and the pore pressure in the more impervious parts of the breakwater such as the core (and the seabed if of sand). This means that for example uplift forces on concrete cappings and geotechnical aspects such as slip-circle stability and settlement cannot be properly evaluated in a scale model at the moment.

Surface tension effects

The surface tension determines the amount of entrapped air in breaking waves. As a consequence scale effects are present in scale models of forces from breaking waves and overtopping/spray. The shape (surface profile) of the waves in very small scale models is also affected.

Stive, 1985, studied the influence of air entrainment in a comparative scale model study of waves breaking on a beach. He recorded wave heights, set-up and vertical profiles of maximum seaward, maximum shoreward and time-mean horizontal velocities and found no significant deviations from the Froude scaling in a wave height range of 0.1 meter to 1.5 meter. This indeed indicates that surface tension scale effects are insignificant even in small scale models except for phenomena where a very accurate reproduction of the profile of the breaking wave is important. The most important example is shock pressures on plane solid walls. A special problem related to shock pressures is the interpretation of the recorded pressures in the model, because the air compressability is not to scale. This problem has been discussed by many researchers, see for example Lundgren, 1969, but it still remains to check model data against prototype measurements before the uncertainty related to shock pressures can be evaluated.

However, the author believes that the order of magnitude of wave pressures on wave walls found from proper scale models is correct. This opinion is based on a study of breakwater failure where damage to the concrete capping with wave walls allowed a rather accurate determination of the wave forces involved. By means of results from scale model tests, performed by DHI, in which wave pressures on the wave wall were recorded, it was possible to estimate the wave climate. This estimate was in very good agreement with the wave climate established by hindcast from meteorological observations.

Effects of mechanical properties of armour units

The relative strength of armour units is dependent on the size of the units, Burcharth, 1981. This has to be taken into account when designing and interpreting the scale models. The importance of this has been demonstrated in a number of papers by NRC, Canada, see for example Timco et al. 1983, who also developed a method of producing concrete armour units with correctly scaled mechanical properties, Timco 1981.

There are different ways of tackling this strength problem in scale models, as discussed by Burcharth, 1983, but in the case of tests with large (in prototype), complex types of unreinforced armour units the method established by NRC seems to be the best. The reliability of the method has yet to be evaluated. This can be done only by comparison with prototype measurements. A promising full scale experiment with instrumented 48 t Dolosse set up by the U.S. Army Corps of Engineers, Vicksburg, might provide very useful data for such a comparative study.

6. STOCHASTIC DESIGN PROCEDURE

It follows very clearly from the foregoing discussion that our quantitive knowledge on the loads and the structural response is limited to such an extent that design based purely on theory is not feasible. It is obvious that it will take years before we have developed a reliable design theory. Until then scale model tests are by far our most important tool.

In this rather unfortunate situation it is reasonable to think of a stochastic or probabilistic design method. However, it is often argued that a probabilistic design procedure is of little value as long as the understanding of the physics is poor. It is of course true that such a design process never gives figures in which to place high confidence as long as we cannot describe the physics. However, it is worth while to recall that the less we know, the more important it is to try to assess the reliability. The probabilistic approach is the only one which gives information on the risk of failure with due consideration to the uncertainty or scatter of the various parameters involved.

It is no excuse not to use the method because we do not know the probability density functions. As engineers we must estimate these functions, just as we estimate safety factors.

To-day's knowledge makes it of course not very easy to assess the probability functions. This is obvious from the Figures 16 and 17, which show typical failure modes and the corresponding fault tree. It is seen that not only the distribution functions for a great number of individual parameters but also the joint distribution functions for correlated parameters must be estimated.

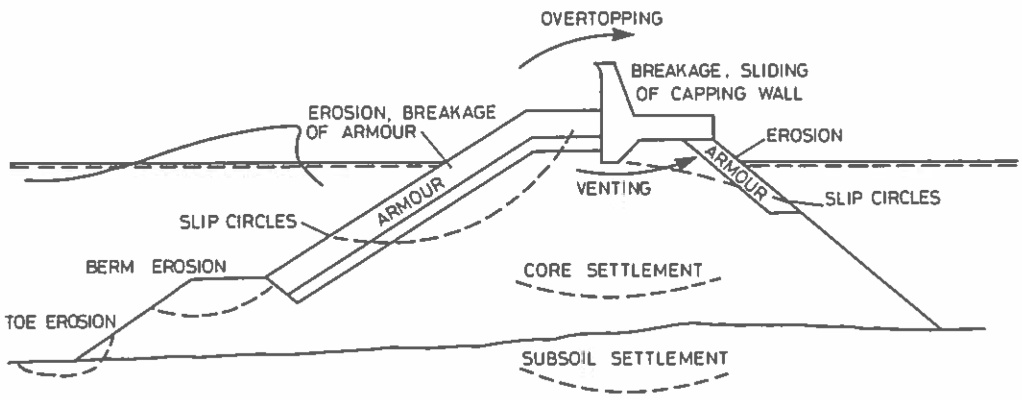


Figure 16. Failure modes of conventional rubble mound breakwater.

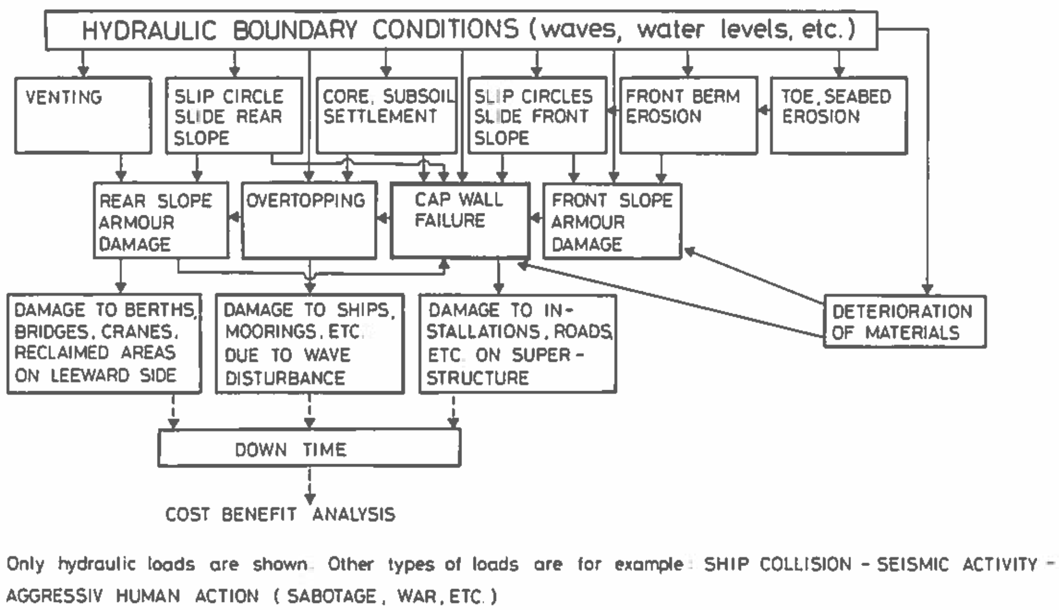


Figure 17. Simplified FAULT TREE for one section of a conventional rubble mound breakwater.

Despite all the problems one should go ahead and for a start restrict the stochastic design calculations to the vital parts of the structure only involving the most important parameters. An example is given by Nielsen et al., 1983, for the design of an armour layer. Systematical scale model experiments should be performed by capable laboratories to support this development.

In the present situation it is very important for the designer of rubble mound breakwaters to understand all the uncertainties he is up against. This might help him in developing designs which are uncomplicated in the sense of clear failure modes and load responses which are not very sensitive to exceedences in the estimated loads.

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Danish Society of Hydraulic Engineering
Seminar on Rubble Mound Breakwaters
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Geotechnical aspects of rubble mound breakwaters

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SYNOPSIS. Geotechnical aspects of rubble mound breakwaters concern the mechanical behaviour of the rock body and foundation subjected to seepage forces, pore pressures and accelerations caused by waves and quakes. They can be evaluated by the application of computer models. Related phenomena and their relevance to present-day design practice are elucidated, and experience gained by the engagement in unique projects is summarised.

OPTIMUM GEOTECHNICAL DESIGN

computer aided probabilistic approach

1. The reliability of a rubble mound breakwater is associated with its functioning under severe loading conditions due to waves or quakes, which are typically stochastic. Probabilistic methods are available to spot weak elements in a design and to define the relative importance of structural components [1].

2. Recently a national Dutch study [2] has been accomplished that deals with the applicability of numerical models on the basis of a comprehensive failure tree for a hypothetical rubble mound breakwater. The outcome shows geotechnical failure (figure 1) third in rank behind breakage and displacement of armour units. It shows precisely which failure mechanism is most probable: the keynote towards an optimum design.

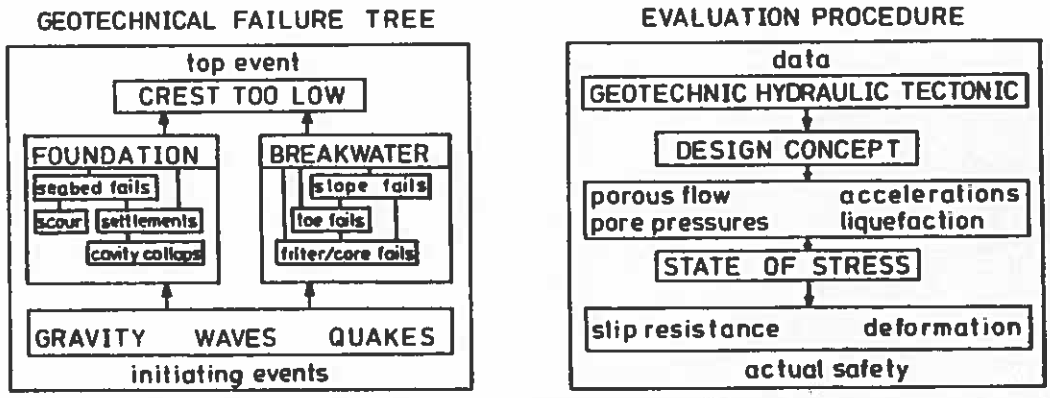

Figure 1. Assessment of geotechnical reliability

3. The determination of the probability of a distinct failure mechanism requires a proper data collection. However, since the data are difficult to obtain, engineering judgement and sensitivity analyses must cover this quandary, which render the results subjective. Therefore, results should be verified by physical tests, and if possible by prototype monitoring.

PHENOMENOLOGICAL RELEVANCE

porous hydro-dynamics

4. Fluids in porous media exhibit phenomena known in hydraulics, but the scale differs and the observation is from a granular matrix which itself may displace or distort.

5. The equation of motion of a small reference volume V of a viscous fluid in the void space of a porous medium can be expressed in terms of the pressure p and the volume velocity w:

$$-\nabla(p+\rho gz) = \rho\dot{w} + \mu\nabla^2 w \quad \text{with} \quad \dot{w} = w_{,t} + w\cdot\nabla w \qquad (1)$$

Here, ρ is the density and μ the dynamic viscosity of the fluid. Integration of the equilibrium equation is not possible in a porous medium because the space domain of the internal pores cannot be described.

6. Consider a larger volume U encompassing so many pores and granular particles that suitable average field variables and their derivatives can be defined; the fluid velocity:

$$v = \iiint w dV / U \qquad \text{with} \qquad U = \iiint dV \qquad (2)$$

should satisfy the requirement that v is independent of U. Treating likewise all others variables and properties yields:

$$-n\nabla(p+\rho gz) = n\rho\dot{v} + nR \quad \text{with} \quad \dot{v} = mv_{,t} + bv\cdot\nabla v \qquad (3)$$

where n is the porosity, m the virtual mass coefficient and b the momentum distribution coefficient. R represents the internal friction force due to viscosity and slip resistance at micro-scale, that is expressed by an empirical law:

$$R = n\mu(v-u)/k \qquad \text{or} \qquad R = \rho gq/K \qquad (4)$$

with k the so-called intrinsic permeability, K the hydraulic permeability and q = n(v-u) the filter velocity relative to the granular matrix velocity u. Rewriting equation (3) leads to:

$$m\rho v_{,t} + \nabla(p+\rho gz+\rho bv^2/2) + n\rho g(v-u)/K - \rho bv*(\nabla*v) = 0$$

and taking the curl assuming u rotation free yields:

$$\nabla*v = (\nabla*v)_0 \exp(-ng(t-t_0)/mK) \qquad (5)$$

Usually $mK/ng>T$ (T: wave period), which indicates that free vorticity in the flow averaged over the reference volume U decays very rapidly for sea waves. Hence, the porous flow can be considered macroscopically irrotational and it can be described in terms of a scalar potential. Turbulence at pore size dimension may exist and can be included in the macroscopic parameter K as a nonlinear effect (porous turbulence).

7. Conservation of mass of pore water can be expressed by:

$$\int_{U(t)} \frac{\partial}{\partial t}(n\rho)dU \quad + \quad n\rho \int_{S(t)} \frac{\partial dU}{\partial t} \quad = 0 \tag{6}$$

U(t) is the volume of a water body encompassing a fixed total mass, and S(t) the moving boundary. The first term represents the field storage due to fluid compression and/or granular deformation (porosity change) and the second represents the storage at moving boundaries. Time dependency in porous flow is mainly related to moving boundaries, which in breakwaters is identified with fluctuations of the internal water table.

8. The aforementioned macroscopic reference may apply to the propagation of shock waves in a saturated porous medium. Then the fluid and solid phase equilibrium is expressed by:

$$-n\nabla(p+\rho gz) = n\rho\dot{v} + n^2\rho g(v-u)/K \tag{7a}$$

$$-(1-n)\nabla(p+\rho' gz) + \nabla\sigma' + n^2\rho g(v-u)/K = (1-n)\rho'\dot{u} \tag{7b}$$

where ρ' is the grain density. The term $\nabla\sigma'$ is related to the intergranular contact forces represented by a macro-scale stress tensor including normal and symmetric shear components. Solving the two-phase equilibrium requires additional relations between the field variables p, σ', u and v, and also ρ and n. Numerical models have been developed suited to solve the problem of shock wave propagation in a saturated porous medium. Since the deformation behaviour of a granular medium is typically nonlinear, especially the shear deformation, the assessment of the dynamic response becomes rather complex.

<u>porous turbulence</u>

9. The motion of a pore fluid is described by equation (3), and the actual flow law by equation (4). Disregarding inertia and convection, and introducing a potential Φ yields:

$$-n\rho g\nabla\Phi = n\mu q/k \qquad \text{or} \qquad -\nabla\Phi = q/K \tag{8}$$

which is known as the law of Darcy.

10. Non-steadiness and nonlinearity are significant for wave induced porous flow in rubble mounds, whereas shown by tests convection is minor [3]. The following relation:

$$-\nabla\phi = mq_{,t}/ng + q/K(q) \tag{9}$$

should be adopted for breakwaters. Many empirical formulas for the K(q) have been suggested [4]. A convenient formula is:

$$K(q) = \sqrt{(2gd/f|\nabla\phi|)} \quad \text{with} \quad f = C\alpha(1-n)/\beta n^5 \tag{10}$$

where, f is the so-called friction coefficient, d the relevant grain size, C the drag coefficient related to the porous Reynolds number ($Re=qd\rho/\mu$); αd^2 is related to the relevant grain cross-section and βd^3 to the volume. This formula covers linear, postlinear and turbulent flow, and it is in agreement with theoretical and experimental results (figure 2).

11. The examination of the extensive research on the subject of non-Darcy flow reveals that no universal equation exists, which holds for all flow regimes. This is valid for the hydro-dynamic flow in armour that cannot be described macroscopically in a proper way. The concept of volumetric porosity is too poor to include the intricate pore geometry. The only acceptable way seems to be numerically modeling a three dimensional Navier-Stokes flow at micro-level. The solution of this challenging problem requires however a great effort.

