Australian Journal of Basic and Applied Sciences, 3(3): 2717-2827, 2009
ISSN 1991-8178

Recent Advances in Stability Formulae and Damage Description of Breakwater Armour Layer

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Abstract: The present paper gives a brief overview of the current state-of-the-art for estimating stability of rubble mound breakwater armour layer, identifying a set of relevant aspects, such as definition of damage, type of armour units, and the sea state. Since all the stability formulae are designed to estimate the appropriate weight of armour units corresponding to a certain damage level, different methods of damage assessment are distinguished, and most well-known stability formulae are discussed with the emphasis on definition of damage.

Key words: breakwater, armour layer, stability formulae, damage description

INTRODUCTION

Determination of the armour layer stability is a matter of concern to many coastal engineers. The uncertainties that come from the complex nature of the wave and nearshore currents, diversity of variables, and the stochastic wave-structure interactions dominate the accuracy of stability estimation. Since 1950s, various experimental studies have been carried out to improve analysis and design of armour layer, seeking to decrease the uncertainties and to avoid overestimating or underestimating armour weight. However, predicting the stability of armour layer still rely to a large extent on the designer's experience.

Different failure modes that can cause damage to a rubble mound breakwater are illustrated in Fig. 1. Armour units hydraulic instability is among the most critical failure modes since it can disintegrate armour layer and consequently initiate progressive failure that is likely to make the breakwater unable to function. This paper aims to briefly overview the major advances in the stability formulae of armour layer with the emphasis on definition of damage and the parameters that affect the hydraulic stability of armour layer, discuss some of widely accepted methods, and address key problems and shortcomings.

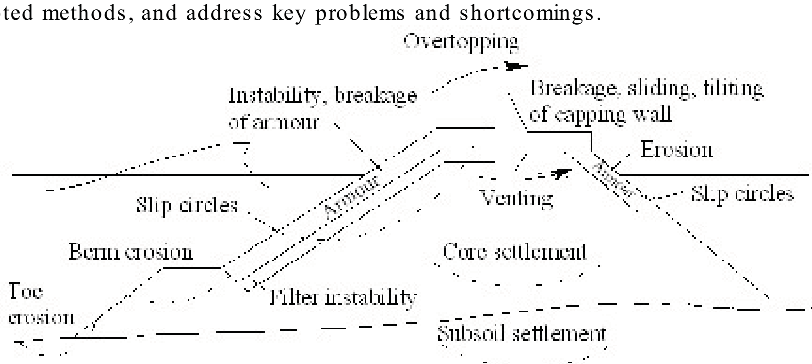

Fig. 1: Breakwater failure modes. Source: Burcharth (1991)

Damage Description:

Generally, the main functions of the armour layer are to decrease incident wave energy and wave run-up and to protect the inner layers of a rubble mound breakwater or a revetment from eroding and washing away. To fulfil these goals, armour units should have enough structural and hydraulic stability in order to stay on the mound layer during the storm attacks, protecting the slope without major damage.

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If incident waves can dislodge enough units from the armour layer, lower layer will directly exposed to the wave action and the mound materials can be extracted from the slope. This critical situation, if the severity of storm does not reduce, will be implicated in the progressive failure of the breakwater depending on the duration of the sea state. Therefore, damage was defined as the amount of displacement of armour units. The term 'displacement' is taken to mean position shifted more than a distance (CEM, 2006). A measurable parameter is essential to evaluate damage level of armour layer, yet it is not easy to give a clear definition of damage.

Damage of Stone Armour Layer:

Surface profiling and counting the number of displaced stones are two main methods of evaluating the damage level. Among pioneers, Iribarren (1938), Hudson (1959), Ahrens (1975), Thompson and Shuttler (1975), and Broderick (1983) measured damage by surface profiling, while Hedar (1960) counted the number of displaced armour units. The details of these measurement methods can be found in Melby (1999), however the method used to determine the damage level were not clearly described in most cases.

Broderick (1983) proposed a dimensionless damage parameter, S, for stone armour layer which is described by:

$$S = A_e / D_{n50}^2 \tag{1}$$

where A_e is the average of eroded cross-sectional area of the armour layer and D_{n50} is nominal diameter of armour stones or in the other words the median equivalent cubical length of the stones:

$$D_{n50} = \left(\frac{M_{50}}{\rho_a} \right)^{1/3} \tag{2}$$

and where M_{50} is median mass of rock grading given by 50% on the mass distribution curve, and ρ_a is mass density of armour units. In the case of concrete units there is only one mass and no grading, and consequently the nominal diameter is described without the subscript 50, i.e. D_n. In fact (1) gives the number of armour stones with median equivalent cubical length, D_{n50}, displaced within a width of D_{n50}.

