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The Lessons From Recent Breakwater Failures

developments in breakwater design

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Hans F. Burcharth

THE LESSONS FROM RECENT
BREAKWATER FAILURES.
DEVELOPMENTS IN BREAKWATER DESIGN

January 1987

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THE LESSONS FROM RECENT BREAKWATER FAILURES. DEVELOPMENTS IN BREAKWATER DESIGN

by

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INTRODUCTION

The design and construction of breakwaters is a classical task in coastal engineering. Breakwaters have been built, damaged and repaired for hundred of years. It is generally believed that coastal engineering science has improved a lot during the last 20 years. The profession was therefore taken by surprise when a series of major breakwater failures occurred very recently in a four year period.

In February 1978 the almost completed Sines breakwater in Portugal was severely damaged and in the following winter partly destroyed. In San Ciprian in Spain the newly built breakwater showed breakage of the main armour layer blocks after the two winter seasons 1979 and 1980. During a storm in December 1980 the new main breakwater of Arzew el Djedid in Algeria was partly destroyed. After two storms in January 1981 the just completed Tripoli breakwater in Libya was severely damaged. Also in the Italian harbour Gioia Tauro under construction extensive damage occurred at the breakwater in 1980. These accidents are examples from a long list of damaged rubble mound structures.

The subsequent analyses of these failures revealed that they are all different and not due to one particular reason, as will be discussed in the following.

If one should generalize it might be said that the design of these structures, all of a conventional type with concrete armour units, is the result of a somewhat uncritical extrapolation beyond experience of the breakwater design tradition, which started with the introduction of the Tetrapod some 30 years ago. The damaged breakwaters are very expensive structures with a typical length of 2 km situated in fairly deep to very deep water on exposed locations facing the North Atlantic Ocean and the Mediterranean Sea. Construction costs are typically in the range 10,000 - 100,000 US \$ per running meter and the cost of repair after some of the major failures are in the same order of magnitude.

EXAMPLES OF DAMAGED BREAKWATERS

The Tripoli breakwater, Libya

The extension of the Tripoli harbour included a new 4 kilometer long rubble mound breakwater built in the seventies along the seaward edge of a reef in shallow water, but close to deep water Fig. 1. Maximum water depth at the toe is only 12 meter at MWL. The maximum fetch exceeds 1000 km.

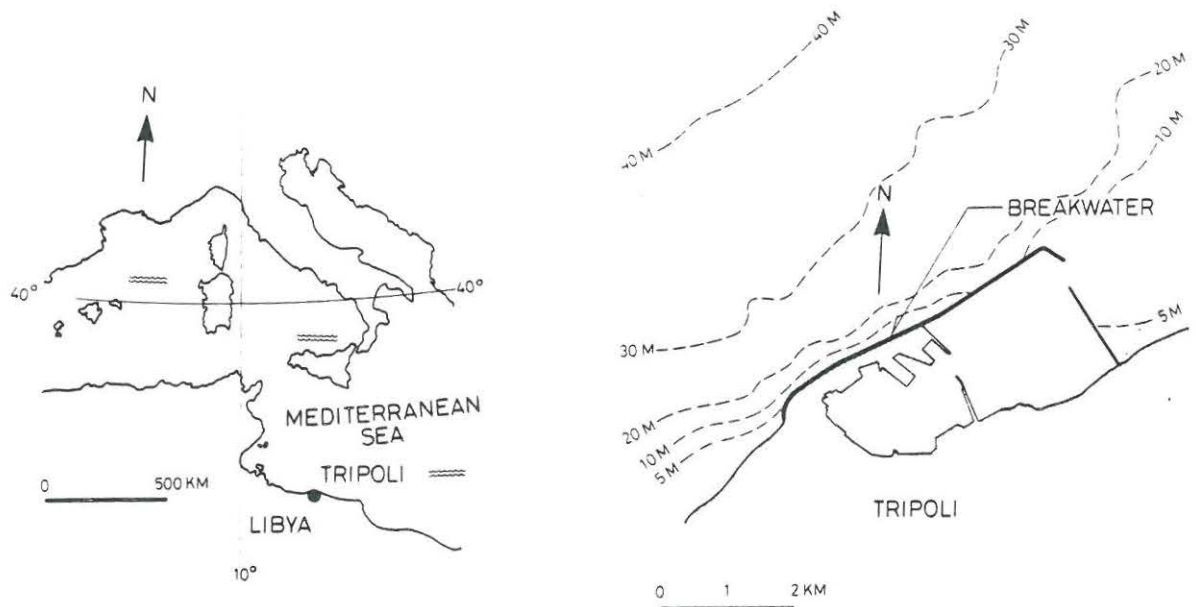


Fig. 1. Location of the Tripoli breakwater.

The area behind the breakwater was reclaimed and used for a double carriage roadway, storage areas and sheds. Fig. 2 shows a typical cross section. The main armour was 18 t Tetrapods supported by a berm of 5-8 t rocks. A concrete capping wall with the crest at 9 meter above MWL was placed directly on the filter layer stones of 2-4 t and the sand used as reclamation ma-

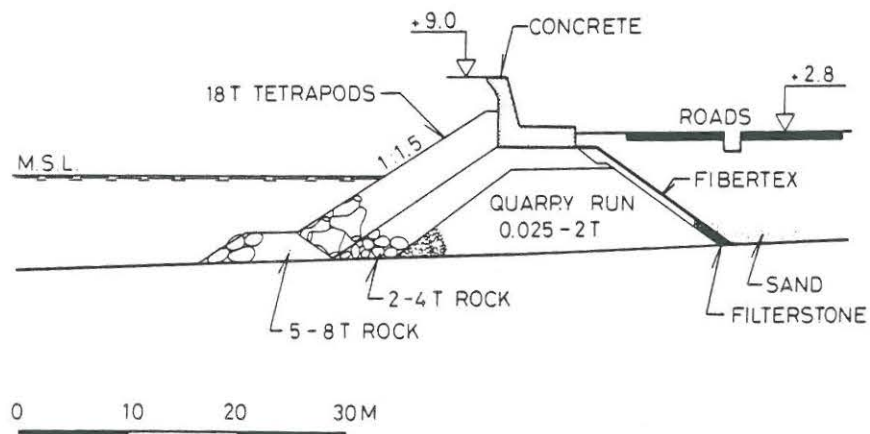


Fig. 2. Cross section of the original design. Tripoli breakwater.

terial was separated from the coarse core material of 0.025-2 t quarry run by a geotextile membrane and a filter layer.

Already during the construction period problems with overtopping and venting under the superstructure leading to sinkholes and damage to the reclaimed area were reported. The breakwater was severely damaged in January 1981 in a storm with an estimated maximum deep water significant wave height H_s of 8-9 meters and peak periods of $T_p = 13.5-14.5$ sec. However, the structure was designed for a H_s of only 4 meters! Naturally the damage was extensive consisting of the following failure modes

- Collapse of sections of the vertical part of the wave wall (designed as a gravity structure)
- Breakage of the Tetrapods
- Venting through the very permeable filter layer and core underneath the concrete base plate leading to blow out of reclaimed sand and complete damage of the roads
- Overtopping leading to flushing and consequent erosion of the reclamation and to damage to the sheds
- Some erosion of the berm.

The breakwater was redesigned on the background of the 100-year return period storm estimated to contain a deep water sea state of $H_s = 10$ m and $T_p = 14-15$ sec. No overtopping of solid water was allowed for sea states with less than three years' return period.

Due to the use of the reclamation area the critical factor in the redesign was to ensure limited overtopping. The chosen solution was in principle a very wide berm in front of the main armour besides many other means as shown on Fig. 3.

As seen from the figure there is a significant difference in size/volume between the original design and the redesign.

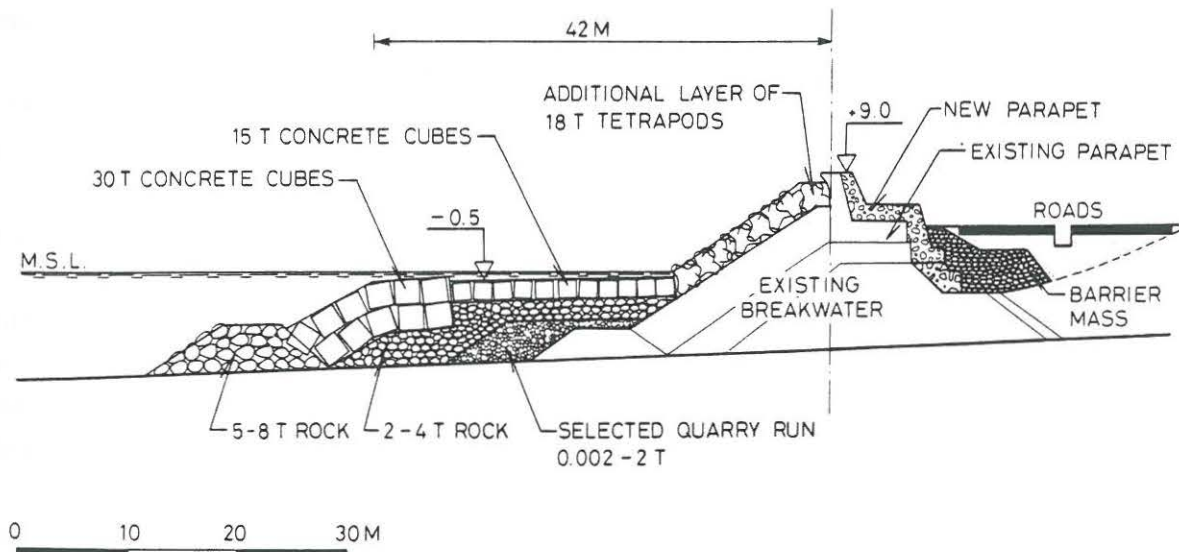


Fig. 3. Typical cross section of the redesigned Tripoli breakwater.

