8. LOW-CRESTED AND SUBMERGED BREAKWATERS IN PRESENCE OF BROKEN WAVES

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Abstract

Functional design of low crested breakwaters requires an accurate prediction of wave transmission and set up in the protected areas. Nevertheless, commonly used formulae do not appear to be reliable enough, especially for structures located in shallow waters. The paper describes results from large-scale model tests conducted on rubble mound breakwaters exposed to breaking waves. Tests were carried out at the "Grosser WellenKanal" of Hannover, Germany. The model dimension, near to prototype, allowed minimizing scale effects connected to wave breaking. Existing formulas on wave transmission have been verified and influence of crest width and breaker index have been highlighted. Moreover the wave set-up behind the structure has been analysed showing the importance of momentum release of breaking waves for submerged breakwaters and of the mass balance for low crested ones.

8.1 Introduction

Detached low-crested and submerged breakwaters are frequently employed, often in conjunction with beach nourishment, in defending eroded coastlines. This because of their small environmental effects combined with obvious aesthetic advantages.

Although both physical and numerical investigations have been performed, no reliable procedures are yet available for their functional design.

Regarding wave transmission, for instance, present formulae (van der Meer 1990a; van der Meer 1990b; van der Meer 1991; Daemen 1991; d'Angremond et al. 1996), derive from data collected from different laboratories where is not certain that the same analysis procedures have been dealt. Moreover, role played by structure permeability, crest width and water depth has not been fully clarified.

Together with wave damping capability, performances of detached breakwaters are significantly affected by phenomenon of wave set-up (Diskin et al. 1970; Dalrymple and Dean 1971; Loveless et al. 1998). Presence of the barrier, in fact, generates a hydraulic head surplus behind the structure driving large long-shore currents in the protected area. This may increase the shoreline erosion instead of reducing it (Murphy ,1996; Browder et al. ,1997; van der Biezen, 1998). For these reasons, a competent economical and functional design method needs the knowledge of relationships linking crest freeboard, wave transmission and set-up behind the structure.

This paper will present results from large scale model tests conducted at the "Grosser WellenKanal" of Hannover (Germany) on low crested/submerged breakwaters mostly located in shallow waters. Within the framework of the EU project "Transnational Access To Major Research Infrastructure" a team of Italian (Calabrese and Buccino of University of Naples "Federico II", Vicinanza of Second University of Naples, Lamberti and Tirindelli of University of Bologna), Danish (Burcharth and Kramer, of University of Aalborg) and Dutch (van der Meer of INFRAM) researchers, coordinated by Calabrese (University of Naples "Federico II"), have worked together on a research study titled "Low-crested and submerged breakwaters in presence of broken waves".

Need of large-scale facility for such kind of study is essential because of the extremely limited water depths (and wave heights, few centimeters!), inevitably used in small scale, which may substantially affect the analysis. Furthermore, it is difficult to correctly reproduce the permeability of the barriers in small scale experiments. Results here discussed derive from preliminary analysis conducted using only a part of the whole data set and reported in Calabrese et al (2002) and Calabrese et al. (2003).

8.2 Experimental set up

The model tests were carried out at the "Grosser WellenKanal" of Hannover, Germany. The wave flume has a length of 300 m, a width of 5 m and a depth of 7 m. The facility is equipped with a piston type paddle for generating regular and random waves. The installed power of the piston type wave generator combined with an upper flap is about 900 kW. The gearwheel driven carrier gives a maximum stroke of ± 2.10 m to the wave paddle. The wave generation is controlled by an online absorption system. This special system works with all kinds of regular and irregular wave trains. Thus, the tests are unaffected by re-reflections at the wave generator and can be carried out over nearly unlimited duration.

The bathymetry in the flume was formed by moulding sand over fill in the channel to the required shape. From deep water near the paddle, the seabed was flat for 105.3 m than it sloped initially at 1:20 for 20 m to change for a more gentle slope of 1:50, and terminated in a 15 m horizontal section. The bed level at the test structure was + 3 m relative to the flume floor at the wave paddle. To minimize effects of any reflection from the end of the flume, an absorbing sand beach with 1:18 slope was built (Fig. 1).

A 1.3 m high rubble mound breakwater was installed on a flat area at the end of 1:50 sand beach.

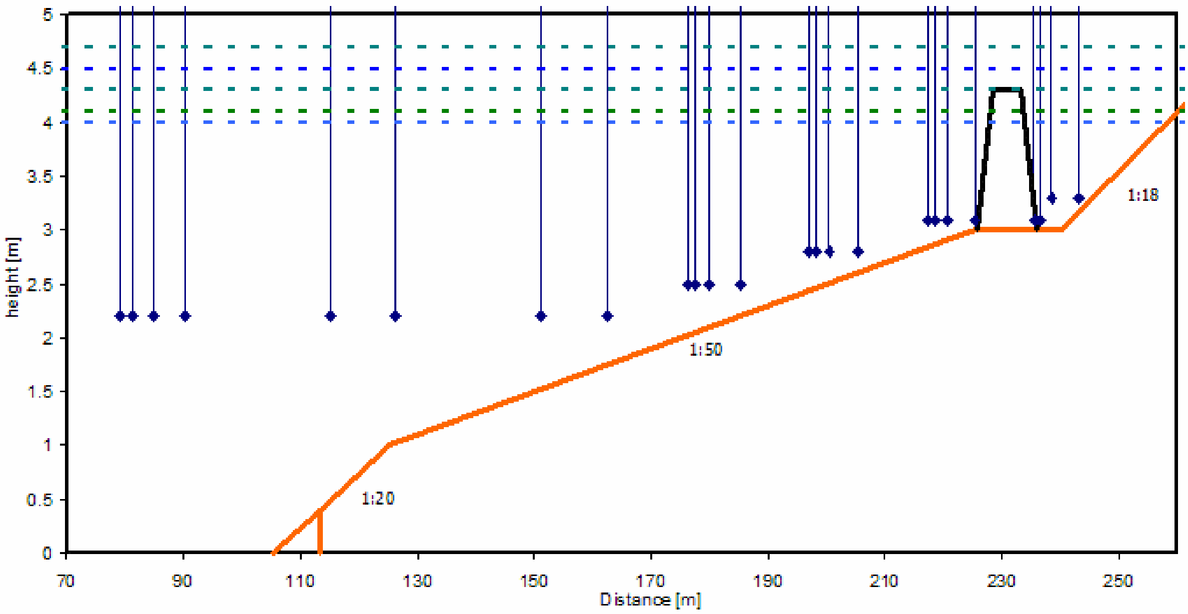

Fig. 8-1 Seabed profile, wave probes position and five different tested s.w.l.

Three different cross sections were tested (Fig. 2):

a) 1 m crown width, with an impermeable sheet in the middle of the structure;
b) 1 m crown width, without impermeable sheet in the middle of the structure;
c) 4 m crown width, without impermeable sheet in the middle of the structure.

The impermeable sheet was located during the installation of cross section a) for a better understanding of permeability effects on hydraulic performance of low crested and no freeboard breakwater (Fig. 3). Afterward the cross section b) was obtained removing the sheet by lifting it up. In this way there was no modification of the cross section shape (Fig. 4).

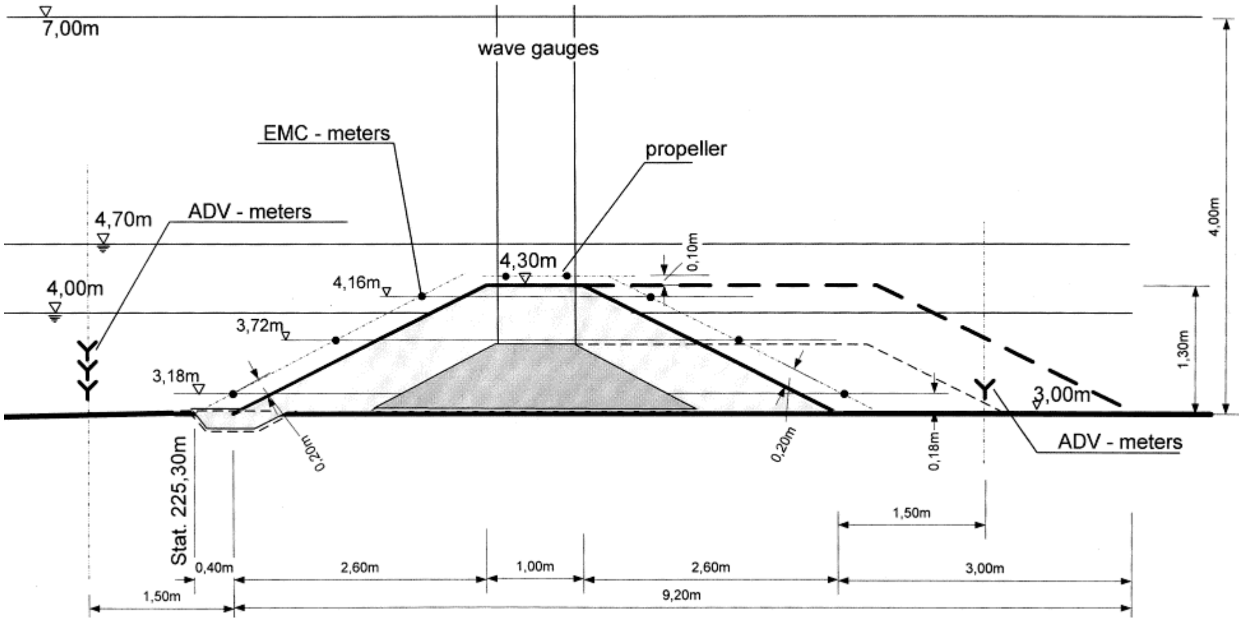

Fig. 8-2. Structure configurations and instruments

Fig. 8-3. Structure installation with an impermeable sheet in the middle

Fig. 8-4. Removal of impermeable sheet from middle of structure

Two different type of rock were used as material for the rubble mound model, Normal Density Rock (armour layer) and High Density Rock (core). Both types were supplied by NCC Industries Norway. The High Density Rock came from NCC quarry at Valberg, Kragerø Norway, the Normal Density Rock came from NCC quarry at Skien, Norway. For both rock types two samples were weighed and measured. Statistics gave the following results:

- W_{50}=30.4kg, D_{50}=0.225m, g_s=2.65t/m3 (armour layer);

- W_{50}=19.0kg, D_{50}=0.184m, g_s=3.05 t/m3 (core).

