CORE MADE OF GEOTEXTILE SAND CONTAINERS FOR RUBBLE MOUND BREAKWATERS AND SEAWALLS: EFFECT ON HYDRAULIC STABILITY AND PERFOMANCE

by

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ABSTRACT

A systematic experimental study in the twin-wave flumes of Leichtweiss-Institute (LWI) is performed on a geocore breakwater and a conventional rubble mound breakwater in order to comparatively determine the hydraulic stability and the hydraulic performance, including wave reflection, wave transmission, wave run-up and wave overtopping. The geocore breakwater consists of a core made of sand-filled geotextile containers covered by an armour made of rock. The geocore is more than an order of magnitude less permeable than the quarry run core of a conventional breakwater. As expected, the core permeability strongly affects the armour stability on the seaside slope, the wave transmission and the wave overtopping performance. Surprisingly, however, wave reflection and hydraulic stability of the rear slope are less affected. Formulae for the stability and hydraulic performance of the geocore breakwater are proposed, including wave reflection, transmission, run-up and overtopping.

1 RESEARCH FRAMEWORK, MOTIVATIONS AND OBJECTIVES

There are several reasons which might lead the engineer and other decision makers in practice to use sand instead of conventional quarry run for the core of rubble mound breakwaters and structures, including among others:

(i) non-availability of rock material in sufficient quantities and at affordable costs;

(ii) sediment infiltration through rubble mound structures which may result in the shoaling of navigation channels and harbour basins, and thus in higher maintenance dredging costs;

(iii) reduction of wave transmission through the structure which might particularly be crucial in the case of long waves.

On the other hand, the use of sand as a quasi-impervious core instead of quarry stone would result in an increase of

(i) wave set–up and run-up at the structure;

(ii) wave overtopping;

(iii) wave reflection,

which might be detrimental to the stability of the structure, to the operation on and behind the breakwater (due to excessive overtopping) as well as to navigation and seabed stability.

Moreover, serious difficulties arise in practice when trying to design and construct the filter to protect the sand core against wash out by wave action. Applying geometrically closed filter criteria would result in very complex, multiple and relative thin filter layers which will not only be very costly and very difficult to build in larger water depths, but also might certainly fail due to the almost unpredictable very

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complex loading conditions of the sand core under cyclic pulsations by waves and entrained air at the interface with the last filter layer. Such failures have indeed been observed under both laboratory and field conditions in the past. Laboratory evidence has also shown that introducing the so-called “geometrically open filter” criteria to design a “hydraulic sand-tight filter” may reduce the number of filter layers. However, the main practical difficulties mentioned above will remain, including those associated with the long-term stability of the sand-core due to the high complexity of the loading and its uncontrollability during the entire storm duration and the life cycle of the structure.

Geotextile filters might present themselves as an alternative to the very complex, costly and uncertain filter made of multiple layers of granular material. However, geotextile mats are not only difficult to install under waves and currents, but also may introduce a shear surface which might be detrimental to the stability of the armour layer.

A more feasible alternative is to use a core made of geotextile sand containers. This will not only allow to overcome the aforementioned core stability problems, but also to provide (i) a better erosion stability of the core; and (ii) an increased stability against seismic loads as compared to a core simply made of loose sand. However, many of the drawbacks mentioned above remain with respect to wave set-up, run-up, overtopping, reflection and armour stability in comparison to a conventional core. Therefore, an extensive research programme was initiated at Leichtweiss-Institute to study both hydraulic performance and armour stability, including the processes involved and the development of prediction formulae for the design of a class of rubble mound structures with a core made of geotextile sand containers (Fig. 1).

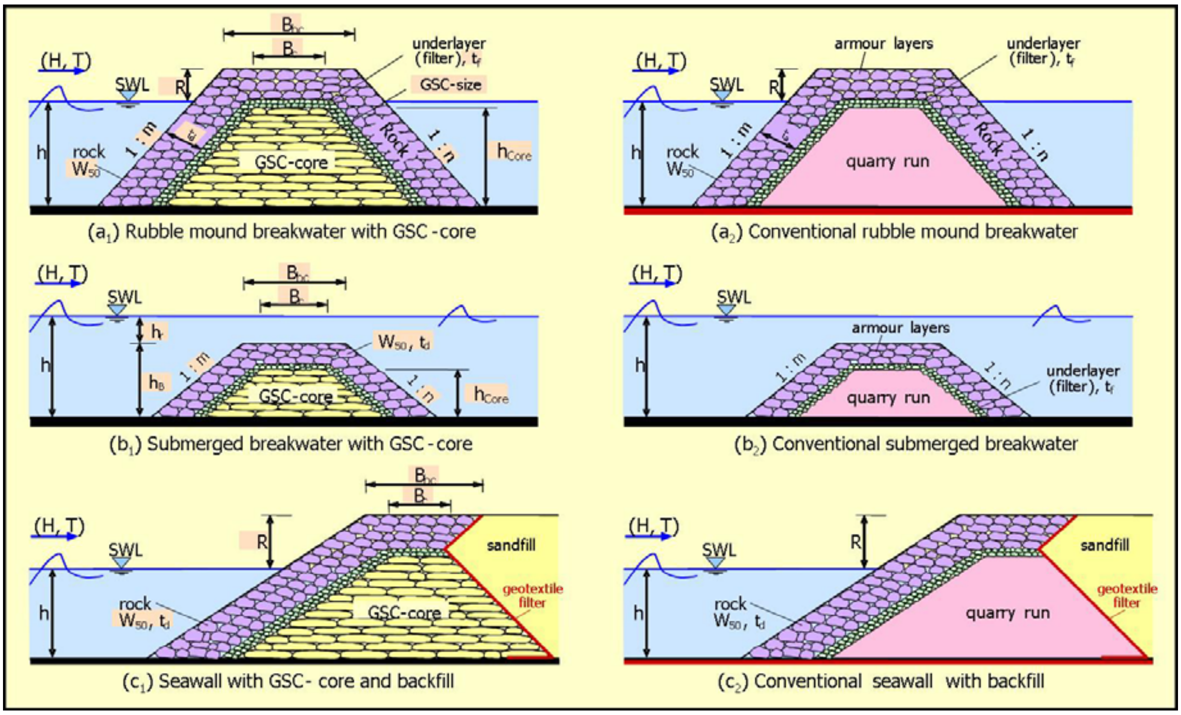


Figure 1. Class of Geocore Structures in Comparison to Conventional Rubble Mound Structures

This paper will, however, address only the first phase of the research programme which is concerned with hydraulic model tests to study in the twin-wave flumes of LWI the hydraulic performance and the armour stability of a rubble mound breakwater made of geotextile sand containers as compared to its conventional counterpart with a core made of quarry stones (Fig. 2).

2 EXPERIMENTAL SET-UP AND PROCEDURE

Both structures, the geocore breakwater and its conventional counterpart, were simultaneously investigated in the twin-wave flumes of Leichtweiss-Institute (LWI), Braunschweig. The twin-wave paddles can be operated synchronously or independently (Fig. 2).

