

CHAPTER 117

STONE MOVEMENT ON A RESHAPED PROFILE

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Abstract

Physical model tests were carried out aiming to provide information on armor stone movements in a berm breakwaters. The following items were examined: displacement threshold, frequency and length of stone displacements. Data were obtained from the observation of cumulative displacements at the end of each wave attack and from video records during the attacks; basic statistical analysis was performed. Threshold conditions, moving frequency and cumulative displacements are expressed as function of a modified mobility number, as stone mobility increases with both wave height and wave period.

Information on mobility and displacement is eventually used to estimate longshore transport and abrasion. The longshore model is based on the assumption that stones move during up- and down-rush in the direction of incident and reflected waves; the model compares favourably with experimental existing results. The abrasion model is based on a proportionality assumption between abrasion volume and abrasion work; the proportionality coefficient characterizing the stone material resistance to abrasion is derived from Latham and Poole mill abrasion tests.

1. Introduction

The berm breakwater concept, i.e. the conscious design of a rubble mound breakwater for dynamically stable conditions, is relatively new. The central idea is to maintain the profile stable in presence of extreme waves accepting some displacements, whose effect on the reshaped profile is irrelevant, and using rock armor of smaller size than required for a traditional (no movement) design criterium. In such a way available rock may be used in some cases where, according to the traditional design, this would not have been possible. Aiming to use available rock, even stones of poor quality may sometimes be used and were actually used (Sigurdarson & Viggosson, 1994) with satisfaction.

Berm breakwaters introduce some new shapes and variants to well known phenomena but also some completely new topics in breakwaters analysis.

In the first class we may include the strongly convex profile where breakers hit the armor layer and its high permeability caused by its greater thickness; these new

shapes require at least a verification of formulae and equivalence criteria used in the case of traditional breakwaters to estimate armor stability, reflection, wave run-up, overtopping and transmission.

In the second class we should include the reshaping process under eventually static or truly dynamic conditions, the abrasion or breakdown of sliding or colliding rock units and the longitudinal transport of mobile stones.

Run-up on a plane (uniform slope) impermeable surface or on a permeable rubble mound is experimentally well documented. Engineers, needing for an estimate of the behavior of more complex profiles, refer normally to the composite slope method proposed by Saville (1958). Herbich & al. (1963) pointed out the need for some correction for very wide berms; when the berm width is greater than 15% of the wave length its relative effectiveness in reducing wave run-up is strongly reduced. Pilarczyk (1990) describes similar results showing that the critical width is dependent on the breaker type. Van der Meer & Stam (1992) propose formulae describing wave run-up on rock slopes where run-up depends on the probability level (frequency of greater values), on the surf similarity parameter and on mound permeability. Ahrens & Ward (1991) express the reduction in run-up due to the berm as function of the berm geometry and observe that, while the reduction in run-up is modest, the improvement in stability of the revetments is substantial.

We have quoted just some of the information about run-up that can be found in the literature, as an example of how the knowledge of traditional breakwater behavior has been transferred to breakwaters with a relevant berm.

Among the other class of problems, the development of the dynamically stable seaward profile of a reshaping breakwater can be predicted by mathematical models based on extensive physical tests, cf. van der Meer (1988).

The movement of stones along the active profile is a peculiar characteristics of dynamic stability and this causes inherently some abrasion of stones and, when wave attack is oblique, some along-structure transport.

The effect of abrasion can nowadays be evaluated in a semi quantitative manner by the method proposed by Latham (1991) and included in CIRIA-CUR (1991) manual. The method is based on the similarity of armor units weight degradation in nature and in a standardized mill abrasion test (Latham & Poole, 1987); the scale factor for the conversion from milling time (measured as thousands of revolutions) to prototype time (measured as years) is given as the product of 9 factors accounting for: incident wave energy (~ 10), zone in the structure (~ 10), waterborne attrition agents (~ 7), mobility of armor in design condition ($\sim 4-10$), size of the armor stones ($\sim 2-10$), meteorological climate (~ 5), stone grading (~ 2), stone initial shape (~ 2), concentration of wave attack (~ 2); the numbers within parentheses are the measures of the ranges of possible factor values and therefore of the relative influence of the factor.

The along-structure transport S can nowadays be evaluated with the formula proposed by Van Hijum & Pilarczyk (1982) which may be written as:

$$\frac{S}{g \cdot D_{n50}^2 \cdot T_p} = 0.0012 \cdot \Delta^2 \cdot \frac{H_s \sqrt{\cos\beta}}{\Delta D_{n50}} \cdot \left(\frac{H_s \sqrt{\cos\beta}}{\Delta D_{n50}} - 7 \right) \cdot \sin\beta \quad (1)$$

The above equation was established in the range $N_s := H_s/\Delta D_{n50} \simeq 12-27$ and is supposed to be usable in the range $10 \div \infty$, since when the mobility index is greater than 50 the formula becomes

$$S \simeq 0.0012 \cdot \pi \cdot H_s^2 \cdot c_{op} \cdot \sin 2\beta$$

i.e. similar to CERC formula.

