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Wave Transmission over Submerged Breakwaters: Performance of Formulae and Models

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ABSTRACT

The wave transmission over submerged breakwaters is investigated using existing formulae and wave models. The objective is to assess their performance and pinpoint research paths for their improvement. Application was made on a case study with two submerged detached breakwaters. It was found that some of the recent relations give satisfactory results of the transmission coefficient, while the predictability of the models tested depends on the wave breaking formulation assumed. In general, wave breaking and porosity of the structure are the most crucial factors that need further study for the improvement of the prediction of wave transmission over submerged breakwaters.

KEY WORDS: Wave transmission; breakwaters; submerged breakwaters; wave models; wave breaking

INTRODUCTION

Coastal protection has always been a field of challenge to engineers due to the complexity of the physical processes involved. In modern times the issue becomes even more complicated, since other non-physical parameters are introduced during the conceptual design of a coastal project. Such considerations may include the environmental and in particular, the aesthetic value of the nearshore landscape. Thus, new forms of the conventional structures are being tested along with new approaches to coastal protection employing mild-type structures. In this framework it is not wonder that low-crested structures and in particular submerged breakwaters, a modified version of the traditional detached breakwater, are increasingly used in projects aiming primarily at combating coastal erosion. The protection afforded by submerged breakwaters to their lee controls the nearshore wave pattern, the sediment movements and finally the morphology of the coastal zone. A prime measure of this protection is offered by the wave transmission over such structures. The commonly used wave transmission coefficient provides the anticipated decrease of a characteristic wave height due to the presence of the submerged breakwater. As expected, the main parameter that affects the transmission coefficient is the freeboard, i.e. the distance between the sea free surface and the crest of the structure. Various semi-empirical formulae for estimating this coefficient are presented in the following section. These are based on data produced during experiments configured usually within a small range of geometric and environmental parameters. At another level, several nearshore wave models, either commercial or academic, were developed in the recent past. Several of these models provide acceptable results as far as wave transmission, reflection, refraction and diffraction is concerned in the vicinity of submerged structures. Some of the widely used models are presented in the relevant section, along with limitations, that reflect the underlying approximations, e.g. the way energy dissipation due to wave breaking is accounted for. This latter process seems to possess a central role in the performance of the wave models with regard to the transmission coefficient.

The present paper describes research aiming at evaluating the semi-empirical formulae and assessing the performance of the wave models, by comparing them against the former. In order to investigate, as far as possible, the physics behind this evaluation, wave models were employed that were based on different governing equations. Thus a Boussinesq-based model, a parabolic mild-slope equation and a nearshore spectral waves model (MIKE 21, 2005) were tested. A case study is also presented, where the above analysis is applied. The project comprises two detached submerged breakwaters located along the mouth of a man-made lagoon in order to protect light structures on the shore. The remaining sections are devoted to the presentation of results, their discussion and conclusions.

WAVE TRANSMISSION FORMULAE

A number of laboratory investigations were conducted in the past to quantify the transmission coefficient, defined by:

$$K_t = H_t / H_i \tag{1}$$

where, H_t, H_i measures of the transmitted, incident waves, respectively.

These investigations produced empirical formulae that have been used widely in engineering applications. However, there are limitations to each one of these due to the laboratory conditions and range of input quantities used in the tests. The physical variables that control in one way or another the transmission coefficient are (Fig.1):

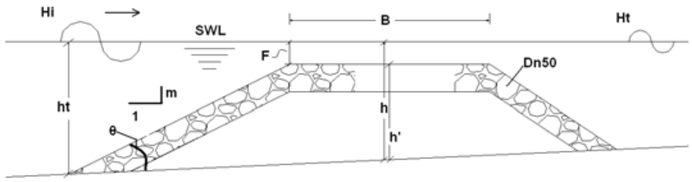

Figure 1. Problem definition

B: crest width of breakwater
F: freeboard (=h-h')
h: water depth (at the axis of the structure)
h': height of structure (at its axis)
h_t: water depth at the (seaward) toe of the structure
m: front slope of the breakwater face (=tanθ)
D_{n50}: nominal rock diameter of armour layer ($=(M_{n50}/\rho_\alpha)^{1/3}$)
H_i: incident wave height (H_{si} or H_{moi}) at the toe of the structure
L: local wavelength
T_p, L_p: period, wavelength at spectral peak
ξ_p: surf-similarity parameter ($=m/\sqrt{S_p}$)
S_p: wave steepness ($=H_i/L_p$)