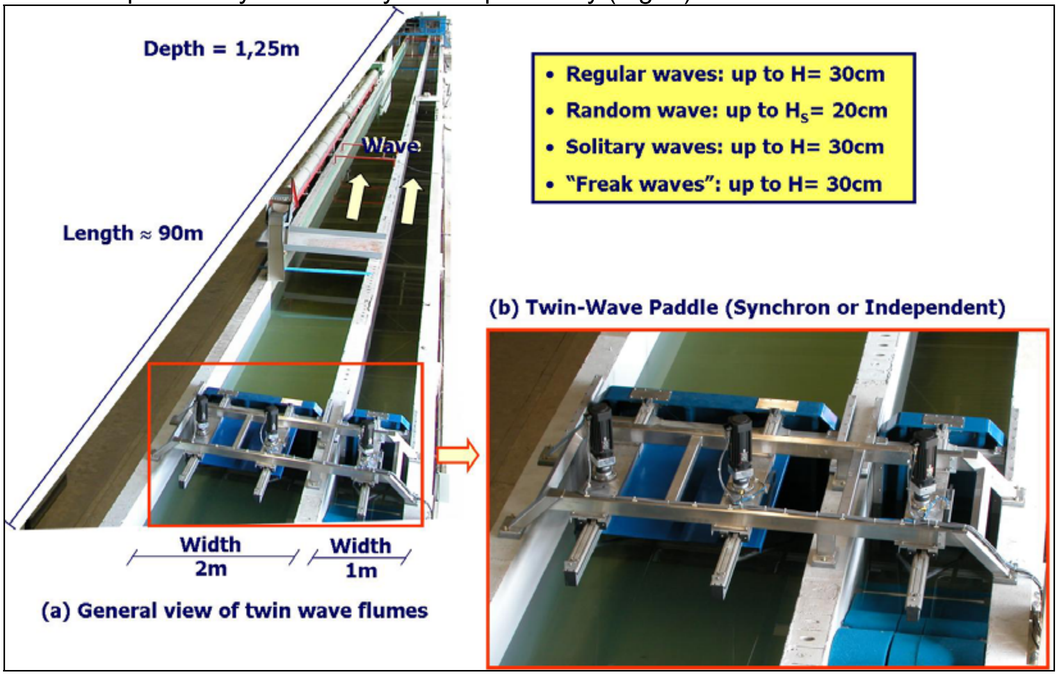


Figure 2. Twin-Wave Flumes of Leichtweiss-Institute (LWI), Braunschweig

The geocore breakwater model was built in the first flume with 2m width, while the conventional model was built in the second flume with 1m width (Figs. 3 and 4).

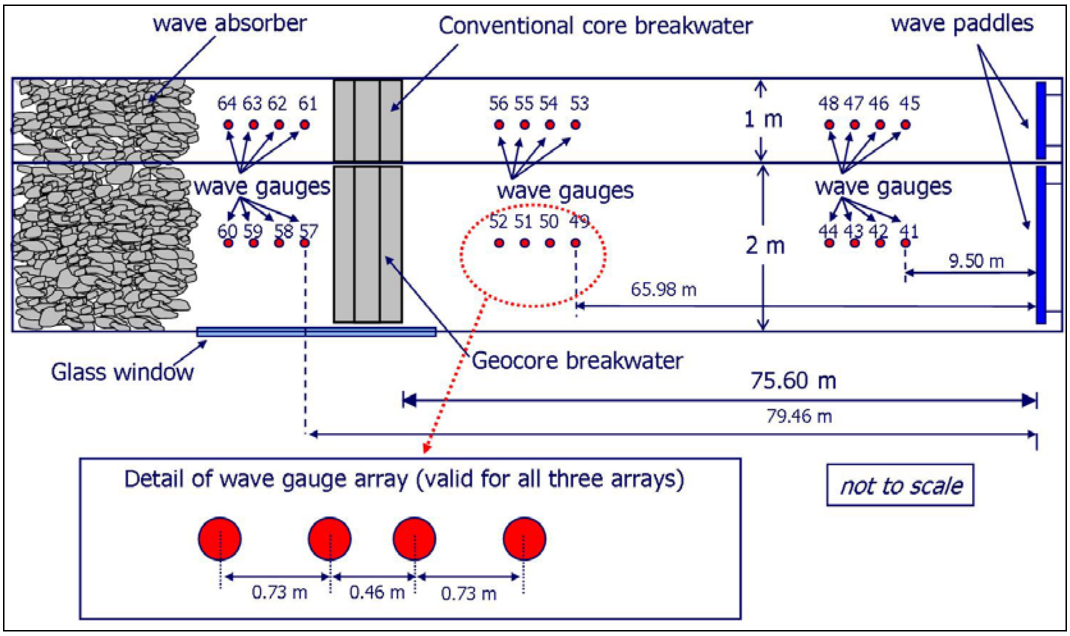


Figure 3. Geocore Breakwater and Conventional Breakwater in the LWI Twin-Wave Flumes

Both models have the same armour, the same geometry, and dimensions, but different cores (Fig. 4). The core of the conventional breakwater is made of stones with D_{50} = 2.3cm, covered by a filter layer with $D_{50} \approx 2.7$cm. The core of the geocore breakwater is made of randomly dropped geotextile containers (0.26 x 0.13m x 0.052m), but without any filter layer. The rock armour units with D_{50} = 5.9cm (0.53kg) are the same for both breakwater models.

Three arrays with a total of 12 gauges in each flume were used for the wave measurements (Fig. 3). The wave gauge array in front of the structure was used for the wave reflection analysis, while the gauge arrays behind the structure were used to determine wave transmission.

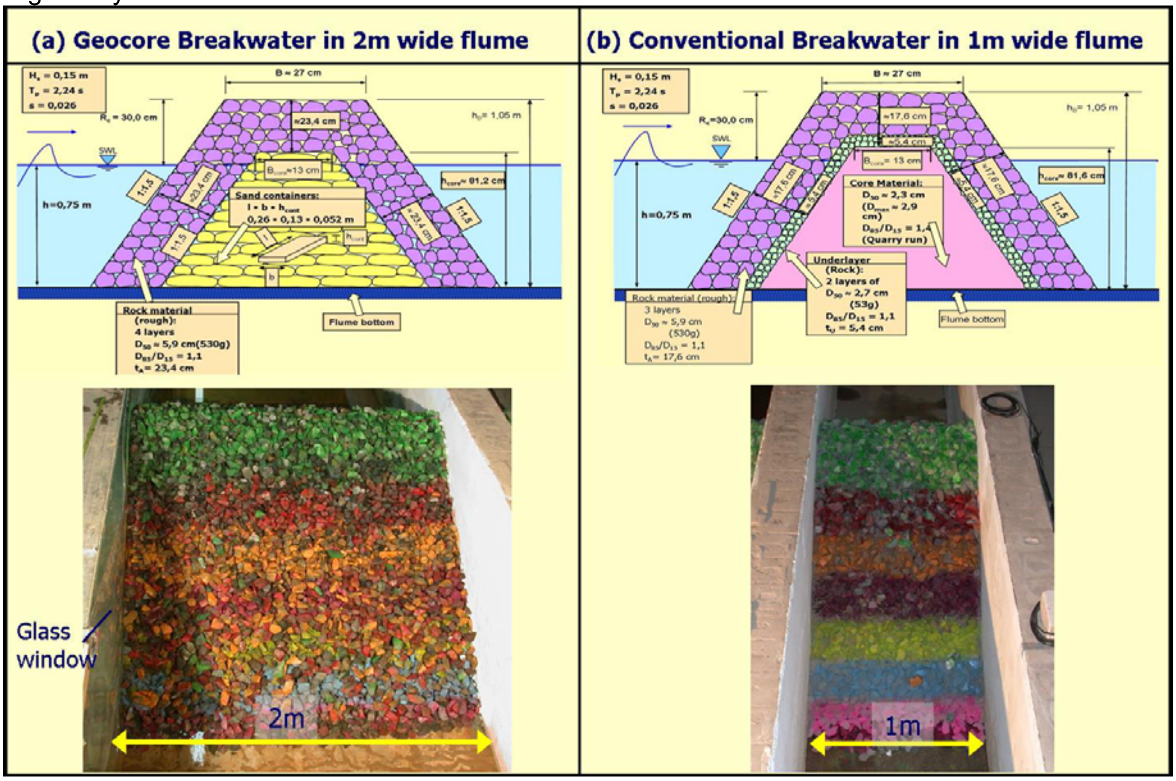


Figure 4. Cross-Sections and Front Views of the Two Breakwater Models in the LWI Twin-Wave Flumes

The construction materials used for both models are illustrated in Fig. 5, showing the shape and size of both geocontainers and rock materials.

Prior to building the rock armour, both breakwater models were subject to controlled steady flow conditions to determine the core permeability. The detailed procedure of the permeability tests is described by Recio and Oumeraci (2006). As a result, Darcy's permeability coefficients $k = 3.9 \cdot 10^{-1}$m/s and $k = 2.27 \cdot 10^{-2}$m/s were obtained for the conventional core and for the geocore, respectively.