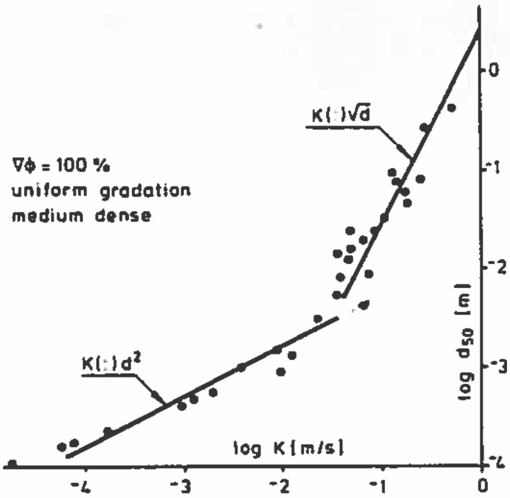

Figure 2. Measured permeability

Internal friction

12. Internal resistance to shear deformation is fundamental to the stability of granular slopes. Tests have shown that loose rock is mobile and compacted rockfill behaves fairly isotropic. This behaviour is reasonably well understood from mechanics on fine granular materials, however, the strength ratio of individual particles and the skeleton is quite different. Intergranular contact points are more likely to crush, yielding a nonlinear internal friction subjected to changes in time due to erosion [5,6,7].

13. Various studies have shown that for rubble mounds the internal friction can be expressed by [8]:

$$\phi = \phi_0 + R'\log(S/\sigma') \tag{11}$$

Here, ϕ_0 is the basic friction angle in the (eroded) contact points, S the equivalent strength related to the particle size and its compressive strength, R' the equivalent roughness related to the rock type and the rockfill porosity, and σ' the actual stress level.

14. The interfaces between subsequent layers of different materials is liable to slip because of reduced interlocking. Including the stress level effect in an available empirical formula [9] yields:

$$\phi' = \delta^{a}\phi \qquad \text{with} \quad \delta < 1 \tag{12}$$

where, δ is the grain size diameter ratio, and a an empirical constant. If the particles are too large to rely on these average formulas, a practicle value for ϕ' can be derived from the number of actual contacts at the interface [10].

15. The mobilisation of the full friction value (ultimate shear strength) requires relatively large strains (up to 0.10). Therefore, since hazardous situations may occur at significantly lower strain level, it is important to use conservative values for the internal friction, or to perform a deformation analysis.

slope stability

16. The analysis of slope stability using the method of slices originally carried out by Fellenius in 1937 has been improved. Bishop examined the influence of various simplifications on the slope stability factor Γ. His suggestion to include horizontal interslice forces provides good results for slopes not steeper than 65°. Even though the method assumes a simple stress state, verification of this assumption for earth slopes has shown a negligible effect on the slope stability factor Γ.

17. Bishop's method has been checked for rockfill slopes [11]. Comparison with a shear failure analysis based on a true stress state shows that the corresponding slope safety factor is slightly higher. However, the importance of the nonlinearity in the friction behaviour is significant, as shallow slip surfaces will not easily develop. Therefore, rockfill slopes can be analysed by Bishop's method adopting nonlinear friction.

18. Porous flow affects the slope stability [10,12]. Resulting pore pressures directly influence the local shear resistance reducing the slope stability factor down to 0.75Γ. For rockfill exposed to waves the local flow field must be considered, which can be determined on the basis of actual wave pressures at the slope, the actual internal water table, and the actual (nonlinear) permeability (figure 3).

pore pressure build up in rubble mounds

19. Pore pressure is due to storage or loading. It dissipates by porous flow. To evaluate the possibility of local excess pore pressures the dissipation period (the drainage capacity) has to be considered. If this period is small compared to the loading period excess pore pressures will be limited. Four processes

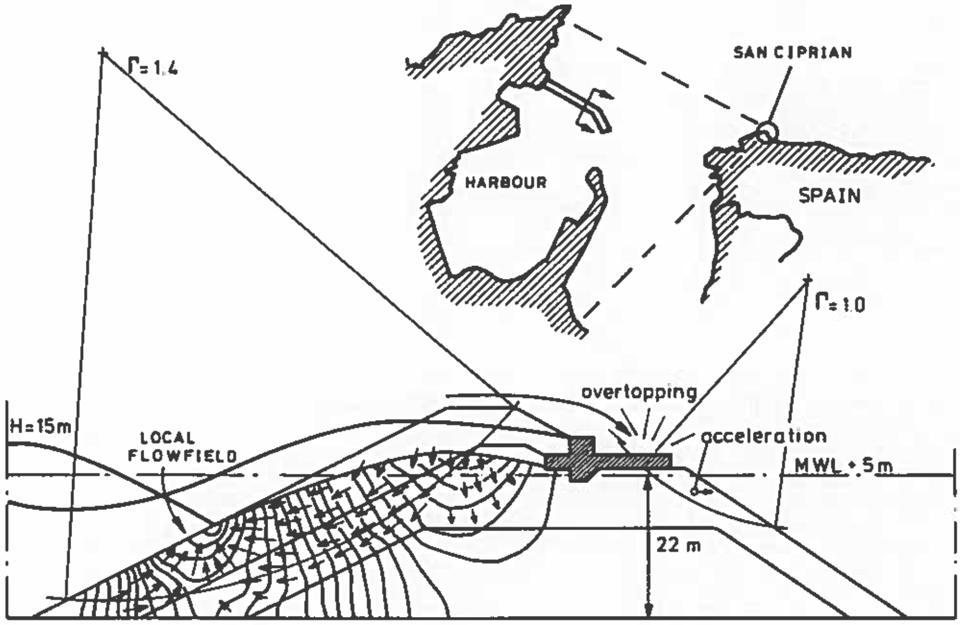

Figure 3. Wave induced local flow field

involving cyclic dissipation and build up of pore pressures are considered: (1) wave induced internal setup, (2) field storage, (3) liquefaction, and (4) dynamic deformation.

__wave induced internal setup__

20. The slope of a breakwater exposed to waves, swell or tides shows a much larger inflow surface at high waterlevel than the outflow surface at low level. The outflow path is also longer, which together results in an extra internal waterlevel setup.

21. The corresponding extra water volume can be represented as the result of an average infiltration (figure 4):

$$I(x) = I_0 \exp(-x/\lambda)$$

$$\text{with} \quad I_0 = \lambda H/(2T(\lambda + H\,tg\gamma)) \quad \text{and} \quad \lambda = \sqrt{(KDT/2n)}$$

The internal setup due to this infiltration can be found by evaluating the corresponding flow field. Two typical cases are

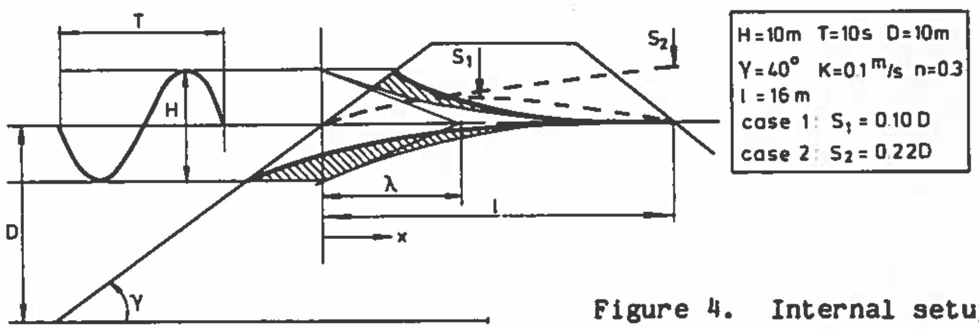

Figure 4. Internal setup

distinguished: one with a constant water level at the inner slope, and one with a backfill. The first case provides the a maximum internal setup at $x=\varepsilon l$:

$$s_1 = \sqrt{(D^2+\eta(1-\varepsilon+\varepsilon\exp(-l/\lambda)-\exp(-\varepsilon l/\lambda)))} - D \qquad (13a)$$

with: $\varepsilon = (\lambda/l)\ln(l/\lambda/(1-\exp(-l/\lambda)))$ and $\eta = \lambda^2 I_o/K$

The second case provides a maximum setup at the backfill:

$$s_2 = \sqrt{(D^2+\eta(1-\exp(-l/\lambda)(1+l/\lambda))} - D \qquad (13b)$$

These formulas have been verified with numerical and physical experiments. The internal setup may reach 0.25D, which is important with regard to the transfer of wave energy and corresponding damage at the lee side.

field storage

22. The response of a two-phase medium to dynamic loading is twofold; each phase takes part. Internal equilibrium is regained after deformation and/or flow of the phases, which takes time. The deformation of a granular medium may exhibit volumetric strains resulting in excess pore pressures, thus generating porous flow. In rubble mounds the drainage capacity is usually sufficient to let excess pore pressures rapidly dissipate. The dissipation process can be approximated by [13]:

$$p = p_o \exp(-3ct/\lambda r) \quad \text{with} \quad c = EK/\rho g \qquad (14)$$

Here, E is the granular matrix flexibility modulus, λ the drainage length, and r the hydraulic radius (figure 5). The consolidation effect is minor, if $T>\lambda r/c$ holds.

liquefaction

23. An aspect of major importance with respect to safety of breakwaters is the potential instability of the toe structure due to wave induced excess pore pressures in the seabed [14]. It constitutes an important failure mechanism [15]. Since many coastal areas contain a top seabed layer of silty or loosely packed sand [3,16,17] consideration of this phenomenon is indispensable and substantial for the design.

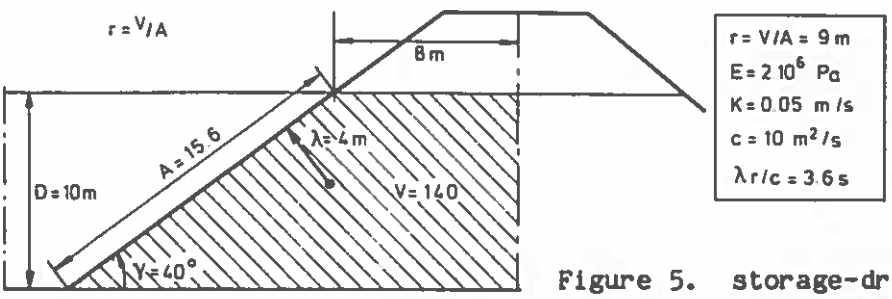

Figure 5. storage-drainage

24. The assessment of this problem is difficult due to the complexity of the phenomenon and the randomness of the loading. The phenomenon however is not new; already in 1885 Reynold describes an experiment on a granular material in which he introduced the concept of dilatancy that expresses the volume change due to shear deformation. In a saturated granular medium this tendency produces excess pore pressures which reduce the shear strength of the granular medium until a liquid state.

25. The actual liquefaction potential of the seafloor can be determined by site investigations which include soil characterisation, measuring the actual porosity and collecting samples. In addition the critical porosity (at zero dilatancy) and the cyclic mobility can be determined by laboratory tests on reconstituted samples, and finally by applying simulation models the test results can be projected on the actual situation which includes drainage and a relevant stress state.

26. The characteristic value of the random wave loading relevant to the phenomenon of cyclic excess pore pressures can be as low as half the hydraulic design wave, indicating the predominance of this phenomenon even for moderate storms [18]. Since experiments show a threshold wave below which no excess pore pressures occur, the probabilistic approach becomes questionable without reference to the time lap of occurrence If anything, excess pore pressure build up due to subsequent waves is essential to the instantaneous geotechnical stability, particularly when the drainage capacity of the toe foundation is insufficient. It decreases the slope stability factor direly (figure 6).

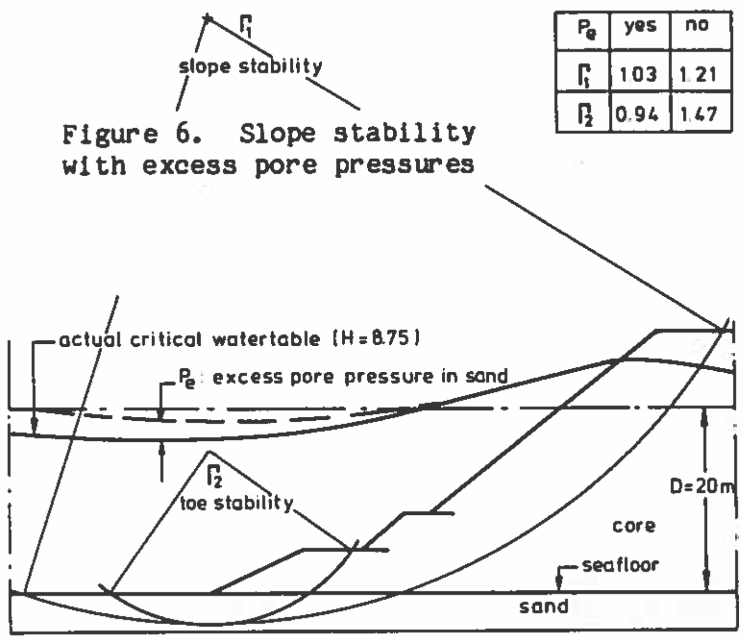

Figure 6. Slope stability with excess pore pressures

27. Solitary waves may subject the breakwater to a tremendous drag that is able to convey the outer slope as if liquefied. These rare events are generally due to unpredictable causes, like earthquakes, vulcanic activity, remote storms and freak waves [19]. Provided the wave loading is known the slope stability can be evaluated .

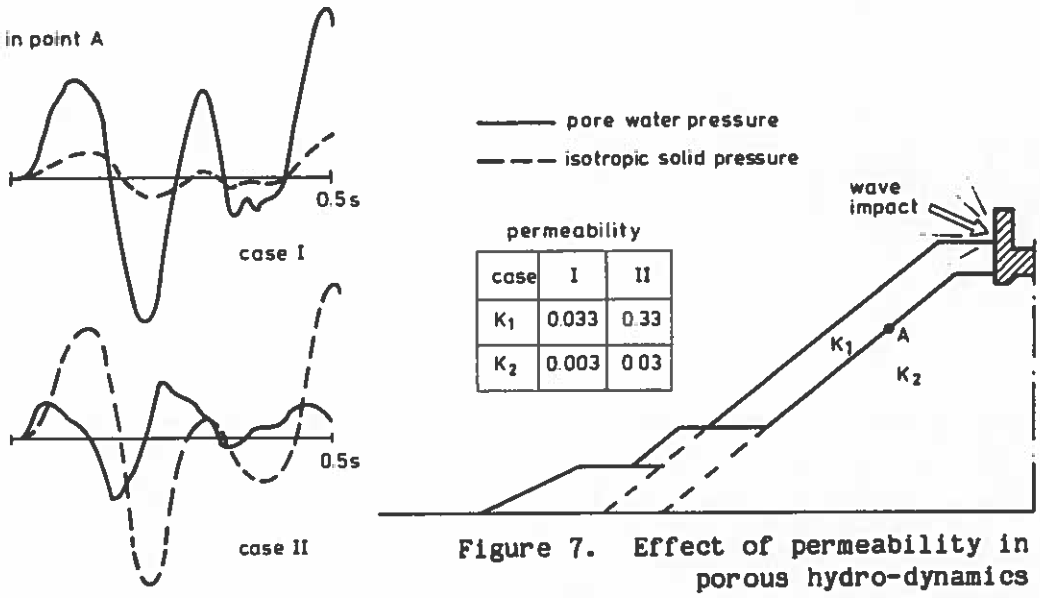

Figure 7. Effect of permeability in porous hydro-dynamics

<u>dynamic</u> <u>deformation</u>

28. The phenomena in poro-hydrodynamics are governed by the multi-phase system (air-water-solid) and the mutual exchange of momentum by interphase friction, pressure and surface tension. A striking example is presented in figure 7 which shows the effect of the permeability on the response of a breakwater to an impacting wave. The result shows the importance of the permeability to dynamic deformation behaviour. Suchlike phenomena cannot be simulated by physical scale models [20].

29. Impact forces caused by water jet surges due to plunging or overtopping waves cause shock waves [21]. This phenomenon has been observed in model tests and it has been verified numerically [12]. It may generate horizontal accelerations in the core and slopes up to 0.1g corresponding to a medium earthquake. The corresponding reduction of the slope stability factor is significant, down to 0.6Γ. A difficulty in evaluating this aspect is the unacquaintedness with the intensity of the impact and reflection. Dynamic forces due to breaking waves against a vertical wall have been measured and the slamming force may reach values as high as three times the semi-static one [26].