The formula returns the volume transport rate; it gives anyways results of poor utility for actual berm breakwaters, since in almost every case $N_s < 7$ and the formula returns no transport at all.

A second formula was proposed by Vrijling & al. (1991):

$$S = 4.8 \cdot 10^{-5} (H_0 T_0 - 100)^2 \quad (2)$$

The formula gives the along-structure transport measured as number of stones per wave and was verified in the range 100–400 of the mobility index used, i.e. in a range of mobility more representative of actual berm breakwaters. The absence of obliquity, for which the formula was criticized, may be interpreted as the effect of the range of the tested obliquity -25° and 50° - wide but in the range of values where the sensitivity is irrelevant because the effect is maximum. The formula may be however easily adjusted including a factor $\sin 2\beta$.

The mobility index used in the formula $-H_0 T_0$, where $H_0 := N_s$ and $T_0 := T_p \sqrt{g/D_{n50}}$ is related to the more usual mobility number N_s by the trivial relation

$$H_0 T_0 = N_s \sqrt{2\pi N_s / (\Delta s_p)} \quad (3)$$

and, for the usual range of wave steepness ($s_p \simeq 0.03$), the range of mobility where the formula was tested corresponds to $N_s = 4-10$.

If berm breakwaters are designed according to Burcharth & Frigaard (1987) recommendations ($N_s \leq 3.5$ or 4.5 in the trunk under oblique and long or respectively steep waves; $N_s \leq 3$ in the roundhead) the formula returns no transport at all and, as the one mentioned before, doesn't provide any information about the accepted "damage".

The present study:

- synthesizes physical model tests performed, cf. Tomasicchio & al. (1992) and Lamberti & al. (1994), aiming:
 - to determine conditions characterizing the incipient movement of stones along the dynamically stable profile,
 - to quantify stone movement frequency and amplitude for a sufficiently wide set of conditions and
 - to establish a relationship between movement characteristics and wave conditions in the range of mobility typical of a berm breakwater;
- shows how this information may be used in order to provide a longshore transport model calibrated in the mobility range typical of berm breakwaters, and to rationalize Latham method for the estimate of armor stone degradation.

2. Model tests

Physical model tests were performed using two different wave flumes, one at Danish Hydraulic Institute (DHI) and the others at ESTRAMED Pomezia Italy in 1993. The test set-up is described in more detail in Lamberti & al. (1994). The tests consisted in a reshaping phase carried out with the design wave attack, followed by stone movement measurements carried out with lower wave intensity. Profile surveys and video records were performed during the investigation. The offshore significant wave height used during reshaping was fixed at $H_{so} = 0.18$ m; due to shoaling and breaking wave height incident on the structure differs somewhat from the offshore one; the significant height of waves incident on the structure H_s was systematically used as the wave intensity parameter. Armour stone size was chosen in order to give a mobility number $N_s = H_s / \Delta D_{n50} \approx 3$ under the reshaping (design) conditions.

For all tests at DHI a fixed initial profile was used (fig. 1); the channel had a constant depth ($h = 0.60$ m), the relative wave height was low ($H_{so}/h < 0.3$) and waves broke essentially on the structure.

In the tests performed at ESTRAMED the channel presented a foreshore slope 1:20 from 0.60 m depth in the generating area to 0.34 m depth, followed by a more gentle 1:100 slope reaching $h = 0.30$ m water depth at the structure toe; the range of offshore significant wave height is the same. Waves were moderately limited by water depth.

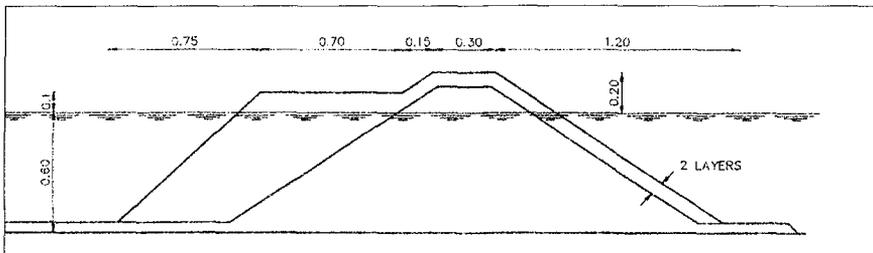


Fig. 1 As built structure in deep water

Each test series started with an initial reshaping phase of the berm-breakwater, composed by 6 wave attacks of 1000 waves each and reaching an almost equilibrium seaward profile of the breakwater. The phase was followed by further 1000 wave attacks of increasing intensity, starting at $N_s \approx 2$ and increasing up to the reshaping wave conditions, aiming to analyze stone movements. During both phases irregular waves were used derived from a Pierson-Moskowitz spectrum with different significant wave steepness.

During the tests the offshore and the incident wave characteristics were measured and the reshaped profiles surveyed, as shown in fig. 2.

Data about displacements were obtained from two sources: video-records of individual stone movements during the wave attacks and observations of the cumulative displacements at the end of each wave attack.