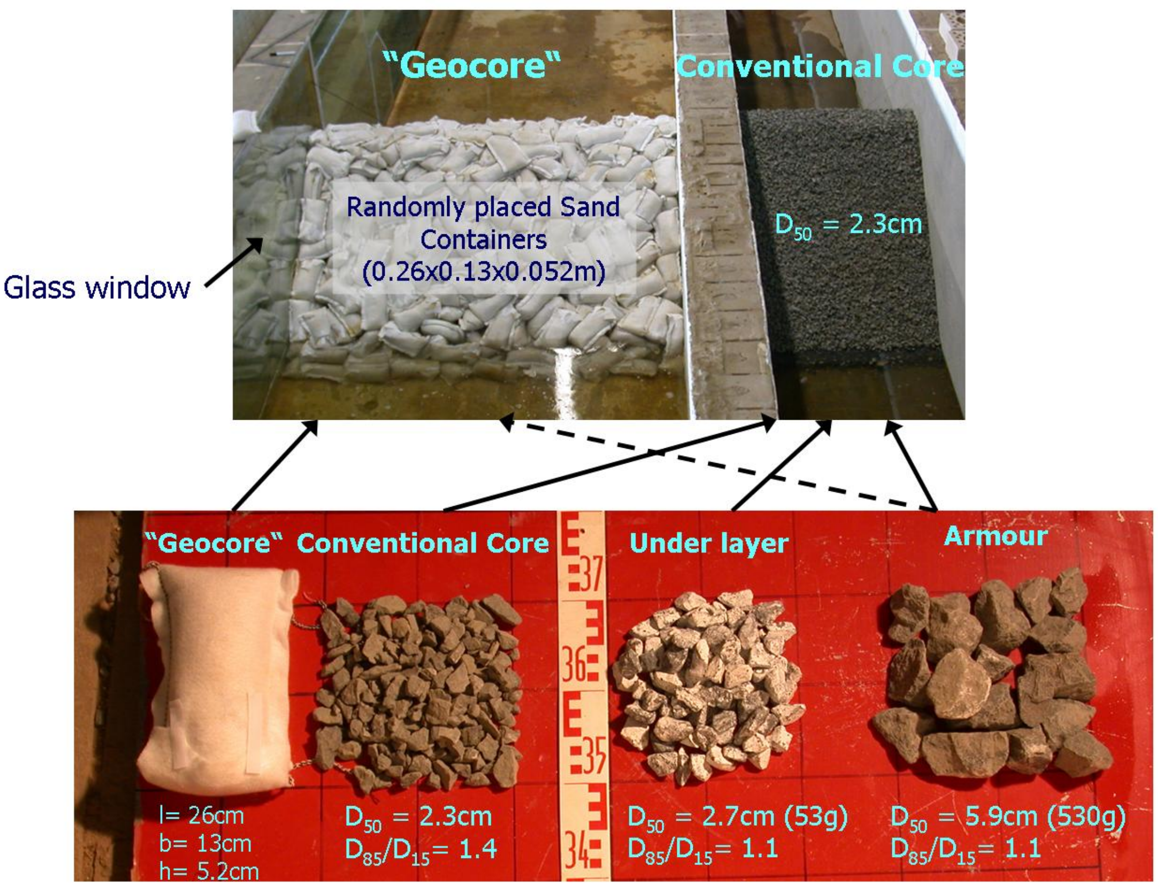


Figure 5. Construction Materials Used for Both Breakwater Models

For the measurement of wave overtopping discharge, a tank (V = 1.36m x 0.46m x 0.25m) on weighing cells is installed behind the structure and linked to the middle of the breakwater crest by a 23.1cm wide overtopping channel. A layer thickness gauge is installed at the breakwater crest near the overtopping channel to identify single overtopping events (see Oumeraci et al, 2007 for more details). Video cameras were used to record the processes in front of and behind both breakwater models as well as through the glass window.

Wave spectra (JONSWAP) with H_s = 0.08-0.20 m and T_p = 1.7-2.55 s were generated in water depths d = 0.25-0.75m. 1000 waves were generated by using active wave absorption at the wave paddles.

3 HYDRAULIC PERFORMANCE: COMPARATIVE ANALYSIS

A comparative analysis of the two breakwater types was first performed with respect to wave reflection, wave transmission, wave run-up, and wave overtopping performance. As a result, prediction formulae are proposed for the geocore breakwater and compared to their counterparts for the conventional rubble mound breakwater alternative.

3.1 Wave Reflection Performance

Wave reflection from coastal structures may severely affect the structure stability by increasing bed scour. It may also increase the erosion of the foreshore and of the neighbouring coastal stretches. Several prediction formulae have been proposed in the past (see "Review" in Oumeraci et al, 2007; Oumeraci and Muttray, 2001; Zannuttigh and Van der Meer, 2007). A comparative analysis of the uncertainties associated with 12 prediction formulae were performed by Muttray (2001), showing coefficient of variation from 10 to 140%. Among the existing formulae, the following one proposed by

Seelig and Ahrens (Seelig, 1983) was found to be most widely used and associated with the lowest uncertainties (CoV<30%):

$$K_r = \frac{a \cdot \xi_0^2}{\xi_0^2 + b} \tag{1}$$

where

ξ_0 = surf similarity parameter $\xi_0 = \tan\alpha \Big/ \sqrt{\frac{H_0}{L_0}}$,

a, b = structure parameter, depending on the permeability, roughness, geometry and water depth conditions (a = 0.5-10 and b = 5-80).

Moreover, it was also found that taking into account the relative water depth kod (with ko = 2π/L0, wave number in deep water) as the primary influencing parameter will strongly decrease the uncertainty (CoV<15%).

$$K_r = c \cdot \frac{\tan\alpha}{\sqrt{k_o d}} \tag{2}$$

where:

c = constant depending on the type of structure;

k_0 = $2\pi/L_0$ = wave number;

The reflection coefficient, obtained from the analysis of the tests with no overtopping for both geocore and conventional core breakwater, is Kr = 0.22-0.54. Plotting the reflection coefficient Kr against the surf similarity parameter ξ_{0m}, calculated using the characteristic wave period $T_{m-1,0}$ and the characteristic wave height H_{m0}, surprisingly failed to provide any correlation between K_r and ξ_{0m} (Fig. 6), although a relatively good correlation between K_r and $T_{m-1,0}$ was observed (see Oumeraci et al, 2007).

Therefore, the data were plotted again using the reflection model proposed by Oumeraci and Muttray (2001) which accounts for the relative depth k_0d as the primary influencing parameter (Fig. 7). This resulted in the following prediction formulae for both breakwaters

$$K_r = 0.43 \cdot \frac{\tan\alpha}{\sqrt{k_o d}} \tag{3}$$

with

k_0 = $\frac{2\pi}{L_o}$ (wave number in deep water);

tan α = steepness of the seaward slope of the structure,

d = water depth in front of the structure.

