Floating Breakwaters Efficiency in Intermediate and Shallow Waters

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Abstract: The efficiency of elastically moored floating breakwaters is studied in the present study using a finite-difference, mathematical model based on the Boussinesq type equations. The flow under the floating breakwater is treated as confined flow separately. The pressure field beneath the floating structure is determined by solving implicitly the Laplace equation for the potential Φ of the confined flow using appropriate boundary conditions. The dynamic equations of sway, heave and roll motions of the breakwater are also solved with the consequent adjustments of the continuity equation. Numerical results are compared with experimental results satisfactorily. The final goal is to study the efficiency of floating breakwaters in shallow and intermediate waters.

Introduction

The main function of a floating breakwater is to attenuate the wave action. Such a structure can not stop all the wave action. The incident wave is partially transmitted, partially reflected and partially dissipated. Energy is dissipated due to damping and friction and through the generation of eddies at the edges of the breakwater. Due to wave energy the breakwater can be put in motion and a radiated wave is produced which is propagated in both directions, offshore and onshore. The movement of the breakwater is specified in terms of the anchoring which defines the degrees of freedom of the breakwater.

There is a number of performance studies dealing with the hydrodynamic problem of floating breakwaters in deep and intermediate water depth. Linear models and analytical solutions have been developed, which describe the full hydrodynamic problem, Drimer et al (1992), Fugazza and Natale (1988), Kriezi et al (2001). Different models have been studied which calculate the forces and the mooring system of a floating breakwater Isaacson and Bhat (1994), Niwiski and Isaacson (1982), Nossen et al (1991), Yamamoto (1982). However a limited number of experimental studies exist: Sutko and Haden (1974) , Tolba (1998).

In the present work a finite difference depth-averaged wave propagation model is developed coupled with a 2DV model for the determination of the pressure field beneath the floating structure in order to investigate the efficiency of floating breakwaters in shallow and intermediate waters.

Basic Hydrodynamic Assumptions

The basic hydrodynamic assumptions for the model formulation are:

[a] The wave propagation model is based on the Boussinesq type equations. The model is applied in shallow and intermediate waters for which $d/L<0.5$ (d=the water depth and L=the wavelength). The floating breakwater produces radiated waves with frequencies of the same order, with the resonant frequencies of the floating structure (period should be around 1-3 sec). Thus a dispersive wave model is required in order to describe both low and high frequency waves and their interaction.

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[b] The water depth assumed in the region of the floating breakwater is the equivalent water depth producing the same specific discharge as the investigated wave in a shallow or intermediate water depth environment due to the differentiation in the horizontal velocity profile, in order to determine the mobilization of the water mass beneath the breakwater. From linear wave theory it is concluded that :

$$heq = \frac{L}{2\pi}\tanh(\frac{2\pi}{L}h) \tag{1}$$

where heq=the equivalent depth in shallow or intermediate water depth and L=the wavelength. Similar expression was also adopted by Losada et al (1997) for the determination of an effective water depth for wave propagation over porous structure.

[c] The semi-immersed body is a rectangle that allows periodic mobilization of the water masses between its keel and the sea bed, regulated by the pressure differences on its upstream and downstream sides. In the case of elastically moored breakwater the downstream wave generation and the upstream wave reflection are dominated by the under-flowing masses, the sway, heave and roll motions of the breakwater, which in turn depend on the equivalent elastic spring coefficient for each motion imposed by the anchoring system.

Model Formulation

Free surface wave motion away from the floating structure.

According to the notation of fig.1 the mathematical model outside the area occupied by the floating breakwater (regions I and III) is synthesized by the following equations, Karambas (1999), Madsen (1991) :

Mass continuity:

$$\frac{\partial \zeta}{\partial t} + \frac{\partial q}{\partial x} = 0 \tag{2}$$

x-momentum:

$$\begin{aligned}\frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + g\frac{\partial \zeta}{\partial x} &= \frac{d^2}{3}\frac{\partial^3 U}{\partial x^2 \partial t} + d_x d\frac{\partial^2 U}{\partial x \partial t} \\ &+ Bd^2\left(\frac{\partial^3 U}{\partial x^2 \partial t} + g\frac{\partial^3 \zeta}{\partial x^3}\right) + Bdd_x\left(\frac{\partial^2 U}{\partial x \partial t} + g\frac{\partial^2 \zeta}{\partial x^2}\right)\end{aligned} \tag{3}$$

where ζ is the surface elevation, q is the volume flux $q=U(d+\zeta)$, U is the depth mean horizontal velocity, d is the water depth, d_x is the bottom slope and B=1/15.