The intensity of stone movements was experimentally defined through the number of stones displaced from the active profile: the convex part of the reshaped profile from the *step* to the *crest* (van der Meer, 1988).

Two non dimensional indexes are used because they naturally appear in the longshore transport and in the abrasion models: the traditional damage index N_{od} , defined as the ratio of the number of displaced stones to the number of the stones in a longitudinal line of the observation area, and the *surface damage level* S_s , defined similarly as the ratio of the number of displaced stones to the number of stones in the upper one grain layer in the active profile portion of the observation area.

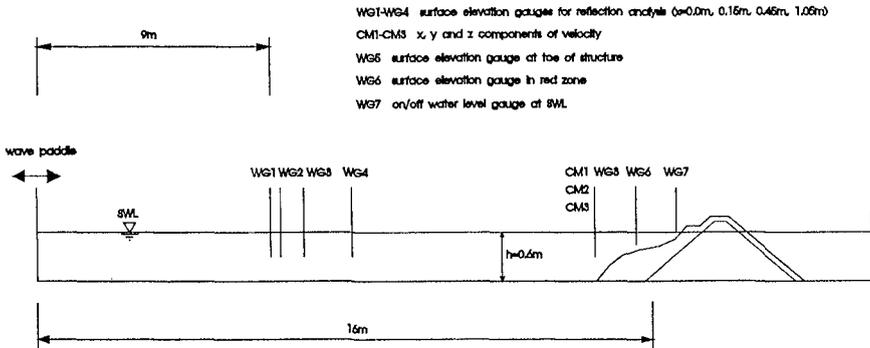


Fig. 2 Model test set-up in deep water

3. Threshold of movement

The threshold of stone movement along the reshaped profile was analyzed in deep and shallow water conditions. Movement on the profile is measured by the damage index:

$$N_{od} := N_d \cdot D_{n50} / B \quad , \quad (4)$$

or by the surface damage level:

$$S_s := N_d \cdot D_{n50}^2 / A \quad , \quad (5)$$

where N_d , B and A are respectively: the number of stones displaced from the active profile at the end of the 1000 waves attack, the width of the observed area and the area itself.

Referring only to DHI model tests, small values of the damage, i.e. $S_s < 0.05$ roughly corresponding to *zero damage* conditions of SPM (1984), were observed to depend mainly on the *stability number* $N_s := H_s / \Delta D_{n50}$ with a minor but clearly observable influence of wave steepness. A good correlation was observed between damage and a modified stability number:

$$N_s^{**} \propto N_s \cdot s_m^{-1/5} \quad .$$

This relation suggests that the relevant wave intensity parameter obtained combining wave height and wave period is the onshore energy flux or the breaker height, cf. Lamberti & al. (1994) and Komar & Gaughan (1972).

The following tests showed that the same relation is satisfied for a berm breakwater also in shallow water conditions (see fig. 3) if as wave intensity parameter the incident wave height is considered, which has exceedence frequency almost equal to the frequency of stone movements, i.e. for practical purposes $H_{1/50}$.

In order to obtain a mobility index that accounts for the above mentioned observed phenomena and exhibits on average conditions the same values of the traditional stability number N_s , the following definition of the modified stability number is introduced.

$$N_s^{**} := \frac{H_{k0}}{C_k \Delta D_{n50}} \cdot \left(\frac{s_{m0}}{s_{mk}} \right)^{-1/5} \cdot (\cos \beta_0)^{2/5} \simeq \frac{0.89 \cdot H_{kb}}{C_k \Delta D_{n50}} \quad (6)$$

where $H_k = H_{1/50}$, $C_k = 1.55$, i.e. the ratio $H_{1/50}/H_s$ according to Rayleigh distribution of wave heights, and $s_{mk} = 0.03$. The last term includes the effect of wave obliquity according to the hypothesis that the onshore energy flux is the relevant wave intensity parameter.