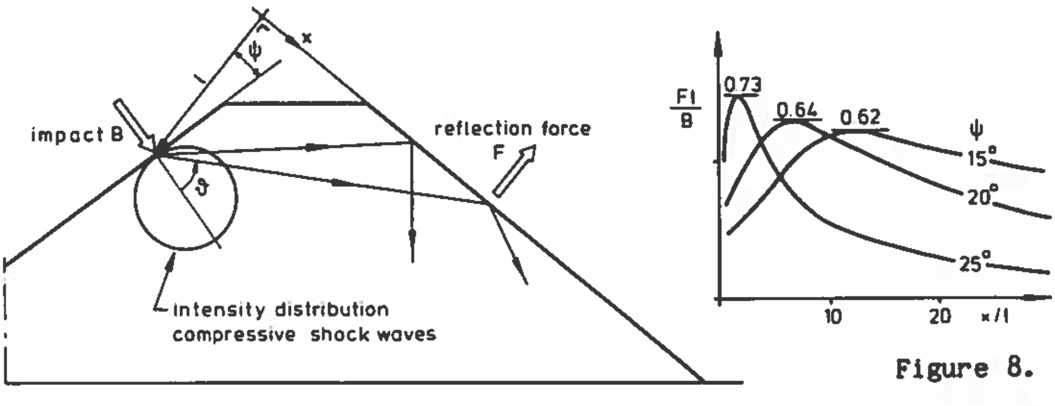

Figure 8.

30. The compressive shock wave upon reflection generates tensile forces at the lee-side slope (dynamic piping) able to lift particles (figure 8). Even if it is strongly diminished by reflection and energy dispersion in the armour layer, high forces can be expected at the lee-side. For a semi-saturated breakwater the intensity of the shock wave will decrease drastically due to internal damping (air compression and related interphase friction due to relative motion). This aspect demands proper consideration for breakwaters with a backfill.

31. To provide some insight in the dynamic deformation of large rubble mounds the two-phase dynamic response to a huge impacting and overtopping wave and one due to earthquake loading have been simulated numerically [12,22]. In figure 9 the cyclic deformations are shown due to an impacting wave; the structure moves biharmonically with a free frequency of about 2 Hz. Plastic deformation zones develop along the slopes under the armour, which cause cyclic settlements. In figure 10 the calculated residual deformations due to a large earthquake (max 0.3g) are shown. The maximum strain is in the order of 0.05. The time history of the cap displacement and the creep at both slopes show the importance of a deformation analysis.

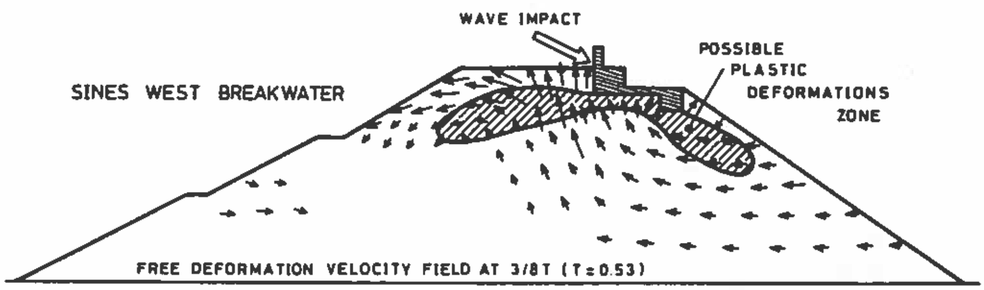

Figure 9. Deformation modes due to a wave impact

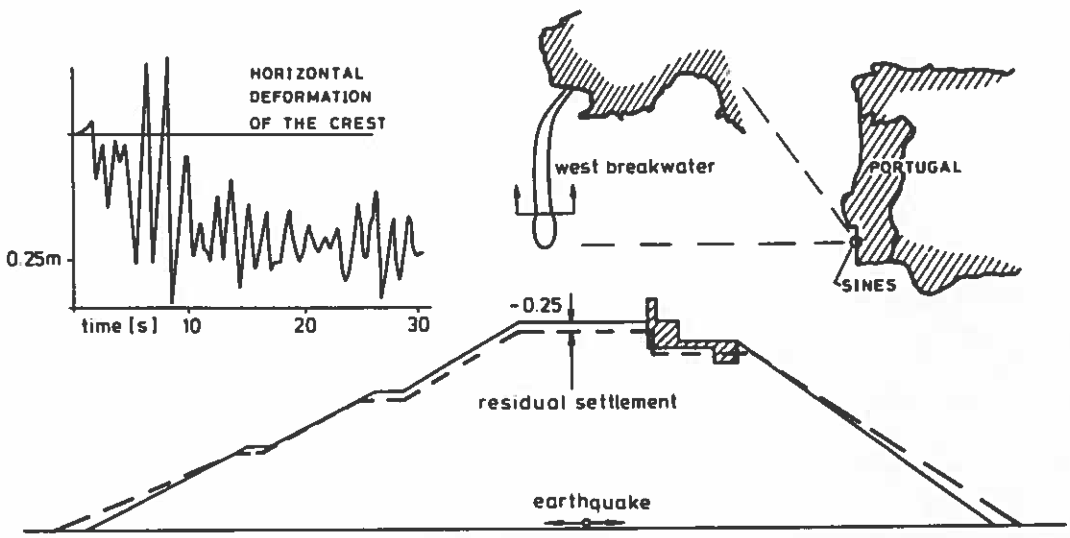

Figure 10. Dynamic deformation due to an earthquake

local stability

32. The hydraulic loading imposed on individual particles by waves, and internal friction and interlocking of randomly placed particles cannot be described in detail. Physical tests seem the only way. A pilot study [10] on the surface particle stability has provided a general comprehension of the characteristics involved; experiments reveal that a certain range of core permeability is optimum for stability of surface particles.

33. The effect of a porous drag force on a surface particle due to outflow from the core is related to the local flow pattern induced by the particle itself when moving. Since a slight motion of the particle causes a decrease in the local flow resistance, a decrease of the initial porous drag force occurs accordingly. Therefore, a surface particle seems to possess a self-healing stability potential due to motion induced inhomogeneity (figure 11). A neighbouring particle does not show similar behaviour. This aspect has been investigated and underscored by experiments [23,24]. Most armour designed for economic weight, optimum interlocking and high porosity has little or no self-healing potential.

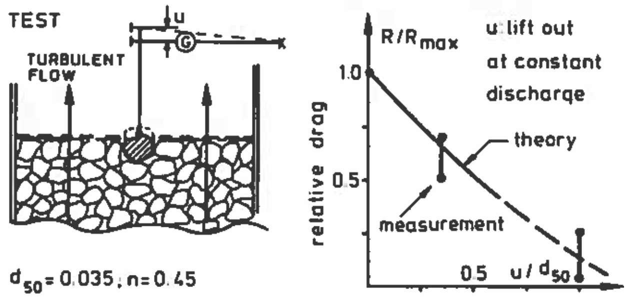

Figure 11. Local stability of a particle

34. The experiments show that for a compacted granular medium porous outflow first activates a group of coherent particles to expand, creating a loose packing, until one favourable surface particle starts moving, giving way to the outflow. Similar behaviour has been observed in sand.

35. That a rubble mound is self-compacting over several storms, may hold for the inner core, but not in general for the most vulnerable parts, the slopes. The local stability of surface particles is jeopardised by up- and downrush drag forces at protruding elements lifted by outflow. The most critical locations are around the transient low water line on the sea-side slope, and at the top lee-side slope. Investigation of up- and downrush drag on protruding particles is recommanded to further study local stability.

gravity

36. The evaluation of gravitational settlement of the rubble mound and compressible layers in the foundation is established by standard formulas and proper site investigation.

37. If a breakwater slope is subjected to dynamic pressures, the weight to be considered in the slope stability analysis

should be the submerged weigth rather than the saturated one because of the absence of pertinent drag forces. For rubble mounds this approach provides a substantial positive effect to the slope stability factor [22].

GEOTECHNICAL EVALUATION

38. Since 1975 the Delft Soil Mechanics Laboratory (DSML) has been involved in studies of large rubble mound breakwaters. A sound expertise has been gained. Some characteristic results with regard to geotechnical aspects of large breakwaters, existent or under construction, are reported here.

storm surge barrier at the Eastern Scheldt

39. The threshold of the movable gates of the barrier (to be completed in 1986) is founded on a rubble mound sill, the composition of which has been extensively investigated. The sill can be conceived as a submerged breakwater withstanding a relatively large fall of 9m steady and 4m cyclic water head. Since the subsoil consists of loose sand with a high liquefaction potential, the stabilization of the seabed has received much attention. DSML has performed the major part of the geotechnical investigations which have been widely reported. The construction has been rewarded by ASCE as an outstanding engineering achievement.

breakwater at Gioia Tauro

40. In 1977 during filling operations a large flow slide onset by a floodwave took place at Gioia Tauro (figure 12). A sand volume of 5500000 m^3 was mobilised, and the reclaimed coast strip for the west breakwater was heavily eroded. Site investigation (borings and density measurement) and laboratory tests revealed that the liquefaction started in the original loose sandbed.

breakwater at Sines

41. The 2 km long immense rubble mound breakwater of the port of Sines was seriously damaged during a severe storm in 1978. For the geotechnical analysis of the original and the repaired profile computer models simulating dynamic pore pressures caused by storm waves and impacts (including Tsunami waves by remote

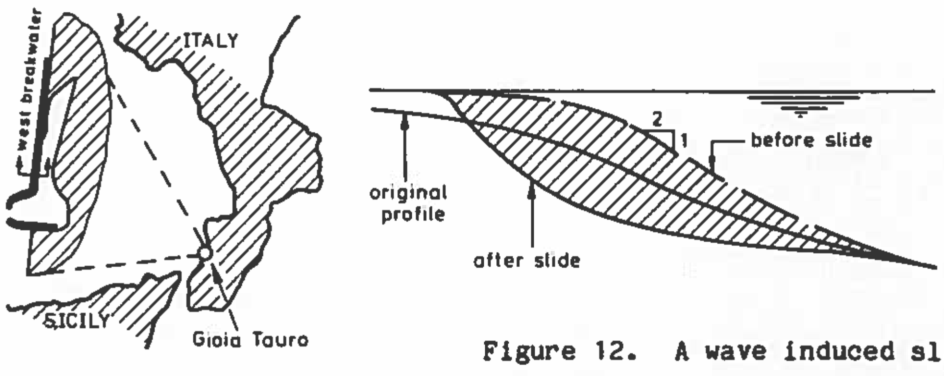

Figure 12. A wave induced slide

tectonic activities) have been used. Physical verification tests have been performed. The study represents an unpreceded illustration of the importance of geodynamics in breakwater design. A comprehensive review has been published [12]. Later the seismic stability has been evaluated and reported [22]. Some results are shown in figure 10.

breakwater at San Ciprian

42. The north and south breakwater at San Ciprian were severely damaged during a storm in 1980. Though a geotechnical analysis showed shallow slip surfaces most vulnerable, the north breakwater was found geotechnically stable (figure 3), except at the lee-side in the case of heavy overtopping. The observed large settlements of the massive superstructure on the south breakwater has been associated with a seabed sand layer under the outer part, which first let the toe settle, next the slope, and finally the top. Similar behaviour has been reported [25]. Unfortunately, no relevant data about the sand were available.

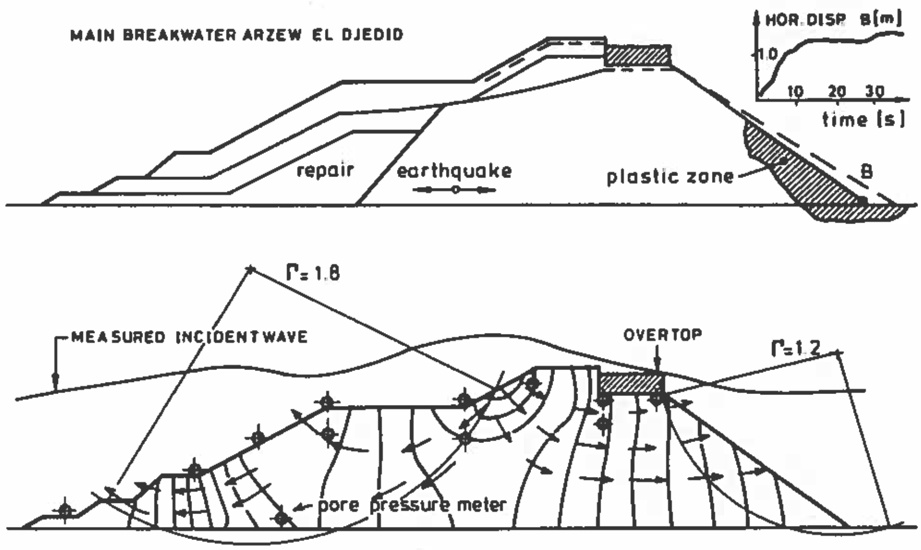

Figure 13. Geotechnical evaluation of a redesign

breakwater at Arzew

43. In 1980 the large rubble mound breakwater at the port of Arzew El Djedid has been severely damaged onset by wave induced armour rocking and breakage. The geotechnical stability of the redesign has been evaluated [26]. Rock characteristics for the erosion-deformation behaviour have been measured, and the dynamic toe- (on seabed sand) and slope stability has been determined under earthquake and wave loading; the latter verified in model tests. The earthquake analysis reveals the potential instability of the inner toe (figure 13). Suggestions for the redesign have been recommended accordingly.

breakwater at Tripoli

44. The breakwater and related roads in Tripoli Harbour were severely damaged during storms in 1981. In the original design

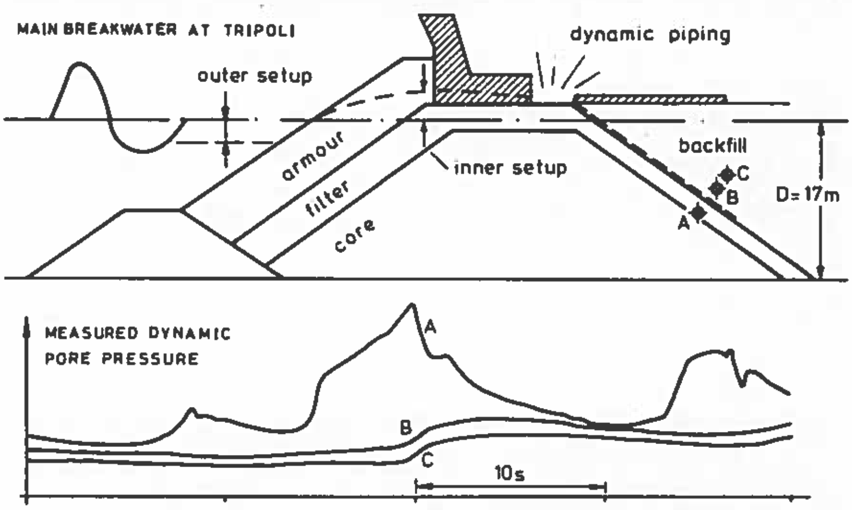

Figure 14. Wave induced backfill pressures

the main filter layer continued to function as a drain under the crest structure towards the backfill. During storms however the wave induced internal setup (figure 14) saturated the entire breakwater body, and easily transferred wave energy damaged the back side. This phenomenon has been verified by experiments. In the redesign the open filter was replaced by a massive barrier of grouted stones up to a depth where gravitational stress prohibits mobility in the backfill [26].

breakwater at Zeebrugge

45. The new harbour of Zeebrugge is protected by two large breakwaters with a backfill (figure 15). The stability of the open filter construction at the separation of core and backfill has been evaluated numerically. The result shows that wave induced porous flow causes internal setup and cyclic water table motion at the filter, which for large storms reach magnitudes leading to outflow from the filter. Wave energy transferred through the saturated core may cause damage at the lee-side. The filter design has been adjusted.

