Risk Awareness – Key to a Sustainable Design Approach for Breakwater Armouring

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Introduction

In prosperous periods people tend to overestimate their capabilities and to accept significant risks. This is a widespread tendency, which led to the financial crisis in 2008. It can be seen back also in the variation in time of investments for flood protection schemes and affects last but not least the designing of breakwaters. A number of major breakwater damages occurred in the late 1970th and early 1980th (a.o. Sines, 1978; Arzew, 1980; Tripoli, 1981). In the period thereafter designers applied larger safety margins. However after more than 25 years without any spectacular failure of a major breakwater; often we see that many of the intuitively applied safety margins are disappearing. This tendency is supported by the new role of consulting engineers in EPC contracts and PPP's. Designers are working more closely with contractors; there is more focus on efficiency. This results in breakwater designs that are close to or right on the edge.

An important aspect in the design of breakwater armour is the determination of the design wave height. Commonly return periods of 50 or 100 years are used, although the lifetime of breakwaters may exceed 100 years. Despite the fact that a design wave height is used which may be exceeded several times during the lifetime of the structure, no safety factors are applied. This is in contrast to the safety factor approach of structural engineering. As a consequence, severe damage may occur if design wave heights are exceeded or if the effect of important design aspects has been underestimated or not considered.

In this paper a number of important design aspects and their possible effect on the armour layer stability are discussed, which are not sufficiently covered by present design methodologies. Guidance is provided on how these aspects should be considered in the armour layer design. Both aspects on the load and resistance side are considered.

Extreme wave conditions

Design wave conditions are mostly determined by extreme value analysis; extreme conditions are extrapolated from severe wave conditions of the past 10 to 20 years. The probability of occurrence of exceptional storm conditions might be underestimated by this statistical approach, as the physics behind an exceptional storm are not addressed.

Storms are associated to low pressure centres. Besides the pressure drop there are a number of factors that may aggravate a storm. These are amongst others a sequence of two or more low pressure systems and the collision of warm or cold air masses with a low pressure system.

The following exceptional storms are examples of storms that have been amplified by such influences:

- The Ash Wednesday storm (6 – 10 March 1962) moved slowly up the East Coast of the US and was fed by cold air masses originating from a high pressure centre over Canada. The storm coincided with spring tide, was associated with heavy snow fall and caused enormous damage.
- The storm that hit Great Britain on 15 – 16 October 1987 was also intensified by the joint occurrence of two low pressure systems. One system approaching the coast and a second system in the western Atlantic. The storm felled 15 million trees across Southeast England and knocked out the electric power in London.
- The Perfect Storm (31 October 1991) was the subject of a book by Sebastian Junger (1997) and of a movie in 2000. A low moved from Canada towards the Atlantic ocean, turned and moved back westward to New England. Cold air from Northwest and warm air from Southeast, the remnants of hurricane Grace were feeding the low. This unusual path provided several days for exorbitant wave heights to build in the North Atlantic.
- The Storm of the Century (12 – 14 March 1993) was intensified by warm and humid air from the Gulf of Mexico. The storm caused a 3.7 m surge in Florida and moved along the US East Coast northward. It was the first weather event to close all major East Coast airports from Washington DC northward.
- France was hit on 26-28 December 1999 by a major storm that developed when two low pressure systems approached the coast. More than 100 people got killed in Western Europe; much of the France’s electric power was knocked out for days.

The above examples indicate that extreme storm events and the associated wave heights result from the coincidence of several factors that are able to generate a storm of historical dimensions. Meteorologists have difficulties in predicting the intensity of these storms accurately a couple days in advance. Engineers should be aware that design conditions determined by a statistical analysis (extreme value analysis) bear significant uncertainties. They can provide some guidance on the likeliness of extreme wave conditions. However, they are unable to predict the largest waves of the coming 100 years with two decimals accuracy. The presence of tropical storms and cyclones will further increase the uncertainties with respect to design waves and surge levels. Coastal structures should be therefore designed in a way, that they can survive a significant exceedence of design conditions (by some 10 – 20%) with limited damage.