Front and rear slopes were kept constant, equal to 1:2. Five s.w.l. have been changed in order to obtain different configurations ranging from low crested to submerged.

To measure the wave characteristics, a set of 27 probes was sampled at 50 Hz (Fig. 1). Wave velocities were measured using the following instruments supported by Coastal Research Centre:

- a set of 4 ADV (Acoustic Doppler Velocity-meter), 3 placed 1.5 m in the front and 1 located 1.5 m at the rear side of the structure, only for the cross section c) there was an extra ADV supported by University of Bologna (Fig. 5);

- a set of 6 EMC (Electro Magnetic Currentmeter), 3 placed on the front slope and 3 on the back slope of the structure (Fig. 6);

- 2 propellers were placed on the top of the model (Fig. 7).

All the velocimeters were sampled at 50 Hz with the exclusion of University of Bologna's ADV that it sampled at 100Hz.

a) b)

Fig. 8-5. Acoustic Doppler Velocimeters: a) 3 in front of the structure, b) 1 at the rear of the structure

a) b)

Fig. 8-6. ElectroMagnetic Current meters: a) instrument detail, b) 3 placed on the front slope

a) b)

Fig. 8-7. Propellers: a) instrument detail, b) 3 placed on the top of the berm

To analyse the results use is made of both time-domain and frequency-domain analysis. For each probe, significant spectral wave height, H_{mo}, have been calculated as:

$$H_{mo} = 4 \cdot \left[\int_{0.5 f_p}^{\infty} E(f) df \right]^{0.5}$$

where $E(f)$ represents the spectral density and f_p the peak frequency.

In calculating transmission coefficient, $K_t = H_{mot}/H_{moi}$, transmitted wave height, H_{mot}, is obtained averaging values detected at three probes placed on the flat bottom behind the structure.

TMA spectra were run giving wave conditions at the structure variable from shoaling to post-breaking (Tab.1). Each test was preliminary run (at least for 500 waves) without the structure, in order to achieve reliable estimates of incident wave parameters, H_{moi} and T_{pi}.

Test	h (m)	H_{mo} (m)	T_p (s)	R_c (m)	B (m)	Test	h (m)	H_{mo} (m)	T_p (s)	R_c (m)	B (m)
1	1	0.6	3.5	0.3	1	25	1.3	0.6	3.5	0	4
2	1	0.8	4.5	0.3	1	26	1.3	0.8	4.5	0	4
3	1	0.9	4.5	0.3	1	27	1.3	1	4.5	0	4
4	1	1	4.5	0.3	1	28	1.3	1	6.5	0	4
5	1	1	6.5	0.3	1	29	1.5	0.6	3.5	-0.2	1
6*	1	0.6	3.5	0.3	1	30	1.5	0.8	4.5	-0.2	1
7*	1	0.8	4.5	0.3	1	31	1.5	1	4.5	-0.2	1
8	1.1	0.6	3.5	0.2	1	32	1.5	1	6.5	-0.2	1
9	1.1	0.8	4.5	0.2	1	33	1.5	1.1	6.5	-0.2	1
10	1.1	1	4.5	0.2	1	34	1.5	0.6	3.5	-0.2	4
11	1.1	1	6.5	0.2	1	35	1.5	0.8	4.5	-0.2	4
12*	1.1	0.6	3.5	0.2	1	36	1.5	1	4.5	-0.2	4
13*	1.1	0.7	4.5	0.2	1	37	1.5	1	6.5	-0.2	4
14*	1.1	0.8	4.5	0.2	1	38	1.5	1.1	6.5	-0.2	4
15*	1.1	0.9	4.5	0.2	1	39	1.7	0.6	3.5	-0.4	1
16	1.3	0.6	3.5	0	1	40	1.7	0.8	4.5	-0.4	1
17	1.3	0.8	4.5	0	1	41	1.7	1	4.5	-0.4	1
18	1.3	0.9	4.5	0	1	42	1.7	1	6.5	-0.4	1
19	1.3	1	4.5	0	1	43	1.7	1.1	6.5	-0.4	1
20	1.3	1	6.5	0	1	44	1.7	0.6	3.5	-0.4	4
21*	1.3	0.6	3.5	0	1	45	1.7	0.8	4.5	-0.4	4
22*	1.3	0.7	4.5	0	1	46	1.7	1	4.5	-0.4	4
23*	1.3	0.8	4.5	0	1	47	1.7	1	6.5	-0.4	4
24*	1.3	0.9	4.5	0	1	48	1.7	1.1	6.5	-0.4	4

* Tests with an impermeable sheet in the middle of the cross section.

Table 1 Wave and structure characteristics

8.3 Wave transmission

8.3.1 Existing formulae for wave transmission

Hydraulic model tests by Seelig (1980), Powell and Allsop (1985), Daemrich and Kahle (1985), Ahrens (1987), van der Meer (1988) were firstly analyzed by van der Meer (1990a). A simple prediction formula has been derived where transmission coefficient, K_t, linearly decreases with the relative crest freeboard, R_c/H_{si}:

$$K_t = 0.46 - 0.3 \cdot \left(\frac{R_c}{H_{si}} \right) \tag{1}$$

Kt is limited between 0.8, for $R_c/H_{si} < -1.13$, and 0.1 for $R_c/H_{si} > 1.2$. The formula does not take directly into account crest width effects.

Daemen (1991) re-analyzed the same data set, excluding data by Ahrens (1987) on reef type structures, as their hydraulic response resulted so much different from conventional breakwaters. The Author introduced a different dimensionless freeboard including the permeability of the armour layer:

$$K_t = a \cdot \left(\frac{R_c}{D_{50}} \right) + b \tag{2}$$

with slope a depending on relative wave height, H_{si}/D_{50}:

$$a = 0.031 \cdot \left(\frac{H_{si}}{D_{50}} \right) - 0.24 \tag{3}$$

Intercept b, represents the transmission coefficient for no freeboard structures ($R_c = 0$); it depends on relative wave height, H_{si}/D_{50}, crest width, B, and incident peak wave period, T_{pi}:

$$b = -5.42 \cdot s_{op} + 0.0323 \cdot \frac{H_{si}}{D_{50}} - 0.0017 \cdot \left(\frac{B}{D_{50}} \right)^{1.84} + 0.51 \tag{4}$$

where $s_{op} = 2\pi\, H_{si}/g\, T_{pi}^2$ represents wave steepnees.

The wave transmission coefficient is assumed not greater than 0.75 and not less than 0.075; limits of the formula are: $1 < H_{si}/D_{50} < 6$ and $0.01 < s_{op} < 0.05$.

Further analysis have been performed by d'Angremond et al. (1996). Data base have been previously filtered deleting tests with high stepness, $s_{op} \geq 0.6$, or breaking waves, $H_{si}/h \geq 0.54$, and structures highly submerged, $R_c/H_{si} < -2.5$, or emerged, $R_c/H_{si} > 2.5$.

Final formula is:

$$K_t = -0.4 \cdot \left(\frac{R_c}{H_{si}} \right) + \left(\frac{B}{H_{si}} \right)^{-0.31} \cdot \left(1 - e^{-0.5x} \right) \cdot c \tag{5}$$

where $x = \tan\alpha/(H/L)^{0.5}$ represents the Iribarren parameter and c is a coefficient equal to 0.8 for impermeable structures and 0.64 for permeable ones. K_t is involved between 0.075 and 0.8.

Recently Seabrook and Hall (1998) proposed, for submerged breakwaters only, the following equation:

$$K_t = 1 - \left(e^{-0.65\left(\frac{R_c}{H_{si}}\right) - 1.09\left(\frac{H_{si}}{B}\right)} + 0.047 \cdot \left(\frac{B \cdot R_c}{L \cdot D_{50}} \right) - 0.067 \cdot \left(\frac{Rc \cdot H_{si}}{B \cdot D_{50}} \right) \right) \tag{6}$$

Authors recommended that caution have to be used in applying the formula outside the ranges: $0 \le B\,R_c/L\,D_{50} \le 7.08$ and $0 \le H_{si}\,R_c/B\,D_{50} \le 2.14$.