It is interesting to compare the redesign with what is probably the first proposal for the breakwater. The proposal, which is shown in Fig. 4, is the result of wave climate estimates and model tests performed in Sweden by Scandiaconsult.

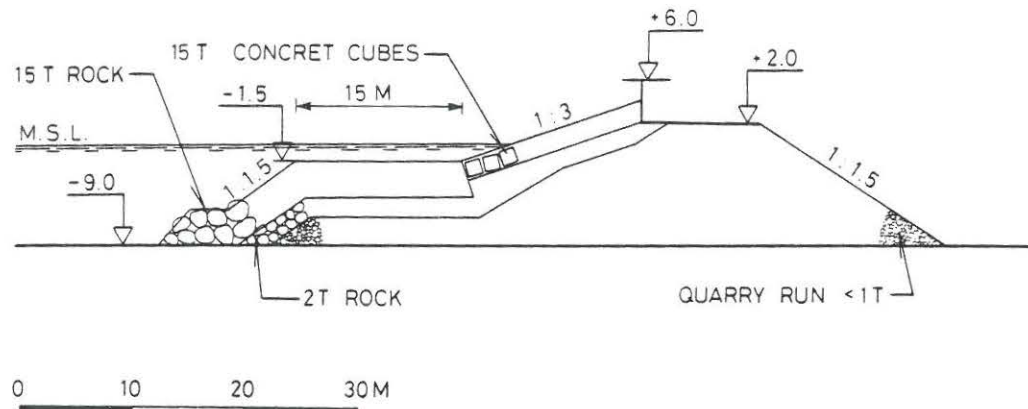


Fig. 4. Early proposal for the Tripoli breakwater.

It is seen that this proposal is very close to the redesign. On this background it is surprising that the much tinier design based on $H_s = 4$ m was chosen for the final design. The actual procedure behind this estimate is not known to the author but the reasoning might have been that the water depth of max 10-12 meters at the toe of the structure would limit the max wave height to app. 8-10 meters, which again (incorrectly) led to a H_s of 4 m, which was used in the model tests.

This sort of primitive reasoning is insufficient because, even in shallow water under strongly depth limited conditions, the deep water waves determine the shallow water waves through wave set-up, refraction, shoaling, breaking, etc., all very dependent on the sea bed topography and the deep water wave characteristics. It should be noted that also the wave periods are important because generally the shallow water wave height increases with the deep water wave period.

In a case like the Tripoli breakwater where overtopping is critical a correct assessment of the near structure wave climate including the SWL is very important because of the response sensitivity to various sea states. This is illustrated in Figs. 5 and 6 which show the influence of the sea bed profile on the near structure wave height and the influence of wave characteristics and the still water level on overtopping, respectively.

It should be mentioned that the parameter on the ordinate axis in Fig. 6 implies that overtopping is inversely proportional to the wave period — contrary to what is generally found for run up. However, the representation can be used in the actual case because the underlying experiments are performed with large waves in relatively very shallow water, where shoaling and breaking of the largest component waves reduce the influence of the period.

Another problem is the design of filters to secure separation of fine backfill material like sand from gravel fractions in cases where large dynamic (alternating/fluctuating) pore pressure gradients are present as for example in Tripoli. In such cases the often used geotechnical stability

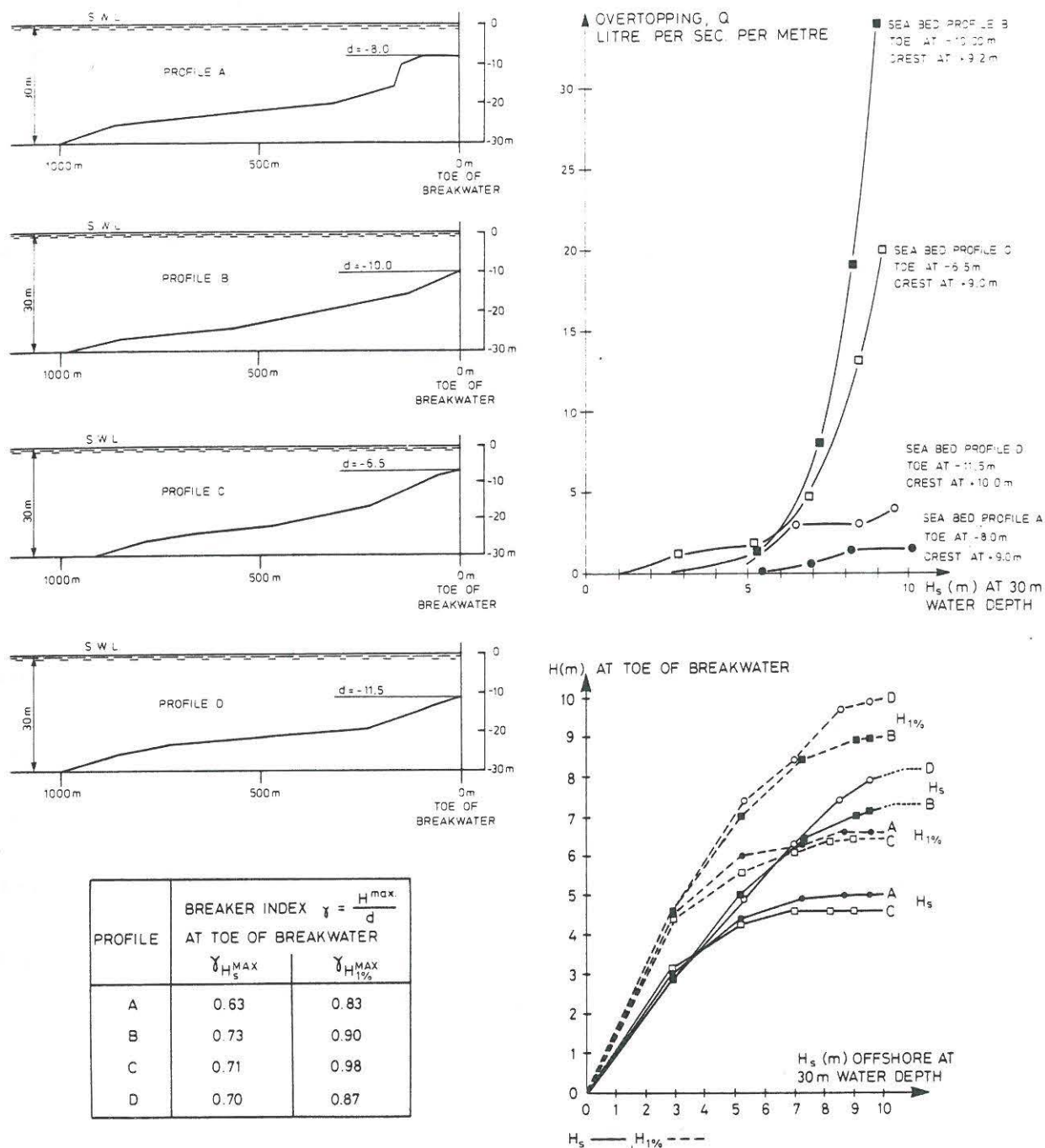


Fig. 5. Example of sensitivity of depth limited wave heights and overtopping to differences in foreshore bottom profiles. Delft Hydraulics Laboratory.

criterion given by Terzaghi for quasistationary conditions is not restrictive enough. On the other hand the criteria given in the Shore Protection Manual seem very restrictive and difficult to fulfill. Geotextile can often solve the problem but generally there is a lack of knowledge on dynamically stable filters. The problem is related mainly to small diameter soil components because the pore pressure gradient increases with decreasing grain diameter. This filter stability problem of venting

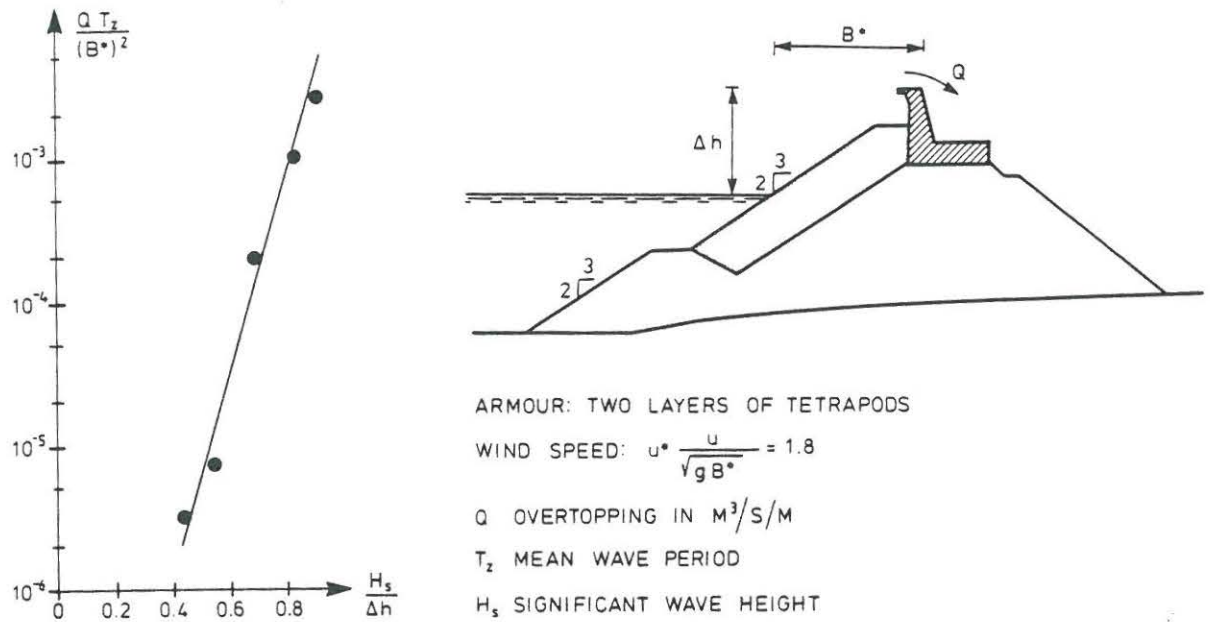


Fig. 6. Sensitivity of overtopping to still water level. Danish Hydraulic Institute.

with related lifting (erosion) of the soil cannot be studied in small scale breakwater models due to scale effects.