As mentioned earlier, the problem of wave transmission behind a submerged breakwater can be regarded as a special case of low-crested structures, where the breakwater crest may lie above the still water level but close enough to it. Several experimental investigations were performed in the past that led to semi-empirical expressions for the transmission coefficient of random waves behind low-crested structures (Allsop, 1983; Daemrich and Kahle, 1985; Ahrens, 1987; VdMeer, 1988). Van der Meer (1990) analysed further the results of these efforts and proposed a simple prediction formula, where K_t depends linearly on F/H_{si}. Daemen (1991) made a similar analysis of the data sets, and later on the two approaches were combined to give the following formula (VdMeer and Daemen, 1994):

$$K_t = -\mathrm{a}F/D_{n50} + \mathrm{b}\ , \quad 0.075 \le K_t \le 0.75 \tag{2}$$

where, $\mathrm{a} = 0.031H_i/D_{n50} - 0.024$

$\mathrm{b} = -5.42S_{op} + 0.0323H_i/D_{n50} - 0.017(B/D_{n50})^{1.84} + 0.51$
for conventional breakwater

$\mathrm{b} = -2.6S_{op} - 0.05H_i/D_{n50} + 0.85$, for reef-type breakwater

Expression (2) is valid for $1< H_i/D_{n50}< 6$ and $0.01< S_{op}<0.05$ and S_{op} refers to offshore conditions. The term reef-type breakwater denotes a shallow structure made of a single layer of rock material.

In the recent edition of the Coastal Engineering Manual (CEM, 2004) the formula by VdMeer and d'Angremond (1991) has been adopted for preliminary calculations of the transmission coefficient. Graphs were produced giving directly the K_t values. These are based on a slight modification of the following simple prediction formula (VdMeer, 1990) derived after analysis of hydraulic model tests by Seelig (1980), Powell and Allsop (1985), Daemrich and Kahle (1985), Ahrens (1987) and VdMeer (1988):

$$\left.\begin{aligned} &K_t = 0.8\ , && \text{for } 1.13<F/H_i<2.0 \\ &K_t = 0.46 + 0.3F/H_i\ , && \text{for } -1.2<F/H_i<1.13 \\ &K_t = 0.1\ , && \text{for } -2.0<F/H_i<-1.2 \end{aligned}\right\} \tag{3}$$

This formula gives a linear dependence of K_t to the relative crest freeboard, while it does not take into account crest width effects.

Following these efforts another wave transmission formula appeared for emerged and submerged structures in the range $-2.5<F/H_{si}<2.5$, d'Angremond et al. (1996):

$$K_t = 0.4F/H_i + 0.64(B/H_i)^{-0.31}(1-\mathrm{e}^{-0.5\xi})\ , \ 0.075 \le K_t \le 0.8 \tag{4}$$

valid for $B/H_i<10$

The previous formula was extended by Briganti et al. (2003) to cover crest widths $B/H_i>10$. The revised formula reads:

$$K_t = 0.35F/H_i + 0.51(B/H_i)^{-0.65}(1-\mathrm{e}^{-0.41\xi}) \tag{5}$$

with range of validity $0.05 \le K_t \le 0.93-0.006B/H_i$

Seabrook and Hall (1998) used results from physical model tests with submerged breakwaters, where various values of freeboard, crest width, water depth and incident wave conditions were applied. Their formula reads:

$$K_t = 1 - \exp(-0.65F/H_i - 1.09H_i/B) + 0.047BF/LD_{n50} - \\ -0.067H_iF/BD_{n50} \tag{6}$$

valid for $0 \le BF/LD_{n50} \le 7.08$ and $0 \le FH_i/BD_{n50} \le 2.14$

More recently, several new formulae were suggested.

Friebel and Harris (2003) developed a "best fit" empirical model based on data sets provided by Seelig (1980), Daemrich and Kahle (1985), VdMeer (1988), Daemen (1991) and Seabrook (1997). Their study confirmed that the transmission coefficient is highly dependent on the non-dimensional freeboard F/H_{si}. To a lesser degree, K_t depends also on the relative crest width B/L or B/h', on the relative structure emergence above sea bed 1-F/h', as well as on the ratio F/B. The proposed formula is:

$$K_t = -0.4969\exp(-F/H_i) - 0.0292B/h_t - 0.4257h'/h_t \\ -0.0696\ln(B/L) - 0.1359F/B + 1.0905 \tag{7}$$

Furthermore, a prediction formula for K_t was developed by using statistical analysis methods (Siladharma and Hall, 2003) applied on experimental results of wave transmission over 3-D submerged breakwaters. The formula given below, was produced after excluding the diffraction term coping with 3-D effects, in order to be able to compare it with other formulae dealing with 2-D configurations:

$$K_t = -0.869\exp(-F/H_i) + 1.049\exp(-0.003B/H_i) - \\ -0.026FH_i/BD_{n50} - 0.005B^2/LD_{n50} \tag{8}$$

It can be seen in Eq. 8 that again the main factor controlling the wave transmission is the relative freeboard F/Hi where $H_i=H_{si}$. Other parameters playing a role in shaping the final value of K_t include the relative crest width B/H_i, the roughness parameter F/D_{n50} as well as an "internal flow parameter" B^2/LD_{n50} where the local wavelength is also taken into account. Calabrese et al. (2003) found that the formula of d' Angremond et al. (1996) gives reliable estimates of the transmission coefficient, thus they upgraded it in order to enhance the dependence of K_t on the breaker index H_i/h and to non-dimensionalise the freeboard F with respect to the crest width B rather than to the incident wave height

H_i. The above presented formulae will be used in the following to perform evaluation and comparisons with wave model results.