Joint probability of design wave height and design water level

In coastal engineering practice, normally the design wave height is chosen as the 1:100 year significant wave height. The associated water level is often taken as Mean High Water Spring plus storm surge plus sea level rise. This is a classical design approach. The actual return period of a 1:100 year wave height combined with this extreme water level may be well beyond 100 years. Therefore – especially in a depth limited situations – this classical approach provides some inherent safety. An alternative, more sophisticated design approach is based on a probabilistic analysis on the combination of extreme wave heights and extreme water levels. This approach became more and more common in recent years. Instead of using the 1:100 year wave conditions in combination with an extreme water level (classical approach), the 1:100 year event may e.g. be defined as the 1:20 year period wave height in combination with a 1:1 year water level (probabilistic approach). Although the probabilistic method may lead to a more economical design – at least in theory – all safety in the design is removed. Without the use of a safety factor on the structural resistance or the possibility to do frequent maintenance, this probabilistic design approach (based on joint occurrence of waves and water levels) is not recommended.

Design wave height in shallow water

In shallow water the wave height exceedence curve becomes less steep than in deep water due to wave breaking. This implies that the difference between the 1:1 and 1:100 year wave heights is significantly reduced compared to the offshore deep water situation. Therefore near design wave heights may occur more frequently in shallow water (with depth limited wave heights) than in deeper water (without wave breaking) as indicated in Figure 1. As a result the structure is more frequently exposed to near design conditions and the storm duration (period of time with wave heights close to the wave conditions at the peak of the storm) will be increased. Damage progression may develop more rapidly. In case of rock armour more damage (displacement of armour stones) should be expected, while in case of concrete armour units breakage due to fatigue will be more likely.

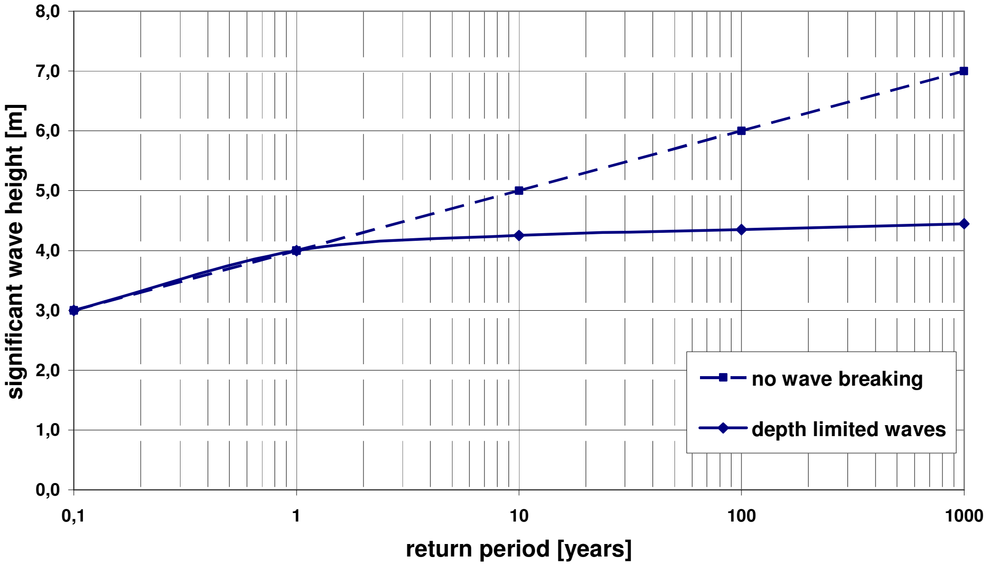

Figure 1. Example of wave exceedence curve in a situation with (shallow water) and without (deep water) wave breaking

Multiple peaked wave spectra

For many coastal projects around the world the wave spectrum of the 1:100 year design storm is assumed to be of the PM or JONSWAP type. These are simplistic, single peak spectra; the JONSWAP spectrum is based on measurements done on the North Sea. Physical model tests of coastal structures are often done with JONSWAP spectra. However, actual wave spectra at specific sites may differ significantly from the PM or JONSWAP shape. Interaction of multiple low pressure systems will result in double or multi peaked wave spectra. An example of such a complex pressure system in the North Atlantic and in the North Sea is shown in Figure 2. The figure shows three different low pressure centres between Denmark and Greenland on 9 November 2007.