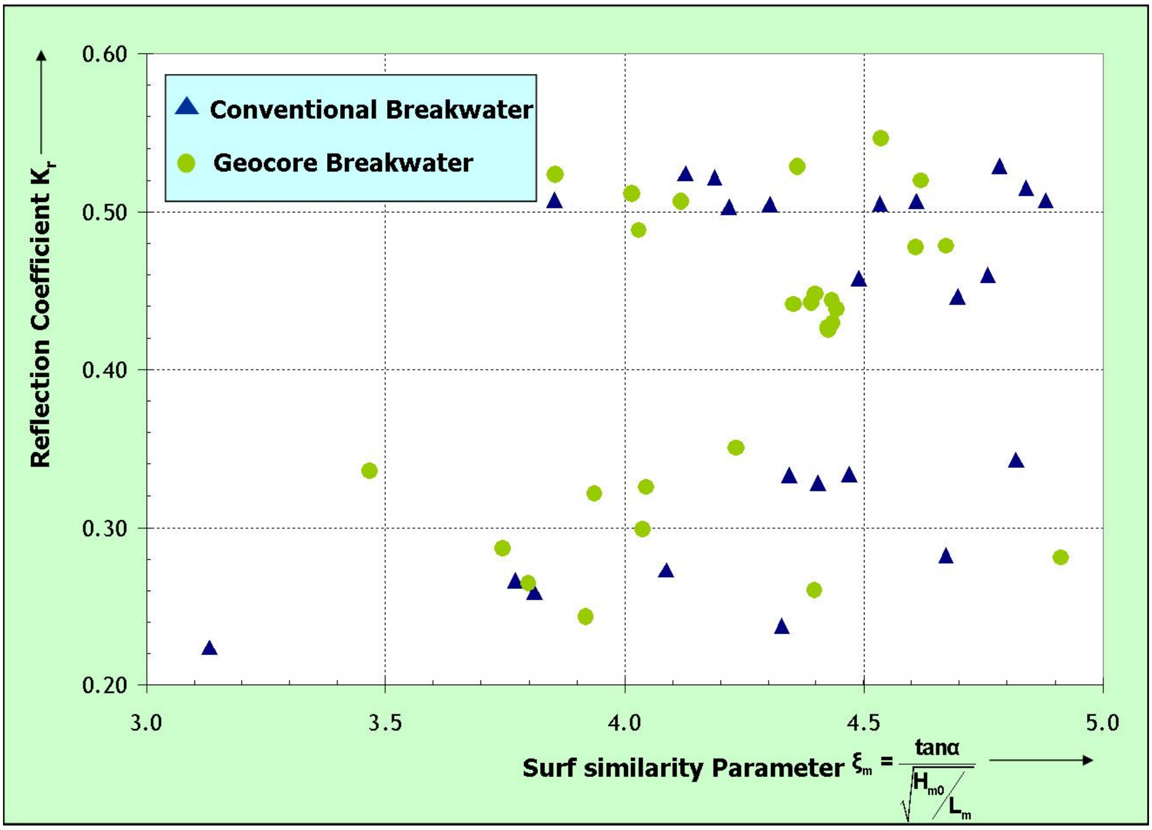


Figure 6. Reflection Coefficient K_r vs. Surf Similarity Parameter ξ_{om}

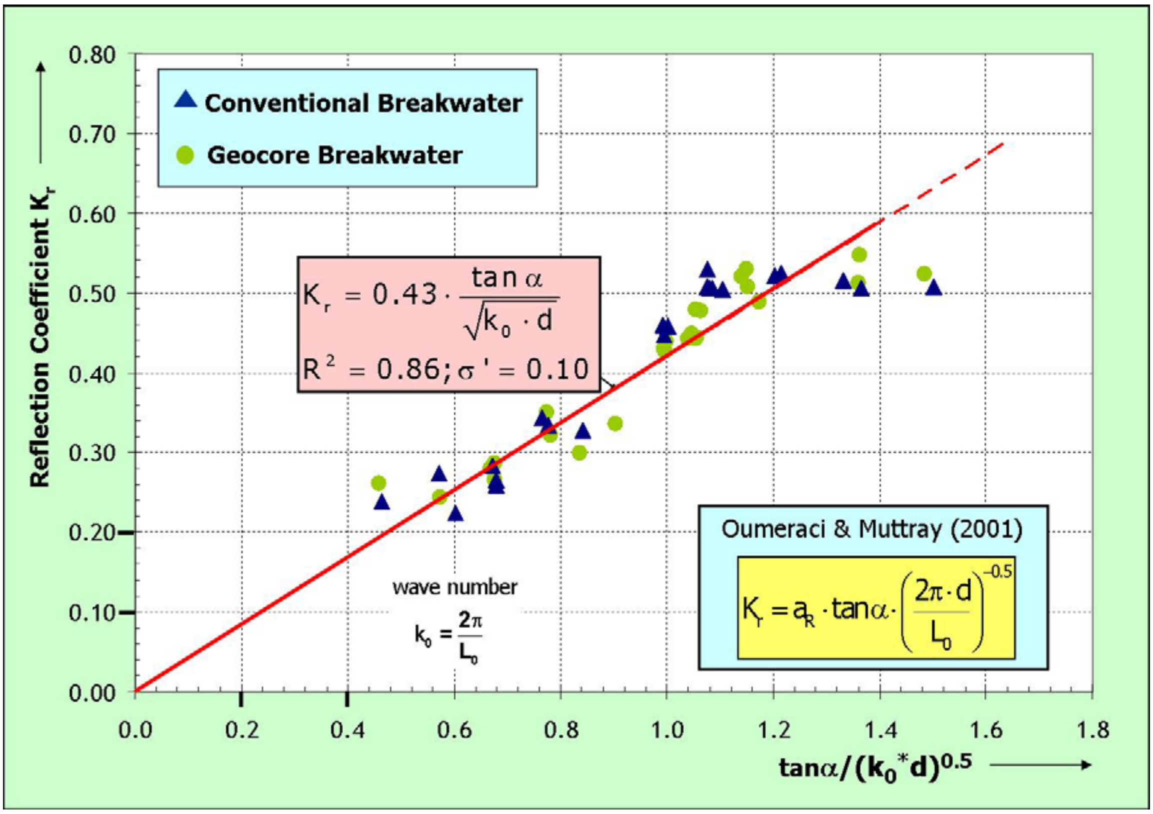


Figure 7 Reflection Coefficient K_r vs. Relative Water Depth k_0d

3.2 Wave Transmission Performance

A rubble mound breakwater is generally aimed at protecting valuable facilities such as harbours but also vulnerable sections of coastline such as sand beaches from wave attack. Its primary function is thus to attenuate the incident waves to a prescribed level. Therefore, wave transmission performance constitutes one of the primary considerations in the functional design. In addition to the transmission around the breakwater head, wave transmission can occur as a result of wave overtopping and by penetration through the structure.

Wave transmission through the structure is essentially determined by the permeability of the breakwater as well as by the period (and steepness) of the incident waves. Longer waves have therefore a much higher transmission potential than shorter waves.

Several models have been proposed in the literature (CEM, 2002; Oumeraci et al, 2007) to predict wave transmission through a rubble mound structure which, however, do not explicitly account for the effect of the core permeability. The parameter which mostly affects the transmission coefficient K_T has been found to be a function of the relative freeboard R_c/H_s and the steepness $s_m = H_s/L_0$ of the incident waves (Allsop, 1983):

$$R^* = \frac{R_c}{H_s}\sqrt{\frac{s_m}{2\pi}} \tag{4}$$

In fact, the best fit in analysing the experimental results for the geocore and conventional breakwater was found by using the modified freeboard R^* according to Eq. (4). As a result, the following formula is obtained (Fig. 8) for the transmission coefficient:

$$K_T = a_r \cdot (R_c^*)^{b_r} \tag{5}$$

where a_r and b_r are constants, which depend on the permeability of the structure. The structure parameter a_r and b_r are obtained for the conventional and geocore breakwater as follows (Fig. 8):