8.3.2 Comparison with GWK tests

For a quantitative comparison between GWK data and existing formulae, bias, b, and root mean squared error, *RMSE*, have been employed as reliability indicators:

$$b = \frac{\sum_N \left(\frac{K_{tp}}{K_{tm}}\right)_i}{N} \tag{7}$$

$$RMSE = \left[\frac{\sum_N \left(\frac{K_{tp} - K_{tm}}{K_{tm}}\right)_i^2}{N}\right]^{0.5} \tag{8}$$

where N represents the number of data and K_{tp} and K_{tm} are respectively predicted and measured values of wave transmission coefficient. Comparative analysis results are shown in Figures 8 and 9.

Graphs show that for structure with the smallest crest width ($B/D_{50} = 4$), Eqs. 1, 2, 5 and 6 are basically undistorted, giving van der Meer formula the minimum *RMSE* (8.5%). It resulted 50% less than Seabrook and Hall (16.4%). Moreover Eq.1 seemed better working for emerged breakwaters. On the other hand for $B/D_{50} = 16$, van der Meer formula (1990) shows significant overestimates due to neglecting of crest width effects. Daemen (1991) and Seabrook and Hall (1998) give under predictions (Fig. 9), which resulted comparable with that by van der Meer. D'Angremond et al. (1996) is almost undistorted and gives a *RMSE* significantly lower.

In Table 2 overall values of b and *RMSE* are reported, for the four discussed formulae.

Analyzed formulas	β	RMSE
Van der Meer (1990)	1.15	0.28
Daemen (1991)	0.86	0.20
d'Angremond et al. (1996)	0.98	0.11
Seabrook and Hall (1998)	0.93	0.24

Table 2 Distortion, b, and Root Mean Squared Error, RMSE for the analyzed formulas

d'Angremond et al. (1996) resulted more reliable than others and appeared to hold also under breaking waves. This is shown in Figure 10, where is plotted the comparison between Eq.5 and the part of GWK data collected in extremely shallow waters ($0.54 \le H_{mo}/h \le 0.61$). It could be useful reminding that the formula was derived for $Hmo/h \le 0.54$.

Nevertheless, it returns a *RMSE* still relatively high, especially for smaller structures (14%). As suggested by Gironella et al (2002), an uncertainty of 10% in determining K_t translates in a 30% uncertainty in predicting sediment transport. Thus, new efforts are needed trying to reduce that scatter.

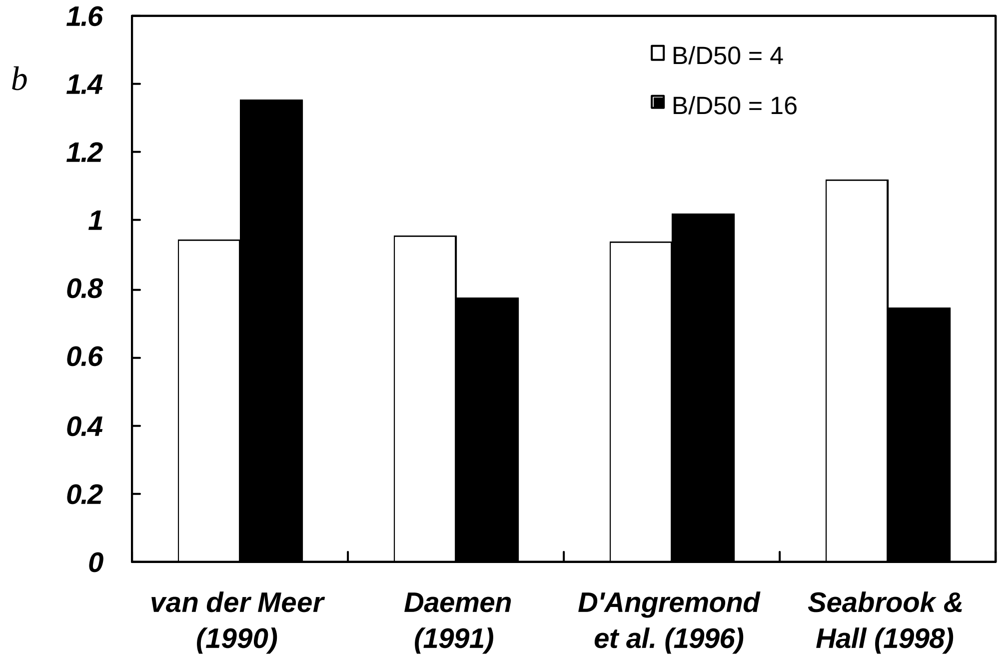

Fig. 8-8. Bias, b, for the analyzed formulae

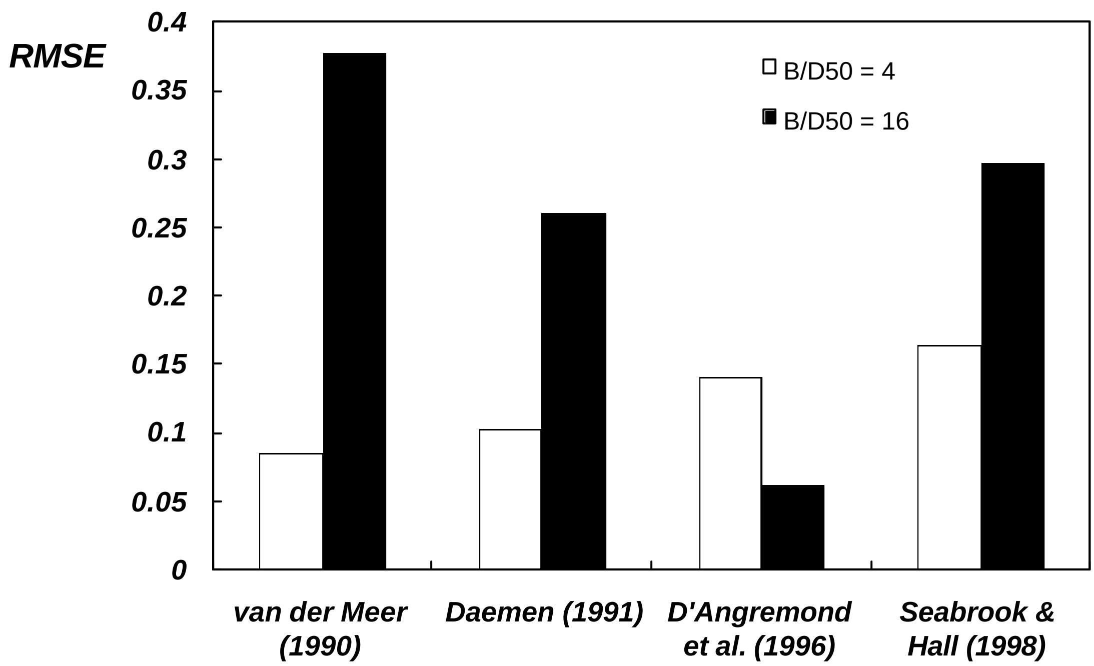

Fig. 8-9. Root mean squared error, RMSE, for the analyzed formulae

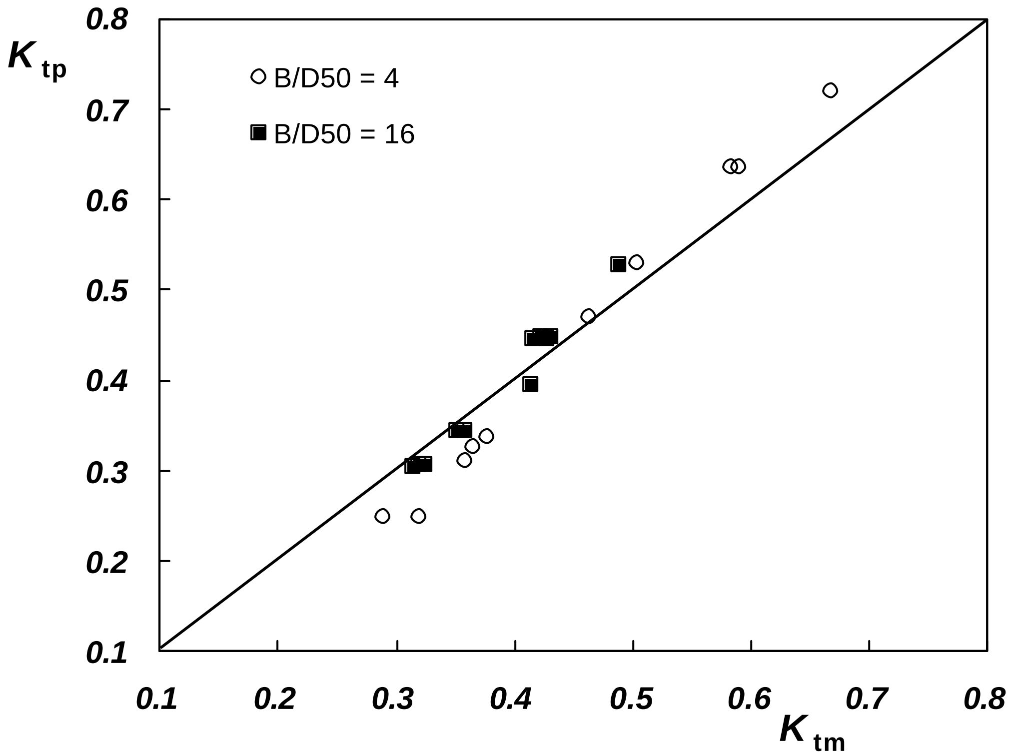

Fig. 8-10. Comparison between experimental data and Eq. 5 for 0.54 < H_{mo}/h £ 0.61

From analysis of data three aspects have to be highlighted:

1. reduction rate of K_t should be not a constant like in Eq. 5. It seemed rather to be a function of berm width, *B*. A quite similar behaviour have been observed in Gironella et al. (2002);
2. a significant role seems to be played by breaker index *H*/*h* especially when the barrier is located in shallow waters;
3. it resulted more convenient to make crest freeboard, R_c, non dimensional by the berm width, *B*, rather than incident wave height.