Although Broderick parameter S is independent of the length of the slope, yet it is not able to differentiate armour stone displacement from profile settlement. Moreover, it does not consider the porosity of armour layer. To measure S, intact profile of the slope is compared with the damaged profile after storm or after specific number of waves in a test, assuming that the profile changes due to the erosion, while the cross-section can be also changed due to the settlement. In order to solve this problem many researchers prefer to count the number of displaced armour stones N in a specified area.

Vidal *et al*. (1995) considered two damage parameters: (I) visual damage parameter, S_V, which is based on counting the number of stones displaced, and (II) profile damage parameter, S_p, which is measured by means of calculating average eroded area on the profiles of the slope. Because of different geometries of trunk and head sections, they proposed separate formulae to calculate damage parameter for each section.
The visual damage in the trunk section was obtained by:

$$S_V = \frac{N \cdot D_{n50}}{(1-n) \cdot X} \tag{3}$$

where N is the number of displaced armour units, n is the porosity of the armour layer, and X is the length of trunk section. The profile damage, S_P, for trunk section was given by the same equation as (1).

Vidal *et al*. (1995) found that in the head of breakwater the region most prone to damage is between levels (SWL+H_s/2) and (SWL-H_s), where SWL is still water level and H_s is significant wave height measured at the toe of the structure. They suggested the arch length $R.\theta$ as the length for the head section covered by angle θ, where R is the mean radius of the damaged area corresponding to the two critical levels. The visual damage parameter for the head of breakwater can be defined as:

$$S_{V,head} = \frac{N \cdot D_{n50}}{(1-n)R \cdot \theta} \tag{4}$$

with

$$R = \frac{B}{2} + \begin{cases} 0.5(H_s + R_c)\cot\alpha; & R_c \leq H_s/2 \\ (0.25H_s + R_c)\cot\alpha; & R_c \geq H_s/2 \end{cases} \tag{5}$$

where α is the slope angle of breakwater, B is the crest width, and R_c is structure freeboard.

For different degrees of damage the accuracy and sensitivity of the damage parameters S_V and S_P differ. If a small number of stones is displaced the visual damage parameter, S_V, is more accurate, but as the damage level increases the profile measurement will be more reliable (Vidal *et al*., 1995).

Burcharth *et al*. (2006) modified the visual damage parameter to link the damage given as number of displaced armour stones N to the Broderick parameter S. To include N in the expression for S, the eroded area can be given as $A_e = V_e / X$ where $V_e = N.D_{n50}^3/(1\text{-n})$ is the eroded volume of the cross-section. The relation between N and S can be explicitly expressed as:

$$S = \frac{N.D_{n50}}{(1-n)X} \tag{6}$$

Damage is generally presented as relative damage, D%, defined as relative displaced units to the total number of units in the armour layer or in a specific zone around Still Water Level (SWL), which is also called active zone. Since in the case of emerged breakwaters most displacements occur approximately in the area from one H_s below SWL to one H_s above SWL, the number of units placed in this zone is often used as the reference number (CEM, 2006). The area between the middle of the crest to one H_s below SWL was considered as active zone in the Shore Protection Manual (1984). These definitions of active zone are not applicable to Low Crested Structures or submerged barriers. Moreover, due to different designs total number of armour units differs for each structure, therefore the results obtained through various studies can not be compared precisely.

Relative damage number, N_{od}, is defined as the number of unites displaced within a vertical strip of width D_n stretching from the bottom to the top of the armour layer (Van der Meer, 1988b). This definition of damage can be easily related to a percentage of damage. N_{od} gives the actual damage, which is related to the number of units in a cross-section with a width of D_n. For a similar damage number, therefore, different percentages of damage may be obtained if the cross-sections are different. The disadvantage of N_{od} is its dependency on the slope length (CEM, 2006).

Unit movement may take place in different ways and each has different contribution to damage. Some of the units may displaced out of layer and completely fail to perform their function. While, some units displaced from their original position may still remain in the eroded area (e.g. two black stones in Fig. 2b) and reach a stable position. In this case, the displaced units may still contribute effectively to the slope protection. Counting method considers all the displaced units as damage regardless of their new position, while (as illustrated in Fig. 2c) profile measurement does not take the units displaced but remains in the eroded area as damage. To reach a conclusion, it should be considered that any type of unit displacement reduces the layer integrity. Hence counting method can lead to damage overestimation on one hand, and profiling method may gives underestimated damage on the other hand.