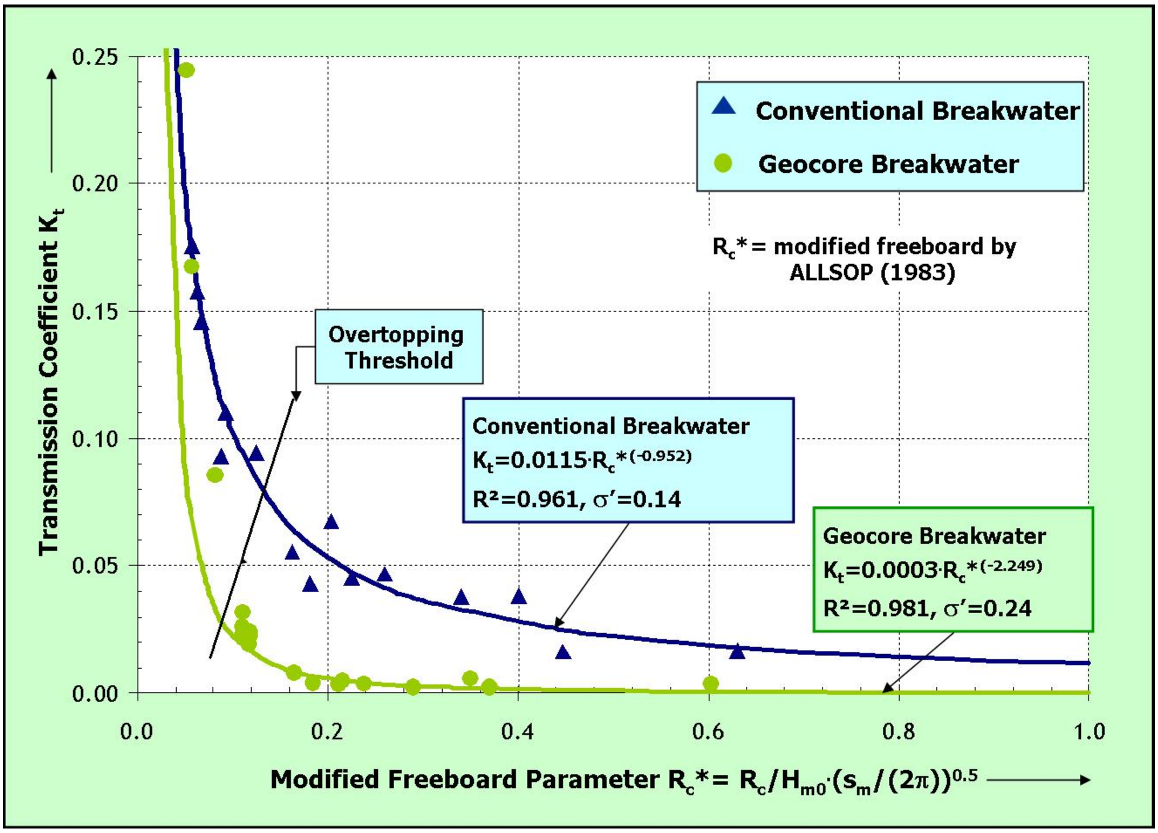


Figure 8 Wave Transmission Coefficient for Conventional Breakwater and Geocore Breakwater

- Conventional breakwater (CoV = ~14%):

$$a_r = 0.115 \text{ and } b_r = -0.952 \tag{6}$$

- Geocore breakwater (CoV= 24%):

$$a_r = 0.0003 \text{ and } b_r = -2.249 \tag{7}$$

It should be stressed that the obtained relationship is valid only for short period waves. For longer waves such as tsunami, the difference between the transmission performance of the geocore alternative and the conventional breakwater will be much more important.

3.3 Wave Run-up Performance

The wave run-up R_u is defined as the maximum elevation during wave attack to which the water surface rises on the seaward face of the breakwater. R_u is important in defining the required height of the structure. Generally, the run-up level exceeded by 2% of the incident waves ($R_{u2\%}$) is used for this purpose. $R_{u2\%}$ generally depends on the wave height, the surf similarity parameter, the geometry and surface roughness of the slope as well as on the permeability of the structure. For less impermeable structures with a rough slope such as the geocore breakwater, most of the energy dissipation takes place at the structure face. As compared to a conventional breakwater, a geocore breakwater is therefore associated with less internal energy dissipation and thus with higher internal set-up and higher wave run-up.

A literature study performed by Schley (2006) has shown that the run-up models for sloping rock armoured structures for which the best fit is obtained with the measured data of both tested breakwater types are (i) those proposed by CEM (2003) for rock armoured slopes (Eq. VI-5-13 and Table VI-5-5 in CEM, 2003), and (ii) the simple linear model developed for rubble mound breakwater by Van de Walle (2003) within the EU-Opticrest project.

Based on the model proposed in CEM (2003) for the conventional breakwater type, the following run-up formula was determined (CoV = 2.8%):

$$R_{u2\%} = 1.217 \times \xi_{om}^{0.274} H_{mo} \text{ for } \xi_{om} = 3.3 - 7.0 \tag{8}$$

while for the geocore breakwater the following formula was obtained (CoV=3.4%)

$$R_{u2\%} = 1.415 \times \xi_{om}^{0.274} H_{mo} \text{ for } \xi_{om} = 4.1 - 6.7 \tag{9}$$

Based on the model proposed by van de Walle (2003), the following run-up formula was obtained for the conventional breakwater type (CoV=2.7%):

$$R_{u2\%} = (0.106 \times \xi_{om} + 1.355) \cdot H_{mo} \text{ for } \xi_{om} = 3.3 - 7.0 \tag{10}$$

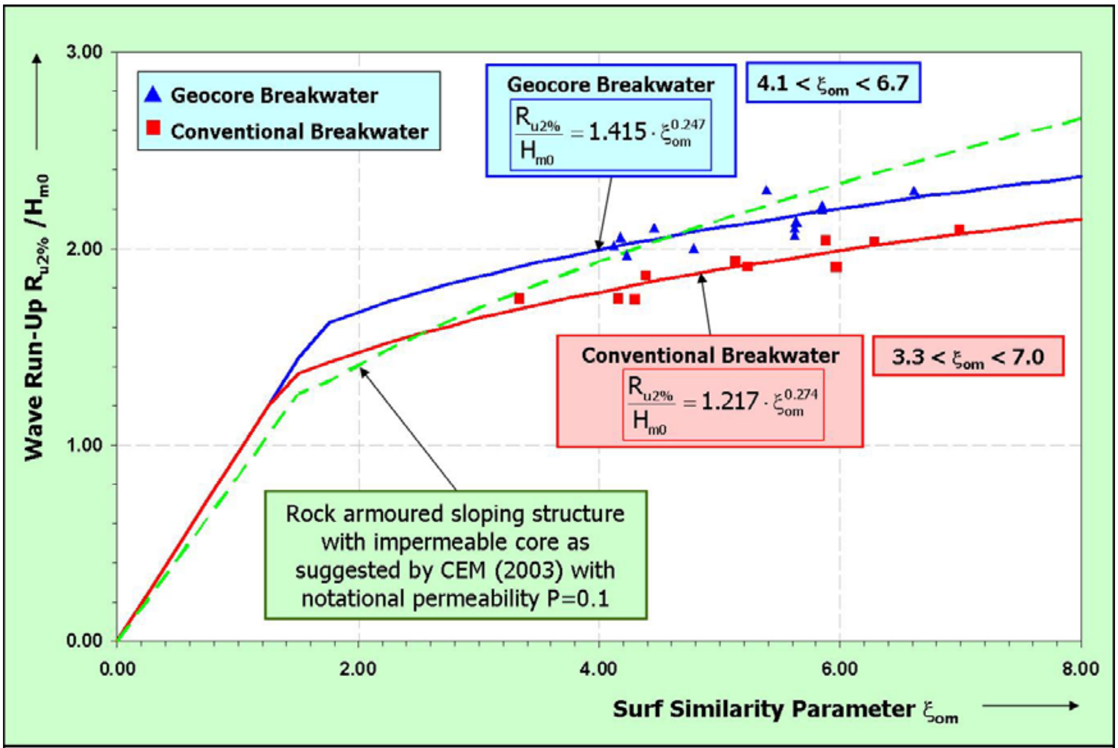
while for the geocore breakwater the following formula resulted (CoV=3.4%):

$$R_{u2\%} = (0.104 \cdot \xi_{om} + 1.583) \cdot H_{mo} \text{ for } \xi_{om} = 4.1 - 6.7 \tag{11}$$