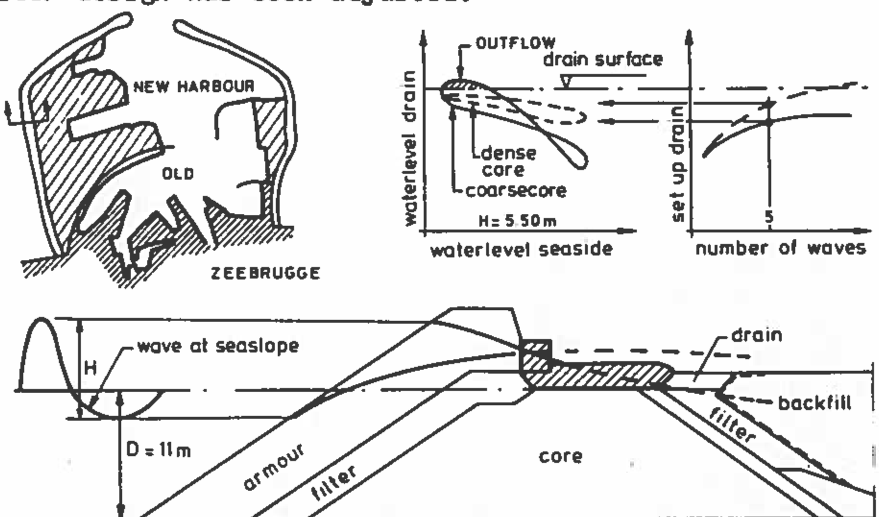

Figure 15. Filter-drain behaviour

breakwater at Mohammedia

46. The new port of Mohammedia is protected by a large rubble mound breakwater into 25m deep sea. The dynamic geotechnical stability of the toe structure partly situated on a loose sand layer has been verified. Data is collected by site investigation (density measurement and sampling) and testing (cyclic undrained triaxial tests). Computer models have been applied to assess the liquefaction potential under wave loading. The background of the approach adopted has been reported elsewhere [18]. The results reveal a marginal toe stability (figure 16). Therefore, relevant improvements have been suggested.

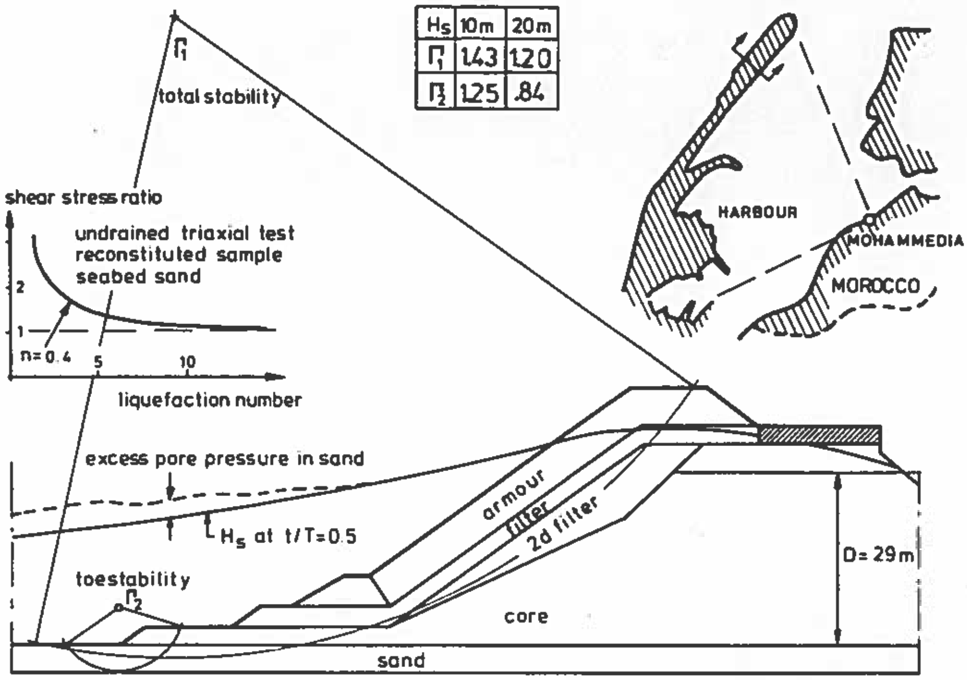

Figure 16. Seaside slope stability evaluation

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Torben Sørensen and O. Juul Jensen

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RUBBLE MOUND BREAKWATERS

EXPERIENCE GAINED FROM BREAKWATER FAILURES

by

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SYNOPSIS. The purpose of this paper is to present some results of studies of breakwater failures and the subsequent experience gained which can be of general importance for the design process of similar structures and for the overall understanding of the physics of rubble-mound structures.

INTRODUCTION

1. The paper deals with the failure of breakwaters and the experience gained. The paper has no intension of being critical of the design or construction of particular breakwaters, but it rather intends to use the valuable information gathered from failures in a general assessment of various aspects of breakwater design and construction. Hopefully, the more information is available in the future the smaller will be the number of severely damaged structures, which many times result in large costs to the client/owner of the particular breakwater structure.

TYPES OF DAMAGE/FAILURE

2. A rubble-mound breakwater can be damaged in several ways depending on its configuration, the wave conditions, and the water level during the storm. The following main modes of damage are identified:

(a) Sliding/settlement of the seaward face due to scouring or morphological changes of sea bed or the toe of the breakwater.
(b) Sliding of the seaward armour layer due to an unstable berm.
(c) Damage due to geotechnical instability of the subsoil or insufficient bearing capacity leading to serious settlements.
(d) Damage to the armour layer by displacement of armour units due to excessive wave forces/moments mainly during wave run-down. Wave run-up is only critical for damage to the armour layer for flat slopes (cotv > 3.5) (ref. 5). For such flat slopes the stones/material in the slope are moved upwards.

(e) In special cases of steep slopes wave forces may cause a sliding of the whole armour layer for a single large wave run-down.

(f) Damage to the crown wall or superstructure due to wave forces. Damage to a breakwater superstructure is often an integrated problem occurring simultaneously with damage to an armour layer in front and erosion under the base of the superstructure.

(g) Damage to the crest and rear side armour layer under excessive overtopping due to high waves and/or high water level. Damage to the rear side armour layer may also be a problem for breakwaters with a superstructure in the case where the configuration of the superstructure is not protecting the rear side armour layer from being hit by voluminous wave overtopping.

(h) Breaking of the armour units due to large dynamic contact forces arising as a result of rocking or wave compaction.

(i) Damage to the reclamation behind a breakwater due to excessive transmission (high permeability) or overtopping.

(j) Damage due to impact of floating objects (vessels, ice etc.).

3. The examples from actual failures in the following will focus on some of these types of failure modes.

General Design Philosophy

4. The designer of a breakwater is faced with a number of problems and uncertainties (ref. 5).

- Traditionally breakwaters have been designed using a design storm or design situation with a corresponding return period of 50 or 100 years. Although the designer perhaps feels that this is sufficiently safe, the real safety is questionable. If we consider that the situation used for design is a true central estimate of for example a 50 years situation, the probability of exceeding of the design conditions is as high as approximately 20% within the first 10 years after construction or 63% within 50 years. Depending on the consequences of exceedance of the design situation such probabilities seem high and in most cases unacceptable especially where either the breakwater structure itself or the structures or facilities protected by it are of high value.
- Instead of designing in the traditional way, it seems more appropriate to consider the acceptable probability of exceedance of the design situation and the economic loss if this happens and then design in accordance herewith. If for example the desired lifetime of a structure is 25 years and the acceptable probability of exceedance of the design event is 5%,

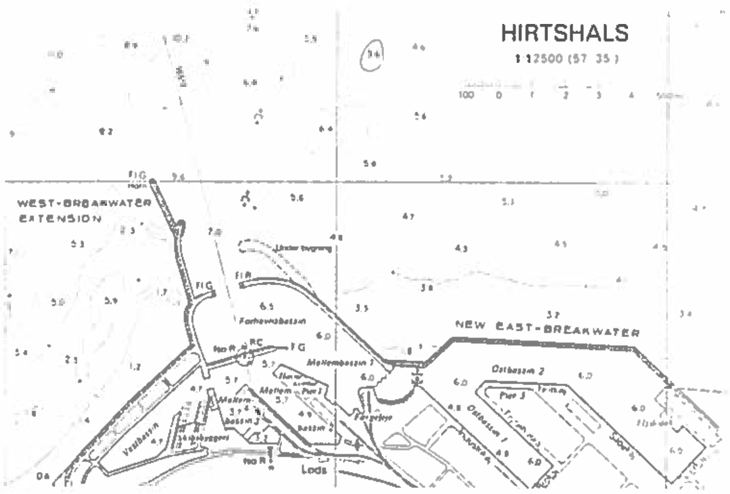
the structure should be designed for a 500 years situation!

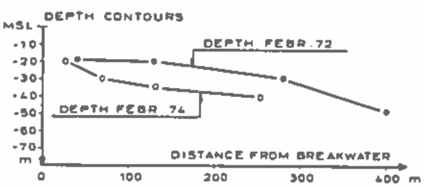
- o Besides these aspects it is necessary to consider the accuracy with which the estimate of the design situation is made. If the estimate of the design situation is uncertain there is a high probability (50%) that the structure is underdesigned compared to what is believed by the designer. It is consequently important to use all possible means such as wave measurements, mathematical modelling for wave hindcast of historical storm and mathematical modelling of wave refraction in order to obtain the most reliable estimate of the design situation.
- o Normally, the designer is further left with a number of complex problems, such that the design is only optimum if the different structural elements of the breakwater designed have the same probability of failure. This is for a number of reasons not as simple as it sounds, because the effect of an increase in wave height is different for the different structural elements. If we consider the stability of the main armour layer on the front side, the damage (number of units being displaced) increases gradually with increasing wave height. On the other hand if we consider the rear side stability of a breakwater, a very small increase in wave height may change the situation from zero-damage to unacceptable damage or complete failure. For a superstructure the situation is even worse, here the situation is either-or. Either the wave impacts are acceptable and cause no displacement of the superstructure, i.e. acceptable situation, or one or more wave impacts are too large compared to the resistance of the superstructure and it is consequently displaced.

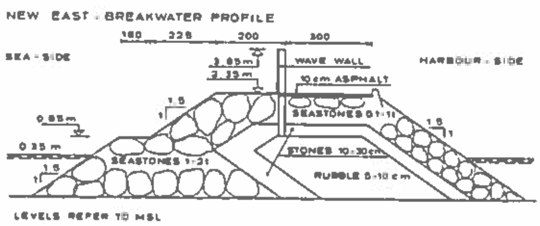
5. These small examples prove the difficulties of obtaining an optimum design and highlights the necessity of obtaining as reliable wave and water level data as possible. Wave (and water level) measurements and studies can not be recommended too many times, they are of vital importance for the design process and should be initiated as early in a project as possible.

Example 1 - Morphological Changes

6. Hirtshals Harbour on the west coast of Denmark was extended in the period 1972-74, as seen on the plan in Fig. 1. The eastern breakwater was under extension in 1973 when a major storm occurred from WNW. The profile of the breakwater is also seen in Fig. 1. During the storm with a water level of +1.35 m (M.S.L.), the breakwater suffered severe damage. The waves were breaking waves, thus the maximum waves which could exist in the actual depth. During the breakwater construction and after the storm it became clear that a major







MODEL TEST PHOTO OF TILTING WAVE WALL

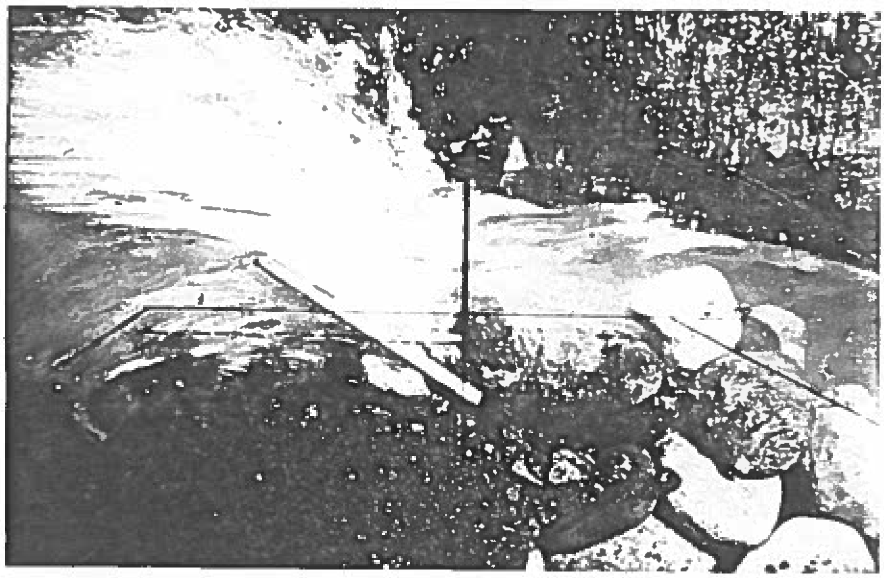


Fig. 1. Hirtshals East-breakwater.

contributing factor to the damage was morphological changes having occurred due to an extension of the main breakwater in 1970-71. Fig. 1 shows typical profiles off the breakwater in 1972 and 1974. The extension of the main breakwater was made in order to reduce sedimentation in the entrance area as the "reservoir" west of the harbour was filled. The extension had the desired effect, i.e. accumulation of sand west of the harbour rather than in the entrance. As a natural side-effect lee-side erosion occurred in front of the west breakwater.

7. Model tests with irregular waves were conducted and they showed almost 100% similitude between model and prototype and on the basis of new model tests the breakwater profile was modified by deleting the wooden wave wall and by making the crest higher and with larger quarry stones. The modified design was finally constructed and has now proven to be acceptable during 10 years.

8. This case documents the need for precise bathymetry charts and for hydraulic model tests, but it further documents the strong need for obtaining the best possible estimates of the morphological changes which are an inevitable result of any harbour construction on a sandy coast. In this case the result was erosion which leads to damage, in other cases the result may be accretion which may result in less severe wave impact and consequently in overdesign if not considered in the design process.

Example 2 - Toe and Berm Problem

9. The stability of a toe or berm on a breakwater is vital for the stability of the entire structure. This may be documented from the two examples nos. 2 and 3.

10. First the breakwater in Tórshavn described in refs. 6 and 7, may be considered. The original profile of this breakwater as built in 1972 is shown in Fig. 2. The breakwater was originally designed using regular waves for model testing and under consideration of a design wave height of H = 4 m and T = 12 s. The failure of the structure occurred for H_s = 3.7 m, T_p = 13.8 s (refs. 6 and 7).