The Sines breakwater, Portugal

The very exposed open coast at Cape Sines some 100 km south of Lisboa was chosen for a major industrial development, Fig. 7. The harbour which includes berths for very large tankers is protected by a 2 km long breakwater constructed in water depths of up to 50 meter. Open fetch distances to the American continents are several thousand kilometers.

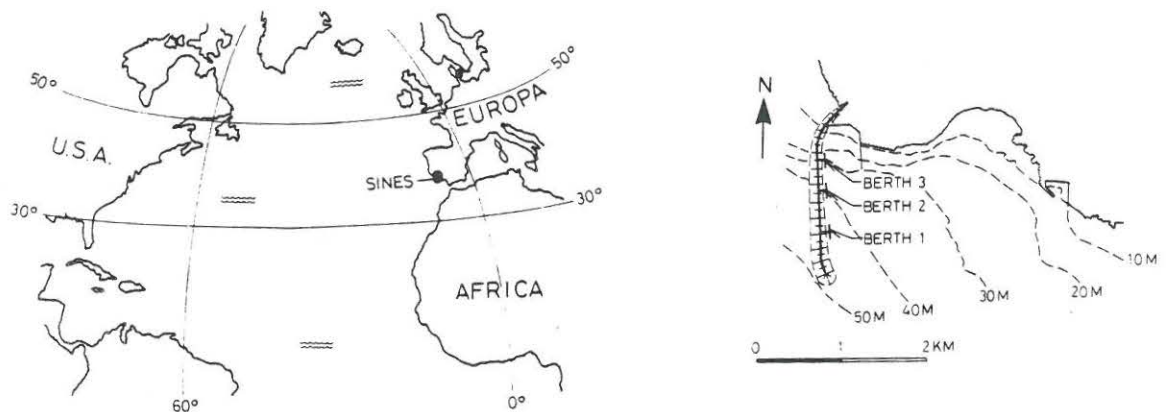


Fig. 7. Location of the Sines breakwater.

A typical cross section of the breakwater is shown in Fig. 8. The main armour was 42 t non-reinforced Dolosse of waist ratio 0.35 and mass density 2.35 t/m^3 . The heavy superstructure with the crest of the wave screen at +19 m above MWL was made of reinforced concrete.

The design criteria are summarized in the following Table 1, as given by Mettam, 1976.

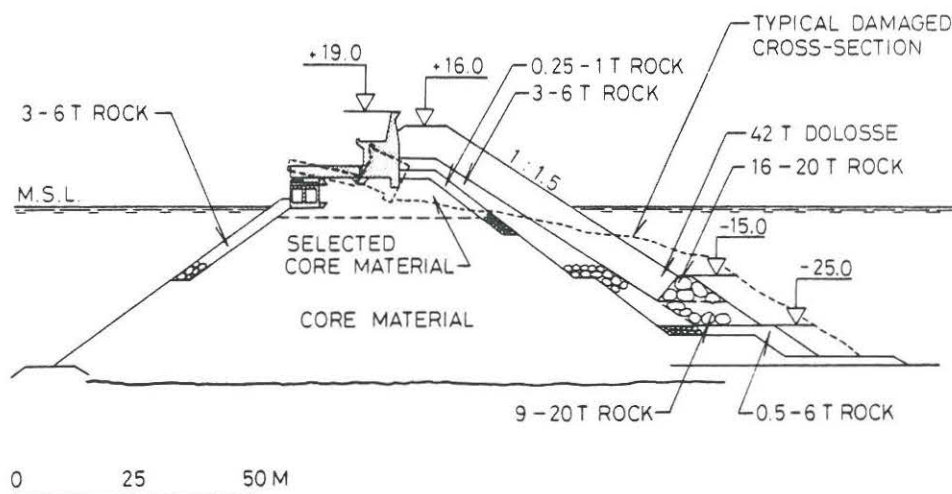


Fig. 8. Typical cross section of the Sines breakwater.

Table 1. Design criteria, Sines breakwater.

Storm return period (years)	Significant wave height H_s (m)	Dolos movement	Overtopping
1	6.5	nil	Begins with H_{\max} individual 10-11 m
10	8.5	Oscillation only	
30	9.5	Beginning of displacement	Severe overtopping with 15-16 m H_{\max} individual
100	11.0	1% damage	

This wave statistics was based on six years of visual observations at a near by location and two years of records off Sines, Campos Morais, 1974. Peak periods up to 16 seconds were considered in the design.

In February 1978 a storm with an estimated deep water H_s of 9 m and peak periods T_p of app. 18-20 seconds destroyed (broke and removed) the major part of the 10,000 Dolosse and eroded the seaward breakwater profile causing the superstructure to be undermined and destroyed. This failure is indicated in Fig. 8. In December 1978 another storm with $H_s \cong 8-8.5 \text{ m}$ and $T_p \cong 16 \text{ sec}$ occurred and in February 1979 the structure was hit again by a heavy storm of

$H_s \cong 9$ m and $T_p \cong 19$ sec. The last two storms destroyed most of the 5,000 Dolosse — some of which had a light steel bar reinforcement — placed as a repair after the first storm.

According to the design criteria such storms should not have caused significant damage to the breakwater.

The succeeding investigations of the failure revealed the following deficiencies:

- The limited mechanical strength of the large Dolosse was not considered in the design.
- The hydraulic stability (against displacements) of the Dolosse armour was overestimated.
- The storm wave climate (long term statistics) including refraction effects was underestimated both with respect to wave heights and wave periods.

The first point covers the fact that the relative strength of the armour units decreases with increasing size of the units, other things equal. This is generally known for all structural elements and was also discussed by Danel in 1960 in relation to Tetrapod armour units.

The second point was in fact discussed in 1974 at the Coastal Engineering Conference, cf. a comparison between the papers presented by Campos Morais and Brorsen et al.

As to the last point it can be mentioned that estimates of extreme wave statistics based on only few years of observation/records involve large uncertainties. 6 years of observation as used for Sines gives a statistical uncertainty in terms of coefficients of variation (standard deviation over mean) of not less than 25% on the 50 year H_s and 35% on the 100 year H_s .

Moreover, the importance of the wave period might not have been fully considered. Generally for deep water wave situations the run up and the damage increase with the wave period (up to a certain limit due to reduced flow velocities) other things equal. This is because more water per wave is brought on to the slope and consequently only a relatively smaller part can be stored in the voids. The result is higher overflow velocities and reduced hydraulic stability of the armour. The reduction in stability by long wave attack is therefore more pronounced for armour where the stability is mainly due to porosity and permeability, e. g. Dolosse. The important effect of armour layer permeability and porosity ("reservoir effect") is discussed in Burcharth, 1983.

For the provisional repair which was performed after the 1979 storm with the purpose of keeping berths no. 1 and no. 2 in operation the following wave statistics was established, Orgeron et al., 1982. Table 2.

Table 2. Wave statistics for Sines. Peak periods up to 24 - 25 sec.

Storm return period years	Significant wave height in deep water H_s (m)
1	8.0
10	10.5
30	11.75
100	13.0

For the provisional repair it was decided to use 90 t grooved cubes (Antifer type) as main armour units placed in a very wide horizontal berm and on a fairly flat slope in front of the still intact wave wall at the inner part of the breakwater, Fig. 9.