WAVE MODELS

MIKE 21 Model

MIKE 21 is a modelling system whereby wave calculations can be carried out (DHI, 2005). Three modules can be employed to perform wave simulations, namely the nearshore spectral wind-wave module (NSW), the parabolic mild-slope equation module (PMS), and the Boussinesq wave module (BW). All above modules were used in this study in one (PMS, BW) or two (NSW, PMS) horizontal dimensions.

The energy dissipation taken into account by these models refers to wave breaking and to bottom dissipation. Dissipation due to breaking refers mainly to wave breaking due to depth limitation as described by the approach of Battjes and Janssen (BJ, 1978) for NSW and PMS modules. Bottom friction is assumed as proposed by Dingemans (1983) for random waves. It is noted that dissipation due to percolation through permeable structures, such as rubble mounds, is not included. This inevitably introduces some error, that may become significant for large values of K_t.

Nearshore Spectral Wind-Wave Module

The governing equations in the model are derived from the conservation law of the spectral wave action density. A parameterization of the latter is performed in the frequency domain by introducing the first two moments of the wave action spectrum as dependent variables. The resulting coupled partial differential equations include the components in the x- and y- directions of the group velocity, as well as a propagation speed representing the change of action in the direction of wave propagation. These propagation speeds are obtained by using linear wave theory. In the NSW formulation the effects of refraction and shoaling are taken into account, while in the source terms the effects of local wind-wave generation and energy dissipation due to wave breaking and bottom friction are included. The effect of current can also be accommodated in the governing equations. However, phase averaged models, such as the NSW used in this study, are not able to describe wave reflection from a submerged structure, introducing thus an additional error. The basic equations, the description of the source terms and to some extent the numerical solution method in NSW are based on the approach proposed by Holthuijsen et al. (1989). The source terms regarding the local wind wave generation are derived from empirical growth relations after Johnson (1998).

Parabolic Mild-Slope Equation Module

This module is based on a parabolic approximation to the elliptic mild-slope equation. This latter equation describes the refraction, shoaling, diffraction and reflection of linear time-harmonic waves on a gently sloping seabed (Berkhoff, 1972). The parabolic approximation adopted is obtained by assuming a predominant wave direction and neglecting back-scatter and diffraction along this direction. Its simplest expression is valid for waves propagating along a predominant direction or within a small angle to it. Kirby (1986), by using Padé approximants, extended its validity to the case of waves propagating at a large angle to the main wave direction. This modified equation is used in PMS module. For given significant wave height, peak wave period, and mean wave direction it is possible to use MIKE 21 Toolbox to obtain the distribution of energy over discrete frequency and direction bands, since in general the wave energy is a function of frequency and direction. This distribution would be specified at the offshore boundary of the model. In the numerical calculation of the wave agitation over the study area, each of the discrete energy components is transformed independently by PMS and the results are linearly superimposed at any inshore grid point.

Boussinesq Wave Module

The BW module is based on time domain formulations of Boussinesq type equations that include nonlinearity as well as frequency dispersion. The latter is introduced in the momentum equations by taking into account the effect that vertical accelerations have on the pressure distribution. The original equations are modified using a flux-formulation with improved linear dispersion characteristics. These enhanced Boussinesq type equations (Madsen et al., 1991; Madsen and Sørensen, 1992) allow simulation of the propagation of directional wave trains up to relative wave numbers $kh \approx 3.1$, whereas the corresponding maximum value applicable to the classical Boussinesq equations (Peregrine, 1967) is $kh \approx 1.4$. The model equations in BW have been extended to take into account wave breaking as described in Madsen et al. (1997). The 1DH BW module used in the present study solves the governing equations by a standard Galerkin finite element method with mixed interpolation. The problem of the presence of higher-order spatial derivatives is treated by writing the Boussinesq type equations to a lower order after introducing an auxiliary variable and an auxiliary algebraic equation. The resulting equations contain only terms with second order derivatives with respect to the spatial co-ordinates (Sørensen et al., 2004).