11. New model tests in 1984 proved that the failure was due to a number of weak points in the design, and that one of the major reasons for the failure of the main armour layer on the front side was the lack of a proper berm below level -5.0 m. It was obvious from the model tests that although the stones with an average weight of approximately 5.0 t should be able to resist H_s = 3.7 m without a severe damage, the quarry run below level -5.0 m was not stable for the largest waves with heights up to H_{max} = 6.8 m, i.e. much larger than the design wave height of H = 4 m originally considered. The erosion below level -5 m lead to failure of the upper part of the armour layer. This example further documents how the damage to a structure is an integrated problem. Once the support of the armour layer fails, the armour layer slides down resulting in more severe wave im-

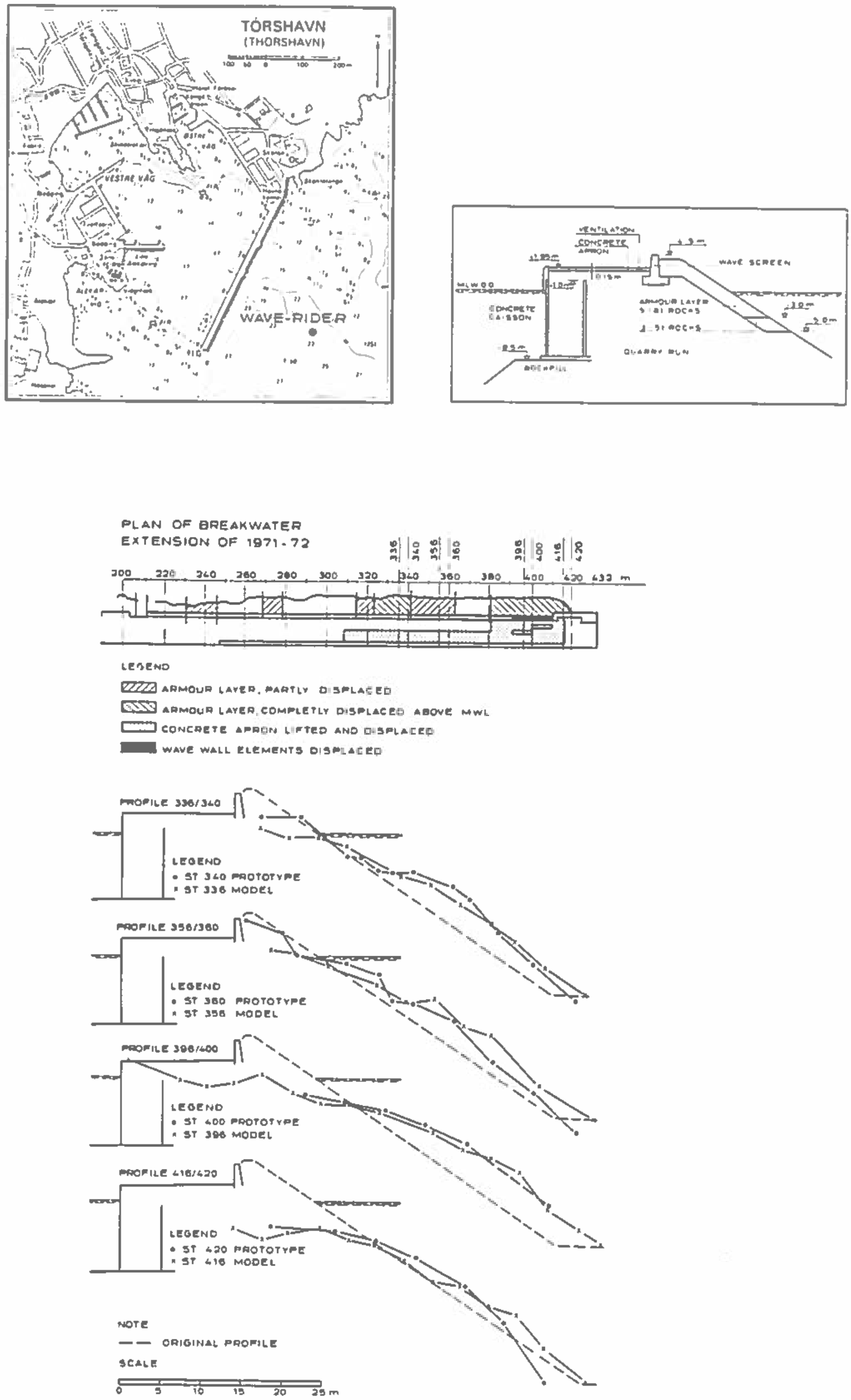


Fig. 2. Torshavn breakwater.

pacts on the crown wall and erosion under the superstructure. The result was severe damage to the wall that would probably otherwise have been limited if the "chain-reaction" had not occurred.

12. The understanding of this type of interrelated stability of the different structural elements of a breakwater is consequently important in breakwater design and the possibility for such effects should be minimized.

Example 3 - Breakwater on a Steep Rocky Sea Bed

13. Breakwater construction on a steep rocky sea bed always constitutes a serious problem for a number of reasons. First the waves on a steep bottom become larger than it is possible for a structure in the same depth, but with a more gentle foreshore slope. Secondly, a sloping bottom and especially if it is rocky means difficulties in obtaining a sufficient "toe hold" for the rest of the structure (berm or armour layer).

14. The breakwater at the Azzawiya Refinery in Libya was constructed in 1977. The profile of the breakwater appears in Fig. 3. The breakwater was a dolos breakwater with the dolos armour layer resting directly on the rocky sea-bed sloping about 1:10 seawards.

15. During a storm in January 1979, the entire seaward armour layer sustained severe damage and a very high percentage of the dolos units broke. The following main conclusion was reached regarding the reason for the damage:

- The poor interlocking of the dolos units at the breakwater toe on the rock bottom allowed the dolos to rock during storm and break themselves or other units upon contact. After large dolos pieces were broken off subsequent wave action washed the debris onto the undamaged units resulting in progression and spreading of the damage.

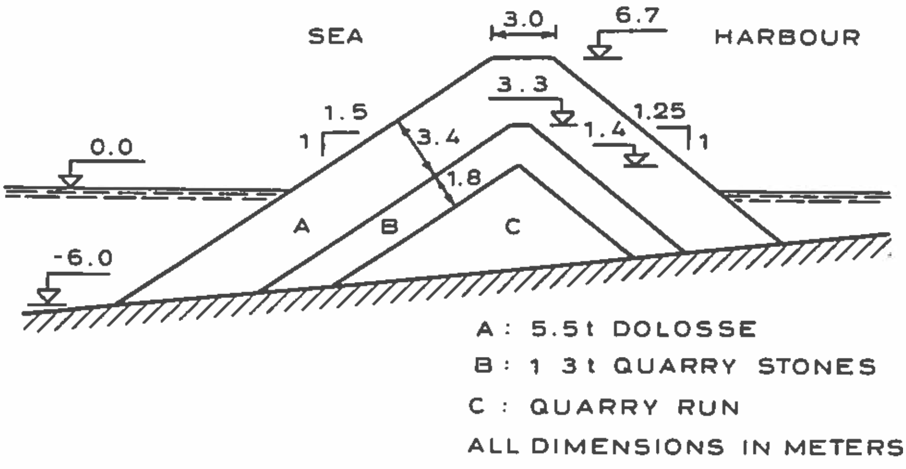
16. This breakwater is thus one of many where the fragility of the concrete armour units has proven to be of vital importance.

17. Many dolos breakwater applications with small dolos as the present of 5.5 t have been successful. This example proves the need for very careful consideration of the use of slender concrete elements such as dolos in cases that on any point are beyond the limits of past experience.

18. Tests were carried out for the repair work of the breakwater and through these tests a repair profile was developed. The profile is shown in Fig. 3. It appears that on the seaward face of the breakwater, the dolos of 5.5 t has been replaced by 16 t concrete cubes. Above level +4.0 m on the front side, on the crest and on the rear side all the non-broken dolos units were used as armouring.

19. The model tests showed that it was very difficult to obtain satisfactory stability of the toe of the structure

ORIGINAL BREAKWATER PROFILE



REPAIR PROFILE

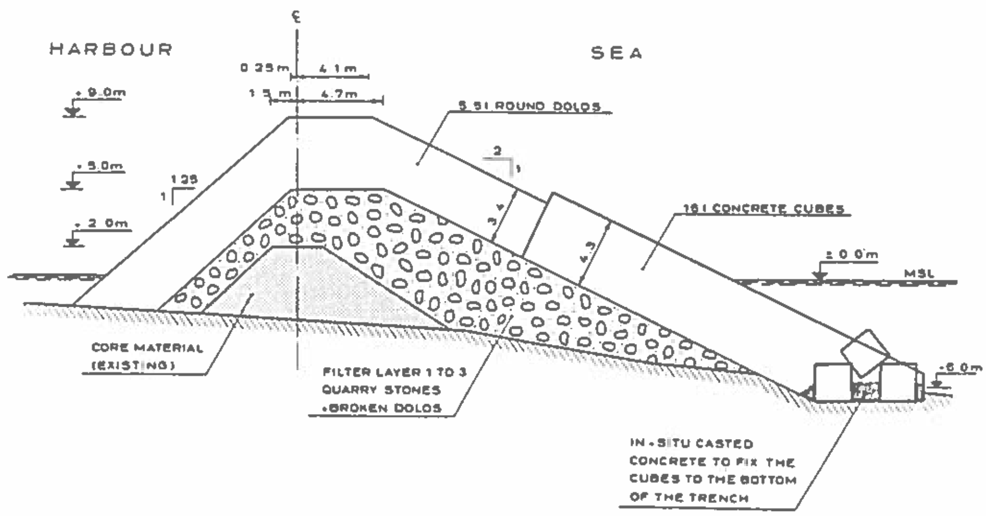


Fig. 3. Azzawiya breakwater.

founded on the rocky bed. Many different types of toe-hold were discussed and some made subject to hydraulic model tests. The type that was actually constructed appears in Fig. 5. It consists of a trench dredged through the cap-rock in which the construction of the armour layer began. After placing of the armour layer the toe of the structure was fixed with underwater in-situ concrete filling the voids in the trench. The repaired breakwater has proved to be stable during the last 5 years after completion of the repair work.

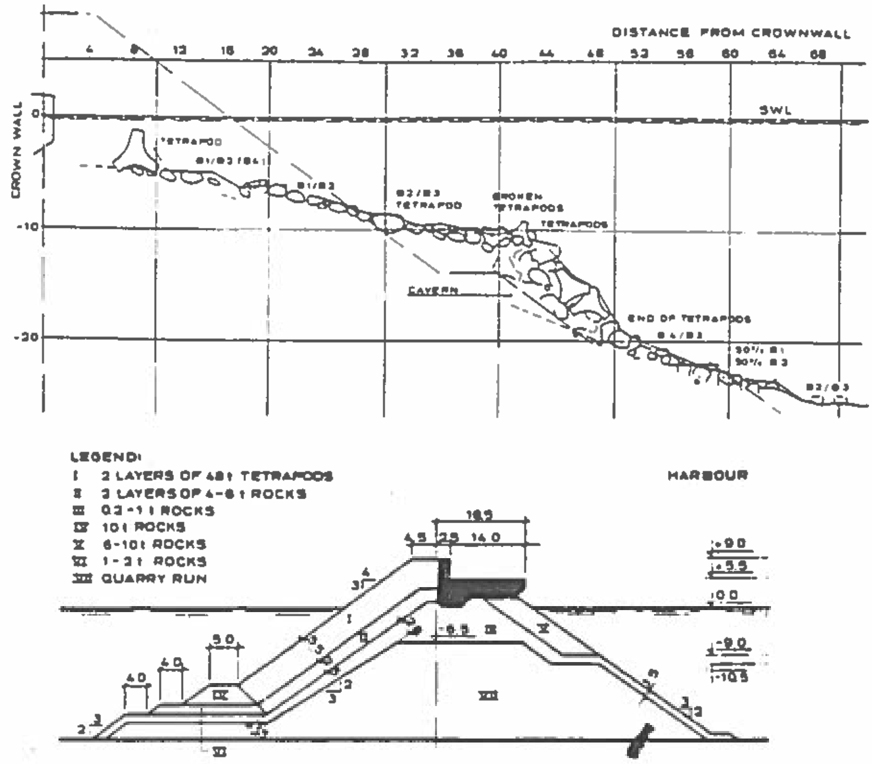
20. This example emphasizes the important aspect of obtaining a safe toe hold on breakwaters on a sloping sea-bed. Especially on a rocky bottom where the toe material can easily slide seawards there is a strong need for a carefully designed toe hold.

Example 4 - Fragility of Large Armour Units

21. During the last 5-7 years there has been a number of examples with severe damage or failure of large deep-water breakwaters armoured with concrete armour units. The main general conclusion when considering all the cases seems to be that the effect of fragility of large concrete units as armouring has not been fully realized in the design process.

22. In Port Arzew El Djedid the main breakwater suffered severe damage in a storm in December 1980 (ref. 1). The original profile of the breakwater, and a typical profile after damage appears in Fig. 4. The severe damage to this structure which involved both the main tetrapod armour layer and displacement and undermining of the concrete superstructure occurred most probably due to insufficient structural strength of the large tetrapods on the very steep slope of 1:1.33.

23. It appeared from monitoring of the entire breakwater that certain sections had resisted better than others, since the complete failure of the armour layer did not take place for the entire structure. However, the diver survey of the interior of the armour layer showed that just below SWL up to 80% of the tetrapods of both of the two layers were broken in certain sections. Model tests using traditional model armour units without scaling of the material strength clearly demonstrated that large settlements/compaction of the entire armour layer occurred during wave action. The settlements were on the average in the order of 2.0 m with values in the range 1.0 to 4.0 m when measured vertically at the wave wall. The settlements began for significant wave heights, H_s, in the range 2.5 to 5.0 m in repeated test runs, which is far below the wave height for which tetrapods were displaced from the armour layer. 2% of the model units were displaced from the armour layer for H_s = 6.5 m, which corresponds to a K_D-factor of 4.5 which seems low for tetrapods when comparing with the factor of 8.0 which was previously generally recommended. The wave heights for which the settlements occur can tentatively be considered as an important parameter for this type of breakwater. H_s = 2.5 and



Arzew Breakwater Cross Section

Fig. 4. Port d'Arzew El-Djedid breakwater.

5.0 m corresponds to "K_D-factors" of 0.25 and 2.0 which are very low for tetrapods. It would appear that proper design should ensure that armour units are solid enough to resist the high contact forces generated by wave compaction of steep armour layers.

24. From observations in the model it appeared that the settlements/compaction occurred during the run-up/run-down process of a single or a few individual waves. The whole armour layer above level -10 m approximately was simultaneously moving causing small adjustments of the positions of each indivual unit which resulted in the compaction of the upper 75% of the armour layer.

25. It is in fact in hindsight easy to understand that the placing of such large 48 t tetrapods will lead to a very loose packing density that is far from the possible maximum packing density, and that the vibrating effect of irregular wave attack will result in compaction.

26. It is very important to notice that most often the compaction occurred without any individual unit being displaced from the armour layer. The model did not similate the fragility of the units and consequently the model did not reproduce the severe damage observed in the prototype.

27. It is highly important for the future breakwater design that the nature of the settlements/compaction of armour layers with different slopes and types of armour units are examined through intensive model experiments and prototype observations.

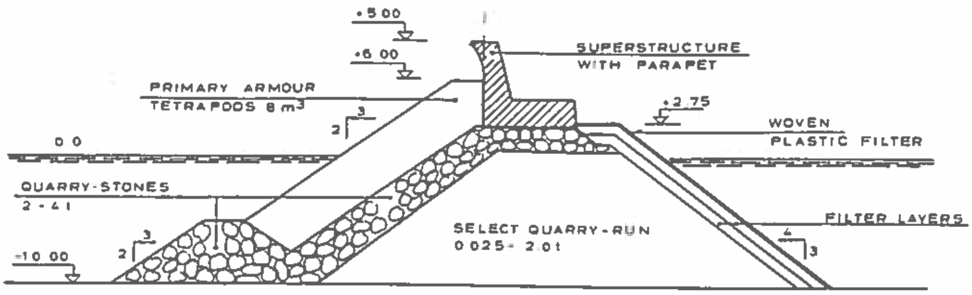
Example 5 - Wave Wall and Problems with Reclamation Filter

28. Many breakwaters have been designed with a wave wall or superstructure, and new designs are still made with superstructures (ref. 2).

29. This type of impermeable element incorporated in an otherwise permeable mound of different materials constitutes a special problem.

1) The superstructure requires a better foundation with minimum settlements compared to a crest of loose elements. The concrete is very fragile and damage to a superstructure can both occur due to excessive wave forces, erosion in front of the wave wall, and especially due to differential settlements causing high contact pressures at contact between individual superstructure elements.
2) The design of the superstructure is a very complex hydraulic geotechnical and construction material problem since the wave forces and the stability of the superstructure are related to the stability of the armour layer in front of the superstructure and to the permeability of the underlying material.