The grooved cube was chosen because of its relatively good mechanical strength and fairly good hydraulic stability. The 90 t presented the handling limit for the available construction equipment. It is interesting to notice that the enormous cross section in Fig. 9 is reasonable stable against sea states only up to $H_s \cong 10.5$ m and $T_p \cong 15 - 18$ sec and thus not completely satisfactory as a final repair. Already at $H_s \cong 8.5$ m significant movements of the cubes starts and for more severe sea states both displacements and breakage of the cubes will occur. The regular

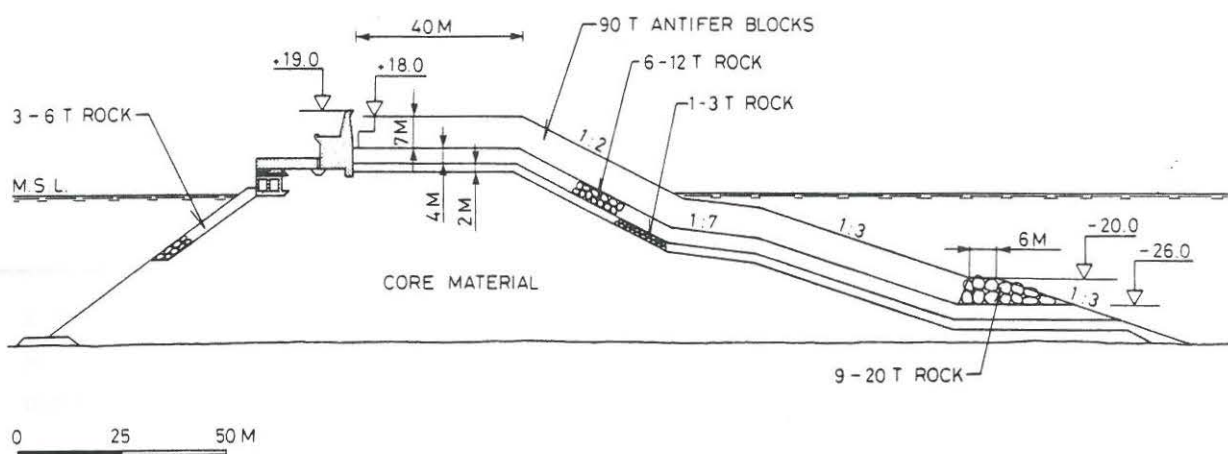


Fig. 9. Provisional repair of part of the Sines breakwater.

pattern in which the cubes are placed increases both the movements and the overtopping. Design conditions for the final repair is a deep water H_s of 13.9 m with peak periods in the range 16-24 sec (100-year return period) and a still water level of +4.0 m. Refraction will increase H_s to be typically 15.3 m at the major part of the breakwater.

The Arzew El Djedid breakwater, Algeria

The large port for export of LNG at Arzew El Djedid is protected by a 2 km long main breakwater constructed in water depths of about 25 m. Fig. 10 shows the location and the lay-out of the port.

The cross section of the main breakwater, which is shown in Fig. 11, has 48 t Tetrapods as main armour and was developed during model tests for a design sea state with $H_s = 9.8$ m and $T_p = 12.5$ sec from north-east corresponding to an estimated exceedence probability of 5% in 50 years (or a return period of app. 1000 years). The breakwater was severely damaged during a

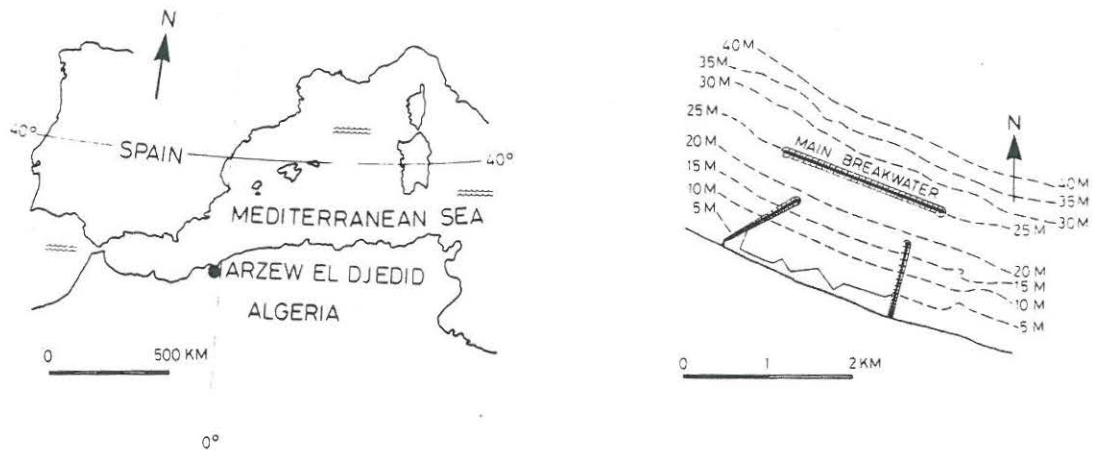


Fig. 10. Location and lay out of Port d'Arzew El Djedid.

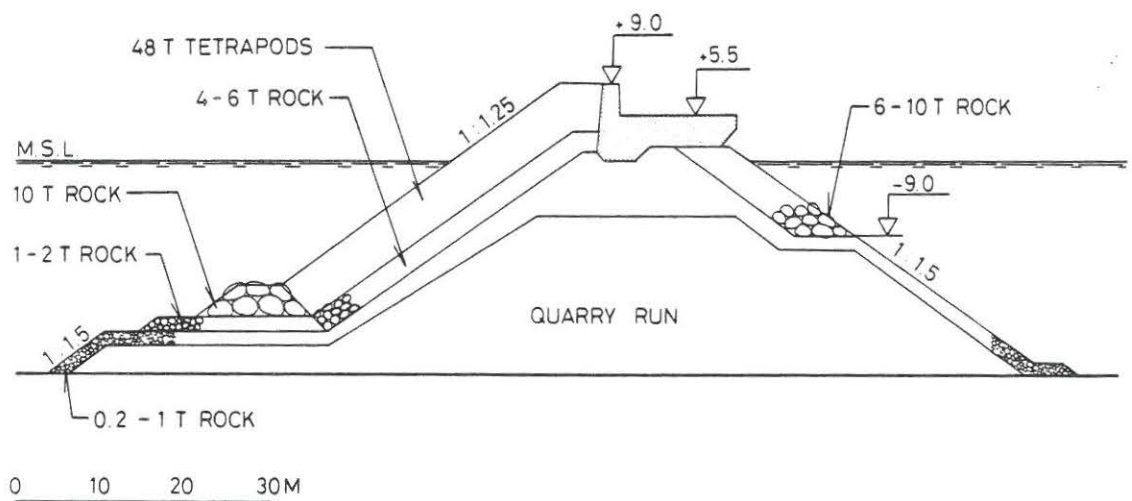


Fig. 11. Cross section of the Arzew El Djedid breakwater.

storm in December, 1980. A study of the damage is presented in Abdelbaki et al., 1983, Sørensen et al., 1985 and in Heijdra et al., 1984.

The sea state off Arzew at the peak of the storm was estimated by DHI to $H_s = 6.7$ m and $T_p = 18$ sec corresponding to a 50 years' recurrence period. In Heijdra et al., 1984, the storm condition is given as $H_s = 7-8$ m, $T_p = 13.5-14.5$ sec. Half of the trunk was severely destroyed with almost every Tetrapod broken and washed down the slope and subsequent undermining and destruction of the concrete superstructure. Along other parts of the trunk the profile seemed almost intact, however, with large settlements in the armour layer and a substantial percentage of broken Tetrapods (up to 80% below SWL) but still more or less in their original position, Fig. 12.

This indicates that the failure of the breakwater was due to breakage of the large Tetrapods and not due to hydraulic instability. It also indicates that most likely the interblock forces created by the settlement and compaction of the armour during wave action caused breakage of

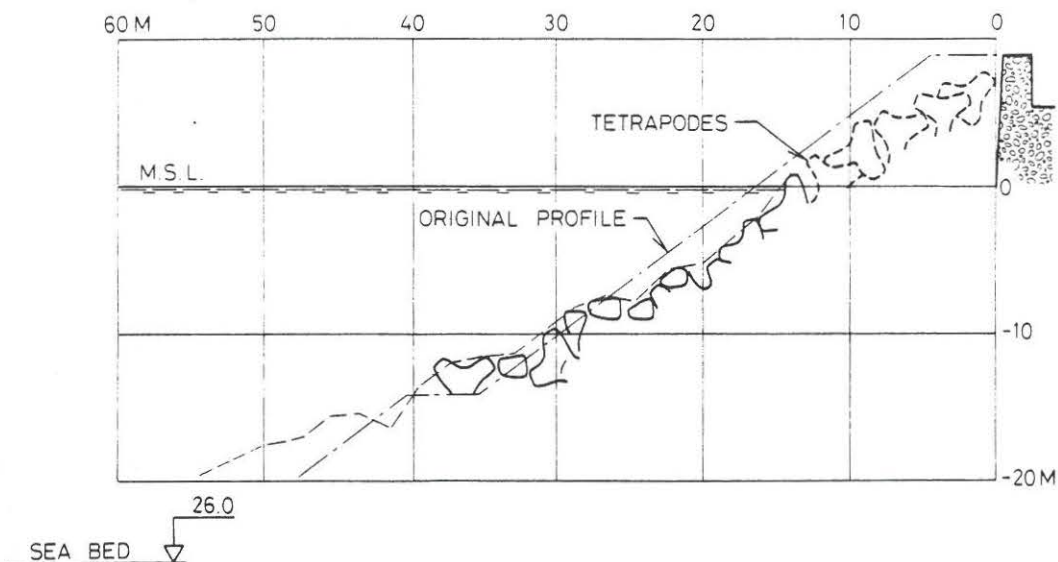


Fig. 12. Typical damaged section with large settlements of the armour layer and many broken Tetrapods still in place. Abdelbaki et al., 1983.

a major part of the Tetrapods. A large settlement of the very steep armour layer was also observed in the model tests. The model armour units were traditional and not scaled with respect to material strength.