On the breakwater sides the continuity equation, equation (2), is modified by the breakwater motion and the specific discharge on the two ends of the structure is composed as follows:

$$q=q_u+q_s+q_{s1}+q_{s2} \tag{4}$$

where the q_u is the under-flowing discharge component referring to the closed conduit flow, q_s is the rate of water masses dislocation due to the sway motion , q_{s1} is the rate of the water masses dislocation due to the heave motion and q_{s2} is the rate of the water masses dislocation due to the roll motion of the structure:

$$q_s = (\zeta + dr)\frac{dx}{dt}$$
$$q_{s1} = \pm\frac{dz}{dt}\frac{B}{2}$$
$$q_{s2} = \pm\frac{d\theta}{dt}\frac{B^2}{8} \qquad (5)$$

In the case of fixed floating breakwater the extra terms in the continuity equation at the breakwaters sides are set to zero.

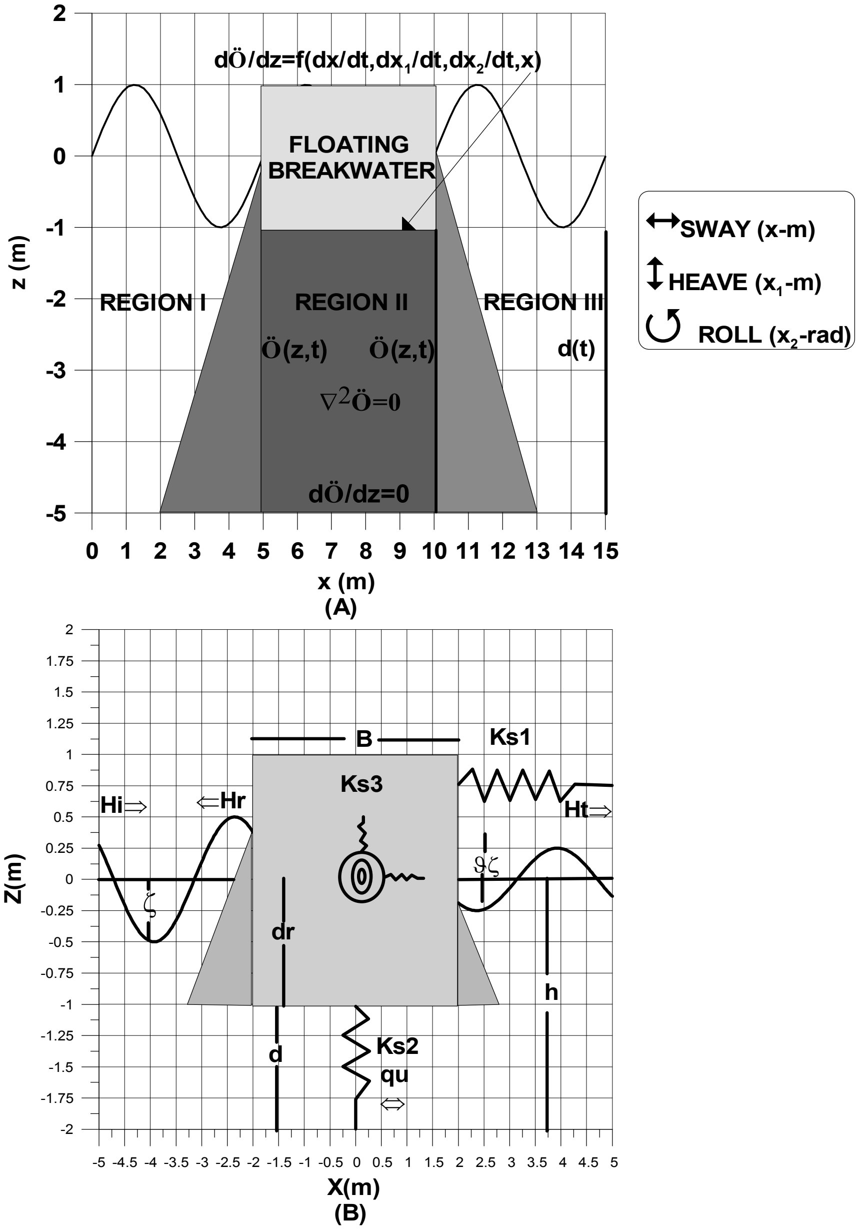

Figure 1. (A) Computational domain and boundary conditions in the region of the floating breakwater
(B) Basic notations of the examined problem

Confined flow in the breakwater area.