8.3.3 A suggestion transmission formula for shallow waters

With the aim to include points 1-3 in a predictive expression, large-scale experimental data have been re-analysed.

Following functional relationship was initially set:

$$K_t = a \cdot \frac{R_c}{B} + b \qquad (9)$$

For intercept b, an exponential formula seemed to be more adequate than power function proposed by d'Angremond et al. (1996):

$$b = a \cdot \exp\left(-0.0845 \cdot \frac{B}{H_{moi}} \right) \qquad (10)$$

where:

$$a = 1 - 0.562 \cdot \exp\left(-0.0507 \cdot x \right) \qquad (11)$$

The angular coefficient, a, was expressed as function of the relative crest width, B/H_{moi}:

$$a = b \cdot \exp\left(0.2568 \cdot \frac{B}{H_{moi}}\right) \tag{12}$$

The influence of the water depth at the structure has been included in scale parameter β:

$$b = 0.6957 \cdot \frac{H_{moi}}{h} - 0.7021 \tag{13}$$

Calibration ranges of Eqs.9-13 are: $-0.4 \leq R_c/B \leq 0.3$; $1.06 \leq B/H_{moi} \leq 8.13$; $0.31 \leq H_{moi}/h \leq 0.61$; $3 \leq x \leq 5.20$. Moreover, it could be of course postulated that coefficients of Eq.13 should be function of the foreshore slope.

Comparison with experimental data, Figure 11, revealed a reduction around 66% of *RMSE* (0.04 vs 0.11), which seemed of course promising.

Further validation of Eq. 9 has been conducted using data by Powell and Allsop (1985) and Seelig (1980), which both refer to conventional breakwaters placed in shallow waters.

Details about tests are available in Daemen (1991).

Data set has been preliminary filtered choosing only those conditions where both Eqs 5 and 9 were applicable.

Comparison is shown in Figure 12. The agreement is quite good and value of *RMSE* resulted around 50% less than d'Angremond et al. (0.09 vs. 0.17, Figure 13).

This appeared satisfactory enough, despite the scatter is significantly larger than in GWK tests. This may be due to some influence of cross section characteristics, which is not accounted in Eq. 9. Powell and Allsop, in fact, used a homogeneous breakwater, while tests by Seelig have been performed using a cross section more similar to that used in GWK. This should explain difference of RMSE between two series (Fig. 14), showing data by Seelig an error quite similar to what encountered at GWK (0.049, Fig. 14).

However these results have to be taken carefully given the limitation of the data series.

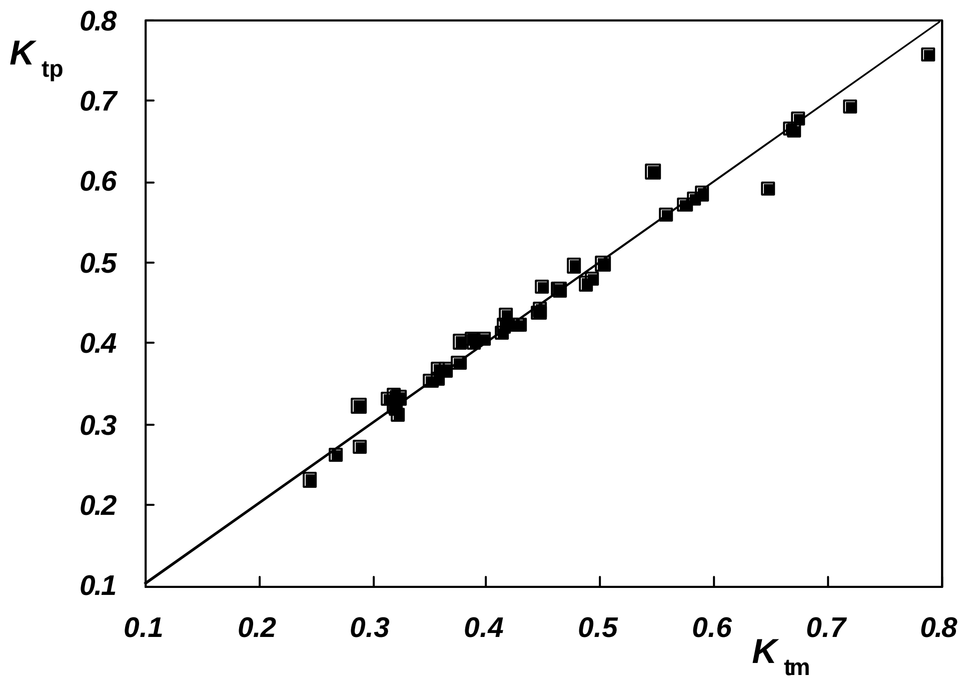

Fig. 8-11. Comparison between GWK's experimental data and Eq. 9

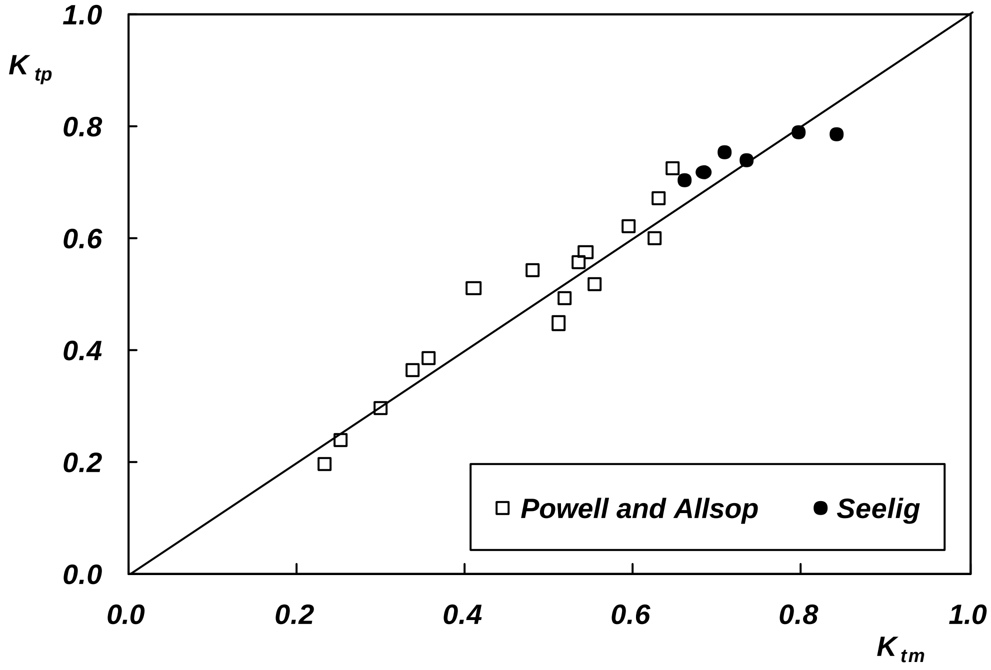

Fig. 8-12. Comparison between Powell & Allsop and Seelig experimental data and Eq. 9

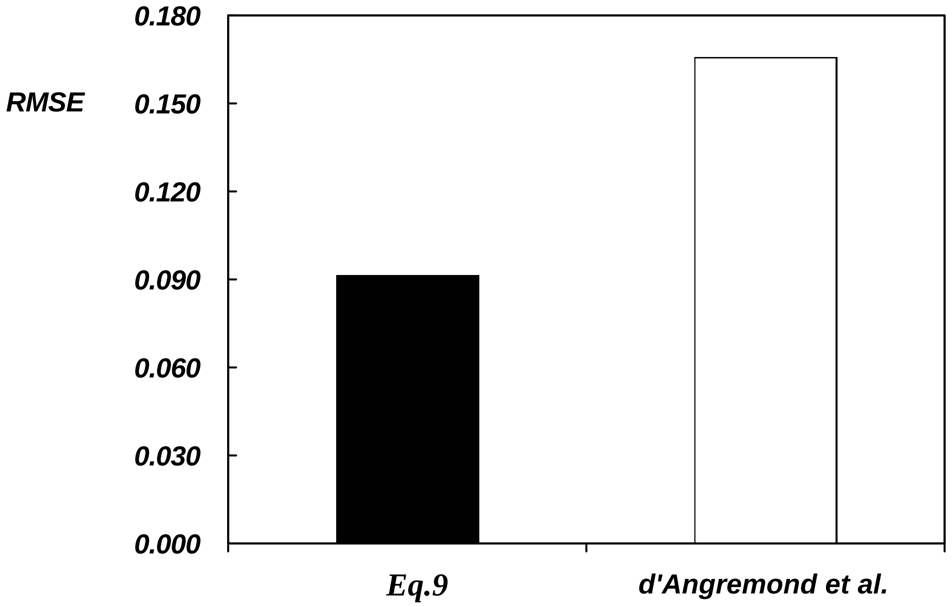

Fig. 8-13. The root mean squared error, RMSE, for the Eq.9 and d'Angremond et al.