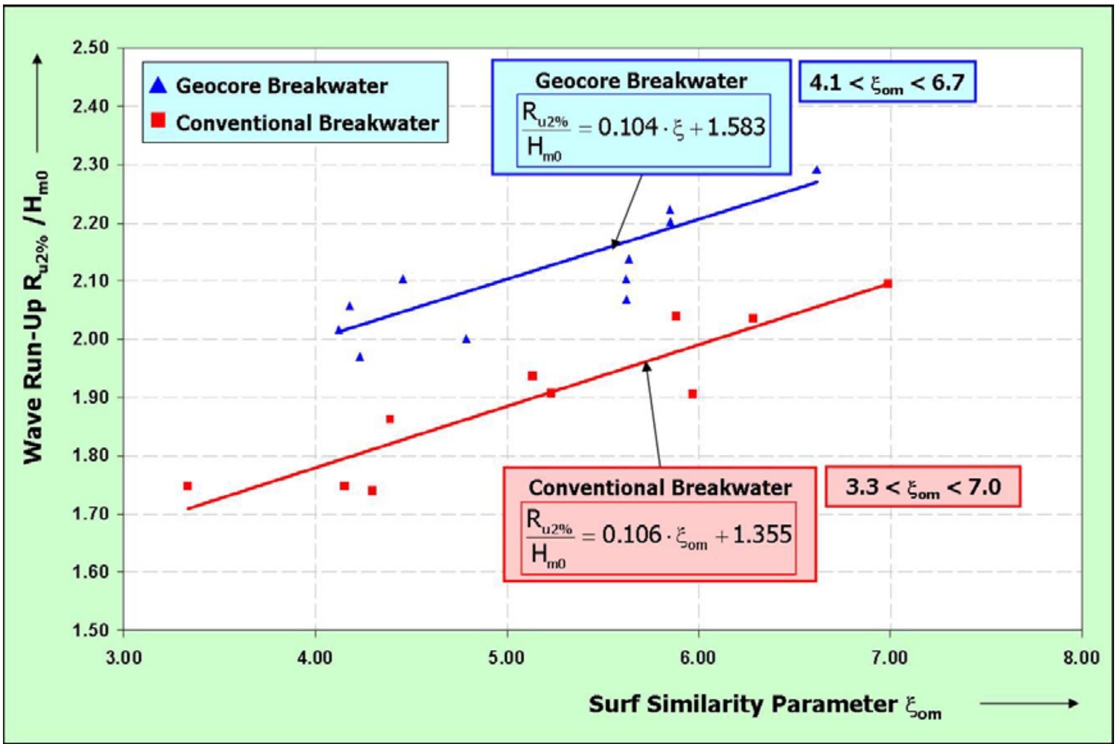
The results using both models to fit the measured data are shown in Fig. 9. The difference between the two breakwater types is in the order of 20%; i.e. the required crest level of a geocore breakwater should be increased accordingly if wave run-up or/and wave overtopping is an important issue.

3.4 Wave Overtopping Performance

Wave overtopping occurs when the maximum run-up level R_{umax} exceeds the crest freeboard R_c. Excessive overtopping of a rubble mound structure may affect (i) the stability of the structure crest and rear slope, (ii) wave transmission past the structure and (iii) the function of the structure, especially when equipment is on the crest and berths for vessels or sensitive reclaimed areas are located behind the structure. Therefore, both safety and function of the structure put restrictions on the wave overtopping discharge to be accepted. Design criteria for defining the crest level are increasingly moving from the requirement associated with wave run-up to allowable wave overtopping discharge.



(a) Based on Model of CEM (2003) for rock armoured slopes



(b) Based on Model of Van de Walle (2003) for rock armoured slopes

Figure 9. Run-up for Conventional Breakwater vs. Run-up for Geocore Breakwater

Many wave overtopping models have been considered to comparatively analyse the measured data for both breakwater types tested, including those proposed by CEM (2003), TAW (2002), Besley (1999), Medina (2002), Bakker et al (2005), etc. (see Schley, 2006). The best-fit of the experimental data for the conventional breakwater and the geocore breakwater was obtained by using the model proposed by TAW (2002):

$$Q_* = a \cdot \exp\left(-\frac{b \cdot R^*}{\gamma_f}\right) \quad (12)$$

where:

$$Q_* = \frac{q}{\left(g \cdot H_{m0}^{\ 3}\right)^{0.5}}$$

q = average overtopping discharge [m³/s·m]

R^* = R_c/H_{m0} = relative freeboard [-]

Using a = 0.2 and b = 2.6 in Eq. (12) for both breakwater types, the correction factor γ_f, initially intended to account for the surface roughness effect, is used to distinguish between the effect of permeability on the average overtopping rate q (Fig. 10):

- γ_f= 0.52 for the conventional breakwater (CoV = 12%)
- γ_f= 0.60 for the geocore breakwater (CoV= 0.20)

Based on the results in Fig. 10 and the results of further analysis (Schley, 2006), the geocore breakwater is associated with three to four times higher overtopping discharges than the conventional breakwater.

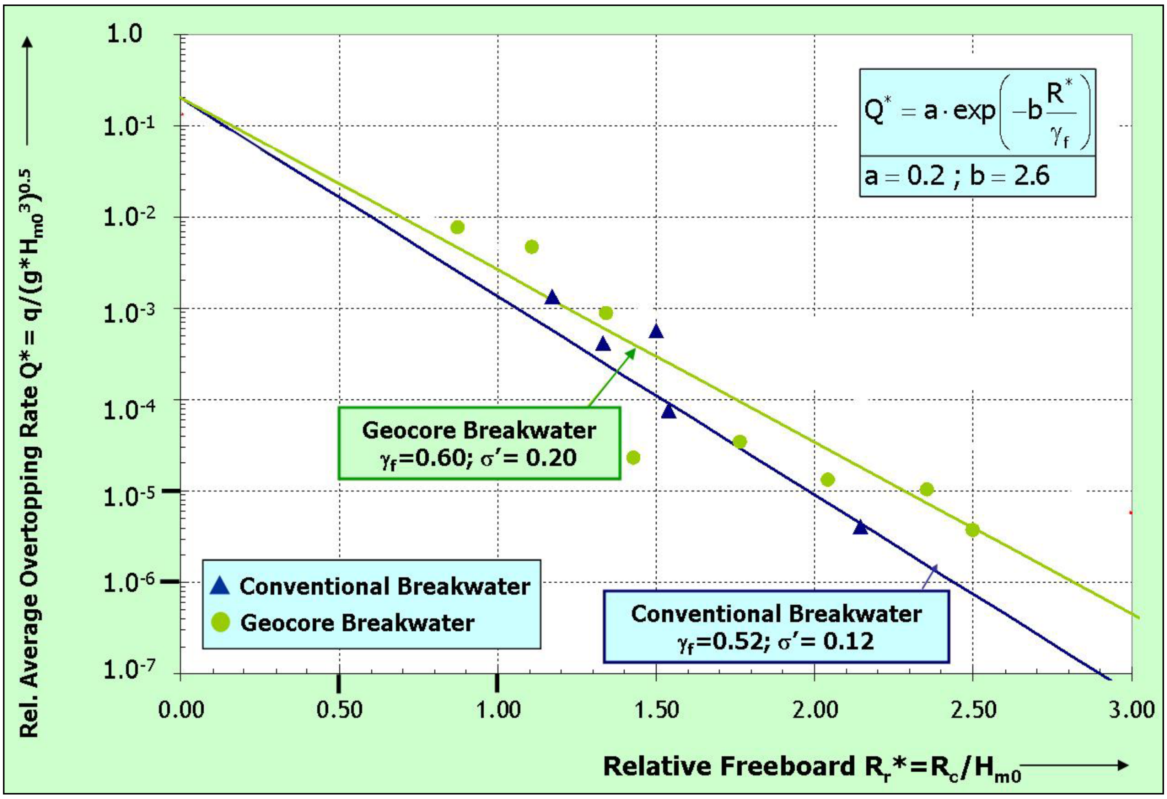


Figure 10. Average Wave Overtopping for Conventional and Geocore Breakwater

4 HYDRAULIC STABILITY OF ARMOUR UNITS

The integrity of both breakwater types is primarily determined by the hydraulic stability of individual armour units on the inner and outer slope. Therefore, the armour was subdivided in six areas on the seaward slope and four areas on the landward slope, while the structure crest was considered as a transition between the inner an outer areas (Fig. 11).

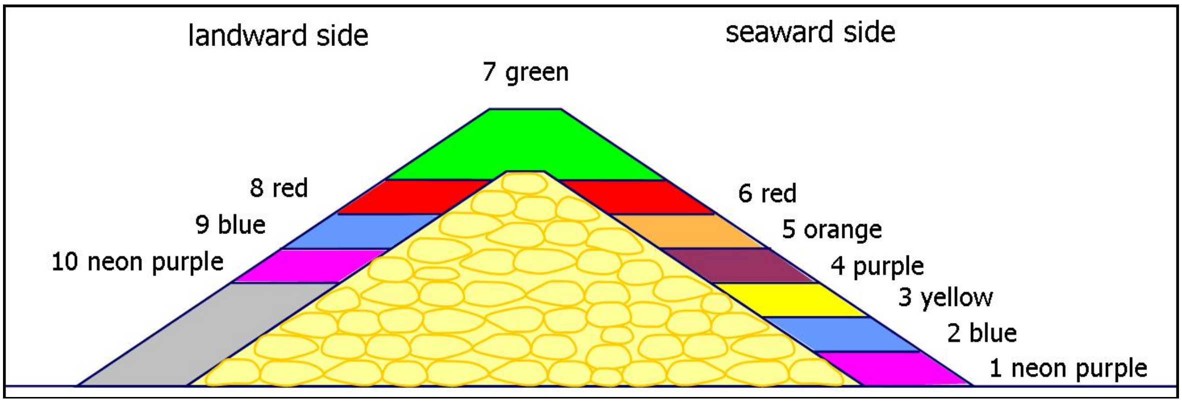


Figure 11. Subdivision of the Rock Armour in Areas with Different Colours for Damage Observation and Analysis.