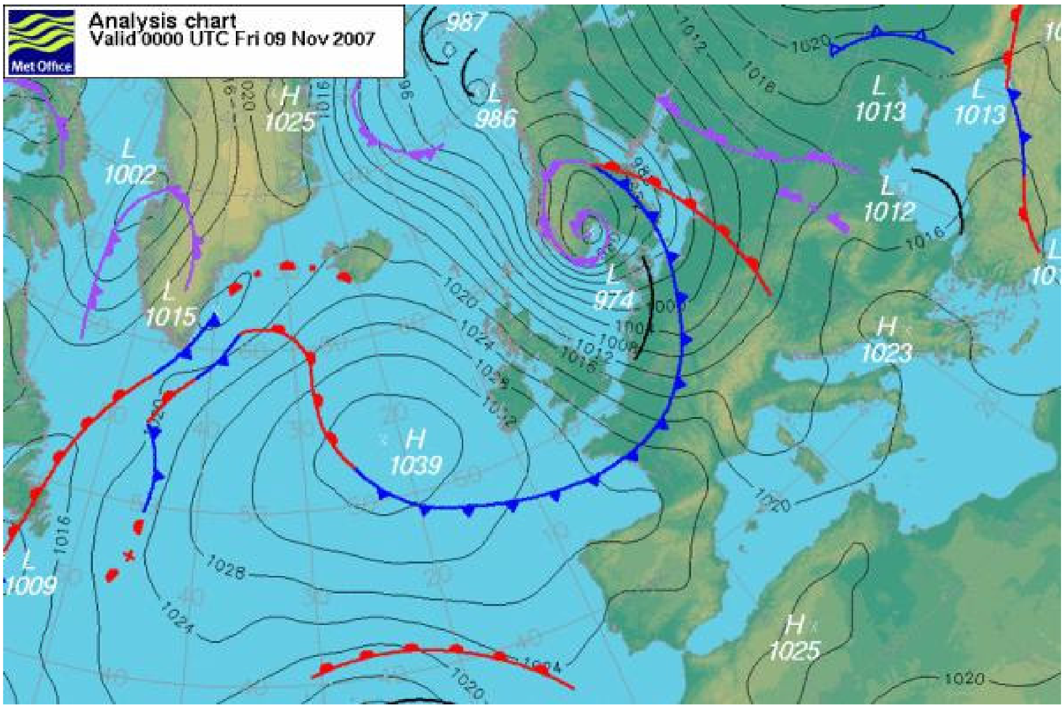

Figure 2. Storm situation with multiple low pressure centres in the North Sea and in the Eastern North Atlantic (Source British Met Office website)

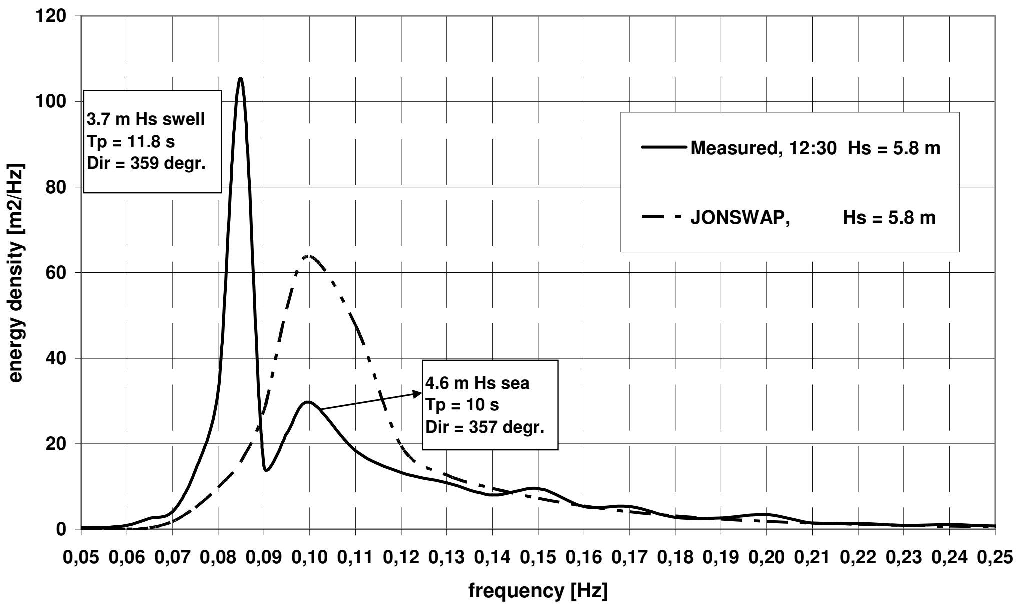

Figure 3. Double peaked wave spectrum measured in the North Sea (Dowsing, UK, 21 December 2003)

An example of a double peaked wave spectrum on the North Sea (recorded on 21 December 2003) is shown in Figure 3. The wave buoy at Dowsing (east coast UK) reported a significant wave height of 5.8 m. However, the wave spectrum shows a double peaked spectrum with a swell component of 3.7 m H_s and a wind sea component of 4.6 m H_s. Both components have nearly the same wave direction (travelling from North to South). This implies that there are two distinct wave fields with peak periods of 11.8 s and 10.0 s. The interaction between these two wave fields will generate a long wave with a wave period of about 65 s. This interaction and especially the resulting long wave may have significant influence on the armour layer stability. Smith (2001) investigated the effect of spectral shape on armour stability. He found that double peaked spectra lead to armour stability outside of the confidence bands of stability equations for single peaked spectra. Therefore the use of single peak spectra (PM or JONSWAP spectra) in physical model tests may underestimate the actual damage that would result from a double peak spectrum.