30. If a breakwater is permeable the transmission through the breakwater may endanger the stability of the reclamation behind the structure if the filter between the reclamation



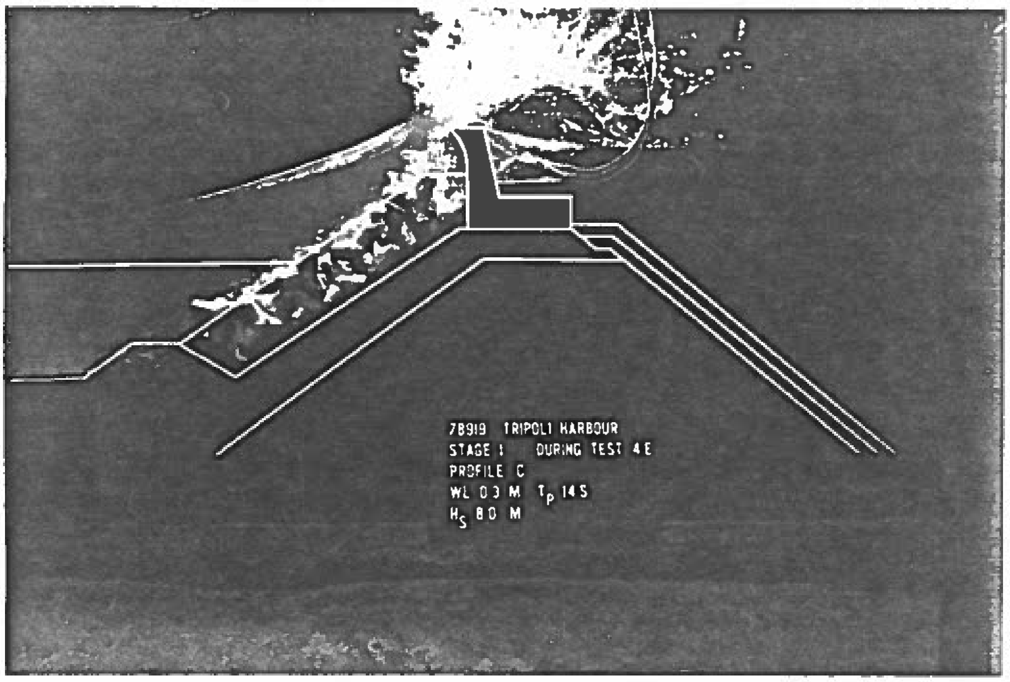


Fig. 5. Tripoli breakwater.

and the breakwater is not properly designed.

31. The main breakwater in Tripoli in Libya was constructed in the seventies. A typical profile appears in Fig. 5. After completion of the breakwater the harbour reported serious problems with overtopping (ref. 4) and "venting" under the superstructure and damages to the reclamation behind the breakwater due to the formation of sink holes.

32. Hydraulic model tests were conducted to find a solution to reduce these problems. It became clear during the tests that the breakwater was severely underdesigned and the model tests made a forecast of severe damage to the tetrapods, the superstructure and the reclamation which indeed happened shortly afterwards.

i Especially it is important to consider the large quarry stones of 2-4 t placed under the superstructure. This layer is a funnel for the propagation of water pressure to the rear side where it created problems for the reclamation. In addition to the large stones, the core material was specified not to have stones with weight below 25 kg ($d \simeq 0.2$ m) which further added to the permeability of the breakwater and to the problems with the reclamation.

ii The filters introduced on the rear side of the breakwater were not adequate to prevent the silty sand used as reclamation material to be washed into the core of the breakwater and thus creating the sink holes on the rear side.

33. This example emphasizes the strong need for high quality hydraulic model tests for any project of importance and it documents that concrete superstructures (as for Port d'Arzew El Djedid) are a very troublesome element to introduce in a breakwater. It should only be introduced if absolutely necessary and then it should normally be founded directly in the quarry run material used for core construction.

34. It further emphasizes the need for careful consideration of the criteria to be used for the lower limit of stone sizes to be used in the core. In many cases a screening of the quarry run can be avoided as done many times for example on the Faroe Islands (ref. 5). The example further emphasizes the strong need for careful consideration a filter between a breakwater and a reclamation. In fact this has in a number of cases proven to be a very critical design aspect which is not always carefully enough examined in the design process. If at all possible it should be avoided that fine fill is overlying coarse quarry run. This solution is an invitation to problems. It is much preferable to use more quarry run material to make the transition in the opposite way i.e. with the coarse quarry run overlying the fine fill. Then the requirements to the filter are less critical.

Example 6 - Gravel Filters

35. Also for the breakwater at Mogadishu in Somalia, gravel filters on the rear side of the breakwater have been the cause of problems. The breakwater project profile appears in Fig. 6.

36. The filter layers were actually constructed with a larger thickness and a flatter slope as seen in Fig. 6. Gradation curves for the two filter layers, the reclamation fill and the quarry run in the breakwater core appear in Fig. 6. The fill behind the breakwater was of two types, i.e. "dredged sand" and "dune sand" as seen in Fig. 6.

37. After completion of construction a loss of reclamation material through the core has taken place resulting in the formation of sink holes in the reclamation behind the breakwater. The sink holes have developed in a narrow band along the breakwater and just above the point where the MWL intercepts the transition between the reclamation fill and the gravel filters. The photo in Fig. 6 shows the sink holes in the pavement.

38. Table 1 shows the ratios between $d_{15,f}/d_{15,b}$ and $d_{15,f}/d_{85,b}$, where f denotes the filter, i.e. the overlaying material while b denotes the base.

Table 1

The above findings are compared with the normal requirements to geotechnical filters (Terzaghi) which states that $d_{15,f}/d_{15,b} \leq 20$ to 25 and $d_{15,f}/d_{85,b} \leq 4$ to 5. It appears that the dredged sand is just at the limit with respect to the $d_{15,f}/d_{85,b}$ criterion. Ref. 3 also deals with the problems of filters.

39. These examples show the difficulties in designing rubble filters and emphasize the importance of making a conservative design, when dealing with fluctuating hydraulic gradients, such as in the transition between a breakwater and a reclamation. Some research has been done by various researchers on the design of granular filters exposed to fluctuating flow, but none of the research is directly applicable to this situation. More applied research is encouraged to obtain safe guidelines, but in the meantime a conservative approach is recommended. In most cases the use of a geotextile in addition to the gravel filters is the most feasible solution.

CONCLUSIONS

40. The examples in the paper speak for themselves, but it seems important to emphasize the need for more wave (and water level) measurements at an early stage of projects. Most often the wave records should be supplemented by numerical hindcast modelling to extend the data base. Information on hydrographic and oceanographic conditions is the key to obtaining the best possible and most reliable design data.

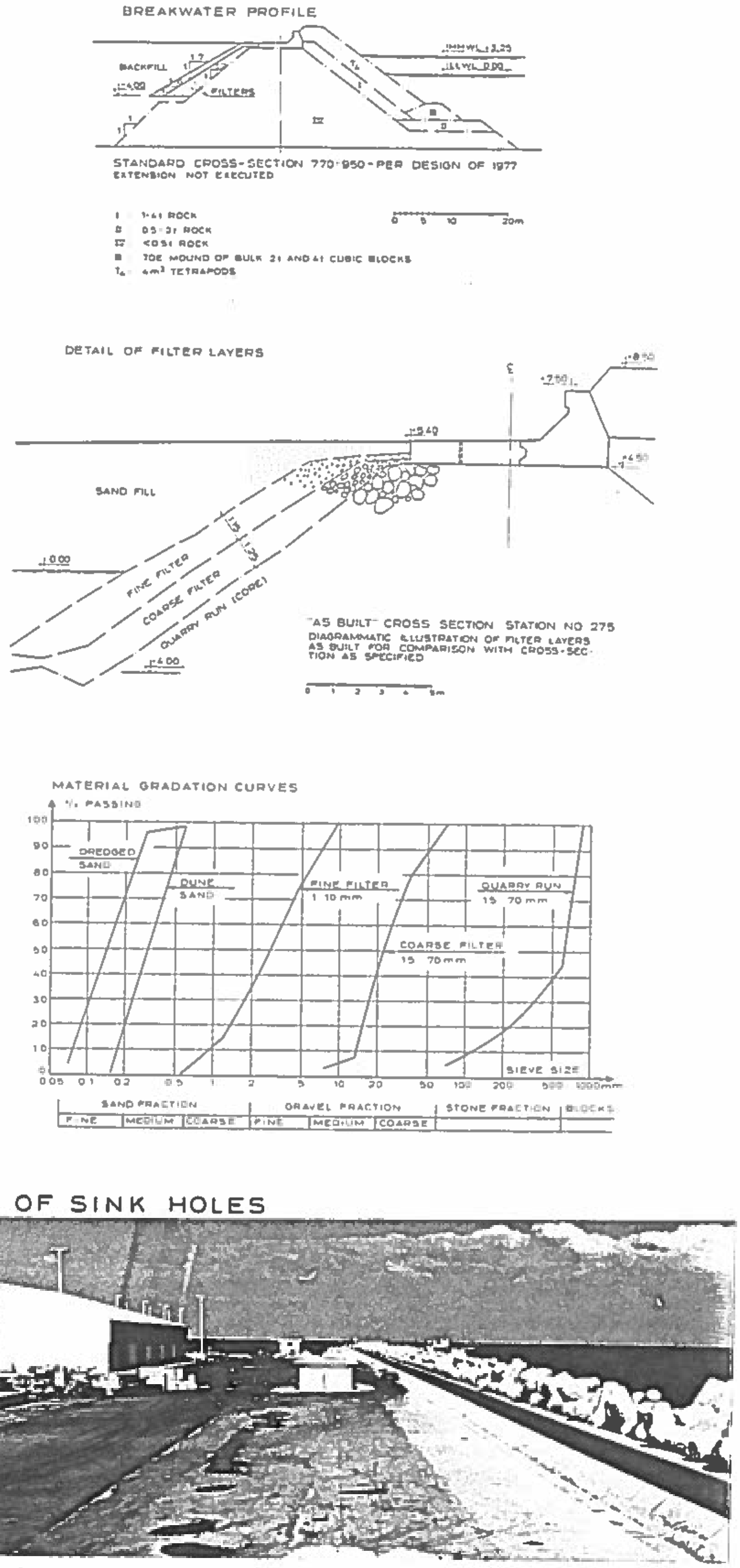


Fig. 6. Mogadishu breakwater.

most reliable design data.

41. The cost of hydraulic model tests is very small compared to the costs of construction, design and supervision etc. and is not the item to economize on in a breakwater project. It has now been proven that hydraulic model tests are a reliable tool in the design process and a close reproduction of the hydraulic effects that occur in nature provided the waves in the model are a true reproduction of the natural conditions (ref. 7). But it should always be kept in mind that the hydraulic model tests reproduce hydraulic effects only.

42. It is further of importance in the design process to incorporate as much experience as possible from previous breakwater projects. In this respect the failures, although unacceptable and costly for the owners, are of such great importance for the engineering community that it is moral obligation to publish such events internationally.

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Table 1. Analysis of gradations.

Material	d_{15} (mm)	d_{85} (mm)	$\frac{d_{15,f}}{d_{15,b}}$	$\frac{d_{15,f}}{d_{85,b}}$
Dredged sand	0.085	0.24	14.1	5.0
Dune sand	0.18	0.48	6.7	2.5
Fine filter	1.2	6.0	12.5	2.5
Coarse filter	15	44	11.3	3.9
Quarry run	170	780		

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Seminar on Rubble Mound Breakwaters, December 4, 1985

A CONTRACTORS VIEW ON RUBBLE MOUND BREAKWATER DESIGNS

BY

Ir. C.F.W. RIETVELD, DIRECTOR ROYAL BOSKALIS WESTMINSTER

1. INTRODUCTION

Regarding the subject " a contractors view on rubble mound breakwater design some might say :
the contractor has no view at all, or his view is completely blurred by the dollar marks in his eyes.
Now, when asked if the contractor has a view the first approach should be to regard the contractor as a partner of all parties involved in breakwater construction.

These parties are :

1. The cliënt who has to supply the terms of reference about what he wants, to provide funding and to maintain the structure after completion.

2. The engineer or consultant who has to translate the cliënts wishes in a design, draw up the contract - at least the technical part of it - advise the cliënt in the tendering process and eventually controls the execution.

3. The contractor for the construction of the breakwater with the required quality , within the required time set by the contract and for the price of his tender sum.

   In this way the obligations and responsabilities are neatly defined and seperated.
   According to this model the main contributions of the contractor are quality, time and costs. (if costs can be seen as a contribution !)
   The subject however is related to the contractors opinion on design.

   So, in the first place the interaction between the design and the elements quality, time and costs of a rubble mound breakwater will be dealt with.

   The subject will not be covered completely, but merely some aspects will be highlighted.

2. QUALITY

The quality of the construction has to be measured against the quality required by the design.
By quality is meant the quality of materials, structural strength and dimensions.
The question is then if this can be achieved in function of :

a. The geotechnical and hydraulic conditions which affect the construction, generally the environnement in which he has to work.

b. the materials available or supplied by third parties.

c. the tolerances accepted on the theoretical dimensions.

d. the workmanship of the contractor.

a. Unfortunately in some cases the design does not include enough or suitable information about geotechnical and hydraulic conditions which can be expected during the construction period or can affect the construction itself.

If for instance the tender documents have the following clause, quoting from a real tender document :

" Soils investigations in the area of site have been carried out over the past four years by three separate foundation engineering contractors.
Some of these bore holes are located on the drawings and the relevant soils reports are included in the contract documents.
It is the contractor's responsability to make such additional investigations and surveys as he deems apporpriate and necessary to satisfy himself.
Neither the engineer nor the employer will accept any responsability whatsoever with regard to the accuracy of any of the above information".

When on top of this the contractor is held fully responsable for all consequences of subsidence or slidings, we are in the case of " mission impossible ".

Wave climate is another element.
For the contractor the design wave is not so important.
On the other hand the wave climate he can expect during 90 % of the construction period is of eminent importance to choose his equipment, assess the workability and develop the working methods.
Also the planning of the construction phases, where more vulnarable parts of the unfinished breakwater are exposed to wave attack, depends on the day to day wave climate.

Currents also can play an important role.
In Zeebrugge for instance it was proven by operational tidal model studies, that to prevent scour the sea bottom had to be protected over 2 km in front of the breakwaters under construction.
Without these protective measures the breakwater would have had a greater construction height with considerable extra costs.

Who would be to blame ? And who to pay ?

Therefor it is strongly recommended that the design should include all these considerations during the construction phase and that the contractor gets all this vital information.

Construction heights should be indicated by the consultant, considering subsidence. Protective measures should be described and paid for.

To illustrate this, figures 1 and 2 give examples from Zeebrugge.