Energy Dissipation due to Wave Breaking

Basic bore-type formulation

Energy dissipation due to wave breaking is the dominant factor for correctly tuning wave propagation models in shallow waters. Hence, the information relevant to the model applications performed in this investigation is put together in the following. The basic formulation due to BJ expresses the energy dissipation rate by the bore-type relation:

$$E_d = -\frac{\alpha}{4} Q_b f_m H_{\max}^2 \tag{9}$$

where, $\dfrac{1-Q_b}{\ln Q_b} = -(H_{rms} / H_{\max})^2$

$H_{\max} = \gamma_1 k^{-1} \tanh(\gamma_2 kh / \gamma_1)$

$H_{rms} = (8E)^{1/2}$

f_m is the energy averaged mean wave frequency

k is the wave number

h is the water depth

E is the total wave energy

In the above expressions, α controls the rate of energy dissipation, Q_b is the percentage of breaking waves in a Rayleigh distributed wave train, H_{max} is the maximum wave height before breaking, γ_1 is a steepness related breaking index, γ_2 is a depth related breaking factor. By increasing γ_1 the steepness related breaking is reduced. For monochromatic waves the fraction Q_b is taken 0 or 1 for non-breaking or breaking waves, respectively. The above basic formulation is applicable to both NSW and PMS modules, with the following values for the three breaking constants:

$\alpha=1.0$, $\gamma_1=1.0$, $\gamma_2=0.8$

The value for γ_1 was suggested by Holthuijsen et al. (1989), while the other two by BJ.

Improvements on the basic formulation

All efforts for improving the basic formulation of the energy decay due to wave breaking refer to the treatment of the three breaking parameters α, γ_1, γ_2 specified previously. The first effort was made by Battjes and Stive (BS, 1985), who specified γ_2 as a function of deep water wave parameters. By calibrating the dissipation model against measurements they obtained (by assuming α=1.0, γ_1=0.88):

$$\gamma_2 = 0.5 + 0.4\tanh(33S_o) \tag{10}$$

where, S_o is the deep water wave steepness (=H_{rmso}/L_{op})

$H_{rmso} = H_{mo}/\sqrt{2}$

L_{op} is the deep water wavelength based on peak frequency

Later, Nelson (1987) suggested a dependence of depth related breaking on the local bed slope according to the relation:

$$\gamma_2 = 0.55 + 0.88\exp(-0.012/\tan\theta) \tag{11}$$

where, $\tan\theta$ is the bed slope ($\geq$0).

The above expressions hold for wave breaking on a beach. For wave breaking over submerged structures with very steep slopes followed by a horizontal berm, incipient breaking as described above is not expected to be accurate. Recent experiments by Johnson (2006) allowed calibration of γ_2 for waves propagating over submerged structures with freeboard:

$$\left.\begin{array}{ll} \gamma_2=1.55 \;, & \text{for } F/H_{mo}\leq 0.5 \\ \gamma_2=1.91-0.72F/H_{mo} \;, & \text{for } 0.5<F/H_{mo}<1.5 \\ \gamma_2=0.8 \;, & \text{for } F/H_{mo}\geq 1.5 \end{array}\right\} \tag{12}$$

These expressions were also used in this study by applying them "externally" to the wave modules NSW and PMS. As noted above, γ_2 caters for the depth-controlled wave breaking. The other part of wave breaking, i.e. that related to excessive wave steepness, is controlled by the factor γ_1. Johnson (2006) proposed an improved expression for the steepness-induced breaking based on integrating over all frequencies and directions the rate of energy dissipation due to whitecapping (Komen et al., 1994).

Surface roller concept

In BW module a different wave breaking concept has been used, called the surface roller concept. In this approach incipient wave breaking occurs if the slope of the water surface exceeds a certain amount, whereby the geometry of the surface roller is determined. The roller is considered as a mass of water not taking part in the wave motion, but carried along with the wave celerity. The influence of the roller is taken into account through an additional convective momentum term arising from the non-uniform vertical distribution of the horizontal velocity (Madsen et al., 1997). In BW it is assumed that incipient breaking occurs when the local slope of the free surface exceeds 20°. Various shape, celerity and period factors are set depending on the type of breaker. If wave breaking and moving shoreline are included in the simulation, then an explicit numerical lowpass filter has to be specified. This is introduced in order to remove high frequency instabilities during uprush and downrush and to dissipate wave energy wherever the surface roller cannot be resolved.

Energy Dissipation due to Bed Friction

The rate of energy dissipation due to bottom friction is formulated in MIKE 21 models by using the quadratic friction law to express bottom shear stress. For monochromatic waves the rate of energy dissipation E_b is calculated by the following relation proposed by Putnam and Johnson (1949):

$$E_b = -\frac{1}{6\pi}\frac{c_{fw}}{g}\left(\frac{\omega H}{\sinh kh}\right)^3 \tag{13}$$

where, c_{fw} is a wave friction coefficient; H is the wave height; $\omega$is the circular frequency

An extension of the above relation due to Dingemans (1983) is applicable to the case of unidirectional Rayleigh-distributed random waves:

$$E_b = -\frac{1}{8\sqrt{\pi}}\frac{c_{fw}}{g}\left(\frac{\omega H_{rms}}{\sinh kh}\right)^3 \tag{14}$$

where, h is the local water depth in both expressions.