GENERAL TRENDS IN THE RECENT FAILURES OF RUBBLE MOUND STRUCTURES

The construction of rubble mound breakwaters in deep water and at more exposed locations led to the adoption of very steep cross section profiles to reduce the volume of construction and to the use of large complex types of concrete armour units, such as Tetrapods and Dolosse, to save weight for easier handling during construction.

It follows from the many recent failures including the examples given above that in the design the following has not been fully considered:

- The very limited relative strength of the large unreinforced complex types of armour units, which in fact restricts the exposure of these units to limits often far below those given by the hydraulic instability.
- The large uncertainty usually related to the determination of the extreme wave climate statistics on which the design conditions are based. This is evident for deep water locations but has also some bearing on shallow water sites with more complicated sea bed topography.
- The very sudden collaps of the steep slopes with complex armour units (often enhanced by a wave wall super structure) not only in case of starting breakage of armour units but also in case of exceedence of moderate displacements of intact units.

- The very large design load exceedence risk if a permanent structure is designed for a sea state corresponding to a moderate return period (less than 100 years).
- Consequences of major failures.

The synthesis of all these points, which of course are strongly related in a balanced design, is that given the limitations in knowledge about the various parameters and processes the safety levels have been far too small.

Because of the stochastic nature of the involved processes a rational design procedure must be based on the probabilistic approach where uncertainties related to all major parameters (environmental conditions, loads, and structural response) are taken into account for the estimation of the risk of failure. The approach is applicable on several levels ranging from identification and assessment of safety factors on few and separately treated important processes to a full stochastic procedure based on the joint probability density functions for the involved parameters, see for example Nielsen et al., 1983, Mol et al., 1983, Franco et al., 1985.

THE STATE OF THE ART

The design procedure contains at least the following stages:

- Collecting of data about environmental conditions and establishing the related long term (extreme) statistics.
- Determination of design values for environmental conditions (waves etc.).
- Generation of alternative designs (from theoretical considerations and experience).
- Model testing of alternatives including optimization and selection of final design.

For many years it has been pointed out that of the four points mentioned above the first two represent the largest problems. It is the opinion of the author that this situation has changed within the last years. This is mainly due to the development of reliable *numerical models for hindcasting* of sea states from meteorological observations which means that even for locations where wave records are practically non-existing or only covering short periods a reliable long term statistics can be established as long as synoptic weather maps of reasonable quality are available. Very often meteomarine observations from ships are available too and can be used for calibration of the numerical model.

Moreover, a very important aspect is that the *uncertainty* related to the extreme event found by hindcast or records can be estimated. This in fact means that even when poor conditions for hindcasts are present this can be taken into account by applying a correspondingly relevant safety level, Wang et al., 1983, Battjes, 1984, Burcharth, 1985.

As an example it can be mentioned that the uncertainty given as the coefficient of variation (standard deviation over the mean value) on a 50-year return period significant deepwater wave height estimated from 15 hindcasted data set of extreme storms within a 20 years period will be in the order of 25-30% for areas like the Mediterranean Sea and the North Sea.

At the moment the largest problems related to the design process are the determination of the wave loads and the structural response (wave-structure interaction), inherent in the two last of the four points mentioned above.

Generally formulae to predict the loads and the response are few and cover only simplified conditions. Moreover, they do not contain any information on related uncertainty although the uncertainty is almost always considerable. An example is the Hudson formulae, which predicts the response of — but not the load on — armour layers of bulky units placed in two layers on filter layers on a straight slope. The wave height is the only hydrodynamic parameter in this formula which is not valid for complex armour units and broken profiles.

Another example is the formulae for wave loads (chock forces) on vertical walls which by comparison give such a dispersion in the results that the reliability generally speaking is questionable. Besides the fact that wave loads and corresponding structural response are of stochastic nature no generally applicable theoretical formulae can be developed as long as our knowledge about wave structure interaction is as limited as it is to-day. The existing formulae are empirical or semi-empirical and based on model scale experiments and few prototype observations/records.

Scale model tests are by far the most important design tool and necessary for practically all breakwater designs except some simple traditional minor structures.

But also in scale models there are serious problems for example related to armour layer stability. This is because both the load and the response in terms of distribution of stresses in the units and movements of the units must be determined and converted to prototype scale which is difficult as will be discussed in the following.

NEW DEVELOPMENTS

Armour unit integrity

The many failures of breakwaters where large complex types of armour units were used have resulted in a return to the use of bulky cubic blocks because of their relatively larger mechanical strength.

However, from a hydrodynamic point of view they are less efficient in dissipating wave energy due to the rather limited porosity and permeability of a pack of such units. Moreover, the strength of large massive blocks is also limited and their fragility is often enhanced by thermal stresses created in the hardening process due to big temperature differences generated by the heat of hydration. Although thermal stresses can be avoided one still has fairly tight limits for allowable movements (impact velocities) of large cubes (~ 50 t or more) if no breakage shall occur.

The higher the porosity and the permeability the more efficient is the armour layer if the forces can be resisted without breakage of the units. Fig. 13, which shows the three main categories of armour units, illustrates the importance of permeability/porosity.

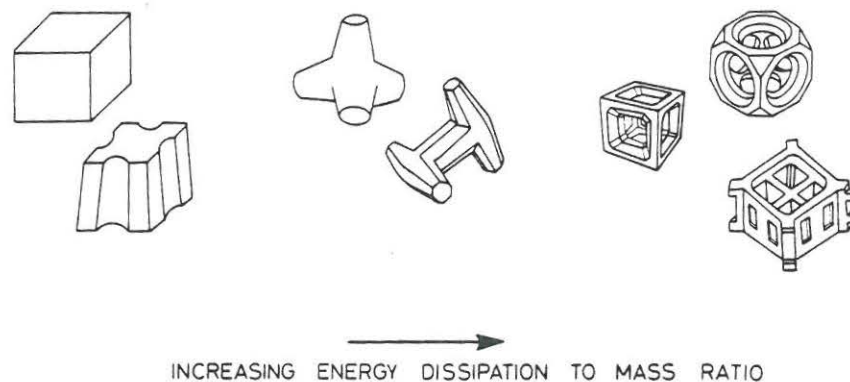


Fig. 13. Hydraulic efficiency of types of armour.

The hollowed block types need to be placed in patterns. They are successfully used for sea wall revetments but are not suitable for breakwaters in deeper water facing the open sea.

The massive cubic types can be improved by grooves and holes, Burcharth, 1983, which increases the armour layer porosity and the permeability and reduces the thermal stresses. The mechanical loads on such units can be assessed as done by DHL in Holland by measuring the impact speeds in hydraulic scale models and compare with prototype impact tests performed in the dry taking into account also fatigue, Silva 1983, Burcharth 1983, 1984.

The randomly placed complex type of units like Dolosse and Tetrapods are the most difficult to deal with in a design where both hydraulic stability and structural integrity must be assured. The problem is very delicate in the case of large units (20 t or more) because if a conservative no-movement stability criterion is adopted (which of course means small impact stresses) then the size (mass) of the units will be almost the same as for the cubic types. Moreover, for the large units the stresses induced by gravity and shake-down settlements can be very high and even exceed the strength of normal unreinforced concrete, cf. for example the Arzew breakwater failure and experience of the Danish Government Coastal Authority with breakage of 20 t Dolosse due to settlements caused by penetration into a sandy sea bed. It follows from this that a rather accurate assessment of both movements and forces/stresses is necessary to obtain a safe and economical design based on complex types of large unreinforced units.

The influence of the geometric scale (size of the armour unit) on the armour unit stresses is illustrated in Fig. 14, Burcharth et al., 1986.

The stresses due to flow forces and static (gravity) forces increases linearly with the geometric scale while stresses due to impacting units increases with the square root of the scale. Thus the relative importance of these stresses depends on not only the geometry of the units but also their scale and position on the slope. From this it is clear that a design criterion solely based on movements of armour units as observed in scale models (e.g. some percentage of the blocks rocking or being displaced) is not adequate unless extensive prototype experience is present (which is generally not the case).

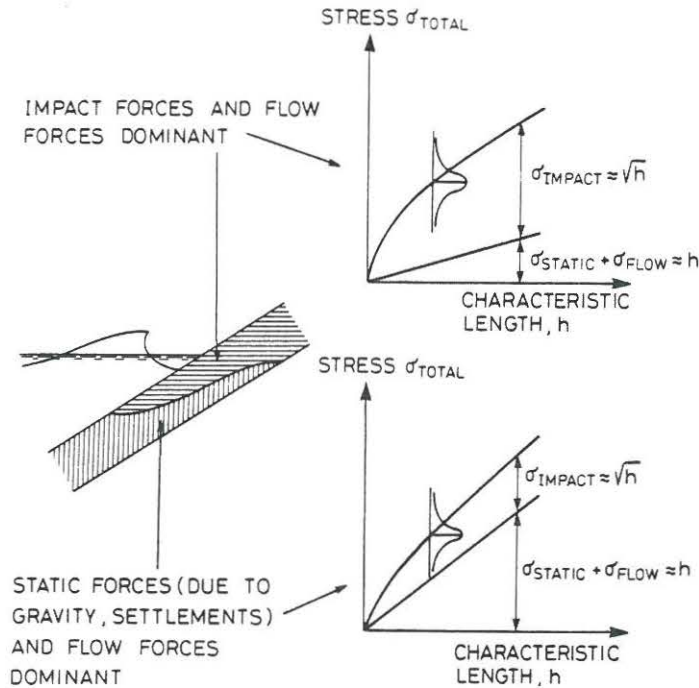


Fig. 14. Qualitative representation of stresses in complex armour units as function of the size (length) of the units.