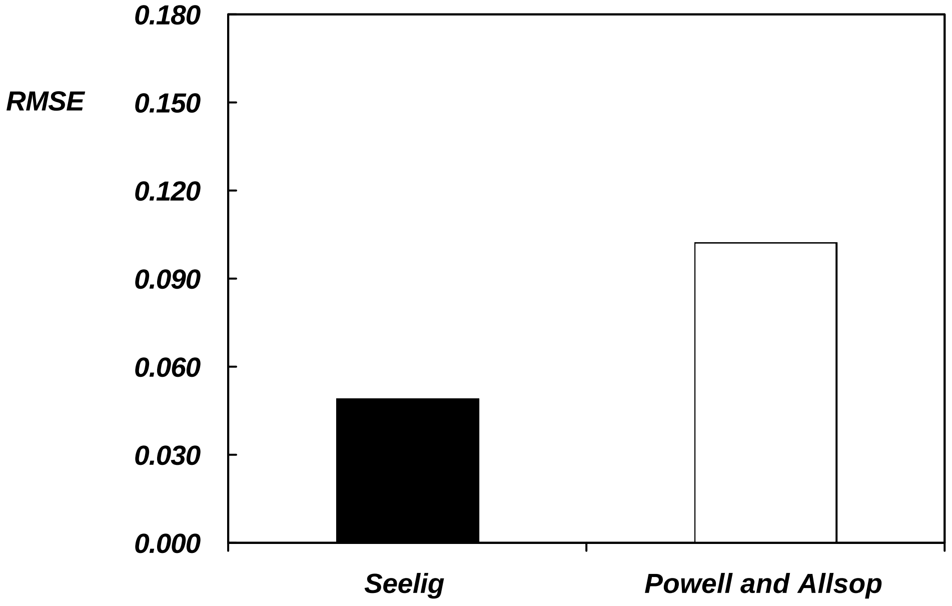

Fig. 8-14. The root mean squared error, RMSE, for Seelig and Powell & Allsop

Detailed analysis is in progress with the aim to validate eq.9 against other data set. First results are encouraging as it is shown in Fig. 15 – 16.

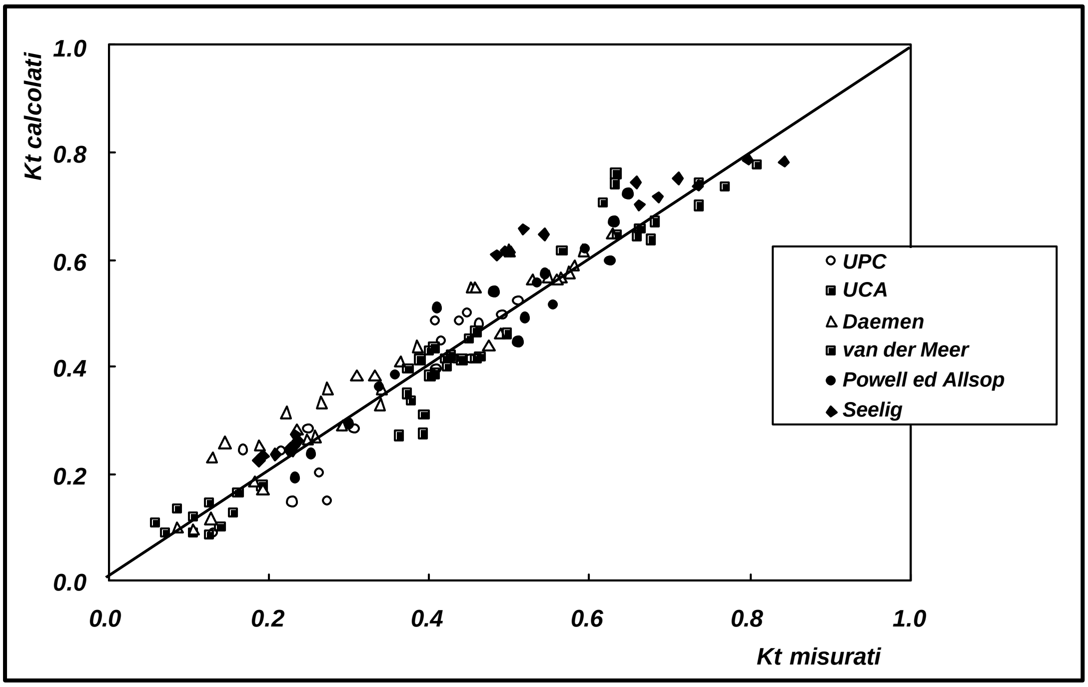

Fig. 8-15. Comparison between UPC, UCA, Daemen, van der Meer, Powell and Allsop, Seelig experimental data and Eq. 9

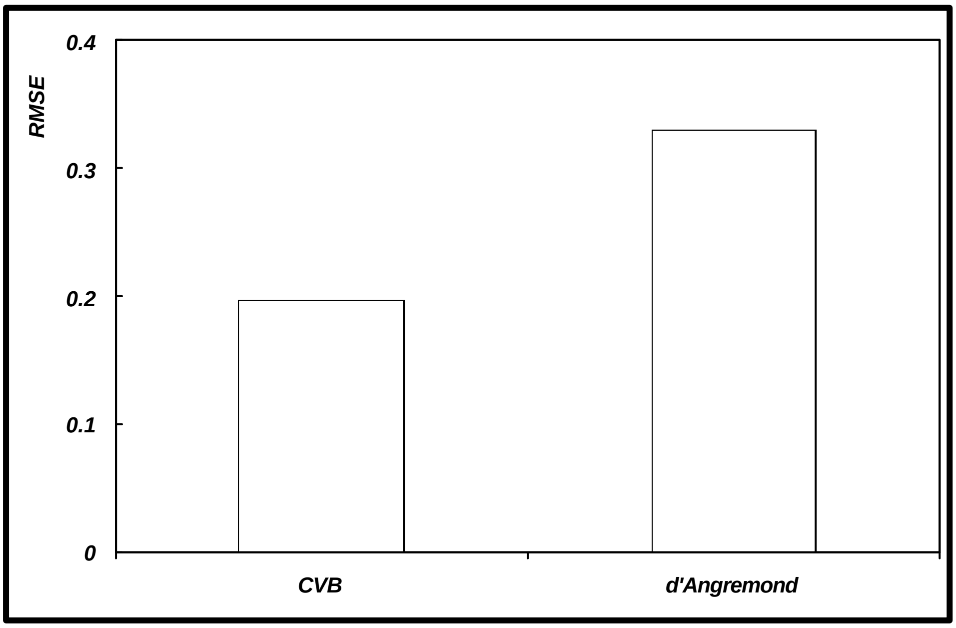

Fig. 8-16. The root mean squared error, RMSE, for the Eq.9 (CVB) and d'Angremond et al. using a larger data set

8.4 **Wave set up**

8.4.1 Existing formulae for wave set up

For a submerged breakwater, where no breaking is considered, an analytical solution for piling up, d, has been given by Longuet-Higgins (1967). His approach applies vertical momentum flux balance above the still water level and yields the following expression:

$$d = \frac{H_T^2 \cdot k_1}{8 \cdot \sinh\left(2 \cdot k_1 \cdot d_1\right)} - \frac{H_t^2 \cdot k_2}{8 \cdot \sinh\left(2 \cdot k_2 \cdot d_2\right)} \tag{14}$$

in which H_t is the transmitted wave height; H_T^2 is the sum squared of incident and reflected wave heights; d is the water depth; k is the wave number; the numerical subscripts denote parameters values before and behind the breakwater. Eq. 14 basically represents the difference between the "set down" (Longuet-Higgins and Stewart, 1964) expected respectively at rear and in front of the barrier.

Dick (1968) measured the set-up for an impermeable rectangular breakwater and found out that Eq. 14 greatly underestimated experimental values. Subsequently, Diskin (1970) performed a study on a two dimensional physical model of a trapezoidal breakwater with an homogeneous cross section. Using regular waves, the Author developed an empirical relationship between set-up, incident wave height, H_i and depth of submergence, R_c:

$$\frac{d}{H_i} = 0.60 \cdot \exp\left[-\left(0.70 - \frac{R_c}{H_i}\right)^2\right] \tag{15}$$

The equation was developed from data in the range:

$$-2 < -\frac{R_c}{H_i} < 1.5 \tag{16}$$

Eq. 16 indicates that set up increases with H_i and that, for a given wave height, it attains a maximum when the water level is just below the crest, $R_c = 0.7\ H_i$. For water levels above and below this value, set-up lowers and tends to zero. In the Diskin's formula neither crest extent, B, or period, T, influences have been considered. Since only one size of rock was tested, Diskin was unable including permeability of the barrier in the formula.