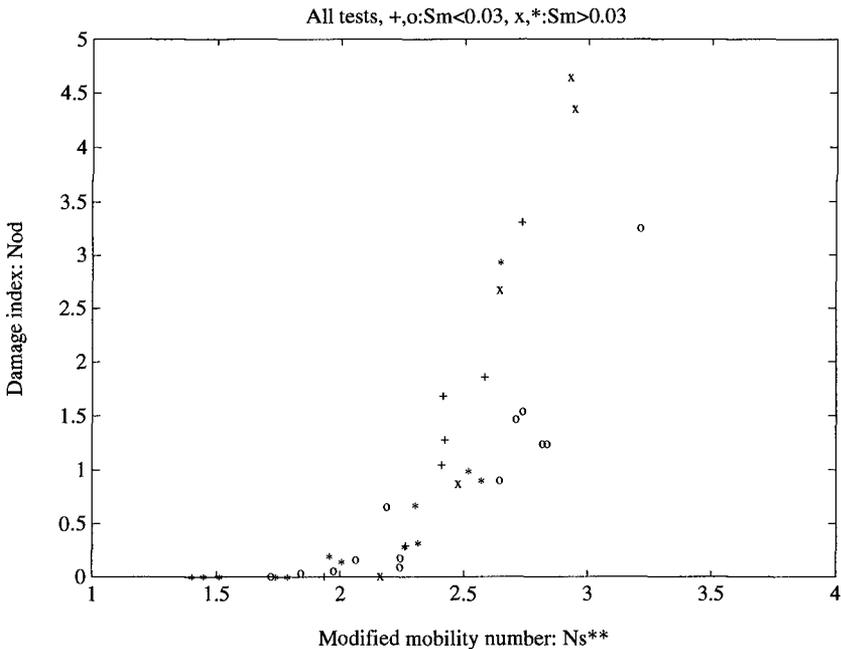


Fig. 3 The empirical relation between the damage in 1000 waves N_{od} and the modified stability number N_s^{**} . +, x : deep water; o, * : shallow water.

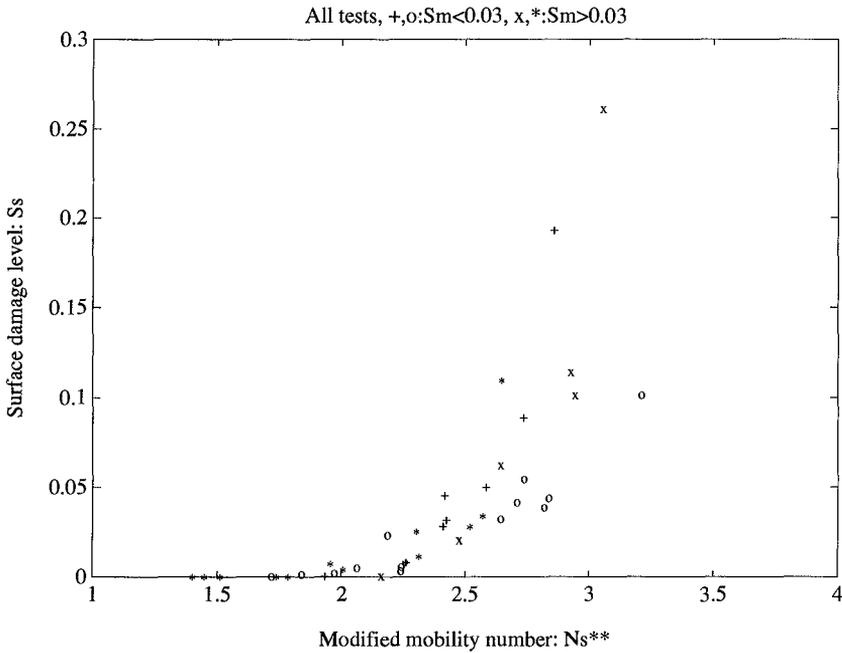


Fig. 4 The empirical relation between frequency of stone movement in 1000 waves S_s and the modified stability number N_s^{**} . +, x : deep water; o, * : shallow water.

Strict threshold conditions correspond to $N_s^{**} \approx 2$, or approximately to $H_0T_0 \approx 33$. There is therefore a rather wide range of conditions where some stone movements occur below the threshold conditions of formulae (1) and (2).

4. Stone mobility

An estimate of the frequency of stone movement can be obtained dividing the damage level by the length of the observation period (≈ 1000 waves).

The analysis of video records provided some simple statistical description of stone displacements; the mean and standard deviation of the interdisplacement period and of the displacement length were evaluated. The mean interdisplacement period resulted in good agreement with the global frequency derived before (frequency equal to the reciprocal of mean interdisplacement period).

Figure 5 shows the relation between the nondimensional mean displacement D_s^{**} and the modified mobility number N_s^{**} for deep water tests. In order to obtain a unique relation between the average displacement l_d scaled with the nominal stone

size and mobility the former should be multiplied by the square of the wave aspect ratio at structure toe (ratio between the vertical and the horizontal dimensions of the particle horbits). This factor includes the effect of both the water depth and the wave period. If the exponent was 1, the displacements would increase respect to wave height as the horizontal dimension of the horbits. The greater exponent was required for a complete compensation of shallow water effects.

The equations representing the average empirical correlation for mean damage level (frequency) and mean displacement length are:

$$N_{od} = 1.8 \cdot N_s^{**} \cdot (N_s^{**} - 2.0)^{2.2}, \text{ for } N_s^{**} > 2.0 \tag{7.1}$$

$$S_s = 0.14 \cdot (N_s^{**} - 2.0)^{2.2}, \text{ for } N_s^{**} > 2.0 \tag{7.2}$$

$$D_s^{**} := l_d t g h^2 (kh) / D_{n50} = 1.4 \cdot N_s^{**} - 1.3 \tag{8}$$

They represent a refinement of those published in Lamberti & Tomasicchio (1994), including more data on shallow water conditions.