The more rational design approaches which are used to-day are:

- Correct scaling of material characteristics of the model armour units, Timco, 1981.
- Measurements (by the use of strain gauges) of forces (bending and torsion) in a cross section of model armour units, DHL, 1980, and Scott et al., 1986, as basis for a FEM based assessment of stress distributions in the units.
- Direct measurements (by the use of strain gauges) of the stresses in large (~ 50 kg) model armour units made of prototype concrete, Nishigori et al., 1986.

The first method is the only one which can give the important information on how breakage of the armour units influences the development of the failure and the ultimate/residual strength of the structure. On the other hand this method is not practical if the problem is to design the armour units in terms of determining the necessary prototype strength (e. g. degree of reinforcement) whereas it is perfect for the checking of a design.

The second method introduces some uncertainty on the actual stress distributions. Until now the method has been used in small scale models with armour units where the surface roughness has not been scaled correctly. A correct roughness is very important in models of large units because stresses due to gravity and compaction can be significant, cf. Fig. 14.

The third method is by far the best for the determination of the stresses, but it is also very expensive as it requires a large scale model test facility.

The newly published results by Nishigori et al., 1986, confirmed the already known fact that rocking of large complex armour units as for example a 50 t Tetrapod can create stresses close to and even bigger than the ultimate strength. In the model with instrumented concrete Tetrapods of 50 kg the authors measured impact tensile strains ϵ_m up to $30 \cdot 10^{-6}$ for rocking units and up to $92 \cdot 10^{-6}$ in a case where a wave caused the Tetrapod to roll down the slope. The density and the modulus of elasticity of the concrete was $\rho_n = 2.3 \cdot 10^{-6} \text{ kg mm}^{-3}$ and $E_m = 2.4 \cdot 10^4 \text{ Nmm}^{-2}$ respectively. Following the similitude relations for impact loaded armour units, Burcharth, 1981, these results can be converted into prototype tensile stresses σ_p as follows

$$\sigma_p = \epsilon_p E_p = \epsilon_m E_p (\lambda_\rho \lambda_\rho \lambda_E^{-1})^{0.5}$$

where ϵ is strain, E is modulus of elasticity, λ_ρ , λ_ρ and λ_E are the length scale, the mass density scale and the scale of elasticity respectively. Indices p and m refer to prototype and model respectively.

Considering the 50 kg concrete Tetrapod as a model of a 50 t prototype ($\lambda_\rho = 10$), with mass density $\rho_p = 2.3 \cdot 10^{-6} \text{ kgmm}^{-3}$ ($\lambda_\rho = 1$) and modulus of elasticity $E_p = 4 \cdot 10^4 \text{ Nmm}^{-2}$ ($\lambda_E = 1.67$) we obtain prototype stress ranges of up to 2.9 Nmm^{-2} and up to 9.0 Nmm^{-2} for rocking and rolling Tetrapods respectively.

The ultimate tensile strength for conventional concrete of good quality is typically 4 Nmm^{-2} and 6 Nmm^{-2} for pulsating loads and impact loads respectively. However, since we are dealing with repeated loads, fatigue must be taken into account, Fig. 15.

Even for a limited number of impacts the strength is reduced considerably as seen in Fig. 15. Impact stresses of 2.9 Nmm^{-2} as found for severely rocking 50 t Tetrapods will in fact cause the unit to break after app. one thousand impacts. Rolling will cause immediate breakage.

It is expected that within few years a large number of results on armour unit stresses will be available both from models as described above and from prototype measurements as for example

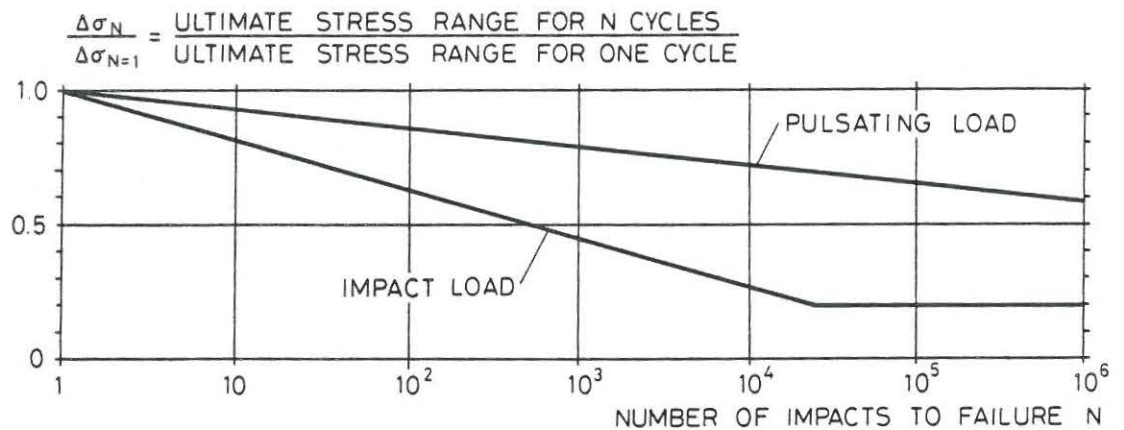


Fig. 15. Fatigue in conventional unreinforced concrete. Uniaxial and flexural stress. Burcharth, 1984.

the ongoing Dolosse research project at Crescent City by U. S. Army Corps of Engineers. Such results are necessary for the development of the now lacking formulae for the structural design of complex types of armour unit and also for calibration of the *numerical models* for structural and hydraulic design of armour layers which will soon be developed. Significant progress in numerical modelling of armour layer stability is made by Kobayashi et al., 1986.

With the knowledge we have to-day it is obvious that in cases where large armour units are needed to obtain hydraulic stability complex types of units like Tetrapods and Dolosse do not represent the optimum solution because of the very limited structural strength.

Reinforcing the concrete is, of course, an obvious way of improving the strength properties. Both conventional steel bar reinforcement, scrap rail reinforcement and fibre reinforcement are some times used in complex types of units. Results from full scale static and impact load tests with Dolosse in the range 1.5 - 30 t indicate that conventional steel bar reinforcement is superior to steel fibres of equal quantity. By using app. 130 kg steel bars per m³ concrete, spalling and not cracking seems to be the limiting factor, Burcharth, 1981 and 1983, Grimaldi et al., 1984. The last reference also states the effectiveness of a 50 kg per m³ concrete scrap rail reinforcement in Dolosse.

Chopped polypropylene fibres are successfully used for the SHED unit, a very slender hollowed cube type unit. However, fibres are not very effective in stiff structural elements like Tetrapods and Dolosse but are of course very useful in slender, flexible beams where large deflections only induce small strains.

Fear of corrosion has prevented many coastal engineers from using steel reinforcement. The influence of crack width on corrosion is still not fully understood. However, because of the many offshore concrete structures in the North Sea some experience has already been collected. Fjeld et al., 1982, summarized the research in Germany, United Kingdom and Norway on the relationship between crack widths and corrosion in bar reinforcement as follows (quote):

1. Reinforcing bar corrosion requires the ingress of chlorides to the reinforcement. Depending on cover and concrete quality this requires 3 to 15 months, but a considerably shorter time at significant cracks. Therefore reinforcing bars corrosion tends to be initiated at an earlier time in the region of the wider cracks than in other locations.
2. The rate of corrosion after initiation depends mainly on the concrete quality and is independent of the crack width.
3. Corrosion products, i. e. Mg and Ca compounds from the concrete, are deposited in the crack and hamper the continuing corrosion. The corrosion rate is continually decreasing and after 1 to 2 years the corrosion is almost stopped.
4. The final amount of corrosion is broadly independent of the crack width, at least if the crack widths are less than 0.5 - 1.0 mm. (End of quotation)

However, Wilkins et al., 1986, points out that concrete which is exposed to air from time to time, as in the tidal/lower splash zone of any structure in the sea, forms a "galvanic interaction

zone", which is likely to be a particularly effective cathode, whereas concrete which is permanently immersed (like most parts of the offshore structures) is unlikely to contribute significantly to accelerated corrosion.

Another effect which might be relevant for the steel bar reinforced armour units is that repeated loads and subsequent deformations in cracked sections will increase the corrosion and reduce the fatigue life, Booth et al., 1986.

Formulae for hydraulic stability of armour

It is well known that the Hudson formula has limitations by not taking into account all the parameters which affect the armour layer stability (e. g. wave period and underlayer permeability). However, the Hudson formula has been very useful for design estimates related to all types of bulky units (e. g. rocks and cubes) but it is not applicable for complex interlocking types of units such as Dolos, among other things because, for such units, the stability does not follow the \cot (slope angle) relationship, Brorsen et al., 1974.

General formulae are still lacking but for *stone armour* layers a new set of formulae mainly based on extensive model test series at the Delft Hydraulics Laboratory is given by Van der Meer et al., 1984.