A more extensive program of experiments have been carried out by Loveless et al. (1998) in the random wave flume at the Hydraulics Laboratory of University of Bristol's Civil Engineering Department. On the whole, eight different models have been tested including variations in crest width, front slope angle and rock size. All the models were homogeneous rubble mound structures, made up on rock of narrow grading. Most of experiments were conducted with regular waves; some irregular wave trains have been also run in order to study how information obtained under monochromatic waves should be transferred to spectral ones. First the Authors noted that Diskin's formula predicted with some accuracy the values of set-up for submerged breakwaters ($R_c < 0$), while it largely overestimated experimental data for emerged or "no freeboard" ones ($R_c \geq 0$). This was attributed to permeability (Diskin's model had median diameter, D_{50}, of 40% less than the smallest of the Loveless models), which is expected to have no influence for submerged breakwaters, but to greatly affect the behavior of emerged ones. Moreover the Authors found out that to adapt monochromatic waves results to irregular waves, the average wave height should be used instead of significant one. Finally following expression has been proposed:

$$\frac{d}{B} = \frac{\left(\frac{H_i \cdot L}{d \cdot T}\right)^2}{8 \cdot g \cdot D_{n50}} \cdot \exp\left[-20 \cdot \left(\frac{R_c}{h_c}\right)^2\right] \tag{17}$$

where h_c is the height of the structure and L is the wave length.

Basically left side of Eq. 17 represents the mean hydraulic gradient needed to drive back the net inshore rate pumped by waves, $H_i\ L/T$, by a dominantly turbulent flow through the structure.

8.4.2 Results for submerged breakwaters

Wave set-up behind submerged structures is essentially dominated by the amount of momentum released by the waves which break on them. A first estimate of it, d_m, could be obtained from the horizontal momentum balance of the volume shown in Figure 17:

$$S_{xx} - S'_{xx} + \Pi_2 - \Pi_1 + P_1 - P_2 = 0 \tag{18}$$

where S_{xx} is the "xx" component of tensor radiation stress; P represents the force exerted by the structure on the volume of fluid; P is the net hydrostatic thrust.

As a first crude approximation, we assumed the obstacle to be impermeable and non reflecting and that forces acting onto the breakwaters are hydrostatic in average. Moreover, as it is shown in Fig. 17, we supposed that surf zone extends from breaking point to inshore toe of the barrier and that wave set up linearly increases across it.

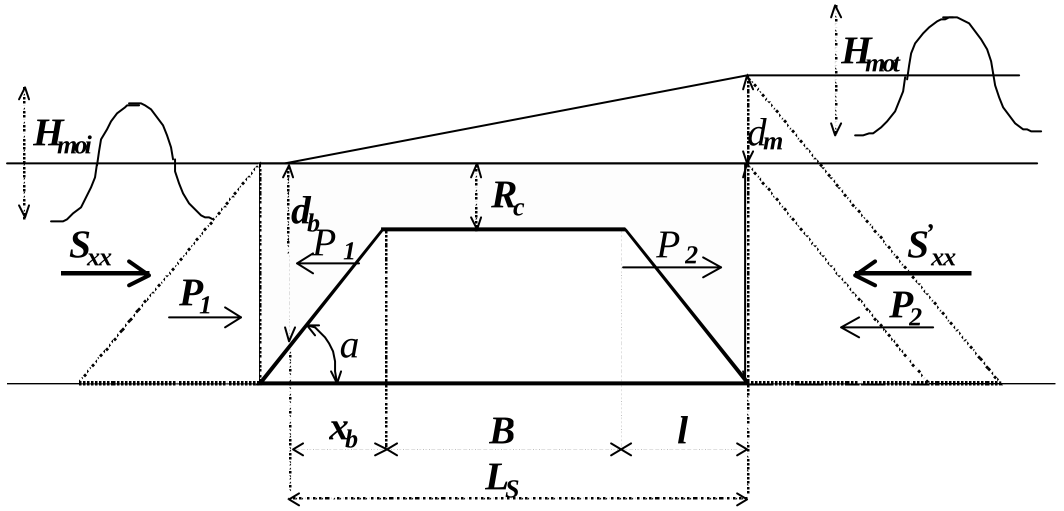

Fig. 8-17. Definitions for energy flux theory for wave set-up on a submerged breakwater

Breaking point is roughly achieved using the linear shallow waters shoaling and the breaking criterion by Kamphuis (1991). This leads to following expression for breaking depth:

$$d_b = \left[H_{moi} \big/ g_b \right]^{4/5} \cdot d^{0.2} \tag{19}$$

$$g_b = 0.56 \cdot \exp(3.5 \cdot \tan a) \tag{20}$$

Under these hypothesis, and assumed for S_{xx} the expression:

$$S_{xx} = \frac{3 \cdot r \cdot g \cdot H_{eni}^2}{16} \tag{21}$$

$$S'_{xx} = \frac{3 \cdot r \cdot g \cdot H_{ent}^2}{16} \tag{22}$$

where H_{en} is the energetic wave height $H_{mo} \big/ \sqrt{2}$ and the subscripts *i* and *t* stand for incident and transmitted respectively. Eq 18 results in a simple second order equation, which can be easily solved for the "momentum" set up, d_m:

$$d_m = 0.5 \cdot \left[-b + \left(b^2 + 4 \cdot c \right) \right]^{0.5} \tag{23}$$

where:

$$b = (2 \cdot d - A) \tag{24}$$

$$A = \left\{ \left[1 + \frac{x_b + B}{L_s} \right] \cdot h_c - \left[x_b \cdot \frac{(d_b + R_c)}{L_s} \right] \right\} \tag{25}$$

$$c = \frac{3}{8} \cdot H_{eni}^2 \cdot \left(1 - K_t^2 \right) \tag{26}$$

Kt is the transmission coefficient $\dfrac{H_{mot}}{H_{moi}}$.

The comparison with data (Fig. 18) suggests that solution given by Eq. 23, despite it relies on quite raw hypothesis, furnishes reasonable estimates of order of magnitude of wave set up for submerged barriers. Nevertheless it tends to give values less than those measured when the reef crest becomes large and, mostly, when submergence reduces. This probably because, under such conditions, the shear stress due to the outflow current over the breakwater, increases; this mainly occurs for small submergences where large velocities are expected.

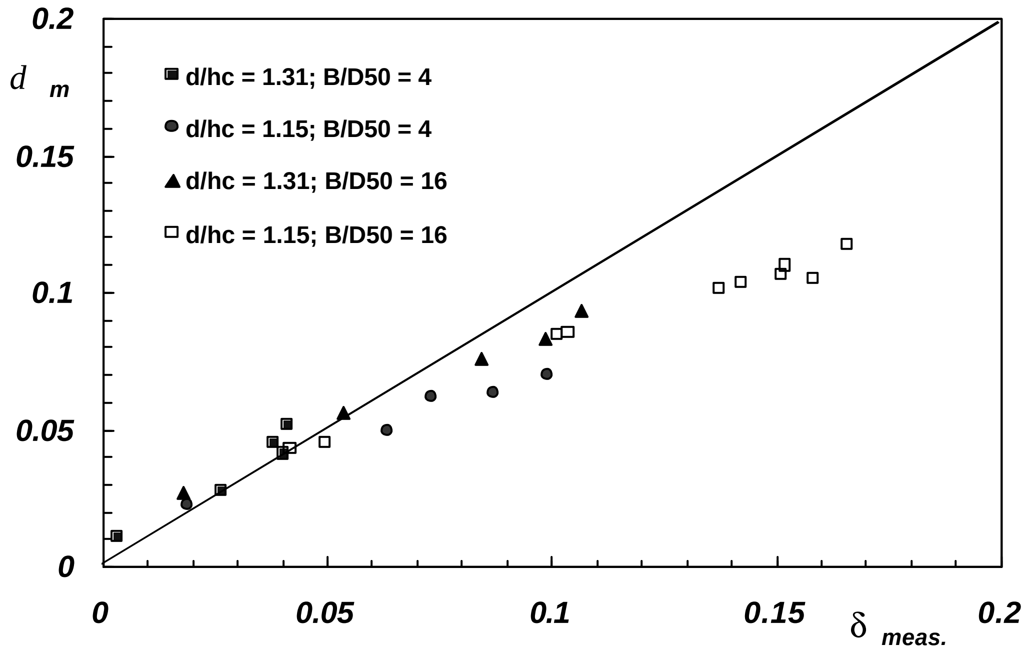

Figure 8-18. Measured wave set up (in meters) vs Equation 23

Hence a further contribution we will call "continuity set up" (Darlymple and Dean, 1971) should be added to the "momentum flux contribution" given by Eq. 23. We will determine this in a straightforward manner, assuming the return flow to be uniform:

$$d_q = \frac{q^2}{f^2 \cdot |R_c|^{\frac{10}{3}}} \cdot B_{eq} \tag{27}$$

Eq. 27 represents the widely used "Gauckler-Strickler" formula for flows, where R_c has been employed as hydraulic radius. B_{eq} is the rectangular equivalent crest width:

$$B_{eq} = B + h_c \cdot ctga \tag{28}$$

Assuming for the flow rate q the expression:

$$q = \frac{1}{8} \cdot H_{en\,i}^2 \cdot \sqrt{\frac{g}{d}} \tag{29}$$

and setting f = 20 (not so far from that commonly used for rock bottom) we obtained a very good agreement with experimental data (Fig. 19).