The damage D to the armour layer was obtained by counting the number of displaced units over a minimum distance $a \geq D_{n50}$ within the considered area and for the whole test duration (1000 waves):

$$D = \frac{N_{od}}{N_a} = \frac{N_{tot} / \left(B_{flume} / D_{n50}\right)}{A_{ref} / A_{stone}} \tag{13}$$

with:

- N_{tot} = total number of displaced armour units (stones) over the entire flume width B_{flume} after every test (1000 waves).
- D_{n50} = nominal diameter of the armour unit
- A_{stone} = cross area of the armour unit
- A_{ref} = Reference area for counting the displaced armour units ($A_{ref} = h_B/\sin\alpha)D_{n50}$ with h_B = height of armoured slope)

The stability of the armour units on both seaward and landward slope was comparatively analysed for the conventional breakwater and the geocore breakwater.

4.1 Stability of Seaward Armour Layer

Four methods are used to analyse the hydraulic stability, including the two methods proposed by Van der Meer (1988), the experimental model proposed by Powell and Allsop (1985) and a power model (Oumeraci et al, 2007). All models show that a higher damage expectedly results for the geocore breakwater and that the damage curves are generally almost self-similar for both tested breakwater types. In addition, the Hudson-formula was also used in order to determine the k_D-value for the geocore breakwater as a compared to its conventional counterparts.

Among the existing methods, the following exponential damage model of Powell and Allsop (1985) provides the best fit:

$$D = \frac{N_{tot}}{N_a} = a \cdot \exp\left(b \cdot N_S^*\right) \tag{14}$$

with:

a = 5.38 and b = 1.2131 for the conventional breakwater

a = 3.70 and b = 1.962 for the geocore breakwater

In Eq. (14) the modified stability number N_S^* is defined as follows:

$$N_S^* = \left(s_m\right)^{-1/3} \frac{H_{m0}}{\left(\frac{\rho_S}{\rho_W} - 1\right) \cdot D_{n50}} \tag{15}$$

where

s_m = wave steepness using the characteristic wave period $T_{m-1.0}$ for the calculation of the deepwater wave length.

However, the damage curves described by the exponential model in Eq. (14) show a steeper increase of the damage D for larger N_S*-value than the observed data. Moreover, the observed damage values in the lower N_S*-range are significantly higher than the predicted curves. Therefore, the following power model was adopted which provides a better fit than the exponential model in Eq. (14):

$$D = \frac{N_{od}}{N_a} = a \cdot \left(N_S^*\right)^b \tag{16}$$

With the background that most of the design application in the engineering practice is generally based on a damage level D ≈ 5%, the parameter a and b in Eq. (16) have been determined (Fig. 12):

a = 5.5 and b = 4.6 for the conventional breakwater ($R^2 = 0.78$)

a = 5.5 and b = 5.2 for the geocore breakwater ($R^2 = 0.72$)

Although the slope angle α was not varied in the model tests, it is expected to affect damage D as follows:

$$D = \frac{a'}{(\cot\alpha)^c} \cdot \left(N_S^*\right)^{b'} \tag{17}$$

where the parameter c describes the increase rate of the damage with steeper slope. The modified stability number N_s* is

$$N_S^* = \left[\frac{D}{a}\left(\cot\alpha\right)^c\right]^{1/b'} \tag{18}$$

Using Hudson's formula, the stability number N_S is defined as:

$$N_S = \frac{H_S}{\left(\frac{\rho_S}{\rho_W} - 1\right) \cdot D_{n50}} = k_D \cdot \cot\alpha \tag{19}$$

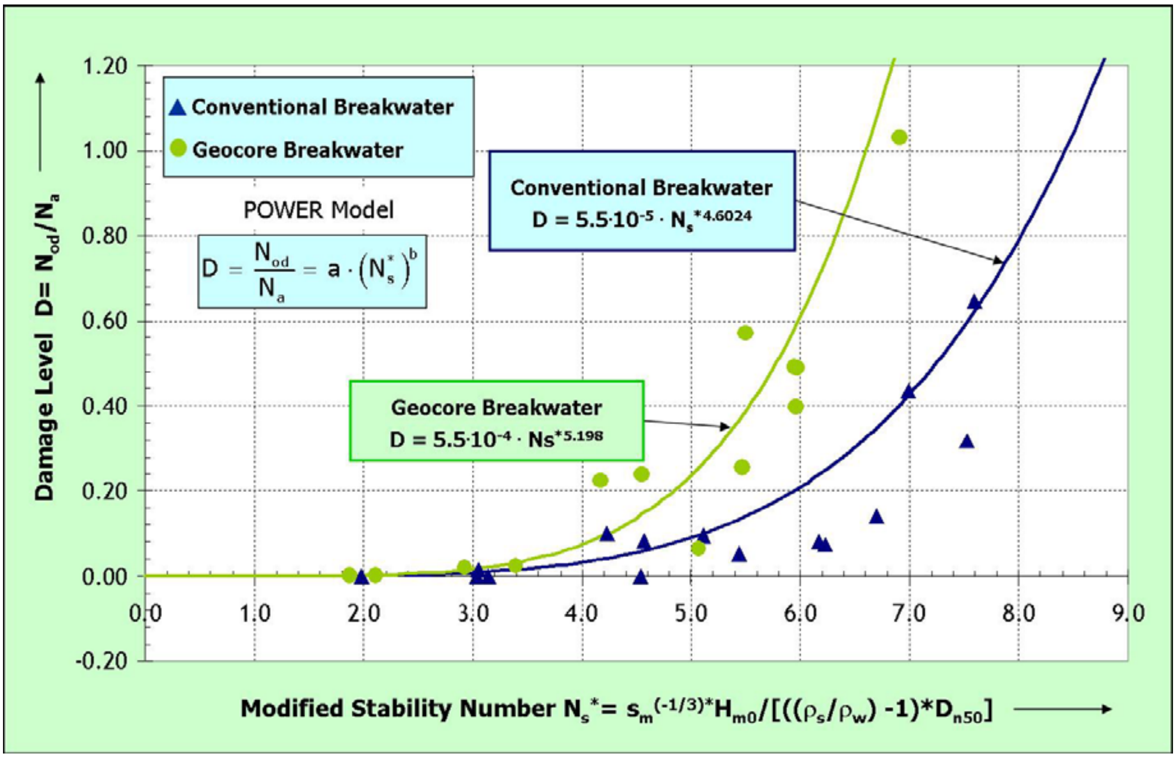


Figure 12. Damage Level D for the seaward armour layer of the conventional and geocore breakwater

Using the modified stability number N_S^* according to Eq. (18) instead of N_S, the k_D-parameter is obtained from Eq. (19):

$$k_D = \frac{s_m}{\cot\alpha} \cdot \left(N_S^*\right)^3 \tag{20}$$

Substituting N_S^* from Eq. (18) into Eq. (20) yields the k_D-parameter:

$$k_D = s_m \left(\cot\alpha\right)^{\left(\frac{3c}{b}-1\right)} \cdot \left(\frac{D}{a'}\right)^{3/b'} \tag{21}$$

Systematic hydraulic model tests by varying the slope angle α, the permeability of the structure and wave steepness s_m will allow to determine the k_D-parameter in Hudson's formula as a function of the damage level D.

Applying Eq. (21) for the two breakwater types and the wave conditions tested, the following k_D-values are obtained for D = 5% (Oumeraci et al, 2007).