Inclusion of directional distribution of wave energy and influence of currents is effected in both NSW and PMS modules through the extension proposed by Holthuijsen et al. (1989).

The friction factor in the presence of waves c_{fw}, can be calculated through the empirical expression c_{fw}=f_w /2, and the following relation (Svendsen and Jonsson, 1980):

$$f_w = \begin{cases} 0.24 \;, & \text{for } a_b/k_n<2 \\ \exp\left\{-5.977+5.213(a_b/k_n)^{-0.194}\right\} \;, & \text{for } a_b/k_n\geq 2 \end{cases} \tag{15}$$

where, k_n is the Nikuradse roughness parameter; a_b is the water particle amplitude at the bottom

The roughness parameter is difficult to determine. In cases with no bed forms it can be estimated by k_n=2.5d_{50}, where d_{50} is the median grain size of the bottom sediments (Nielsen, 1979).

In simulations of short waves in ports and harbours, where BW module is normally used, the effect of bottom friction is relatively unimportant and it can be neglected. For modelling long wave transformations the bottom friction formulation follows the Chézy bed friction law. According to this, the shear stress τ_b at the bed can be expressed in terms of the Chézy number C by:

$$\tau_b = \rho g U|U|/C^2 \tag{16}$$

where U is the depth-averaged velocity , $C=\frac{U}{U_b}\left(\frac{2g}{f_w}\right)^{1/2}$

APPLICATION TO A CASE STUDY

Main Features of the Study Area

The project under study is developed around a focal water expanse comprising a man-made lagoon, occupying an area of about 6.2

hectares on the shores of the Red Sea. It will be used mainly for swimming and related activities. Figure 2 shows the general layout of the Lagoon, containing two submerged breakwaters, the principal role of which is the protection from wave agitation of the bungalows to be built on piles at the shore.

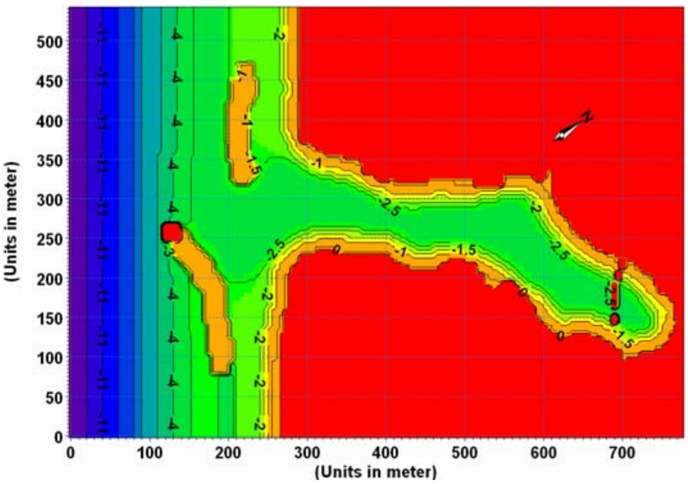

Figure 2. Lagoon reference plan

Input Conditions

Astronomical tides in the area are of the mixed semi-diurnal type. The main input tidal levels considered, were as follows:

Mean Sea Level (MSL)	±0.00
Highest Astronomical Tide (HAT)	+0.80 m
Lowest Astronomical Tide (LAT)	–0.70 m

The site is exposed to waves coming from directions within a small angle sector, from 195° to 230°. The narrow and elongated shape of the shoreline restricts waves from developing fully. The wave data adopted as input to the wave models were:

Deepwater 10-yr: H_s= 2.11 m , T_s=5.8 s , T_p=6.1 s
Deepwater 50-yr: H_s= 2.95 m , T_s=6.8 s , T_p=7.14 s

The above values refer to a water depth of 50 m. In order to obtain the corresponding values at the boundary of the wave model, wave transformations should be taken into account especially those related to refraction and shoaling. Application of the above transformations yields the following wave characteristics at the offshore model boundary, i.e. at a water depth of 15 m:

Return period 10-yr : H_s=1.99 m , T_p=6.1 s
Return period 50-yr : H_s=2.72 m , T_p=7.14 s

For the calculation of H_{si} standard Jonswap and TMA spectra were used whereas linear transformations of both sinusoidal and 5th order Stokes waves was used for the calculation of H_{maxi}. In numerical simulations the calculation of K_t was based on off-shore wave height at the toe of the structure on a typical cross-section of the southern breakwater at the middle of its length.

Since no reliable data on storm surge in the area are available, a rough calculation was performed based on information of wind speed and bathymetry offshore the studied site (Dean and Dalrymple, 1984). The input value taken for storm surge was 0.35 m.