Damage of Concrete Armour Layer:

Profile measurement is not common for concrete (artificial) armour layers. Instead units displaced out of the layer, or units displaced less or more than a specific distance (e.g. 1 or 0.5 D_n) are of counted.

Armour units may move under wave action but stay in their initial location. This type of movement in which the unit is disturbed but not displaced is defined as rocking. While rocking may not significantly affect stone armour units, it can be more important in the case of concrete units, especially slender units such as dolosse and tetrapods, as rocking may cause breakage. Therefore, rocking may be considered as a potential damage (Yagci and Kapdasli, 2003) in the case of concrete units. Rocking is not typically recorded in the laboratory investigations due to the fact that armour blocks could hardly break in the small scale models.

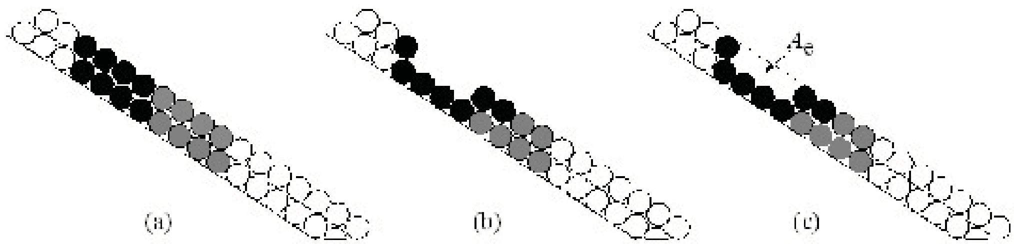

Fig. 2: Different methods of damage evaluation: a) intact profile; b) damaged profile, counting method; c) damaged profile, measurement of eroded area.

Damage Criteria:

Although damage parameters can quantify the degree of damage, they do not provide a physical interpretation of damage level (Vidal *et al*., 1995). Therefore, damage criteria are required to relate the damage observed to the damage measured. Losada *et al*. (1986) classified observed damage into three levels: initiation of damage, Iribarren damage, and destruction. Later, an additional damage level called start of destruction was proposed by Vidal *et al*. (1991). The definitions of these damage levels are as follows:

Initiation of Damage:

A certain number of armour units are displaced to a new position at a distance longer than D_n. This damage leaves holes larger than the average pore size on the armour layer.

Iribarren Damage:

The number of units removed from upper layer is large enough that a unit in the lower layer can be dislodged.

Start of Destruction:

This type of damage is defined as the initiation of damage to the lower armour layer that a number of units are displaced by the wave action.

Destruction:

Material from the secondary (or filter) layer is removed. The armour units leave the slope continuously, and if the severity of the sea state does not reduce, the mound will be destroyed after a sufficiently long period.

It is not reasonable to design armour units for no-damage at all, which requires very large and massive armour units. Instead the term "zero-damage" or "no-damage" defined as "initiation of damage", corresponding to a degree of damage less than 2% by counting or 5% by profiling (SPM, 1984), is used for estimation of armour unit's weight. The "no-damage" criterion is taken typically to be when S is between 1 and 3, and "failure" is assumed when S is greater than 10 (Van der Meer, 1987). Indeed, the term "failure" represents "start of destruction" criterion because after this point damage (hydraulic instability) can turn to a rapid destruction of the structure. For concrete armour units, N_{od}=0.5 was suggested to be the "start of damage" (Van der Meer, 1999).

Stability Formulae:

Many empirical stability formulae have been proposed to predict the stability of armour layer since the 1930s, indicating the importance of this area of research in the field of coastal structure. More than 20 formulae, for instance, were proposed between 1933 and 1988 (Koev, 1992). One should bear in mind that the stability formulae of armour layers are based on small scale model experiments and consequently are influenced by the scale effect. These formulae are good measures for preliminary design, but the uncertainties of formulae and test conditions should be considered (CEM, 2006). For large structures, design codes generally recommend model testing before actual construction.