Figs. 20~21 show the comparison with models by Diskin et al. (1970) and Loveless et al (1998). Since the set up behind submerged breakwaters is strictly connected to energy loss of waves traveling over the structures, it seemed reasonable to employ H_{en} in applying the models (which are valid for regular waves) to irregular waves. However it was noted that the use of significant wave height led to large overpredictions. This also agree with suggestions of Loveless et al. (1998). No significant differences have been noted in the Loveless method when mean or peak periods were used.

Diskin's formula generally agree with data, although the scatter is much larger than in Figure 19. This is mostly due to neglecting the effects of crest width, *B*: the formula tends to overpredict data relative to the narrow crested barrier and to under predict those for the structure with wider extension.

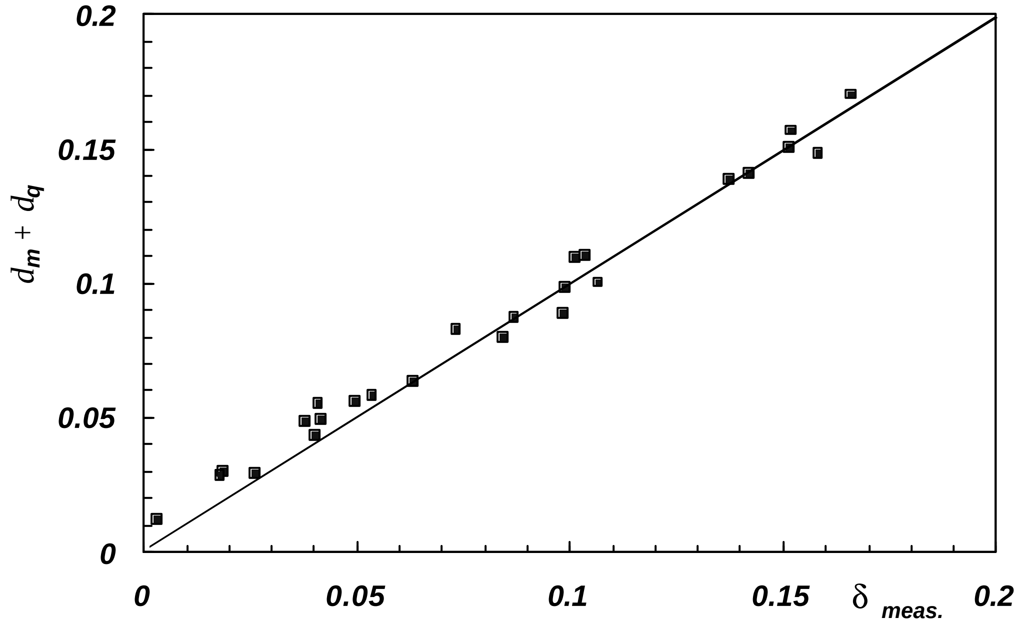

Fig. 8-19. Comparison between measured and predicted set up (values in meters)

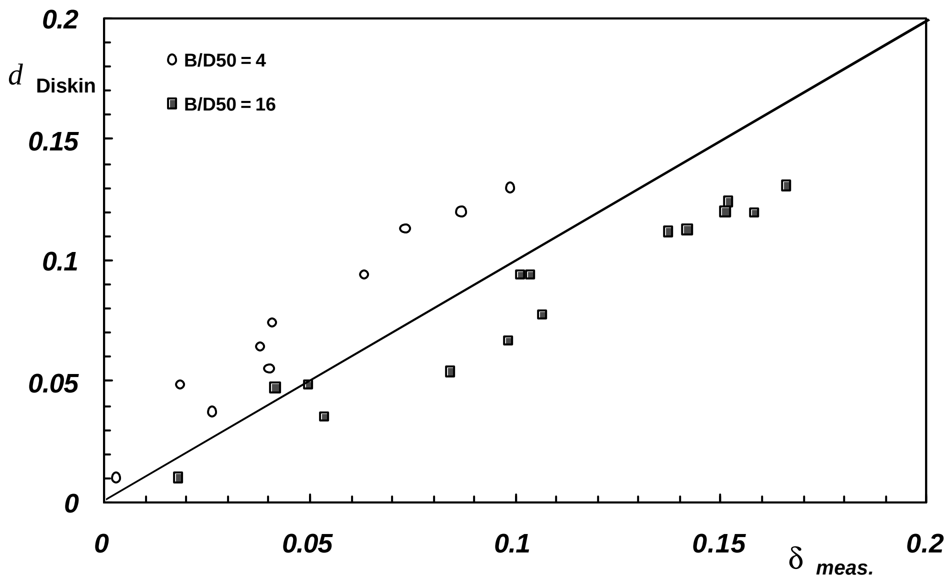

Fig. 8-20. Comparison between experimental data and Diskin's formula (values in meters)

On the other hand the model by Loveless (Fig. 21), gives accurate prediction for $B/D_{50} = 4$, while greatly overestimates the data for $B/D_{50} = 16$.

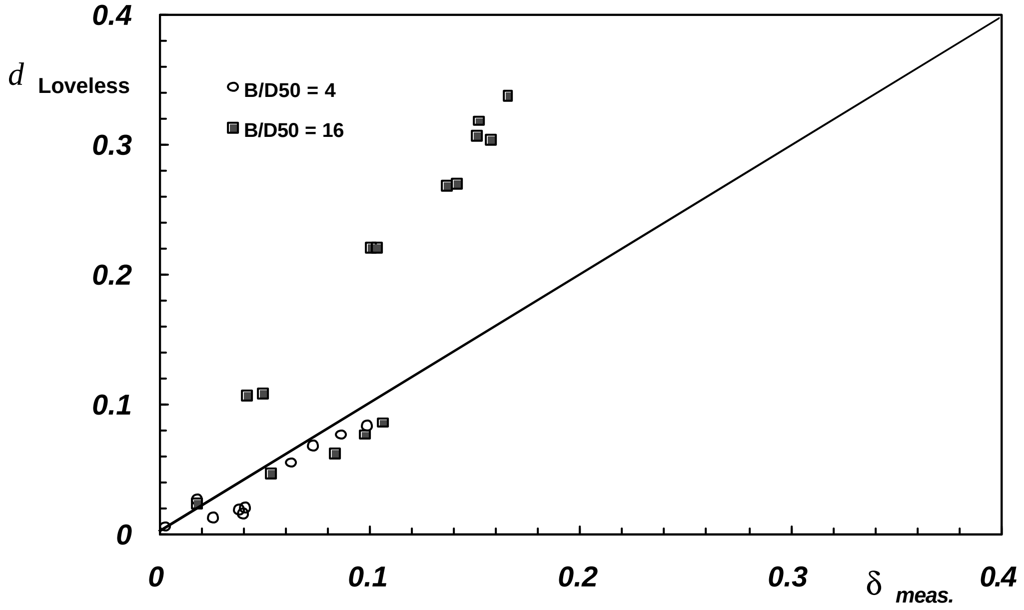

Fig. 8-21. Comparison between experimental data and Loveless formula (values in meters)

8.4.3 Results for low crested breakwaters

Figs. 22~23 show the comparison between Diskin's and Loveless formulae and experimental data for low crested breakwaters.

Diskin's expression predicts data with some accuracy, especially for wide crested breakwaters.

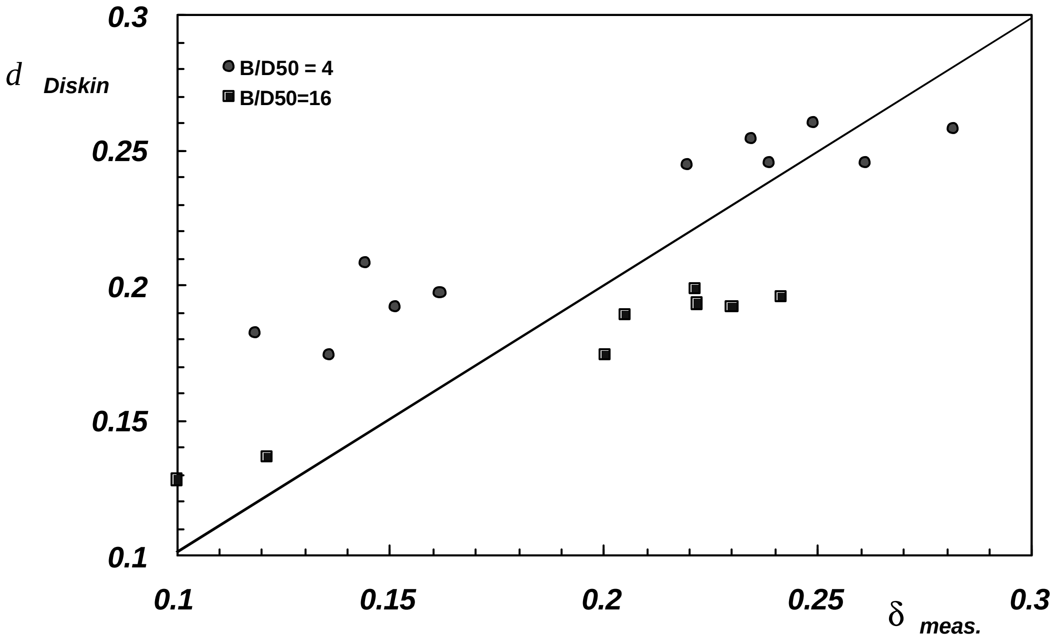

Fig. 8-22. Comparison between experimental data and Diskin's formula (values in meters)

Regarding Loveless formula, the graph suggests that it tends to overestimate measured values for the larger breakwater, although it correctly reproduces the trend of the measures.