Design waves on a steep foreshore

The significant wave height in a depth limited situation is typically of order Hs/h = 0.5. A wave height to water depth ratio Hs/h = 0.55 to 0.6 is occasionally applied for design. However, a depth limited significant wave height can be also affected by the seabed slope and by the occurrence of breaking waves (in other words, by the percentage of incoming waves that are breaking or broken). Both aspects are not addressed by simplistic breaker criterion of type Hs/h = constant. This may result in a significant over- or underestimation of the actual wave height.

Parametric surf zone models have been developed by Battjes and Jansen (1978) and many other authors (see for example Battjes and Jansen, 2008). These models are based on a Rayleigh wave height distribution in deeper water and predict the distorted wave height distribution due to wave breaking as the waves propagate into shallow water. The significant wave height inside the surf zone can be derived from the modified wave height distribution, the occurrence of wave breaking and the effect on the significant wave height inside the surf zone are explicitly addressed in these models.

Goda (1975, cited in Goda, 2000) proposed a parametric surf zone model that includes the effect of bottom slope. He derived from this model design diagrams for various bottom slopes and a set of equations for a crude estimate of significant and maximum wave heights inside the surf zone. These equations have been included in the British Standard (BS 6943: part 1).

The Goda model was applied for assessing significant wave heights in the surf zone for a wide range of boundary conditions (seabed slope 1:50 to 1:6, wave steepness $H_{s,0}/L_0 = 0.005$ to 0.04). The model was somewhat modified, wave set-up and surf beat were excluded, a linear shoaling approach was applied. A total of 882 simulations was performed, the results were stored in a data base. Two governing parameters for the local wave height inside the surf zone were determined by dimensional analysis:

- A dimensionless wave parameter $B = (H_{s,0}/h)\,(h/L_0)^{1/4} = H_{s,0}\,L_0^{1/4}\,h^{-5/4}$ (further referred to as “breaker parameter”);
- The seabed slope $\tan(\alpha)$.

An equivalent deep water wave height, $H_{s,0}$ has been applied for this analysis. The actual offshore wave height has been corrected with respect to wave refraction on the foreshore and possible wave diffraction by islands and headlands (see Goda, 2000). The breaker parameter includes further the local water depth, h and the deepwater wave length, L_0 (determined from the peak wave period of a single peak spectrum).

The variation of significant wave heights inside the surf zone is plotted in Figure 4 against the breaker parameter B for different seabed slopes. The lines in Figure 4 are an empirical best fit to the wave height prediction of the surf zone model. The deviation between the actual wave heights in the surf zone and the best fit as plotted in Figure 4 is presented in Figure 5. Results of hydraulic model tests performed by Hovestad (2005) (cited in Muttray and Reedijk, 2008) are also plotted for comparison.