Formulae for Rock Armour Units:

Iribarren Formula:

Rock armour units rely to a great extent on their weight in order to gain the hydraulic stability. Therefore, stability formulae generally evaluate the minimum required weight of armour unit to resist the maximum wave forces allowing a reasonable degree of damage. A qualitative stability ratio can be defined as the wave forces (i.e. drag force F_D and lift force F_L) divided by the restoring force (CEM, 2006), i.e. armour buoyant weight:

$$\frac{F_D + F_L}{F_G} \approx \frac{\rho_w D_n^2 v^2}{g(\rho_a - \rho_w) D_n^3} = \frac{v^2}{g \Delta D_n} \tag{7}$$

where ρ_w and ρ_a are mass densities of water and armour unit, respectively, $D=(\rho_a/\rho_w)-1$ is relative buoyant density of armour unit, D_n is the unit nominal diameter, and v is instantaneous flow velocity. For a breaking wave height of H, the fluid velocity can be computed using the wave celerity $v \approx (gH)^{1/2}$ erefore the stability ratio can be obtained by:

$$N_s = \frac{v^2}{g \Delta D_n} = \frac{H}{\Delta D_n} \tag{8}$$

N_s is called stability number and appears in most of stability formulae.

Iribarren (1938) proposed the following formula based on simple relations:

$$\frac{H}{\Delta D_n} = K(\tan\phi \cos\alpha \pm \sin\alpha) \tag{9}$$

where φ is the angle of repose of the armour, a is the slope angle of the structure, and K is a coefficient that depends most on the shape of the armour units and damage level.

Hudson Formula (Shore Protection Manual 1984):

After extensive tests with monochromatic (regular) waves, Hudson (1959) introduced a combination of influences, K_D, and replaced (cosα - sinα) with $(\cot\alpha)^{1/3}$. He assumed that in the case of rubble mound structures φ=1. This reduces (8) to well known Hudson formula:

$$\frac{H}{\Delta D_n} = (K_D \cot\alpha)^{1/3} \quad \text{or} \quad M_{50} = \frac{\rho_a H^3}{K_D \Delta^3 \cot\alpha} \tag{10}$$

where M_{50} is medium mass of units. The formula does not cover the slopes steeper that 1:1.5 or gentler than 1:4. The coefficient K_D represents all the factors that may play a role in the stability without clarifying the factors and the way they affect the stability. Many experimental tests were carried out and the results published by U.S. Army Corps of Engineers (USACE) in the Shore Protection Manual (SPM). In 1984 edition, SPM proposed more conservative design recommendation compared to SPM 1977. The values of K_D introduced by SPM (1984) are considerably smaller (more conservative) in the case of breaking waves. It seems that after a large number of breakwater failure during the late 1970s (e.g. Sines breakwater in Portugal), USACE aimed to increase the safety margin of armour layer.

Hudson formula has been widely used because of its simplicity, however it has many shortcomings. The formula does not take into account important parameters such as wave period, wave spectrum shape, groupiness of waves, porosity (permeability) of the structure, relative crest height, damage level (loss of stability), and sea state duration. Instead, all of these unnoticed parameters are represented by K_D, a key parameter that is required to be obtained through experiment. Consequently, many experimental studies were carried out to estimate K_D for different types of armour units under breaking and non-breaking waves, but these experiments were frequently resulted in modification of Hudson formula and a large number of stability formulae were devised.

Van Der Meer Formula:
Rayleigh Sea States:

Most of earlier stability formulae are based on physical model experiments using monochromatic (regular) waves where the same wave with the same force (wave height) is applied on the structure during each wave attack. Regular waves result in a fast equilibrium in response to the repeated forces with significant wave height (CEM, 2006) and, in consequence, the duration of tests with regular waves is not likely to influence the damage level. This explains why the effect of storm duration (or number of waves) and the spectrum shape were not considered in the earlier stability formulae (Van der Meer, 1988a). Wavemaker developments, around 1970, made it possible to generate random (irregular) waves. Thompson and Shuttler (1975) conducted an extensive investigation on the stability of rubble mound revetments under irregular wave attack. Van der Meer (1988a) re-analysed their data and discovered a clear relation between damage level, wave period, and duration of the sea state.

The relation between the structure slope tanα and wave steepness $s_{om}=2\pi H_s/gT_m^2$ can be described by the surf similarity parameter (Iribarren parameter), ξ_m, (Battjes, 1974):

$$\xi_m = T_m \tan\alpha \Big/ \sqrt{2\pi H_s/g} \tag{11}$$

where T_m is the mean wave period (calculated from the spectrum or from the wave record), H_s is significant wave height at the toe of the structure, and g is gravity (9.81 m.s^{-2}). Surf similarity parameter is often used to distinguish different breaker types on a beach or structure.