On contrary for $B/D_{50} = 4$ experimental data are quite far from Loveless predictions.

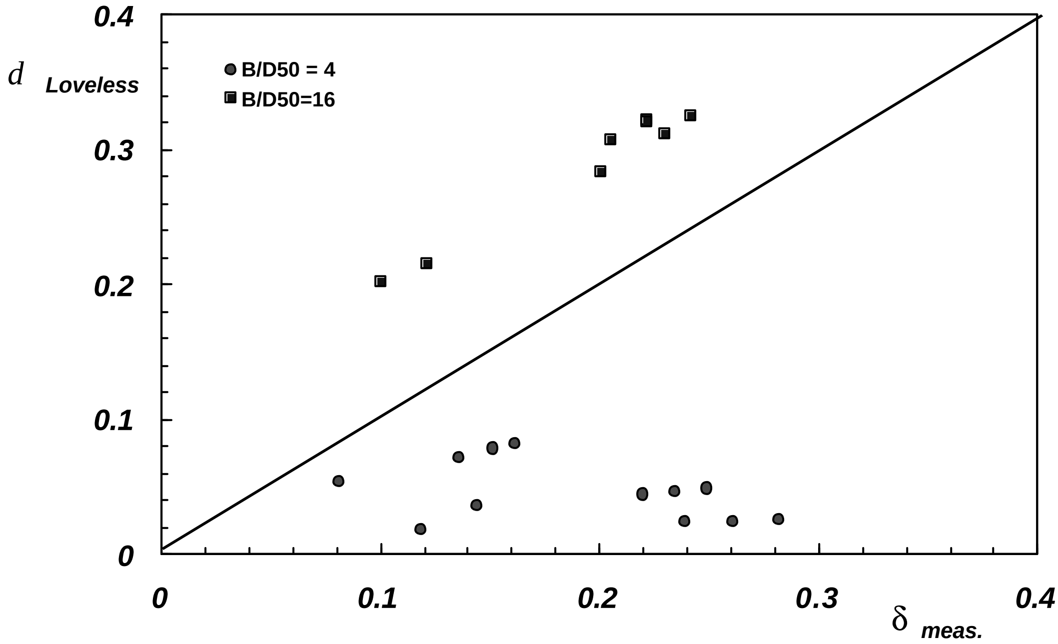

Fig. 8-23. Comparison between experimental data and Loveless formula (values in meters)

The analysis of experimental data seemed to reveal that wave set up behind overtopped but not submerged breakwaters is controlled by "the hydraulic behavior" of the barrier. This means that it is ruled by the mass balance. We expect that overtopping flow rate will be returned offshore partly flowing over the structure and partly through it depending on permeability and freeboard of the breakwater. Assuming a dominantly turbulent flow back, we may set (for a first crude estimate):

$$d = l \cdot \frac{q^2 \cdot B_{eq}}{8 \cdot g \cdot (d + d)^3} \qquad (30)$$

Eq. 30 basically represents a Darcy-Weisbach formula, where q is the flow rate for unit of width, and λ is a friction coefficient. If we set:

$$q = \sqrt{g \cdot H_{moi}^3} \qquad (31)$$

we expect that friction coefficient depends on crest freeboard, R_c, as well as on the inherent permeability of the breakwater. Further, this influence of R_c should be twofold.

From one side, when crest freeboard increases a reduction of l is expected due to a reduction of flow rate q, which is not considered in Eq. 31. On the other hand the larger is the freeboard, the lower is the proportion of the entire discharge will pass over the breakwater. This increases the overall resistance that structure offers to the backwash.

In Fig. 24, friction coefficient evaluated as (measured values of set up have been employed):

$$l = \frac{d}{\dfrac{q^2 \cdot B_{eq}}{8 \cdot g \cdot (d + d)^3}} \qquad (32)$$

is plotted against the non-dimensional crest freeboard $\dfrac{(R_c - d)}{H_{moi}}$.

This parameter seemed to be particularly effective, as it allowed to retain all the information also for $R_c = 0$.

Figure seems to suggest that when the freeboard is relatively low, effects of reduction of water mass flowing back over the structure (i.e. increasing of (R_c - d) should be prevalent respect to the reduction of the flow rate q. So we note a net increase of the friction coefficient, and hence of the wave set up, being all the other parameters the same. On contrary for large freeboards all the water mass flows back through the breakwater, and effects of q reduction reveal to be prevalent.

Unfortunately no definitive conclusion can be derived, since data are few and the effects of structure extent and, mostly, of the intrinsic permeability cannot be assessed. However accounting the crudeness of the assumption have been done, the results appear encouraging.

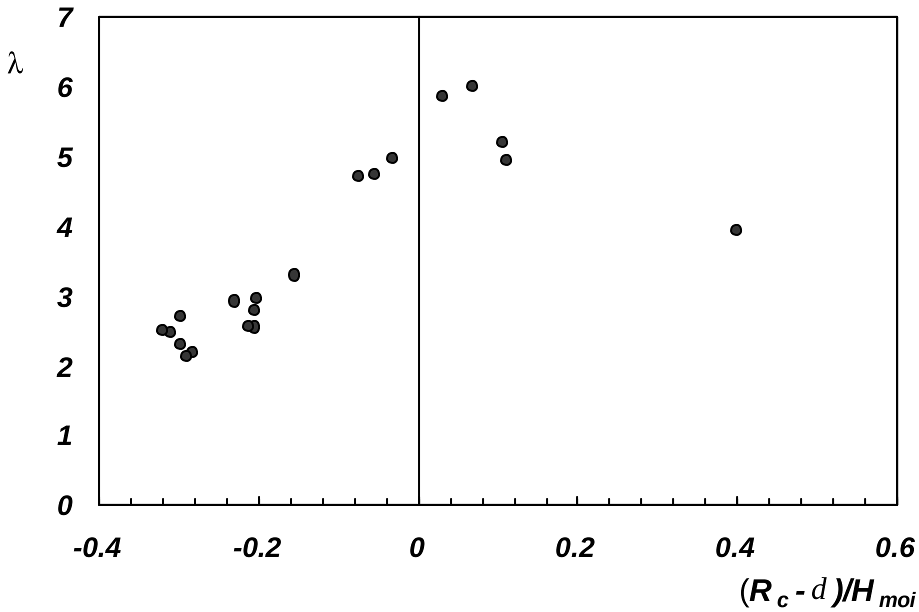

Fig. 8-24. Friction coefficient against non dimensional crest freeboard

8.5 Preliminary conclusions

The paper has presented results of large scale model tests on wave transmission and set up at low-crested/submerged breakwaters. The structures were located in relatively shallow waters under wave conditions ranging from shoaling to post breaking.

Analysis indicated that, among existing transmission formulae, d'Angremond et al. (1996) gives the more reliable estimates of transmission coefficient, K_t, especially for wide crested structures.

Nevertheless the scatter between data and predictions is still significant.

Some explanations of this scatter has been given:

- breaker index should influence wave transmission in the nearshore, although it is usually disregarded in currently used formulae;
- crest width should affect the reduction rate of K_t.

These observations have been quantitatively included in Eq.9 allowing to considerably reducing the differences between predictions and measures (*RMSE* = 0.04). Comparison with data by Powell and Allsop (1985) and Seelig (1980) generally confirmed the validity of proposed approach, despite an influence of cross section characteristics arose.

Further analyses are in progress with the aim confirming/improving Eq. 9 as well as checking its advantages relative to others predictive methods. Particular efforts will be addressed in enlarging data set and in pointing out the effects of permeability.

Analysis of data on wave set up revealed that for submerged breakwaters rise of mean water level should be dominated by the loss of wave momentum connected to wave breaking. When crest width increases and, mostly, when degree of submergence reduces, a “continuity set up contribution” should be added taking account the shear stress accompanying the return flow over the structures.

Behaviour of overtopped but not submerged breakwaters seems to be essentially controlled by the hydraulic of the system and namely by the balance between overtopping flow rate and the backwash over and through the barrier.

A comparison has been done with prediction formulae by Diskin et al. (1970) and Loveless et al. (1998) which are valid for regular waves. Results show that using the energetic equivalent wave height, H_{en}, the former predicts with some accuracy the data although a certain scatter is still present. On contrary the agreement with Loveless formula appeared rather poor.

8.6 Acknowledgement

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