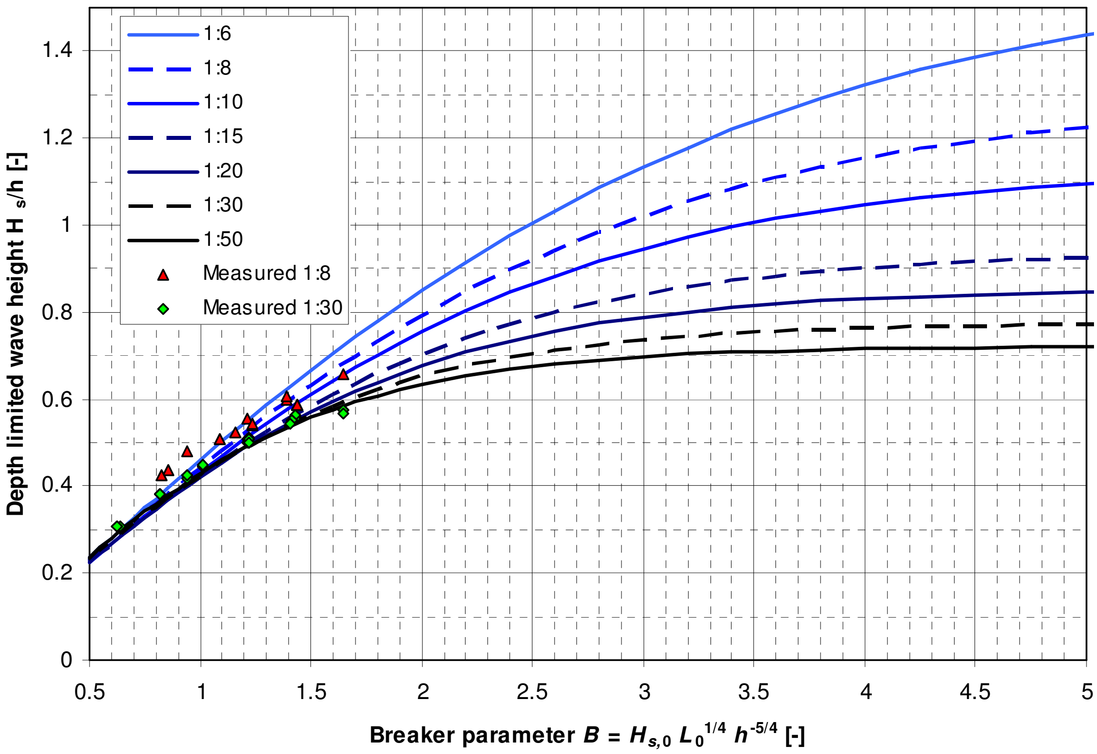

Figure 4: Significant wave height inside the surf zone H_s/h vs. breaker parameter B

Figure 4 indicates that the significant wave height, H_s inside the surf zone may vary from about 0.3 h to more than 1.4 h. The wave height is closely linked to the local water depth. It is further largely affected by the seabed slope tan(α) (especially in shallow water, B > 1.5) and by the percentage of breaking or broken waves. The latter is described by the ratio of equivalent deepwater wave height and local water depth, $H_{s,0}/h$. The wave height in the breaker zone is finally influenced to some extent by the wave transformation (wave shoaling) in shallow water, which is described by the relative water depth $(h/L_0)^{-1/4}$.

Figure 4 can be applied for assessing depth limited significant wave heights. The possible effect of wave set-up and surf beat should be determined separately and should be included in the local water depth. It is important to note that the local wave height will be also in a depth limited situation largely affected by the offshore wave conditions. A proper analysis of offshore wave conditions is thus inevitable also in case of depth limited design conditions. A simplistic breaker criterion (like $H_s/h = 0.6$) is completely inappropriate for assessing a depth limited design wave height.