- k_D = 1.90 for the conventional breakwater
- k_D = 1.24 for the geocore breakwater

This suggests that the required weight of the armour unit for the geocore breakwater is more than 1.5 times larger than that of a conventional breakwater.

This is in line with the results available in the literature suggesting a difference of 50-60% in terms of the required weight of the armour units for a difference in the core permeability of the same range as in this study (e.g. Burcharth et al, 1998).

4.2 Stability of the Shoreward Armour Layer

Noticeable damage of the armour units in the seaward slope starts to occur only when the relative freeboard reaches or exceed a value of 1.5 ($R_c/H_{m0} \geq 1.5$). The rear side armour stability increases with increasing relative freeboard, and thus with decreasing overtopping discharge, while the front armour stability generally increases (without parapet wall). Most of the published results are rather project specific, so that generic models are not yet available to predict the hydraulic stability of the rear side armour. A brief review of the available prediction formulae is given by Oumeraci et al (2007).

Among the most valuable experimental results available in the literature, those published by Jensen (1984) show the closest similarities to the results obtained in this study for both breakwater types tested. However, Jensen failed to provide any prediction formula for the damage level D as a function of the relative freeboard. Based on the results of the experiments performed for both types of breakwater, the following prediction model is determined (Fig. 13):

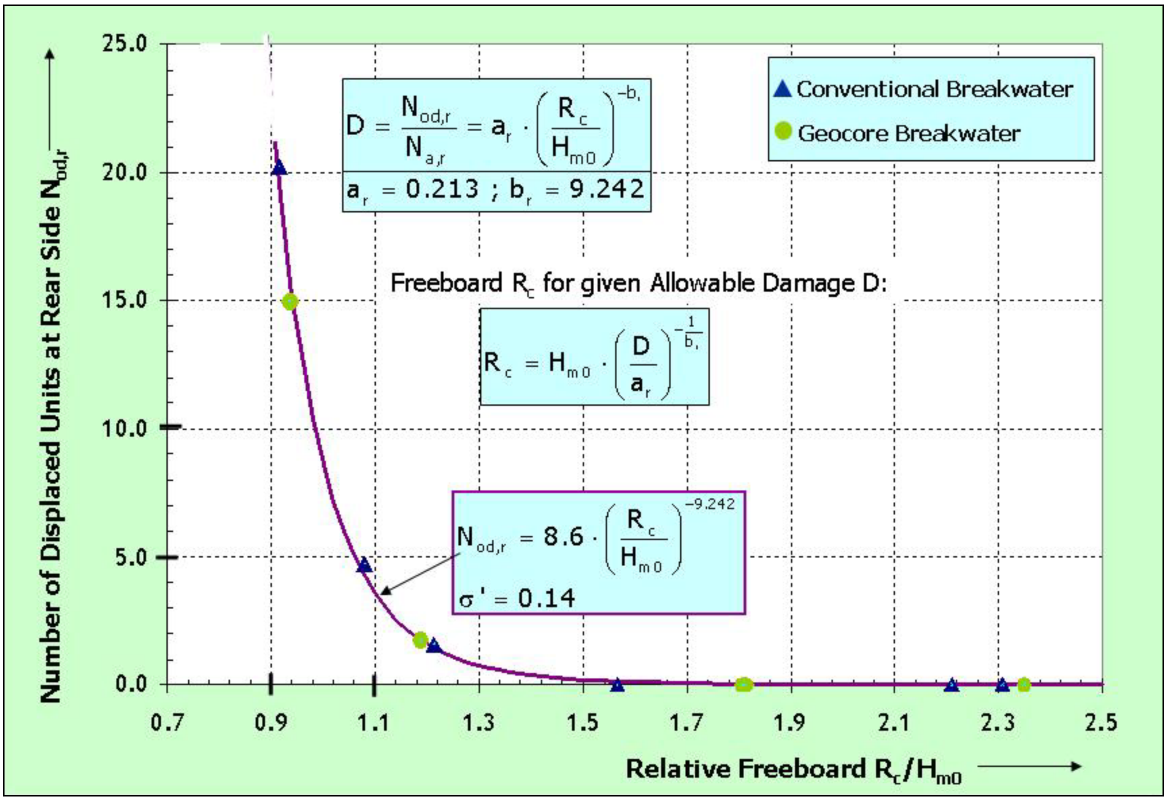


Figure 13. Stability of Rear Armour for the Conventional Breakwater and the Geocore Breakwater

$$N_{od} = a \cdot \left(\frac{R_c}{H_{m0}}\right)^{-b} \tag{22}$$

with a = 8.6 and b = 9.242 (CoV = 11.4%).

This shows that no significant difference is obtained between the geocore breakwater and the conventional breakwater. It is rather surprising in view of the differences in terms of wave overtopping. Defining the damage level D (see Eq. 13):

$$D = \frac{N_{od,r}}{N_{a,r}} \tag{23}$$

where $N_{a,r} = N_a$ due to the same slope on the seaward and shoreward side. Eq. (23) can then be rewritten as:

$$D = a_r \cdot \left(\frac{R_c}{H_{m0}} \right)^{-b_r} \tag{24}$$

where a_r and b_r are given (Fig. 13). Hence, the required relative freeboard R_c/H_{m0} can readily be calculated as a function of the allowable damage level D:

$$\frac{R_c}{H_{m0}} = \left(\frac{D}{a_r} \right)^{-\frac{1}{b_r}} \tag{25}$$

5 SUMMARY, CONCLUDING REMARKS AND OUTLOOK

A rubble mound breakwater with a conventional core made of quarry run (conventional breakwater) and its similar counterpart with a core made of geotextile sand containers (geocore breakwater) has been simultaneously tested under the same incident wave conditions in the LWI twin-wave flumes to study the differences in terms of hydraulic permeability, hydraulic performance and hydraulic stability.

The permeability of the geocore breakwater was found to be 14 times less permeable than that of the conventional breakwater with a quarry run core.

No significant difference could be observed in terms of wave reflection performance. For both breakwaters, the surf similarity parameter surprisingly failed to describe the wave reflection. A much better description was achieved by using the relative depth parameter k_0d with $k_0 = 2\pi/L_0$ proposed by Oumeraci and Muttray (2001).

As expected, the difference in terms of wave transmission is essentially determined by the wave steepness and the relative freeboard. Therefore, a modified relative freeboard, including both parameters as proposed by Allsop (1983), was found to be most appropriate to describe wave transmission. The full potential of the geocore breakwater will particularly emerge when used for the protection against long waves.

For surf similarity parameter $\xi_{mo} > 3$, which represents the values of interest for the design of rubble mound structures, the geocore breakwater is associated with a 20% higher run-up. In this practical range, the simple linear model proposed by Van de Walle (2003) is found to be the most appropriate to describe wave run-up.

The difference between the two breakwater types in terms of wave overtopping strongly depends on the relative freeboard. For common design freeboard ($R_c/H_{m0} < 1.5$) the difference is less than expected. The wave overtopping model of TAW (2002), originally derived for sea dikes, with corresponding correction factors ($\gamma_f = 0.52$ for conventional breakwater and $\gamma_f = 0.60$ for geocore breakwater) was found to be the most appropriate.

Regarding the difference in terms of the seaward armour stability, 50-70% larger armour units are required for the geocore breakwater with an allowable damage level D = 5%. This is in the range of the published results on the effect of the core permeability. Very surprising, however, is that no significant difference in terms of the armour stability of the rear slope is observed, even for a relative freeboard larger than 1.5.

Despite the case study character of the experimental investigations performed in this project, the results have shown that the geocore solution may indeed represent a feasible alternative with a wide application potential, especially in areas where rock is not available in large quantities and at low costs as well as in areas where the protection against long waves is a major issue.

Moreover, the geocore concept should be extended for other classes of structures such as seawalls, artificial reefs, groins etc. The advantages of such a solution are expected to be particularly revealed in the case of reclaimed land protected by seawalls (Fig. 1b).

The results have also revealed that the research results yet available about the effect of the core permeability on the hydraulic performance and the hydraulic stability is still not conclusive, particularly with respect to the stability of the rear slope, wave reflection and wave overtopping.

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KEYWORDS

rubble mound breakwater
core permeability effect
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armour stability
wave reflection
wave run-up
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wave transmission