Plunging waves (breaking, $\xi_z < 2.5 - 3.5$):

$$\frac{H_s}{\Delta D_{n50}} \cdot \sqrt{\xi_z} = 5.7 P^{0.14} \left(\frac{S}{\sqrt{N}} \right)^{0.2}$$

Surging waves (non-breaking, $\xi_z > 2.5 - 3.5$):

$$\frac{H_s}{\Delta D_{n50}} = 0.83 P^{-0.2} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot \alpha} \xi_z^P$$

where

H_s significant wave height

Δ $\rho_a / \rho - 1$

ρ_a mass density of stones

ρ mass density of water

$D_{n50} = (W_{50} / \rho_a)^{1/3}$

W_{50} = 50% value of mass distribution curve

$\xi_z = \left(\frac{g}{2\pi H_s} \right)^{0.5} T_z \tan \alpha$

T_z average wave period

α slope angle

P permeability coefficient dependent on underlayer permeability

S A/D_{n50}^2 non-dimensional damage level

- A eroded cross-sectional area of profile
N number of waves

It was found that grading of the armour stones, the shape of the spectrum and the groupiness of the waves had insignificant influence on the stability.

Armour units

A very promising armour unit which combines very good hydraulic stability with good structural strength and easy production is the HARO unit, developed by HAECON, Belgium, and used in Zeebrugge Harbour, De Rouck et al., 1986, Fig. 16.

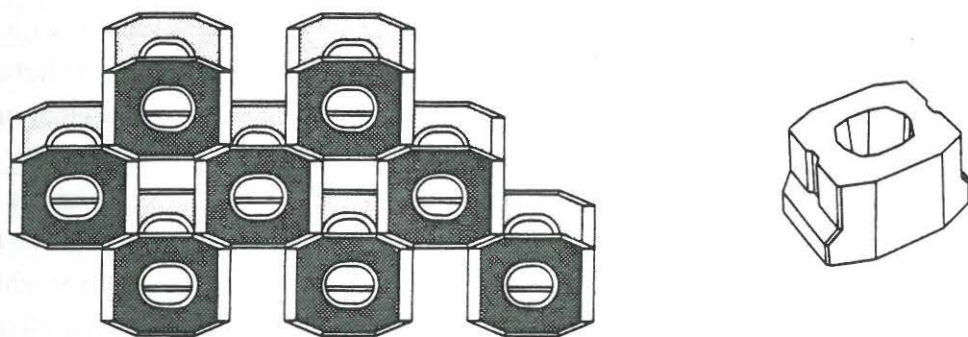


Fig. 16. The HARO unit by Haecon. Principle of placement if placed in regular pattern (random placement can be used too).

The significance of the unit is that it can be placed randomly and still maintain a very good hydraulic stability (e.g. for a double layer $K_D \cong 6-13$ for 0-0.5% damage). This is due to the large porosity of 55% (volume of pores over total volume) and the large permeability, which also gives a low run-up. With reference to Fig. 13 the HARO unit fits in between the complex types (Dolosse etc.) and the hollowed blocks (which need regular placement).

The concept of the complex types of units is still very useful for less exposed locations and we might see a come back also where large blocks are needed as soon as reasonable design methods are developed.

It is worthwhile to mention the Japanese concept, Fig. 17, for the use of complex types of armour units on exposed locations where very long waves are present as for example along the Pacific Ocean coastlines. The purpose of the armour units is to dissipate the wave energy and thus reduce the overtopping and the chockforces on the caissons.

The Japanese name for complex types of armour units is 消波工 (SHOHAKO), which means "structures to dissipate waves". The concrete used for the armour blocks has a fairly low cement content and a low compression strength ($\sigma_{cyl} = 18-21 \text{ Nmm}^{-2}$ dependent on block size) and is always designed to be less strong than the concrete used for the caisson. Because of the low cement content it is believed that harmful thermal stresses are avoided whereby an overall good tensile strength is obtained.

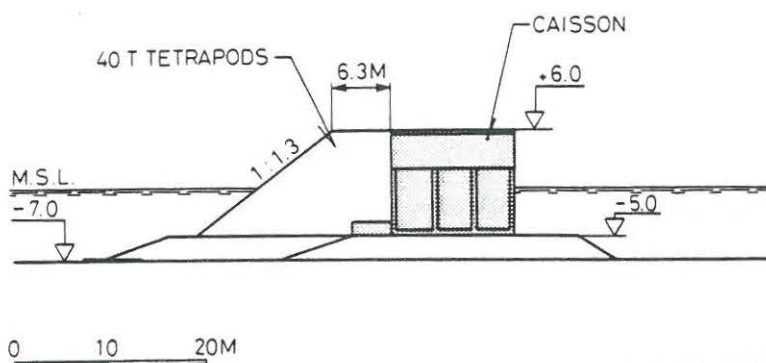


Fig. 17. Example of Japanese practice for the use of complex types of armour units.

Another recent development is the Accropode by SOGREAH, France, Fig. 18.

It is an unreinforced concrete unit but strong because of its chunky shape with many short and thick "fingers" which provide a macro roughness and thereby a good intertangling with neighbour blocks. Sogreah recommends it to be placed at random in a single layer on very steep slopes of 1:1.33 and recommends K_D values of 10 to 12. These figures lead to a very economical armour layer. The Accropode armour layer stability will be further tested in various laboratories in the near future. The promising behaviour of the Accropode is not primarily due to the porosity which is limited because of the single layer concept, but to the special shape which provides an excellent intertangling which is enhanced by the large gravity prestressing effect acting on steep slopes. At the same time the units are strong enough to resist considerable movements without breakage.

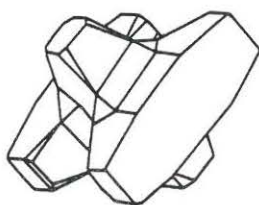


Fig. 18. The ACCROPODE unit by SOGREAH.

Berm Breakwater

Within the last years several breakwaters have been designed and constructed as berm type — or sacrificial type — breakwaters. The idea of this type is that rocks considerable smaller in size than traditional armour stones can be used because such material when displaced by waves eventually will form a stable equilibrium s-shaped profile provided that the sufficient amount of material is available, Fig. 19. The berm breakwater can be constructed without the use of expensive equipment, and maintenance and repair is as simple as the construction. For a more detailed discussion of the concept see Baird et al., 1984. A sacrificial breakwater will always be stable in head-on seas if the crest of the mound is not eroded in its full width. This principle holds for all sizes of stone materials since even a sand beach stabilizes, though with a very flat profile. The ef-

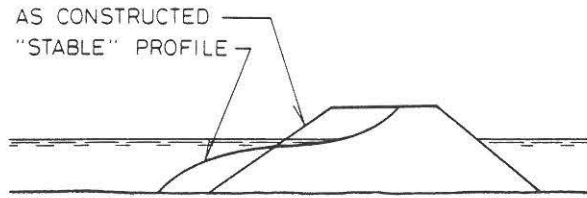


Fig. 19. Principle of the berm breakwater.

efficiency of a berm breakwater is very dependent on the permeability of the mass of stones facing the sea. This again put restraints on the grading and size of the stones, i. e. too much fine material will block the pores and reduce the stability. A large test series run by Delft Hydraulics Laboratory has formed the empirical basis for a computer program for the prediction of the seaward berm breakwater profile for head-on wave exposure as function of H_s , T_z , D_{n50} , D_{85}/D_{15} (grading), Δ , N , and the water depth, Van der Meer, 1986. Very little has been published about berm breakwater stability in oblique waves and about stability of berm round-heads. However, model tests of the breakwaters for the St. George Harbour, Pribilof Islands, Alaska, indicate that for a reasonable stable trunk design oblique waves are not more dangerous than head-on waves. Moreover, the tests also showed that satisfactory stability of berm type round-heads can be obtained, Gilman et al., 1986. However, a general analysis of the behaviour of berm breakwaters in oblique waves and of the behaviour of berm round-heads is still lacking.

SAFETY ASPECTS IN THE DESIGN

Independent on the existence of codes and recommendations it is essential that the designers operate on the basis of an exceedence risk concept with due consideration to the uncertainties related to environmental loads, structural response, construction, and maintenance, Burcharth, 1985.

The *encounter probability*, E , i.e. the probability that the design sea state is equalled or exceeded during the structural lifetime, L , should be used instead of a selected return period, R . The relationship is given by $E = 1 - (1 - R^{-1})^L$, which is depicted in Fig. 20.

The *uncertainty on estimates on design wave climate* should be properly evaluated. Fig. 21 shows a typical example for the estimation of uncertainty on H_s in deep water off the Libyan coast in the Mediterranean Sea. The underlying data are the peaks of 14 hindcasted storms in a 20 years' period.

The *scatter in the hydraulic model test results*, which to a great deal reflects the uncertainty related to the structural response, is very often significant and must be considered in the design. Fig. 22 shows an example of test data for a study of the hydraulic stability of Dolosse.