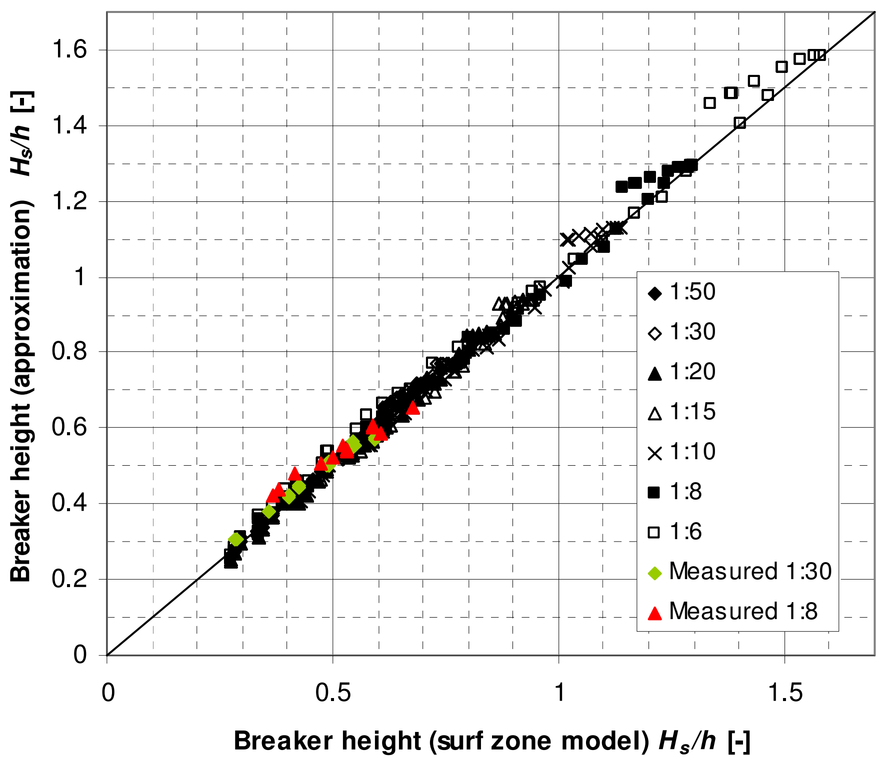

Figure 5: Comparison of results from surf zone model and empirical fit function

Wave breaking on the foreshore

The effect of breaker type on armour stability has been addressed by van der Meer (1988). Van der Meer considered the wave breaking on the slope of the structure (i.e. he derived the breaker type from the ratio of breakwater slope and wave steepness). In case of a breakwater with a relative steep slope the breaking will be initiated at the toe of the structure and will be more related to the foreshore slope than to the breakwater slope. However, the foreshore slope is not addressed in any design formula for rubble mound breakwaters. It is well implicitly included in the design formula for vertical breakwaters (Goda, 2000).

Muttray and Reedijk (2008) investigated the effect of foreshore slope in a situation with breaking waves and almost identical wave conditions at the toe of the structure (a rubble mound breakwater with rock armour). The resulting damage to the armour layer varied significantly in these tests and it has been concluded that the occurrence of plunging breakers (that is mostly associated with a steep foreshore) will probably result in more severe wave loads and in larger damage. However, no predictive equations are available for assessing the frequency of plunging waves in front of a breakwater.

The surf zone model of Goda (2000) was applied by Muttray and Reedijk (2008) for this purpose. The occurrence of wave breaking can be derived directly from this model. The associated breaker types were assessed by the breaker type classification of Battjes (1974) (with surf similarity parameter $\xi_0 = \tan\theta / \sqrt{H_0 / L_0}$ and deep water wave length $L_0 = gT_p^2 / (2\pi)$):

$$\xi_0 < 0.5 \qquad \text{spilling breaker}$$
$$0.5 \le \xi_0 < 3.3 \qquad \text{plunging breaker} \qquad (1)$$
$$\xi_0 \ge 3.3 \qquad \text{surging breaker}$$

The modified surf zone model of Goda (2000) (with linear shoaling approach and exclusion of surf beat and wave set-up) was applied to the 882 surf zone simulations (see above) for assessing the breaker types. The results are plotted in Figure 6. As expected the occurrence of plunging breakers varies with the surf similarity parameter and with the breaker parameter. The latter is closely related to the occurrence of breaking waves. It can be seen from Figure 6 that:

- Spilling breakers are expected for $\xi_0 < 0.25$ to 0.7 (depending on B), no plunging breakers are predicted in this range;
- Plunging breakers are predicted for the range 0.25 to $0.7 < \xi_0 < 3$ to 5; the percentage of plunging breakers is increasing with B;
- Surging breakers are expected for $\xi_0 > 3$ to 5 (depending on B), no plunging breakers are predicted in this range.

The transition from spilling to plunging waves is marked in Figure 6 as "lower bound" and the transition from plunging to surging waves as "upper bound". Lines with typically 1%, 5%, 10%, 20% and 30% plunging breakers are indicated. The results of the model simulations are also plotted; the scatter should be noted.