Sea level rises as ocean temperature does. During the past century the global mean sea level rose by a value between 10cm and 20cm. The rate of level rise is expected to be accelerated due to increased CO_2 emissions in the atmosphere. Following the median scenario adopted by the Intergovernmental Panel for Climate Change, a central estimate of the sea level rise was deduced of the order of 0.20m. This value was taken as input to the models.

The input data for bed friction energy dissipation are:

Breakwater area: k_N=0.0125 m , d_{50}=0.005 m
Remaining area : k_N=0.0003 m , d_{50}=0.00012 m

It has to be noted that since this project is not yet materialized, no calibration of the model could be performed. However, a further stage will involve physical modeling of the submerged breakwaters and some model calibration should be feasible. In order to accommodate the resulting uncertainties and be on the safe side at this stage a conservative low value for the bed friction at the breakwater area was adopted as above. An additional point for this selection was that no decision has been reached yet regarding the construction material of the breakwaters. In case these would be made of natural rock, the friction coefficient could be estimated by the relevant formula of Madsen and White (1975).

RESULTS AND DISCUSSION

Application of the previously mentioned input conditions to the case study under consideration gave the K_t values presented in Figs. 3 and 4, as obtained by the formulae and models respectively. These figures refer to various wave conditions for which the corresponding transmission coefficient is given. The wave conditions are decoded as follows:

wave condition #1: 10yr H_s through linear transformation from deep to shallow water
wave condition #2: 10yr H_{max} through linear transformation from deep to shallow water (coincides with TMA-spectrum transformation and breaker index 0.8)
wave condition #3: 50yr H_s through linear transformation from deep to shallow water
wave condition #4: 50yr H_{max} through linear transformation from deep to shallow water (coincides with TMA-spectrum transformation and breaker index 0.8)

The wave transmission formulae are those presented previously with the following remarks. The expression of d'Angremond et al. in Fig. 3 includes its extension due to Briganti et al. (2003) to cover wide crest widths (Eq.5). The formulae of VdMeer and Daemen, of Seabrook and Hall and of Siladharma and Hall involve the nominal diameter D_{n50} of the armour layer of the breakwater. This is calculated through the relevant expression due to VdMeer and Pilarczyk (1991). This latter relation takes into account the local water depth. In the graph of Fig. 3 the results associated with the above formulae were obtained for water level at the lowest astronomical tide prevailing in the study area.

It can be seen from the graph of Fig. 3 that for all four wave conditions the eight formulae give results that behave in a more or less consistent manner. Indeed, a "central" part of the results is formed by excluding the formulae of CEM and VdMeer and Daemen. The CEM gives under any conditions higher K_t values by as much as 50% than the average of the values of the "central" part. Also, VdMeer and Daemen and Calabrese et al. underestimate for three wave conditions the K_t value. The underestimation by VdMeer and Daemen for 2 out of 4 wave conditions is of the same order of magnitude with the overestimation by CEM. These initial findings are in accord with the fact that both formulae resulted from the first efforts to address the problem by involving only a few simple parameters, e.g. CEM's expression for K_t is based only on the ratio F/H_i (Eq.3), without taking into account other important factors such as the crest width, the water depth, etc. The results by d'Angremond et al. are regarded to behave favourably enough, partly due to the fact that their formula includes the surf-similarity parameter having to do with the wave breaking mode. This is confirmed by others, as e.g. by Calabrese et al. (2003), Daemrich et al. (2001), Mai et al. (1999). Siladharma and Hall's relation behaves relatively smoothly for the wave conditions tested and it involves the diameter D_{n50}, a fact that may include indirectly some effects of the structure porosity. This relation is actually an improvement of the older formula by Seabrook and Hall. The results of a single formula closely located mid-way between the two extremes under any wave condition tested are those of Friebel and Harris. The good behaviour of this formula is confirmed through comparison with experiments by Penchev (2005). Based on the previous discussion the formulae retained for further comparison with the wave models are those of d'Angremond et al., Siladharma and Hall, Friebel and Harris.