Based on the study of Thompson and Shuttler (1975) and on an extensive investigation on rubble mound breakwater exposed to irregular, non-breaking, non-depth-limited waves Van der Meer (1988a) proposed a comprehensive stability formula for rock armour layers that takes more influencing factors into consideration, namely damage level, number of waves, and permeability of the structure. Van der Meer formula is given by the following expressions:
for plunging (breaking) waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2 P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \tag{12}$$

for surging (non-breaking) waves:

$$\frac{H_s}{\Delta D_{n50}} = 1.0 P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha} \quad \xi_m^{P} \tag{13}$$

where the coefficient P is the notional permeability parameter, S is damage level, N is number of waves which takes duration of sea state into account, and α is seaward slope angle of the structure. The transition from plunging to surging waves can be calculated by a critical value of ξ_{mc}:

$$\xi_{mc} = \left(6.2 P^{0.31} \sqrt{\tan\alpha}\right)^{1/(P+0.5)} \tag{14}$$

The formula covers the slopes with cotα between 1.5 and 6. The significant wave height, H_s, in this formula is defined as the average of the highest 1/3 of the waves, $H_{1/3}$, in a time series, or obtained from the spectrum: $H_s=4(m_0)^{1/2}$, where m_0 is the zeroth moment of the energy density spectrum.

Vidal *et al.* (2006) showed that the average wave height of the 50 highest waves reaching a structure in its useful life, H_{50}, can describe the damage in Rayleigh-distributed sea states during the structure service lifetime and therefore can be applied instead of the number of waves. They took H_{50} into account to transfer Van der Meer Formulae into the following equations:

$$\frac{H_{50}}{\Delta D_{n50}} = 4.4 P^{0.18} S^{0.2} \xi_m^{-0.5} \text{ ; for plunging waves} \tag{15}$$

$$\frac{H_{50}}{\Delta D_{n50}} = 0.716 P^{-0.13} S^{0.2} \sqrt{\cot\alpha}\, \xi_m^{P} \text{ ; for surging waves} \tag{16}$$

Non-rayleigh Sea States:

In shallow water, wave height distribution at the breakwater toe is not Rayleigh-distributed. Substitution of $H_{2\%}$, wave height exceeded by 2% of highest waves in a sea state, for the H_s was proposed by Van der Meer (1988a) on depth-limited foreshores. For a non-Rayleigh distribution, (12) and (13) can be re-arranged with the ratio $H_{2\%}/H_s$=1.4:
for plunging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7 P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \tag{17}$$

and for surging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.4 P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha} \quad \xi_m^{P} \tag{18}$$

Note that the ratio $H_{2\%}/H_s$ is equal to 1.4 in Rayleigh distributed sea state, but in shallow water this ratio decreases due to wave breaking (CEM, 2006) and the actual wave heights should be known.

Permeability:

The armour layer stability is obviously influenced by permeability of the structure, depending on the size and grading of sublayers, filter layers, and core stones. The larger permeability results in the higher stability. The reason for the higher stability is more water can penetrate into the more permeable structure during wave run-up and consequently the smaller forces will be on armour units. Notional permeability coefficient, *P*, was aimed to take permeability into account, but it does not represent the porosity. Fig. 3 illustrates examples of *P*.

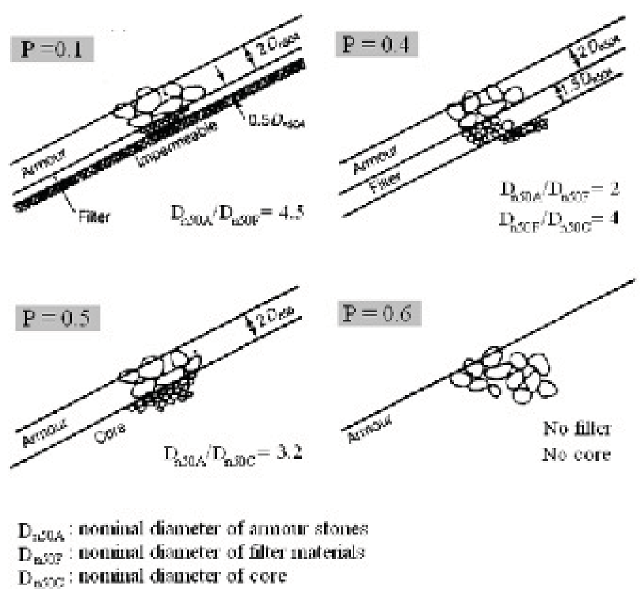

Fig. 3. Notional permeability coefficients. Source: Van der Meer (1988a).