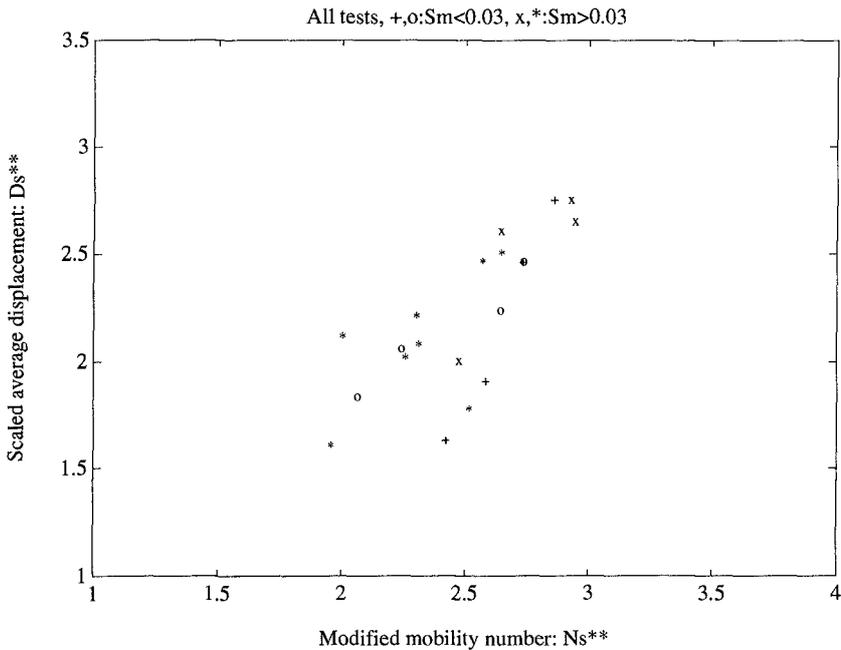


Fig. 5 The empirical relation between mean displacement length and the modified stability number N_s^{**} . +, x : deep water; o, * : shallow water.

5. The longshore transport model

The two equations (7.1) and (8), or other similar ones giving the frequency and mean length of displacements, combined with the hypothesis that under oblique wave attack the displacement takes place in the direction of wave propagation, or an equivalent one, can be reworked in a long-structure transport formula for very low mobility levels, for which eq. (1) and (2) are not accurate.

A particle will pass through a certain control section in a small time interval Δt , if and only if it is removed from the updrift area of extension equal to the longitudinal displacement length l_d (if the intervall is small the probability of multiple movements is small of 2nd order, and the corresponding event may be treated as having 0 probability). The number of particle removed from this area is N_{od} in 1000 waves per one diameter long strip, i.e. the number of particles traversing the control section

$$S \cdot \frac{\Delta t}{T_m} = \frac{l_d \cdot \sin \beta_{kb}}{D_{n50}} \cdot \frac{N_{od} \Delta t}{1000 T_m}$$

i.e.:

$$S = \frac{l_d}{D_{n50}} \cdot \frac{N_{od}}{1000} \cdot \sin \beta_{kb} \tag{9}$$

where the characteristic wave obliquity at breaking point is evaluated from the wave characteristics at the measure point according to the procedure (the breaker index γ is set equal to 1.42 according to Komar & Gaughan, 1972):

$$H_{kb} = (H_k^2 \cdot c_g \cdot \cos \beta \cdot \sqrt{\gamma/g})^{2/5}$$

$$c_{kb} = \sqrt{g H_{kb} / \gamma}$$

$$\sin \beta_{kb} = \sin \beta \cdot c_{kb} / c$$

For the sake of simplicity we have assumed in the presentation that the length of displacements and the time between displacements are deterministic, but the result is asymptotically exact whichever are the distributions of the displacement length L_d and of the residence (between movements) time T_r , provided they are independent: the mean velocity of each mobile particle (time derivative of the mean particle position) is asymptotically $E(L_d)/E(T_r)$. The same relation is exact at any time if the event process of movements is Poissonian.

Fig. 6 shows the experimental data of Burcharth & Frigaard (1987) and van der Meer & Veldman (1992), interpreted according to the structure of formula (9). Fig. 7 shows the comparison between formula (9), which the substitutions corresponding to (7.1) and (8), and the experimental results. No calibration was necessary and the comparison is a verification of the approach. The four point which do not conform to the general trend refer to van der Meer data with 50° obliquity, for which no offshore equivalent conditions exist due to the great inshore obliquity.