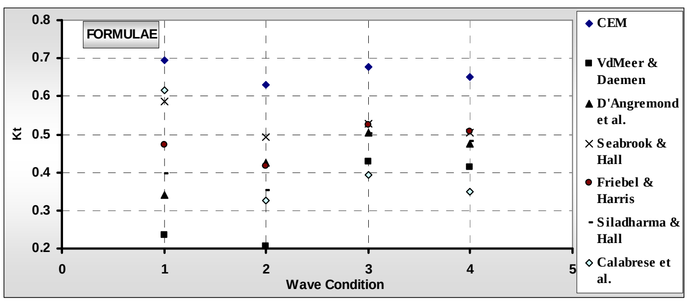

Figure 3. Transmission coefficient K_t by Formulae

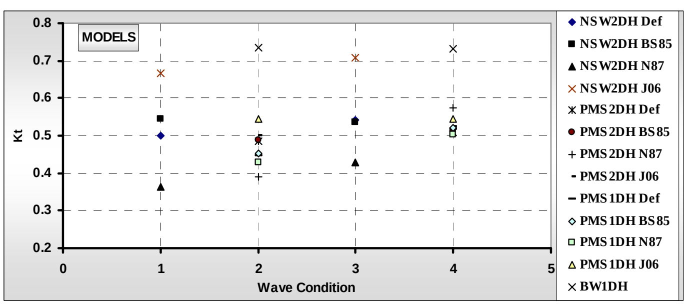

Figure 4. Transmission coefficient K_t by Numerical Models

Figure 4 presents the results obtained by the models tested under the same as above four wave conditions. As mentioned in a previous section three different modules were tested: NSW, PMS, BW. Module PMS was used under both one-dimensional and two-dimensional options. Each of those options as well as NSW module were run for four different representations of wave breaking, namely those of Battjes and Janssen (1978) with the default values of α, γ_1, γ_2 used in MIKE 21; the formulation by BS; and those of Nelson (1987) and of Johnson (2006). All these formulations are presented in the text accompanying Eqs. 9~12. Thus 13 models were used in total for comparison. It has to be noted here that there are some differences in the incident wave conditions at the breakwaters due to the slightly different wave transformation procedures adopted by each model. Calculations performed for the incident wave height at the structure toe showed larger deviations for the breaking formulation by Johnson (2006), that appears to underestimate wave breaking. It is evident that the way the process of wave breaking is taken into account plays a significant role in the final value of K_t produced by the model. On the other end the wave breaking formulation by BJ is found to somehow overestimate the amount of wave breaking. This has been confirmed by Zanuttigh et al. (2003) -for the default values used in MIKE 21- and also by Johnson (2006). Calculations performed for monochromatic waves showed that module BW overestimates K_t, as shown in Fig. 3, while this does not happen when spectral waves were used. Regarding the wave breaking formulations embedded in modules NSW and PMS it appears that the one proposed by Nelson (1987) predicts lower values of K_t than the other models tested.

A comparison of results produced by formulae and numerical models is presented in Fig. 5. The graph refers to wave conditions #1, #2 (panel A), and to conditions #3, #4 (panel B). The K_t values obtained by the more reliable formulae, as pinpointed previously, are included, namely those by d'Angremond et al., Siladharma and Hall, Friebel and Harris. Inspection of the upper panel of this figure reveals that in general PMS wave module behaves consistently and reliably, as compared to the formulae with best fit to experimental data. This conclusion was also reported by Johnson (2006). Also, NSW behaves acceptably, especially with the classical or Nelson's breaking formulations. Of these formulations it is evident that Johnson's underestimates wave breaking and consequently overestimates K_t especially for low freeboard values. Model BW overestimates also K_t when regular waves are used as input. In the case of high incident waves (lower panel of Fig. 5) it can be said that, again, PMS module performs satisfactorily followed by NSW, while BW overpredicts wave transmission (for regular waves). Also, Johnson's and Nelson's breaking formulations do not help models to perform reliably in most of the cases tested. Regarding the comparison of 1DH versus 2DH wave models it is shown that both can perform reliably enough. An interesting feature noted between formulae and models with respect to the effect of crest width to K_t, is that B attains in the formulae an optimum value for which K_t becomes minimum, whereas in the wave models increase of B tends to decrease monotonically the value of K_t.

A representative model output giving H_s values in the study area is given in Fig. 6. The plot refers again to wave condition #1 and is provided by wave model PMS 2DH using default values for the constants in Battjes and Janssen wave breaking formulation. Figure 7 gives a cross-section of the seabed from deep to shallow water along with the corresponding values of the significant wave height for the same as above wave condition #1. A cross-section of the submerged breakwater can be seen, where the wave height is drastically diminished due to breaking. Four wave breaking formulations are shown, identical to those associated with modules NSW and PMS. In this figure model PMS 1DH is presented. It can be seen that the wave transmission associated with Johnson's breaking formulation is appreciably higher than the transmission predicted by the mid-way breaking models due to BJ (default values of constants) and BS.

CONCLUSIONS

The main conclusions of the present study are the following:

(a) Wave transmission over submerged breakwaters is a complicated phenomenon that is not yet fully described by either empirical formulae or wave models.

(b) Recent semi-empirical formulae perform satisfactory by taking into account factors such as crest width, wave breaking, breaker type, magnitude of the armor stones, etc.

(c) Out of the wave models tested, the parabolic mild-slope module (PMS of MIKE 21) showed the most consistent and reliable performance. However, it has to be noted that in many cases the combination of input conditions and bathymetry may lead to a different model as the most reliable one.