Formulae for Concrete Blocks:
Double Layer Armour:

Quarry stone has limited maximum size that results in the slope angle reduction, but concrete blocks can be fabricated as heavy as 20-30 tons. Thus, it is not reasonable to decrease the slope steepness when using concrete blocks. Breakwaters with artificial interlocking armour units are generally built with steep slopes in the order of 1:1.33 to 1:1.5. Therefore this slope angle is usually chosen for tests on the rubble mound breakwater armoured with concrete blocks. Van der Meer (1988b) conducted a study on cubes, tetrapods, and accropode and proposed stability formulae for each one. Since the investigation was limited to only one cross-section, the slope angle and the breaker parameter, ξ_m, are not present in the stability formulae.

Cubes and Tetrapods:

Van der Meer (1988b) expressed the stability formula for cubes with relative damage level, N_{od}, caused by irregular waves with significant height H_s and mean period T_m composed of N waves with the wave steepness s_{om} by:

$$\frac{H_s}{\Delta D_n} = \left(6.7\,\frac{N_{od}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} \tag{19}$$

and for tetrapods:

$$\frac{H_s}{\Delta D_n} = \left(3.75\left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5} + 0.85\right) s_{om}^{-0.2} \tag{20}$$

For “no-damage” criterion (N_{od}=0), equations (18) and (19) reduce to (21) and (22), respectively:

$$\frac{H_s}{\Delta D_n} = 1.0 \quad s_{om}^{-0.1} \tag{21}$$

$$\frac{H_s}{\Delta D_n} = 0.85 \quad s_{om}^{-0.2} \tag{22}$$

Since no-damage is a very strict criterion, it is more cost-effective to design the armour units for the “start of damage” criterion. The start of damage for rock layers was taken to be S= 2 to 3 and for concrete units N_{od} = 0.5 (Van der Meer, 1988b).

Dolos:

Holtzhausen and Zwamborn (1992) investigated the stability of Dolos and recommended the stability formula with respect to the relative damage, N_{od}:

$$N_{od} = 6250\left(\frac{H_s}{\Delta^{0.74} D_n}\right)^{5.26} s_{op}^{3}\, w_r^{20 s_{op}^{0.45}} + E \tag{23}$$

where w_r is waist ratio of Dolos with the applicable range of 0.33 to 0.4, and E is Error term. The higher waist ratio gives the stronger Dolos. The error term E describes the reliability of the formula. It is assumed to be normally distributed and has a mean value of zero, and a standard deviation σ(E):

$$\sigma(E) = 0.01936\left(\frac{H_s}{\Delta^{0.74} D_n}\right)^{3.32} \tag{24}$$

Single Layer Armour (New Generation):

The design of the new generation of armour blocks were developed by improving the interlocking. This type of concrete armour units are generally placed in single layer and are complex in shape. Accropode as the first single layer armour unit was invented by Sogreah in 1980. It has been used widely around the world in more than 100 projects. Later, Core-Loc was introduced by the US Army Corps of Engineers in 1994, and more recently, in 2003, Xbloc was introduced by Delta Marine Consultants (DMC, 2009).

As a result of high integrity, single layer blocks can be stable to higher wave heights compared to double layer systems (Van der Meer, 1999). Consequently, "start of damage" in a single layer system occurs at very high stability numbers and is usually followed closely by a sudden "failure". Design stability number may be obtained by applying a safety coefficient of about 1.3 to 1.5. As illustrated in Fig. 4, the storm duration and wave period show no influence on the stability of accropode (Van der Meer, 1988b). Other interlocking armour blocks, e.g. Core-Loc and Xbloc (DMC, 2009), exhibit similar stability behaviour, however limited test results have been published (Van der Meer, 1999). The stability, therefore, can be described simply by a fixed stability number. Table 1 presents stability numbers, related to the damage criteria, for Accropode, Core-Loc, and Xbloc.