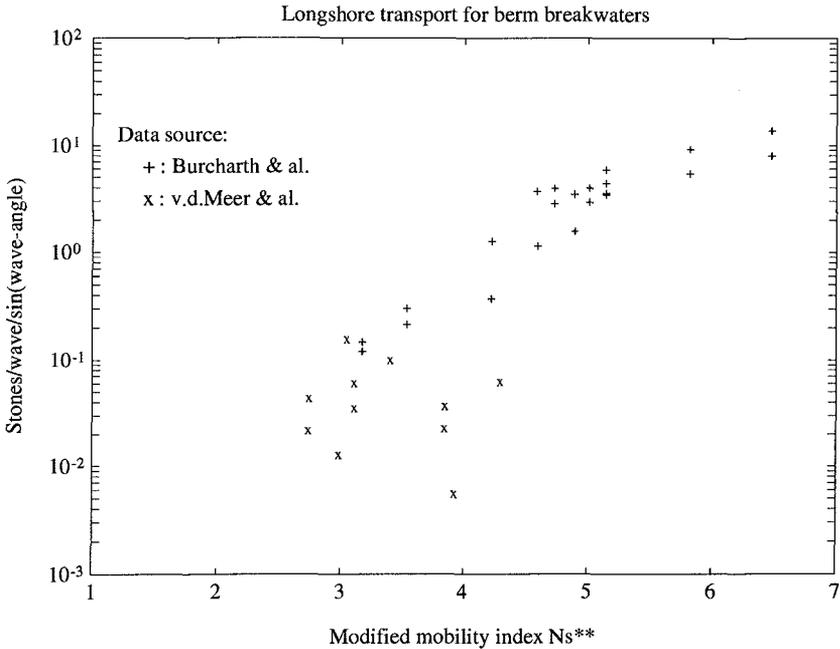


Fig. 6 Relation between the measured longshore transport at berm breakwaters and mobility.

6. The abrasion model

Information on mobility and displacement is eventually used to estimate armor stone abrasion for a given wave climate. The model is based on a proportionality assumption between abrasion volume and abrasion work, which is an ancient hypothesis due to Reye (1848) widely used in tribology; the proportionality coefficient, characterizing the stone material resistance to abrasion, may be easily derived from Latham and Poole mill abrasion tests.

The essence of Reye hypothesis is that between the two factors of the abrasion work, friction stress and sliding distance, a perfect compensation is possible: i.e. if both are altered but the product does not change so does erosion. Or when the energy is dissipated in the impacts, the hypothesis assumes that all the dissipated energy is spent for the disgregation of small volumes of material near the contact. In both cases it is evident that if the material resistance is greater than the applied local stress, part of the work may be converted into heat and not into abrasion. Some scale effects are therefore to be expected, since stress increase considerably with the size of the units while the material resistance remains unchanged.

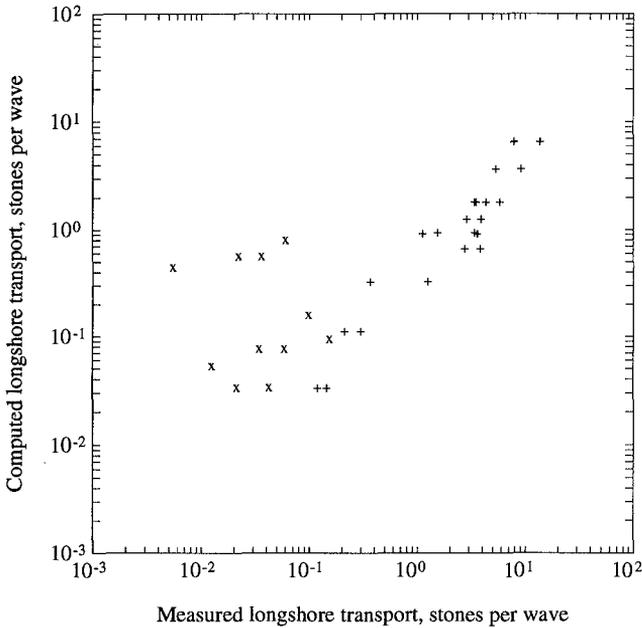


Fig. 7 Comparison between formula (9) and measured longshore transport in berm breakwaters

In the original Latham & Poole (1987) mill a Φ 176 mm cylinder is half-full with rock grains of size 25-32 mm submerged in water and is kept turning at 26-27 rpm speed (a second version of the mill -Latham, 1991- has Φ 195 mm and rotates at 33 rpm giving 1.5 higher abrasion per rotation); due to the internal friction the surface of the granular forms an angle with the horizontal equal to dynamic internal friction angle φ_d ; the gravity center results eccentric and the resisting moment, the work dissipated by abrasion per revolution \mathcal{L}_u , and the relative volume loss can be easily expressed

$$\begin{aligned}
 b &\simeq 4R/3\pi \cdot \sin\varphi_d \\
 \mathcal{L}_u &= W \cdot b \cdot 2\pi \simeq 8/3 \cdot (\rho_a - \rho_w)g \cdot V \cdot R \cdot \sin\varphi_d \\
 -\Delta V &= k \cdot \mathcal{L}_u \cdot N
 \end{aligned}
 \tag{10}$$

where b is the eccentricity of rock gravity center, W and V are respectively the submerged weight of the grains and their volume, ΔV is the volume lost by abrasion and k is the proportionality constant characterizing rock material; $1/k$ is homogeneous to a stress and is presumably proportional to material resistance. Combining the equations and assuming that during the test $N = 1000$ revolutions are

carried out, since by definition k_s in this case is equal to the relative volume reduction, one obtains

$$l_{test} = 8/3 \cdot R \cdot N = 235 \text{ m (260 m in the 2nd version)}$$

$$-\Delta V/V \approx \ln \frac{V_0}{V} = k_s = k \cdot l_{test} \cdot (\rho_a - \rho_w)g \cdot \sin\varphi_d \tag{11}$$