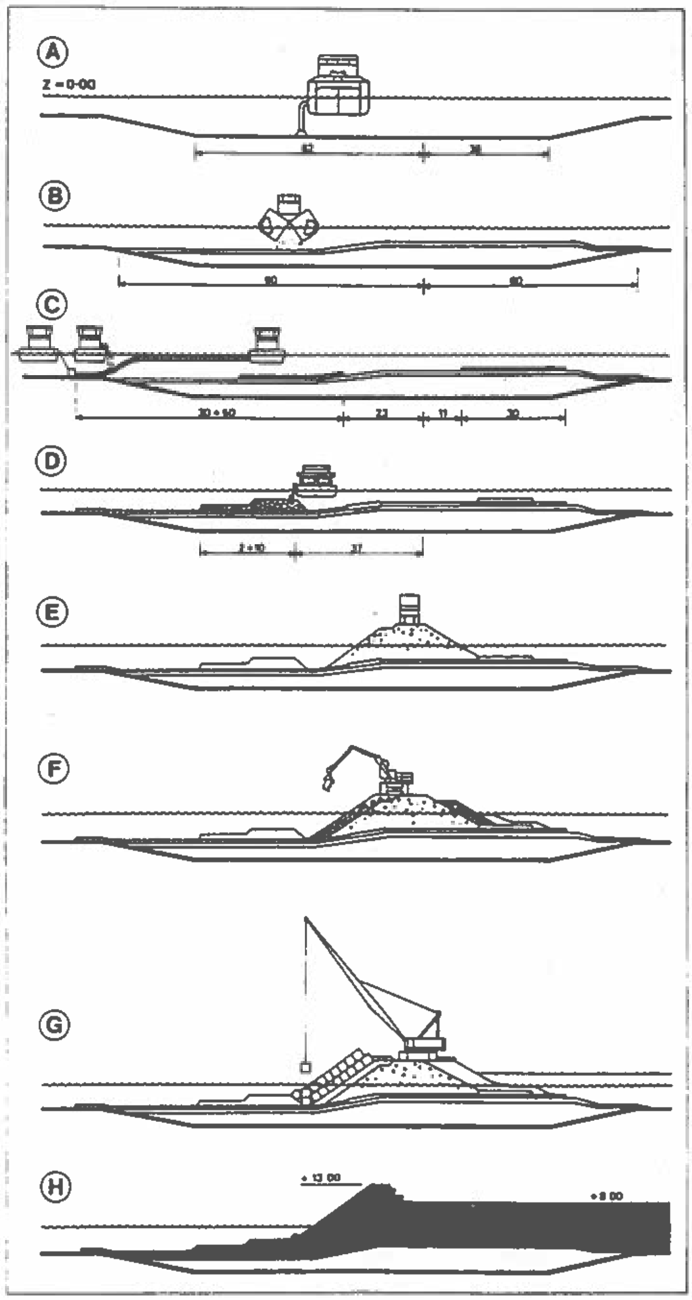
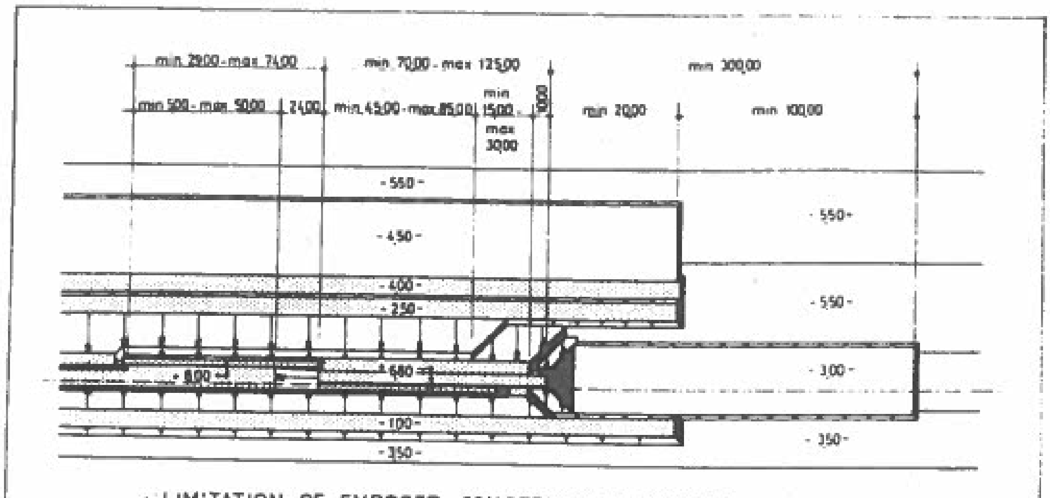


FIG. 1

All the different construction phases are such that each can be executed in a continuous process and is stable in itself.



LIMITATION OF EXPOSED CONSTRUCTION PHASES.

FIG. 2

The vulnarable core, which as the turtles head sticks out, should be as short as possible. So short in fact that if the weather forecast is unfavourable this part can be protected in one 8 hour shift by heavy quarry stone to survive the storm.

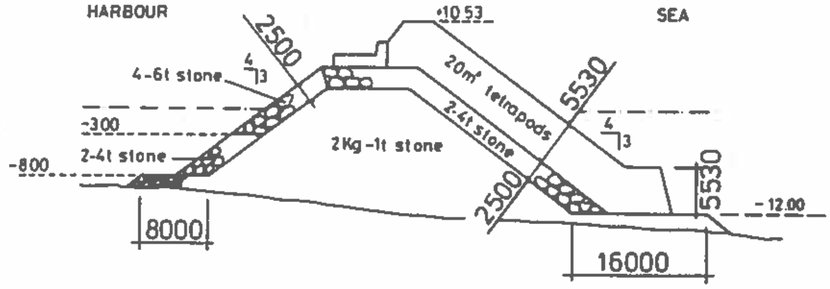
The maximal lengths of exposed structure are part of the contract and the temporarily protections are paid for by the cliënt.

b. Regarding materials quarry rock is the most important.
When new quarries, wether or not indicated by the cliënt, have to be opened it would be rather a coincidence when the overall quality and grading of the blasted rock meets exactly the prescriptions of the contract.

Therefor in the design stage this fact must be recognised and it must be known beforehand wether or not deviations are essential for the structure and alternatives must be studied.
If not a lot of time and money can be lost.

On the other hand a quick easy decision to go on with poor materials might be regretted afterwards.

c. The dimensions of the breakwater are a main feature of the design.



TYPICAL CROSS-SECTION THROUGH BREAKWATER
ANONYMOUS

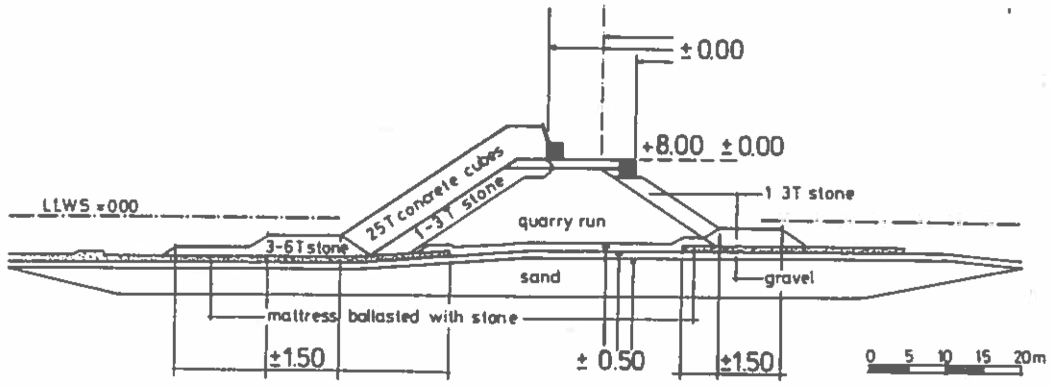
FIG. 3

When given in this way (fig. 3 real example) there might be a problem. A breakwater consisting of quarry rock and concrete blocks can not be built by the milimeter in an environnement like the sea.

Of course tolerances must be given which are realistic.
Too small tolerances are even dangerous because when they can not be met the stability of the structure might be affected.

In the design overlaps, connections, thickness of filter layers, dredging depths should be such that with realistic tolerances the structure is sound.

Figure 4 gives an example of this for the Zeebrugge breakwaters.



TYPICAL CROSS SECTION BREAKWATER WITH TOLERANCES

FIG. 4

The horizontal tolerances of ± 1,5 m are related to the waterdepth, working method, type of material and positioning system.
It should be noted that the depth at Zeebrugge is only 6m below datum.
For greater depths the tolerances might be greater or other working methods (more expensive) have to be adopted.

d. About the workmanship of the contractor we can be very short.

That is his full responsability.

However some contractors have to be protected against themselves.
A very low tender sum might be the result of a lack of workmanship and the consultant must be very careful in his advice to the cliënt about awarding the contract in such a case.

3. THE FACTOR TIME.

Time is a funny thing. One minute spent with your first love is a completely different experience than one minute spent at the dentist's.
In breakwater construction there also seems to be two different time scales.
The time spent between the start of the prefeasibility study and the production of the tender documents and the time given to the tenderers to present their offers are measured in different units.

The same holds for the time between tender date and signature of the contract and the time left to execute the job.

Nevertheless every month counts in the overall feasibility of the project.

A breakwater is mostly the initial project for a harbour development and by definition on the critical path.
One would expect that the construction time would get much attention.
Normally the construction delay is fixed in the contract and the contractor is free to shorten this if he sees an advantage to it, but is not given an incentive to do it.

When the economic importance of the construction time would be recognised, the design of the breakwater can influence this.
Two examples may serve to illustrate this.

For a classic design the seaward slope of our rubble mound breakwater is protected by concrete blocks.

As these blocks tend to be heavy and the placing pattern demands accuracy the progress of the dam front is defined by the time needed to place the blocks.
Each block takes about the same time to place and thus the progress depends on the number of blocks per meter dam to be placed.
A steep slope with a small number of heavy blocks permits a quicker progress than a gentler slope with a greater number of smaller blocks.

For instance when a slope of 1 : 1 $\frac{1}{2}$ with cubic blocks of 30 tons and a slope length of 28 m has an average progress rate of - 5,03 m/day, a slope of 1 : 2 with blocks of 22,5 tons would only progress an average of 3,76 m/day (using the same set of equipment).

This leads to a difference of 25 % in construction time.

It might even be profitable to overdesign the block weight to save time.
Later on it will be demonstrated that it might also save money apart from savings through time.

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Building a breakwater is in fact a transport and handling problem.

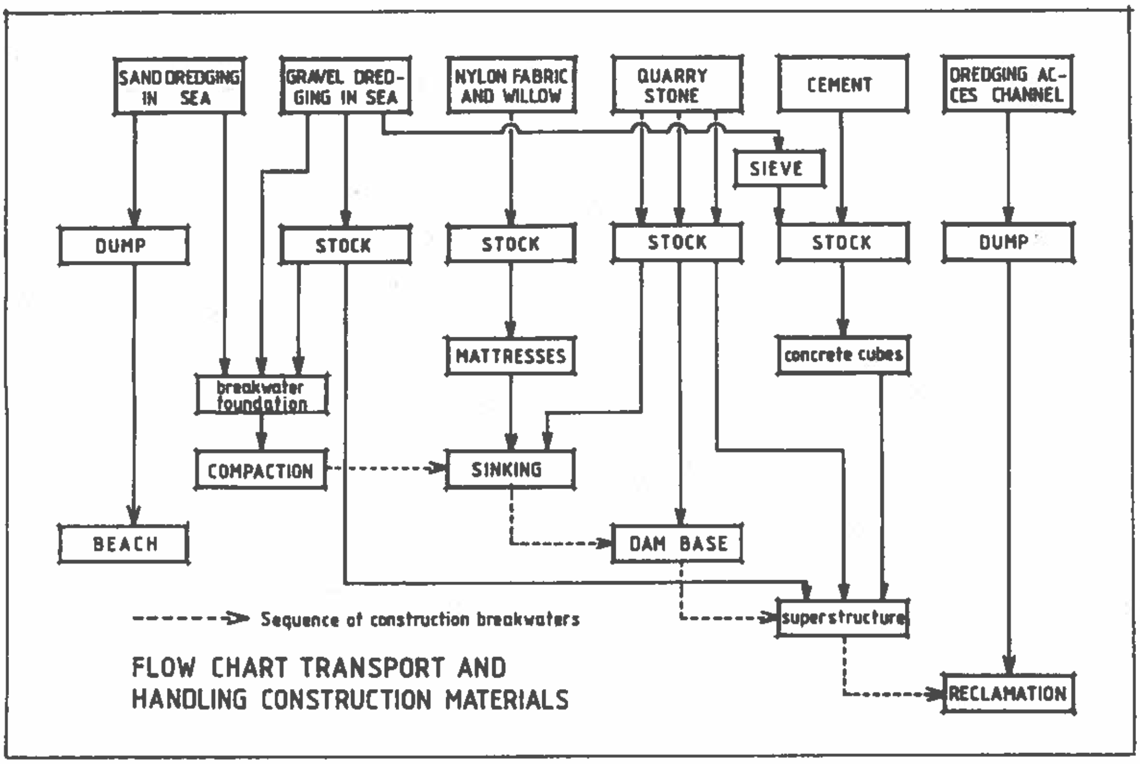

FIG. 5

Simplicity and uniformity in the breakwater design, limiting the different types of materials and the grades of quarry stone will help to speed up operations.
Also good facilities for transport roads, stock areas and handling will be a great help.

When not available the contractor sometimes has to spent a considerable part of his contractual time to build his own facilities.
In that case these facilities most probably do not serve in the future port project and have to be demounted after completion of the breakwater.

Therefor it may be advisable to have a general infrastructure constructed in a seperate contract while the design of the breakwater is still under way.

The ideal situation is that the facilities for the breakwater construction will serve as a part of the future port.

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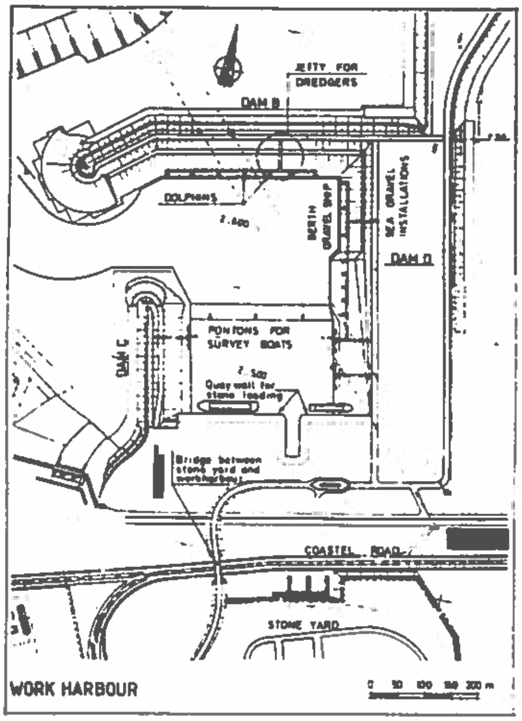


FIG. 6

The design of the work harbour in Zeebrugge meets both objectives.
The harbour was disigned as a future service port for tugs, pilots, salvage and eventual Ro-Ro traffic.

Stock areas and roads fit in the overall port plan.
The construction took place in 1978 when the design of the breakwaters was still under way.

4. COSTS.

There is a lot to say about costs. First of all the factor costs differs from place to place and makes every breakwater project unique.
It makes absolutely no sense to compare unit rates between projects.
Also the construction costs differ between different contractors because of their local presence, type of equipment , expenrience etc...

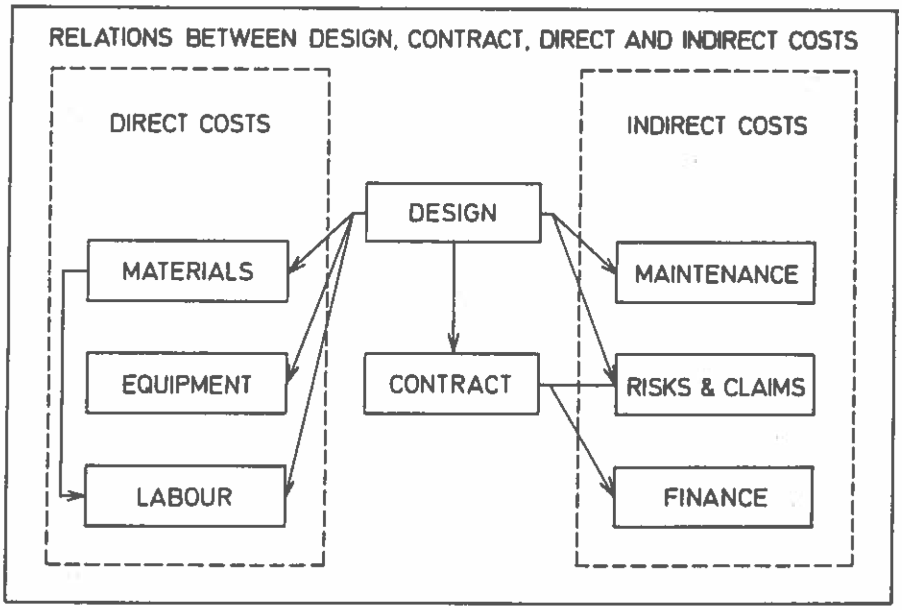


FIG. 7

Apart from all this the design itself has many influences of all kinds on the direct and indirect costs.