(d) Wave breaking is the most significant single factor affecting wave transmission. This leads to the conclusion that the crest width plays an equally significant role in determining the wave transmission coefficient. Johnson's formulation underestimates in general the amount of wave breaking.

(e) An important factor missing from most existing methods that predict the transmission coefficient is the percolation process through the porous body of the structure. This need should be covered by future research.

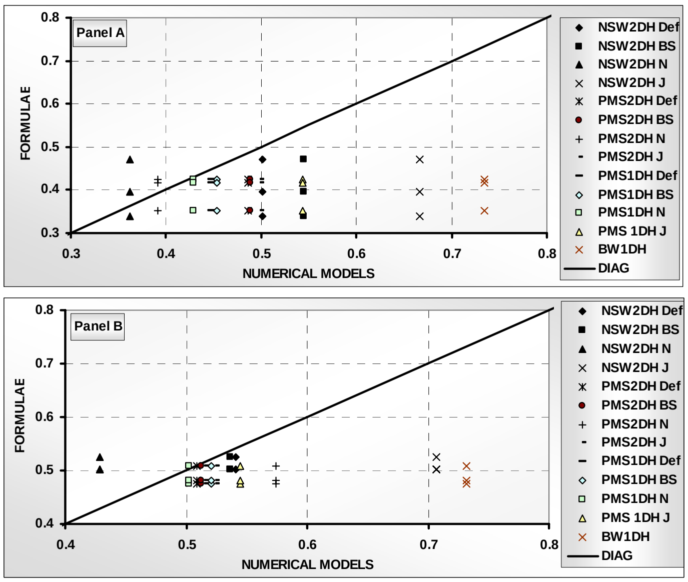

Figure 5. Comparison of transmission coefficient K_t: Formulae vs Numerical Models: panel A 10yr H_s, panel B 50yr H_s

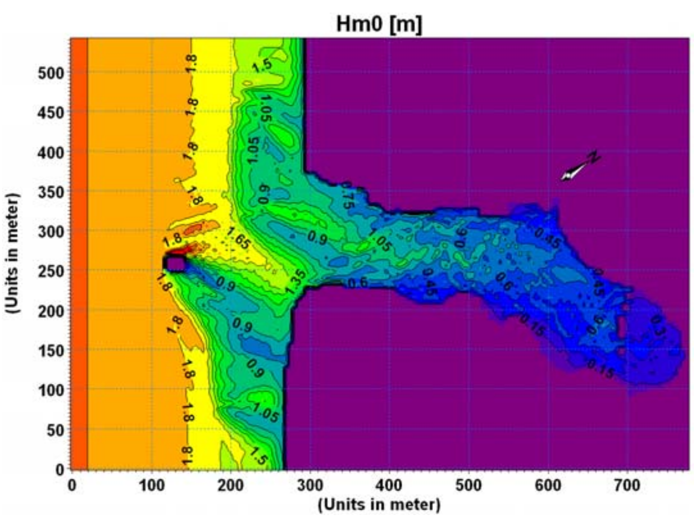

Figure 6. Significant wave height H_s PMS 2DH (wave condition #1)

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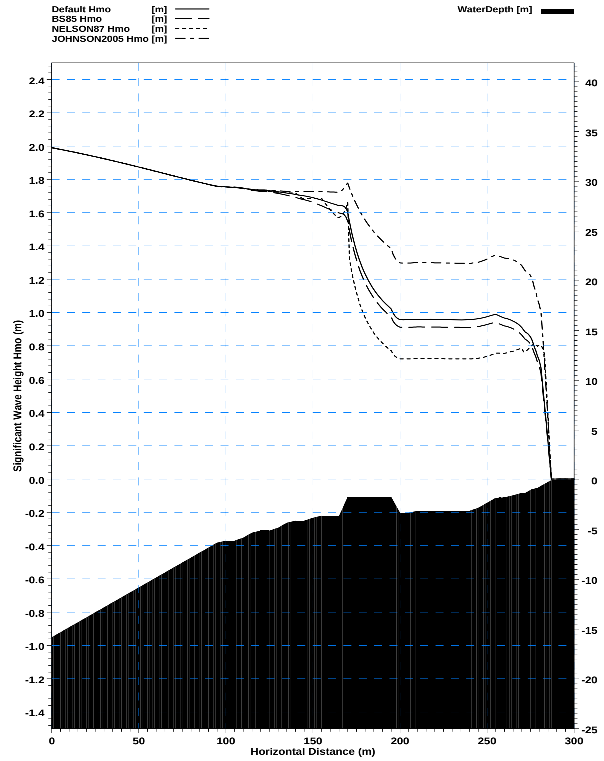

Figure 7. Significant wave height H_s PMS 1DH (wave condition #1)

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