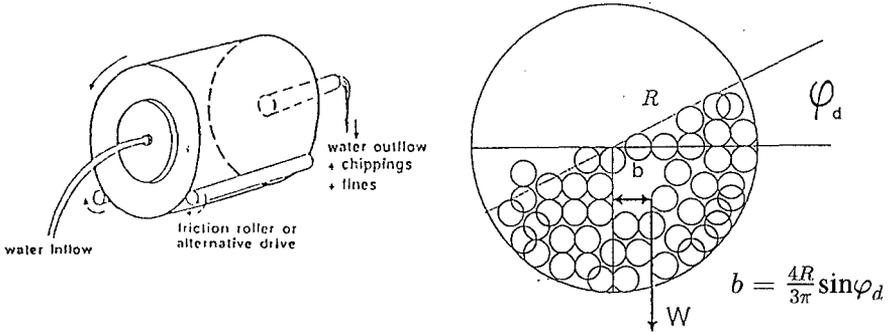


Fig. 8 Geometric scheme of Latham & Poole mill abrasion test

In prototype, a stone sliding for a length l over the profile having slope α is accompanied by the dissipation work

$$\mathcal{L} = l \cdot (\rho_a - \rho_w)g \cdot V \cdot \cos\alpha \cdot \tan\varphi_d \approx l \cdot (\rho_a - \rho_w)g \cdot V \cdot \sin\varphi_d$$

since $\cos\alpha/\cos\varphi_d \approx 1$. According to Reye hypothesis and to relation (11) the relative loss of volume is

$$-\Delta V/V = k_s \cdot l/l_{test} \tag{12}$$

In the lifetime of the breakwater $-L-$ the total displacement length of a stone on the active profile $-l_L-$, reminding that the damage level $-S_s-$ was evaluated over 1000 wave periods, is

$$l_L = \int_0^L l_{avg}(t) \cdot S_s(t) \cdot \frac{dt}{1000T}$$

or, since no movement occurs if wave intensity is below threshold,

$$l_L = \sum_{\text{storms exceeding threshold}} l_{avg} \cdot S_s \cdot \frac{N_w}{1000} .$$

The number of storm exceeding threshold is in the case of practical interest great enough to assume the number of storms N_{st} deterministic and to approximate the sum by its expected value, and therefore

$$l_L = N_{st} \cdot E\left(l_{avg} \cdot S_s \cdot \frac{N_w}{1000} \mid \text{threshold is exceeded}\right)$$

Let λ be the mean frequency of storms and P_* the probability that a generic storm intensity h exceed threshold h_* , the expected number of storm is

$$N_{st} = \lambda_* \cdot L := \lambda \cdot L \cdot P_*$$

Let $F_{H_s}(h)$ be the climatic distribution of storm wave height and let assume that the distribution has exponential behavior provided the threshold is exceeded:

$$F_{H_s}(h) \simeq 1 - \exp\left(-\frac{h-h_0}{h_1}\right) \quad \text{and} \quad P_* = \exp\left(-\frac{h_*-h_0}{h_1}\right)$$

The conditioned distribution, provided that the threshold is exceeded, has similar distribution with the parameter $h_0 = h_*$. Assuming finally for the sake of simplicity that both the number of waves per storm and the wave steepness are constant, that water is deep so that $H_k/C_k = H_s$, letting ξ be the running value of the modified mobility parameter and

$$\xi_1 := \frac{h_1(s_m/s_{mk})^{-1/5}}{\Delta D_{n50}} \quad (13)$$

the total displacement length in lifetime is

$$l_L = N_{st} \cdot N_w \cdot D_{n50} \cdot f_1(\xi_1) \quad (14)$$

where

$$f_1(\xi_1) := \int_{2.0}^{\infty} (1.4\xi - 1.3) \cdot 0.14\xi(\xi - 2.0)^{2.2} \cdot \exp\left(-\frac{\xi-2.0}{\xi_1}\right) \cdot \frac{d\xi}{1000\xi_1}$$

$$- \Delta V_L/V \simeq \ln \frac{V_0}{V} = k_s \cdot L \cdot \left[\frac{\lambda_* \cdot D_{n50}}{l_{test}} \cdot N_w \cdot f_1(\xi_1) \right] \quad (15)$$

The function f_1 may be approximated without any practical loss of accuracy as:

$$f_1(\xi_1) \simeq 0.0020 \cdot \xi_1^{3.0} \quad (16)$$

The factor in brackets in eq. (15) represents the conversion factor of lifetime into equivalent number of thousands of revolutions and should be equal to the reciprocal of Latham (1991) conversion factor X . Actually only some of the factors influencing stone degradation appear in our model; these are: 1-size, 4-incident wave energy, 9-mobility of armor in design concept.

Factor X_1 is according to Latham (1991) directly proportional to the nominal diameter, whereas in our formula stone size acts as inversely proportional.