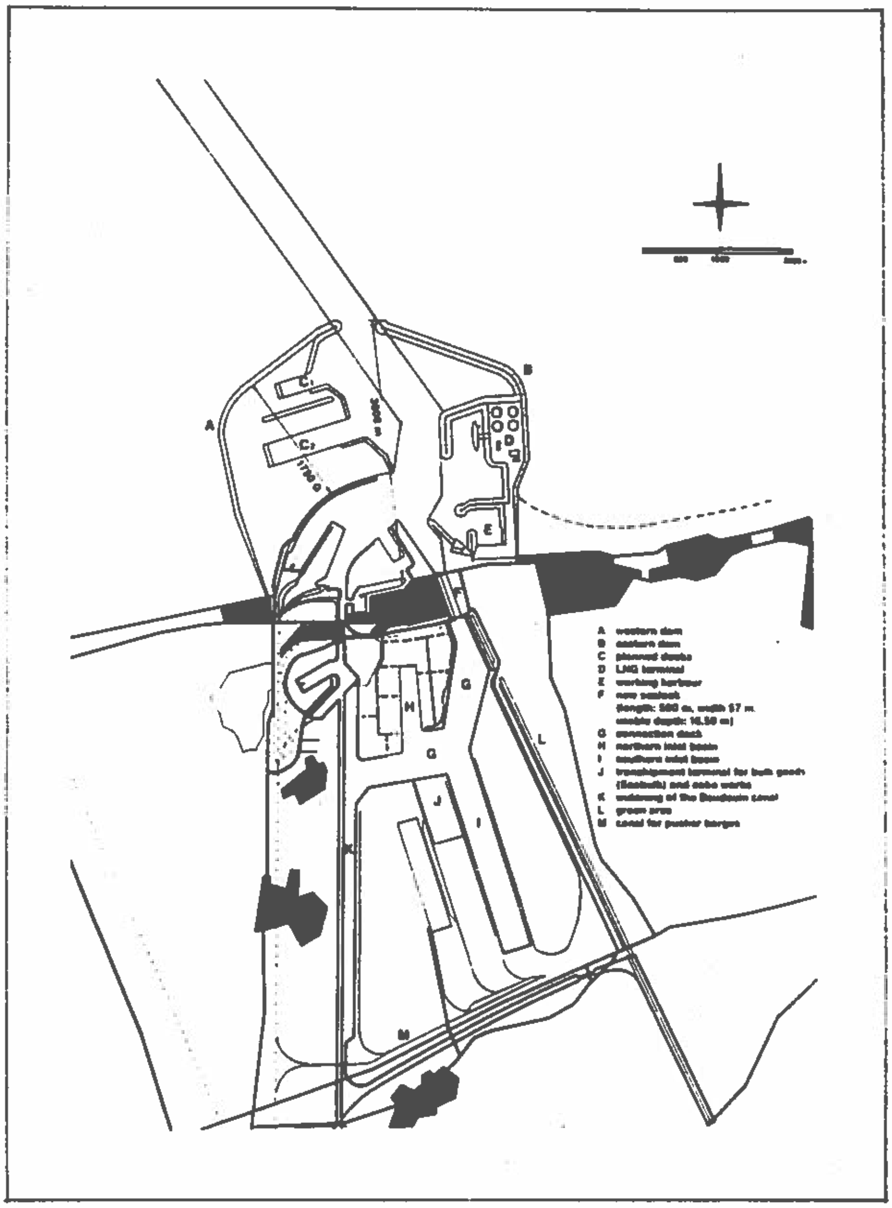


FIG. 8

To illustrate these influences some examples will be taken from Zeebrugge where recently some 14 km of rubble mound breakwater were constructed.

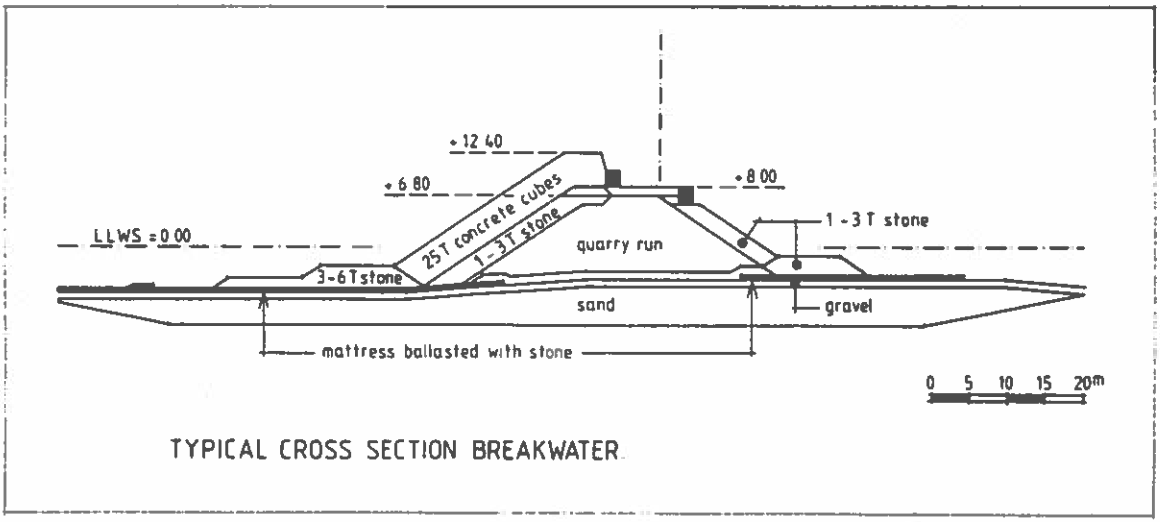


FIG. 9

The typical cross section is shown in figure 9.

Starting with the cost of materials it must be emphasized that all figures relate to Zeebrugge and have no general validity.

CONSTRUCTION COSTS BREAKWATERS ZEEBRUGGE	
Costs of materials per m^3 breakwater in % of core material	
Material	%
Sand	8
Gravel	67
Core 2/300 kg + 1/3 t	100
Secondary layer 1/3t	95
Armour layer 25t cubes	72
Berms 1/3t and 3/6t	120

FIG. 10

Per m3 of breakwater there is not a big difference between the several types of rock. The armour layer and gravel are relatively cheap and sand is very cheap.
Therefor sand was used up to the maximum level that current and wave action would permit.

CONSTRUCTION COSTS BREAKWATERS ZEEBRUGGE

	Costs in ‰ of total/m			Superstructure
	Materials	Execution	Total	alone in % per m'
Soil replacement + compaction	0	92	92	
Bottom protection	139	201	340	
Toe protection (berms)	58	42	100	
Core	204	63	267	57
Secondary layers	65	26	91	20
Armour layer	41	45	86	18
Cap construction	12	12	24	5
	519	481	1000	100

FIG. 11

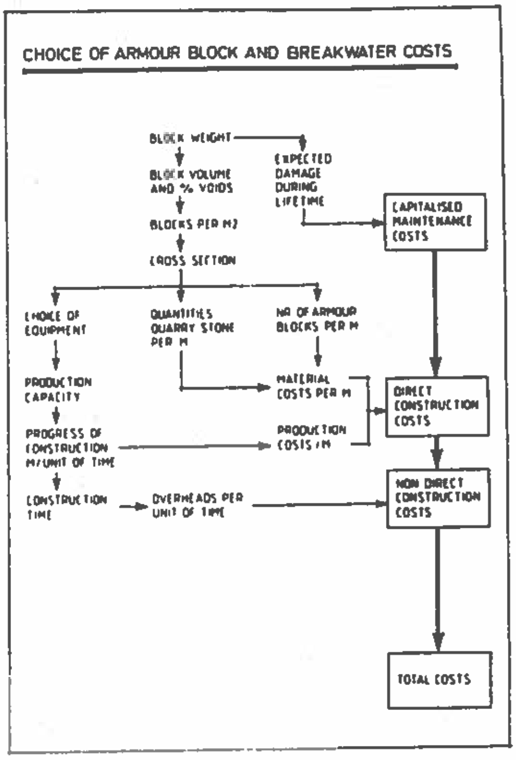
In the cross section of the breakwater the breakdown of the costs per element is shown in fig. 11.

As can be noted the core takes almost 60 % of the total costs of the superstructure and the armour layer only 18 %.

. During the past years much attention has been given to the design and behaviour of the armour layer.
The focus of these discussions being the stability and of course this is a very important matter because the armour has to protect the whole breakwater.
Much attention has also been given to invent block forms with high K_D values to reduce the costs of the armour layer.

Although the armour layer forms a minor part of the total costs, the choice of its elements has a great influence because it affects not only the construction costs but also the construction period and the maintenance costs.

This flow sheet demonstrates the repercussions of the choice of block weight on overall costs.



If for instance a 25 tons cube would suffice for stability reasons an overdesign to 30 tons cubes would give the following result.

	25 T	30T
capitalised maintenance costs	4 %	1 %
direct construction costs	87 %	88 %
total costs after overheads	100 %	97 %
construction period	100 %	95 %

In this special case the overdesign would save 3 % in money and 5 % in time.

Looking for an optimal economic solution, several block forms and block weights have to be introduced and run through the schedule.

Coming back to the core of the breakwater, more attention should be paid to its economics.

Economy can be achieved by

- partly using cheap materials like sand and gravel
- partly using waste materials coming from constructions to be demolished.
- using ungraded quarry run, as grading is costly.

These solutions call for a flexible design as the core material may differ from place to place.

Another economic aspect relates to the wave resistance of the exposed core during construction. When material losses or downtime have to be limited the material used for the core can be adapted to the wave conditions.
It might be economic to use a heavier grade of rock when wave action exceeds certain limits. The more costly material might pay for itself, cutting downtime.
This also has to be regarded in the design as the permeability of the core will be affected and so the pressure field inside the core .
The savings that can be obtained are illustrated with an example from Zeebrugge.

The total costs for the construction of the superstructure can be approximated by

$$C = Q \times u + n \times C_F + (n-x) \times Co$$

in which

Q = total quantity of stone in T

u = unit rate in fr/T

n = total number of available working days

C_F= fixed costs of operations in fr/day

$(n-x)$ = total number of workable days, x = days of delay due to unfavourable weather ; $(n-x) = \frac{Q}{p}$, p being the daily production in T.

Co = operational costs in fr/day

With for example : C_F = 7 milj. fr/day

C_o = 3 milj. fr/day

$Q = 6 \times 10^6$ T

p = 6000 T/day

the result is shown in the following table :

Quarry stone grade	Limiting wave height m	% of exce-dance	$n-x=\frac{Q}{p}$ days	n days	x days	u fr/T	$\overline{u}$ fr/T	C milj.fr	c %
2/300 kg 2/300 + 1000/3000 kg	1,2 2,5	20 5	1000 1000	1250 1053	250 53	600 800	600 632	15.350 14.163	100 92
		15		-197	-197		+32	-1.187	-8

As can be seen a cost reduction of 8 % could be achieved.

Labour costs are mostly dictated by law or national agreements between unions and employers.
In some areas where labour is abundant and cheap, special designs are required.
An example of this is the enclosure dam of the Feni river in Bangladesh. Sophisticated hydraulic engineering backed up this project but the design focussed on the maximal utilisation of manpower.
Roughly 12.000 men were employed at the crucial closing operations.

The purpose of the enclosure was the safety of lowlands along the Feni- estuary.
The tidal range averaged 4 m and heavy tidal currents had to be faced over a readily eroding bottom.

The final gap was 1200 m long and was closed by building up vertically a dam consisting of portable bags filled with clay.

Even for 12.000 man the walking distance was to great to carry all the bags in place in one tide. So first stock piles in the form of streamlined piers were laid in the closure gap.
To prevent scouring the bottom was protected with bambu mattresses, balasted with manually placed or dumped stones.

As many of our future breakwaters will have to be built in third world countries where foreign currency is scarce, mechanical equipment almost non existent and labour abundant our designers have a challenge in inventing hand made breakwaters.

Regarding the indirect costs (fig. 7) as risks, finance and the contract itself, those interested in these subjects can find a fine article by Mr. L. Fletcher in Terra et Aqua nr. 26.
The only point mentioned here about risks is the risk of storm damage during construction.
The occurance of storms can not be controlled, neither by the cliënt or engineer , nor by the contractor.
The easy solution is that the cliënt puts this risk on the shoulders of the contractor and that the contractor tries to find an insurance company to cover this risk and its premium will go into the tender Sum.

This may also be the most expensive solution.
In Zeebrugge the matter was discussed with the cliënt and the insurance companies and it was decided not to insure storm risks.
In stead maximum dimensions of exposed area during construction were agreed (see fig.2) and protective measures in case of unfavourable weather forecasts.
The temporary measures are paid for by the cliënt as well as the eventual repair of storm damage under the condition that the contractor respects the maximal exposure and the protection required by storm.
If not the contractor would be responsable.

Future maintenance and repair is the responsability of the cliënt but should be regarded as an element in the design.
It should be kept in mind that in many areas, once the contractor has left the site, little or no specialised equipment is available.
Placing additional heavy armour blocs on the seaward slope of a breakwater is not an easy job, especially not if the breakwater itself is unaccessable.
Normally the design is based on 2 to 5 % damage under design conditions, but how sure we are about the reality ?
Evenso 2 % damage means 1200 armour units in Zeebrugge.

Those who have attended the breakwater conference in London in october 1985 might have drawn the following conclusions.

- the collectors of the basic design data tell us they do not have enough data.
- the data processors say that they do not really know what they are doing.
- the modelling engineers list many limitations of their modelling techniques.
- the consultants use a variety of formulas in which they do not really believe.
- and finally the contractor starts constructing a rubble mound which might be heap of rubble before the job is finished.

And most astonishing of all, in this chorus of grief one very important voice is missing, namely that of the cliënt.
It is he who is putting a lot of money on the table in the expectation to obtain an important piece of infrastructure on which the development of a whole region may depend.

One of the biggest credits of this conference was that everyone was honest about it and did not pretend to master the problem.
Of course with the synergy of all and the many collegues who are working so hard that they could not be present, the matter will be solved.
But this will take some time.

In the mean time breakwaters will have to be constructed.
Keeping all this in mind we might have to introduce a concept in our designs which could be named.

" the most repairable construction ".

With regards to the design this means

- accessability, either from land of from the waterside
- keeping in mind that the costs of repairs and the local availability of equipment are closely related to the tonmeters to be employed
- rather overdesign than underdesign. Sometimes this will cost nearly nothing as might be the case for armour elements.
- repair stocks for elements that are difficult to obtain or will take a long time to produce
- a regular surveying system
- funding for future repairs.

Knowing our limitations but providing for it as stated before we might make our cliënts more happy than they must be now.

5. INTEGRATED DESIGN.

In the previous paragraphs some points have been raised that might demonstrate that the contractor could contribute in the design of rubble mound breakwaters.
Now the question is how to have access to these possible contributions, especially where in many cases the design phase and the construction phase are completely seperated.
In the design phase the project will be optimized on the basis of assumptions regarding the execution.
During the tender stage the tender price will be made, optimizing the costs on the basis of a fixed design or on an alternative of which the technical aequivalency is difficult to assess.
If design and construction could be optimized in one integrated design procedure, important savings can be obtained without loss of quality.
Also the feedback would be more accessable.

One way of doing this is the so-called frame contract.
This type of contract has been succesfully applied over the past 2 decades in the Netherlands and in Belgium for large and complicated projects.
After preliminary studies the cliënt starts a preselection procedure between interested contractors and after selection of the best group of contractors the design continues in teamwork between cliënt, consultants and contractors.

In the mean time preparational works can be carried out.
The total project will be subdivided in partial contracts for each phase of which the design is finished.

In this way optimal economic solutions can be found and if necessary new techniques can be developped.
The cliënt keeps flexibility in his project and can adapt the lay out or the design during the construction.
Also a lot of time can be saved as design and construction go hand in hand.
In Zeebrugge for instance the contract was signed in september 1976, including design.
After preliminary surveys and studies the works started in june 1977.
The working harbour, stock areas, roads and other facilities were completed in 1978.
The final layout and design of the main breakwaters was only available in 1979.
So about 3 years time were saved and this might be vital in the feasibility of such a project.

Examples of this kind of contract are the storm surge barrier in the Eastern Scheldt in the Netherlands and the outerport extension in Zeebrugge, Belgium.

A similar procedure was followed recently for the construction of the new Eastern breakwater at Calais in France.
On the basis of a preliminary design the cliënt invited pre-selected contractors to bid and to offer alternatives.
The most interesting offer was then futher developped together with the succesful contractor.
This procedure profits from the contractors contributions but takes a lot of time , because the design was virtually done twice.

6. FINAL REMARK

Finally as costs are an important matter, in our discussions with the cliënt in the prefeasibility stage we have to remember the " law of π ".

$$\frac{\text{final costs}}{\text{first (gu)estimate)}} = \pi$$