It is worthwhile to mention that the consequences of design load exceedence are very dependent on the *structural failure mode*, which to a great extent we can decide ourselves, as has also been practice for many years in earthquake engineering, Fig. 23. The most ductile failure is that of a traditional rubble mound with moderate sloping armour of non-interlocking blocks like

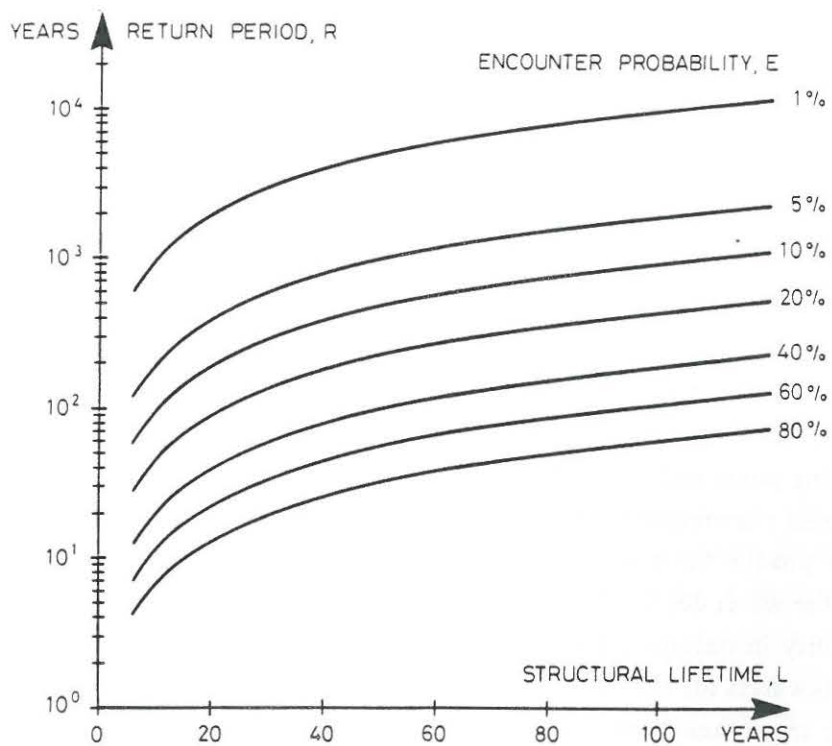


Fig. 20. Encounter probability.

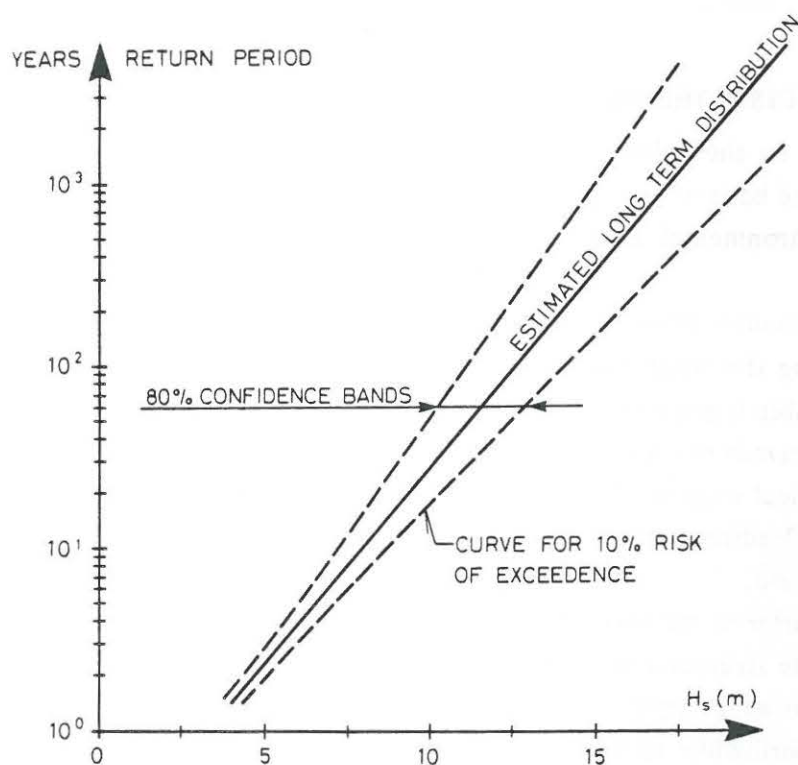


Fig. 21. Example of 80% probability control curves for a data set of 14 storms in a 20 years' period.

DISPLACED DOLOSSE IN TEST AREA AROUND M.W.L.

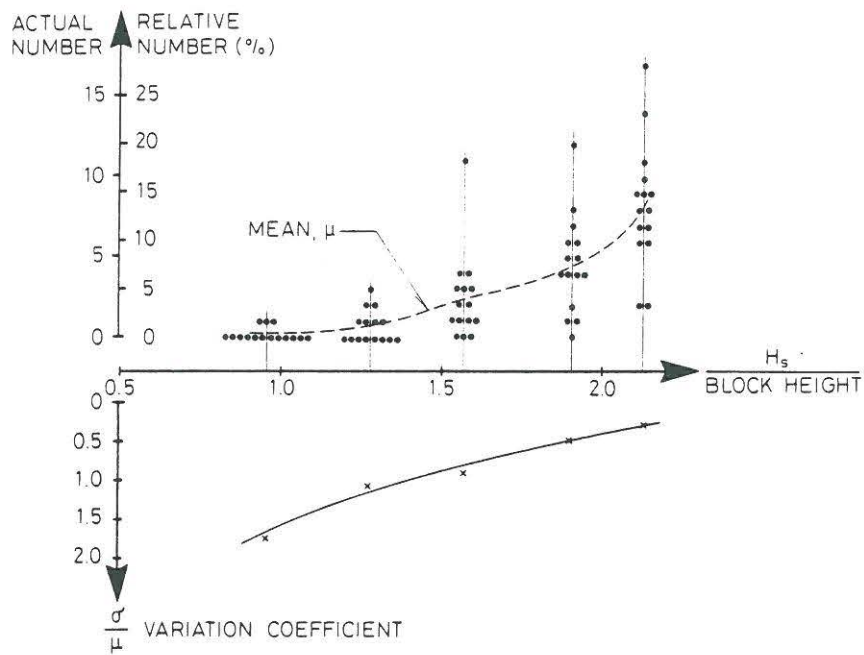


Fig. 22. Example of scatter in armour stability tests. Burcharth, 1986.

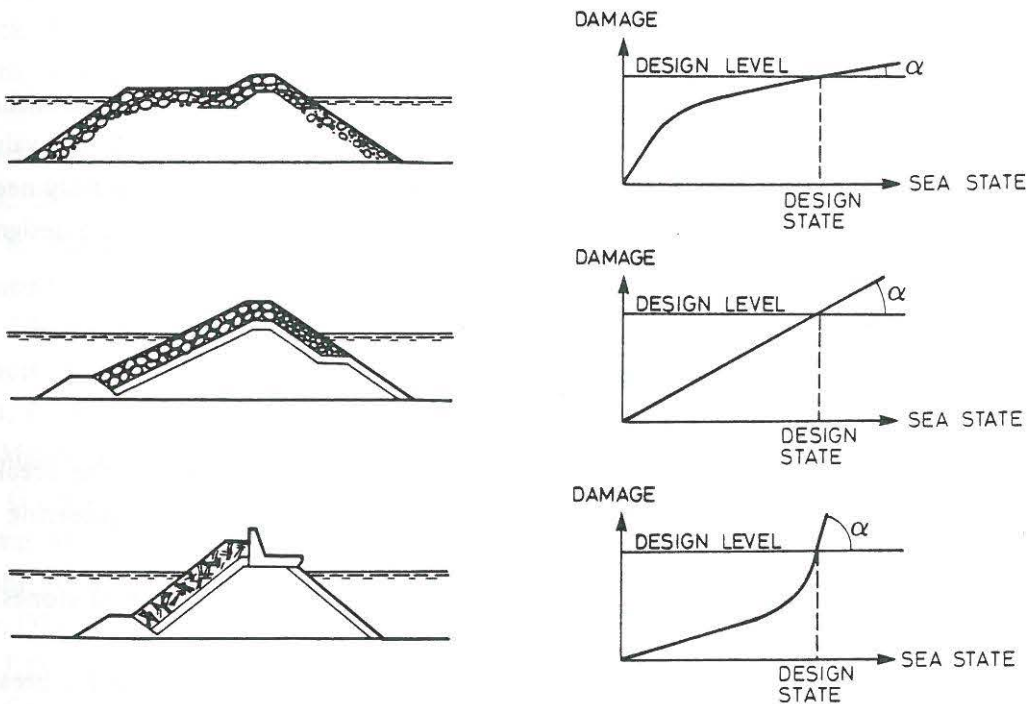


Fig. 23. Failure modes for various types of breakwaters.

quarry rocks. The most brittle failure is associated with steep slopes of interlocking unreinforced complex types of concrete armour units in front of a wave wall. The figure illustrates the very different residual strength/capacity of the three structures. This must of course be reflected in the actual choice of the design parameter figures.

The geotechnical stability (e. g. slip circle analysis) should always be investigated for breakwaters in *deeper* water.

DESIGN CODES FOR BREAKWATERS

Design codes exist for offshore structures and nearly all types of structures on land. The question about the needs for breakwater design codes is often raised but such codes has not yet been brought into practice for the following reasons:

1. The tools (design procedures, formulae, etc.) for the design of larger breakwaters are not yet developed to a level where it, generally speaking, is possible to ensure designs corresponding to predefined performances.
2. It is extremely seldom that human life is endangered or lost because of breakwater failures.

Recommendations for the design, construction and inspection of breakwaters can be very useful but the time for codes has not come yet.

CONCLUDING REMARKS

The paper deals mainly with problems related to rubble mound breakwaters. Although there has been progress in the understanding and knowledge of these problems and also valuable new contributions it is the opinion of the author that the most important and certainly needed recent development is the growing understanding of the necessity to implement in the design process much more realistic approaches to the analysis of the safety of the structures.

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