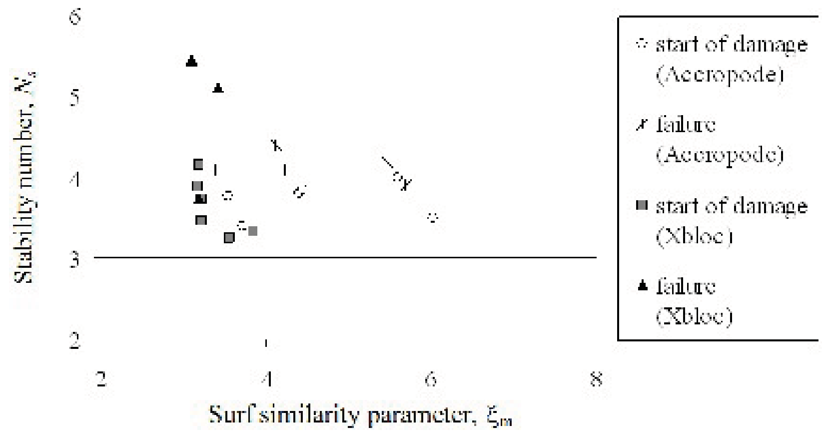

Fig. 4: Hydraulic stability of interlocking armour units results based on model test by Van der Meer (1988b) and DMC (2009).

Table 1: Stability numbers corresponding to damage criteria

Type of armour	Start of Damage	Failure	Design
Accropode	3.70	4.10	2.50
Core-Loc	-	-	2.78
Xbloc	3.50	4.00	2.80

Stability of Low Crested and Submerged Structures:

Conventional breakwaters are generally designed to allow small amount of overtopping. As a result front slope will be mainly influenced by the wave action. On the other hand, when crest is placed at lower levels that the structure is frequently subjected to overtopping, damage will also occur to the crest and the rear slope. However as an advantage, structure will be more stable in view of the fact that wave energy can pass over the crest and consequently the armour units in front side can be smaller compared to the non-overtopped structures (Van der Meer and Pilarczyk, 1990).

Givler and Sørensen (1986) carried out about 45 stability tests on the submerged structure under regular wave attacks. Van der Meer (1988a) performed 2-D model tests on low crested structure exposed to non-breaking irregular waves. Van der Meer and Pilarczyk (1990) analysed the work of Van der Meer (1988) and the Givler and Sørensen (1986) and proposed the following expression for the stability of low crested and submerged breakwaters:

$$\frac{h_c}{h} = (2.1 + 0.1S)\exp(-0.14N_s^*) \tag{25}$$

where $N_s^*=N_s\, s_p^{-1/3}$ is Ahrens (1984) spectral stability number , h is water depth at the structure toe, and h_c is structure height.

It can be concluded from (25) that in the case of submerged breakwaters the stability mainly depends on the relative crest height (structure freeboard), damage level, and the spectral stability number (Van der Meer and Pilarczyk, 1990).

NOTATIONS

- α Slope angle of the structure
- Δ Relative buoyant density of armour unit $((\rho_a/\rho_w)-1)$
- φ Angle of repose of the armour units
- ρ_a Mass density of armour units
- ρ_w Mass density of water
- ξ_m Surf similarity parameter (Iribarren parameter) for irregular waves
- A_e Averaged cross-section eroded area
- B Crest width of the structure
- D_n Nominal diameter of armour unit
- D_{n50} Median value of D_n
- $D_{\%}$ Relative damage
- g Gravity [m/s^2]
- h Water depth at the structure toe
- h_c Structure height
- H_{50} Average wave height of the 50 highest waves reaching a breakwater in its service lifetime
- $H_{2\%}$ Wave height surpassed by the 2% highest waves in a sea state
- H_s Significant wave height
- M_{50} Median mass of rock grading given by 50% on the mass distribution curve
- n Porosity of the armour layer
- N Number of waves of a sea state
- N Number of displaced armour stones
- N_{od} Relative damage number
- N_s Stability number $(H_s/\Delta D_n)$
- N_s^* Ahrens (1984) spectral stability number
- P Notional permeability parameter of the breakwater
- R_c Structure freeboard
- s_{om} Wave steepness
- S Damage parameter
- S_V Visual damage parameter
- S_p Profile damage parameter
- T_m Average wave period in a sea state
- V_e Averaged eroded volume of the cross-section
- w_r waist ratio of Dolos
- X Length of trunk section

ACKNOWLEDGMENTS

The support of University of Malaya and IOES (Institute of Ocean and Earth Sciences), is gratefully acknowledged by the authors.

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