Factor X_4 is also a scale factor as it is inversely proportional to wave height; it also represents the integrity of blocks.

Factor X_9 depends on the ratio of the two mentioned scale parameters.

Actually the are at least two different scale effects: one for the external abrasion agents, such as chemical attack or waterborne attrition agents, and one for abrasion due to armor stone movement. The first type of abrasion acts on the surface of the stone and erodes a layer of thickness independent from stone size, and hence the scale effect on the relative lost volume is inversely proportional to its size; the factor representing the associated scale effect is X_1 . In the second type, stresses on the contact among stones increase proportionally to stone or wave size, whereas the material resistance remains constant; this produces an effect on relative volume loss proportional to size and X_4 is the factor representing this scale effect.

In any case since the two types of abrasion have different scaling factors and the effects are naturally added on stones, the effects should preferably be evaluated separately in the prototype and in the model. The model we propose can be used to evaluate one of the types of abrasion, provided the k_s values derived from small scale tests may be significant. For the other type existing statically stable breakwaters can provide useful information.

Example applications (Lamberti & Tomasicchio, 1994) show that Latham's method, including abrasion due to external agents, returns greater degradation of stones (one order of magnitude greater $\Delta V_L/V$) but is less sensitive to mobility (half order of magnitude). Latham method is calibrated with the aid of several real world cases.

Some scale effects relative to Reye hypothesis are evident from the comparison between the 1st and the 2nd Latham mill: a factor 1.1 in the mill size or sliding length and 1.4 in peripheral velocity produces a factor 1.5 in abrasion intensity for the same material. This indication could suggest to scale up the degradation proportionally to the impact velocity scale, i.e. for a stone rolling on a mound, proportionally to the square root of the length scale. Since the length scale from the mill to reality is almost 100, the scaling would compensate approximately for the pointed out difference. More definite information is anyway required about the scale effects due to impact velocity and stress.

7. Conclusions

A careful analysis of stone mobility shows that:

- the wave height and wave period combination which describes properly stone mobility is the combination proportional to onshore energy flux ($\propto H^2T$); according to the same concept also the effect of wave obliquity can be easily included in the wave intensity parameter; as a consequence a stability number modified respect to the traditional N_s is introduced.
- a wave height greater than the significant one should be used as wave height parameter in order to have comparable stability results in deep and shallow water conditions;
- the true threshold of stone movement corresponds to mobility conditions significantly lower than defined by previous researchers: in the range of mobility conditions typical of berm breakwater operation ($N_s = 2 - 3$) the mobility is low but appreciable;
- relations are provided giving the non dimensional frequency of stone movements and their average length as function of the modified stability number.

The proposed longshore transport model, derived without any special calibration from frequency and length of displacements, gives results in good agreement with measurements performed on berm breakwater models.

The proposed conceptual method for the evaluation of armor stones abrasion on berm breakwater due to stone movement gives only reasonable results and requires some adjustment in order to account for the scale effects relative to impact velocities and stresses.

Acknowledgments

The present study was partially supported by the research and technological development programme in the field of Marine Science and Technology (MAST) financed by the Commission of the European Communities, contract MAST-0032.

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Notations

The following symbols and notations are used in the paper:

- c = wave celerity, $[m \cdot s^{-1}]$;
- c_g = group velocity, $[m \cdot s^{-1}]$;
- C_k = ratio between the characteristic and significant wave height;
- D_{nxx} = nominal diameter of armor stones corresponding to fractile $xx\%$:
 $(W_{xx}/g\rho_a)^{1/3}$, $[m]$;
- $E()$ = expected value;
- H = wave height, $[m]$;
- H_s = significant wave height, $[m]$;
- $H_{1/n}$ = mean wave height of the $1/n$ fraction of the highest waves;
- L = wave length, $[m]$;
- L_d, l_d = length of displacement, stochastic variable and mean value, $[m]$;
- N_s = armor stone mobility (stability) number: $H_s/\Delta D_{n50}$, $[\]$;
- N_s^{**} = modified stability number, $[\]$;
- S = along-structure transport measured as bulk volume, $[m^3 \cdot s^{-1}]$, or as number of stones per wave, $[\]$;
- s = wave steepness: H_s/L_o , $[\]$;
- T = wave period, $[s]$;
- W_{xx} = weight of which $xx\%$ by weight of the armor stones are smaller, $[kg]$;
- β = angle of wave attack (between wave front and shoreline), $[\]$;
- γ = breaking index: ratio between breaking wave height and water depth, $[\]$;
- Δ = relative density of armor stones = $(\rho_a - \rho_w)/\rho_w$, $[\]$;
- ρ_a = mass density of armor stones, $[kg \cdot m^{-3}]$;
- ρ_w = mass density of water, $[kg \cdot m^{-3}]$;
- \cdot_b = at wave breaking point;
- \cdot_k = characteristic or effective value;
- \cdot_m = relative to spectrum mean frequency;
- \cdot_o = in offshore conditions;
- \cdot_p = relative to spectrum peak frequency.