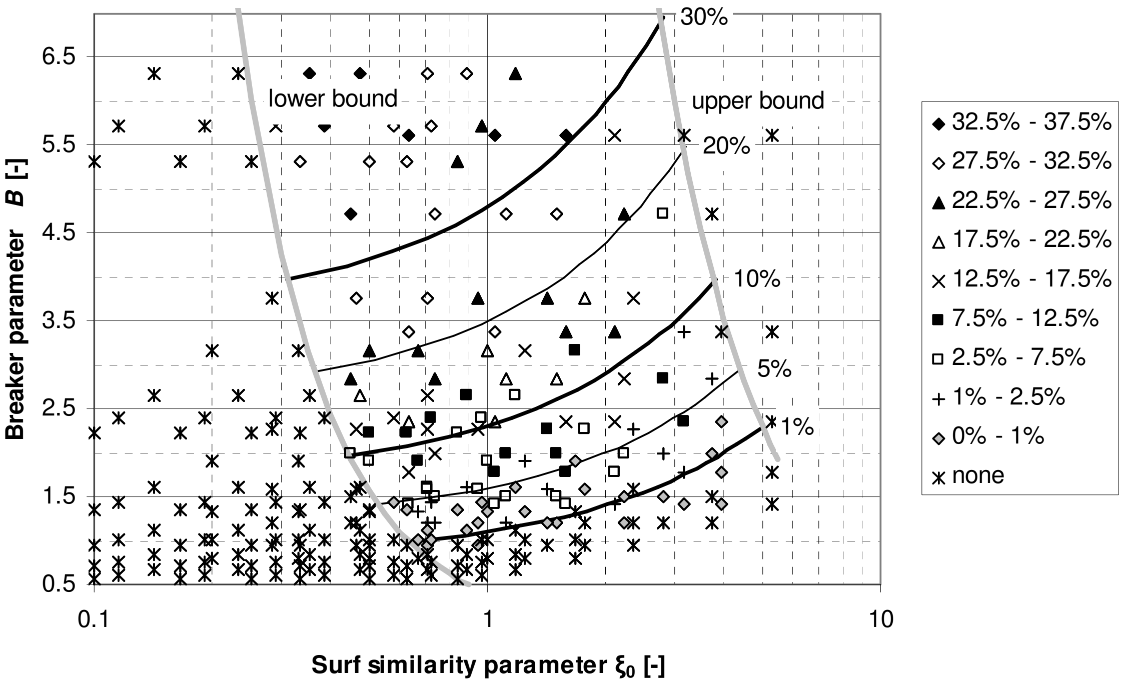

Figure 6: Occurrence of plunging breakers (percentage of all incoming waves)

Figure 6 can be applied as a design aid. The percentage of plunging breakers at the toe of a breakwater (= ratio of plunging waves near the breakwater and total number of incident waves) under design conditions can be assessed from this figure (based on parameters ξ_0 and B derived from design waves in deeper water, seabed slope and local water depth). The lines in Figure 6 are indicative; especially the upper and lower bound mark in fact a gradual transition from plunging to surging and spilling breakers. Therefore, a designer should be aware that plunging breakers could affect the stability of the breakwater not only in a situation where the coordinates ξ_0 and B are inside the marked range of plunging breakers in Figure 6 but also in a situation where they are just outside this range.

A relative small number of plunging breakers may have an adverse effect on the breakwater stability. Muttray and Reedijk (2008) observed about 33% larger damage in a situation where 0.5% to 1% of the incoming waves were plunging breakers (at the toe of the breakwater). It is not yet known whether a larger percentage of plunging breakers will also result in larger damage. A conservative armour layer design and design verification by hydraulic model tests are strongly recommended in a situation with plunging breakers at the breakwater toe.

Dependence of armour strength on wave exposure

The stability of breakwater armour depends on the placement pattern, i.e. the packing density and relative orientation of neighbouring armour units. The stability of loosely packed armour units placed on relatively steep slopes (1: 1.5 and 3:4 V:H) may increase over time as wave action will lead to settlements. This compaction of the armour layer will improve the armour stability at the water line and underwater while it may lead to gaps in the armour layer at the transition between slope and crest (and thus to a reduced stability in the upper part of the slope and at the crest).

When a breakwater design is confirmed in physical model tests, the wave heights are mostly increased stepwise before the design wave height is reached. This test procedure may overestimate the actual stability of the armour layer. Firstly, the strength of the armour layer in the model may be increased by the settlements before the design storm is tested. Secondly, the settlements may cause breakage of armour units, especially if large concrete armour units of slender shape are applied.

In prototype a breakwater may be exposed to (near) design conditions during construction or just after construction. An example is the main breakwater of Puerto Caucedo that faced near design conditions during hurricane Ivan in September 2004 only 1 month after handover. The hurricane waves caused significant settlements of the armour layer. It should be carefully considered if the consequences of a near design storm during construction or just after construction need to be assessed. If so, the armour stability before settling should be addressed in the physical testing programme.

The possible effect of broken armour units on the armour layer stability should be also addressed in hydraulic model tests. Breakage of armour units will mostly occur near the water line and in the most upper part of the slope (where settling of the armour layer will result in loosely packed armour units). De Rover (2008) performed a fundamental study on the effect of broken Xbloc units on the armour layer stability. According to this study the breakage of individual units and clusters of several broken units have little effect on the armour layer stability. However, it is uncertain if these results will be generic and if they are also applicable for other types of armour units. Therefore, it is recommended that confirmative model tests should also include tests where a number of armour units has been removed from the armour layer or has been replaced by broken units.

Effect of core permeability on armour layer stability

Rubble mound breakwaters normally have a permeable quarry run core. Wave energy is dissipated in the core by turbulent flow. If the core is less permeable, increased reflection occurs, leading to higher loads on the armour layer. Therefore a core with low permeability requires larger armour than a core with high permeability, as described in Burchart (1998) and Reedijk et al. (2008). The required nominal diameter of the breakwater armour for a low permeability core may be over 40% larger than has been determined for a permeable quarry run core. Therefore the effect of the permeability of the core is substantial.

This effect is often not addressed adequately during design and construction. During the design stage the core is normally assumed to be permeable. When the breakwater design is verified with physical model testing, the core is scaled in such a way that turbulent flow is also maintained in the scale model. When the design has been verified by model testing, breakwater construction can take place. Depending on the source of rock material for the core (which sometimes is dredged material), many fines may be present in the core material. The core permeability of the constructed breakwater may therefore differ substantially from the value assumed by the designer. The designer should therefore consider which material will be used for core construction and provide adequate specifications. The contractor should make sure that the core material fulfills the specifications. If there is uncertainty on the permeability of the core, sufficient safety should be applied in the design of the armour layer.

Check list

The checklist as given below provides a summary of the items as discussed in this paper. It should be noted that there are other important design items which are not given in this list.

No.	Item	Action
1	*Wave Loading*	
1.1	Is the project location exposed to frequent extreme storms?	• Increase safety by using design return period > 1:100 years • Possible deviation of extreme storm events from typical distribution of major storm should be carefully considered • Upper limit of a 90% confidence interval might provide more realistic design
1.2	Does the design wave height strongly depend on water level? (depth limited situation)	• Consider armour fatigue and damage progression, add safety by increasing armour size • Consequences with respect to frequency of near-design conditions and storm duration should be considered. • Armour unit weight should be typically increased by 15% to 30%.
1.3	Is the water level strongly correlated with storm occurrence?	• Consider the dependence in probabilistic design • The results of a probabilistic approach (based on the joint occurrence of waves and water levels) should be cross-checked against design conditions derived by a classical engineering approach. Reasonable safety margins should be included.
1.4	Is there a steep foreshore? (>1:30)	• Depth limited wave heights on a steep foreshore may exceed a typical H_s/h ratio of about 0.5 significantly and may be of order $H_s/h = 1$. • Design wave heights should be checked against Figure 4.
1.5	Is the breaker index 0.25 < ξ_0 < 5 (based on foreshore slope and offshore wave steepness)	• Plunging breaker are likely to occur at the toe of the breakwater; • Occurrence of plunging breakers should be checked against Figure 6; • Armour unit weight should be typically increased by 15% to

		30%.
1.6	Do multiple peaked wave spectra occur?	• Determine effect on armour stability and overtopping in physical model • If the wave directions of sea and swell spectra coincide the possible generation of long waves should be considered. • The effect of long waves might be a temporary water level rise by up to 10% to 20% of the offshore wave height, $H_{s,0}$. • A possibly increased design water level should be considered in case of multi-peak spectra
2	*Structural Resistance*	
2.1	Can the structure be exposed to near design wave heights during or shortly after construction?	• Use armour type and dimensions that provides adequate strength before/without settling
2.2	Can the core permeability be reduced by choice of material and dimensions?	• Consider effect on armour stability during design • Provide adequate specifications • Actual core material used should comply with specifications

Conclusions

It is concluded that although present design formulae include the most important design aspects, still there are a large number of design aspects which are not addressed in these formulae. Depending on experience and insight of the designer these other aspects might be considered intuitively in a breakwater design. However, this is not a sound practice. Therefore, there is an urgent need for a more complete design guidance on breakwater armouring.

In breakwater design safety factors are mostly not applied on the load or resistance side. Therefore, missing relevant issues in the design process may lead to severe damage or complete failure. In this paper a number of design aspects has been discussed. Still there are more aspects which are not yet sufficiently covered by design guidelines, such as

- What is the effect of wave exposure on fatigue of concrete armour units in the long term?
- How large is the risk that concrete armour units will break due to rocking and what is the effect on the armour layer stability?
- How significant is the effect of armour crest level and armour slope on the stability of interlocking concrete armour units?
- Is the effect of concrete density properly described by the relative density, which is included in the stability number? Will the effect of concrete density be the same for interlocking and non-interlocking armour units?
- How much effect has the placement pattern (packing density and quality of placement) on the stability of concrete armouring? How can we make sure that the placement in lab tests and in prototype will be comparable?

The designer needs to be aware of various relevant design aspects that are not included in common design formulae. This paper provides some guidance on a number of items. A design verification by physical model testing is strongly recommended, especially for breakwaters that are designed on the edge. Overload testing should be part of a standard testing programme. The use of (partial) safety factors might be considered in a future, more complete breakwater design guideline. However, even with a more sophisticated design guideline, the insight and experience of the designer will be crucial for an economical, constructible and safe breakwater design.

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