The modified momentum equation (oscillatory flow) for the closed conduit between the breakwaters keel and the sea bed is presented in the following equation:

$$\frac{\partial q_u}{\partial t} = -d\,\frac{\delta P}{\rho B C m} \qquad (6)$$

where δP is the pressure difference on the breakwater sides and Cm the dynamic mass coefficient (0(Cm)=1.5-2) according to experimental evidence, Fugazza and Natale (1988), and d the difference between “efficient” water depth and the breakwater draught (d=heq-dr). Cm remains the basic calibration parameter regulating the downstream wave amplitude. It is deduced from experimental studies, Sutko and Haden (1974), that the assumption of oscillating flow, equation (6), is valid when the ratio B/L, where B is the breakwater width and L the wavelength, is between 0.1-0.4. For the calculation of the pressure on the breakwater sides the pressure distribution according to Boussinesq is adapted, Peregrine (1972):

$$p_{bouss} = \rho g(\zeta - z) + \rho\left(\frac{(d+z)^2 - (d+\zeta)^2}{2}\right)\left(\frac{\partial^2 U}{\partial x \partial t}\right) \qquad (7)$$

Pressure field determination in the floating breakwaters region.

The pressure field beneath the floating structure is determined solving the Laplace equation for the flow potential Φ, in a 2DV, non-fixed computational domain with time variable boundary conditions (region II, fig. 1):

$$\nabla^2 \Phi = 0 \qquad (8)$$

The boundary conditions used in the model are the following:

At the sea bed (Newmann type boundary condition, Fugazza and Natale (1988)):

$$\frac{d\Phi}{dz} = 0 \qquad (9)$$

At the right and the left boundary of the 2DV computational region the potential is a function of time and depth as the potential of the flow is distributed in the specific boundaries in every time step :

$$\Phi = f(h,t) \qquad (10)$$

The distribution of the flow potential in the flow depth at the two breakwaters sides follows the following expression :

$$\frac{\partial \Phi}{\partial t} + gz + \frac{p_{bouss}}{\rho} = 0 \qquad (11)$$

where p_{bouss} is the pressure as predicted by the Boussinesq theory (given in the next section).

At the top boundary which coincides with the floating breakwaters keel for the general case the derivative of Φ normal to the boundary is a function of the horizontal, the vertical, the rotational velocities of the floating breakwater and the horizontal displacement (Newmann type b. c., Fugazza and Natale (1988)):

$$\frac{d\Phi}{dz} = f\left(\frac{dx}{dt}, \frac{dx_1}{dt}, \frac{dx_2}{dt}, x\right) \tag{12}$$

In the case of a fixed floating breakwater $\frac{d\Phi}{dz} = 0$ while in the case of breakwater in heave motion:

$$\frac{d\Phi}{dz} = \frac{dx_1}{dt} \tag{13}$$

The boundary conditions are also shown in figure 1.

After the solution has converged at each time step, we can dissociate the pressure term and finally calculate the pressure in every computational node using the linearized Bernoulli's equation:

$$\frac{\partial\Phi}{\partial t} + gz + \frac{p}{\rho} = 0 \tag{14}$$

since the altitude at every computational node is known in every time step. The integration of the pressure beneath the floating structure provides the buoyancy acting on the floating breakwater.

Dynamic properties of the floating breakwater.

The dynamic problem is formulated separately for each component of the breakwater motion (sway, heave, roll) and a linear superposition is assumed.

Horizontal motion (sway): In the case of "spring" mooring the horizontal motion of the breakwater is described by the Newton's law in x direction:

$$m\frac{d^2x}{d^2t} = F - K_{s1}\,x \tag{15}$$

where F is the total hydrodynamic force, due to the pressure and the mass flow, on the two sides of the breakwater, K_{s1} is the equivalent horizontal spring coefficient and m is the breakwater mass (m=B dr ρ_w).

Assuming hydrostatic pressure distribution on the two sides of the floating breakwater we obtain the following equation for the total hydrodynamic force:

$$F = \rho_w\left[(dr+\zeta)\left[\frac{g(dr+\zeta)}{2} - \frac{dx}{dt}\left|\frac{dx}{dt}\right|\right]_l - (dr+\zeta)\left[\frac{g(dr+\zeta)}{2} + \frac{dx}{dt}\left|\frac{dx}{dt}\right|\right]_r\right] \tag{16}$$

Vertical motion (heave): The vertical motion of the breakwater is described by the Newton's law in z direction:

$$m\frac{d^2z}{dt^2} = B_u - Rst - K_{s2}z - W \tag{17}$$

where B_u is the buoyancy acting on the floating structure which is known from the 2DV model, K_{s2} is the equivalent vertical spring coefficient, W is the weight of the structure, Rst is the resistance force due to the vertical motion of the structure in the water:

$$R_{st} = C_d \rho S \frac{dx_1}{dt}\left|\frac{dx_1}{dt}\right|/2 \tag{18}$$

where $O(C_d)=2$.

Rotational motion (roll): The rotational motion of the breakwater is described by the Newton's law in x-z plane:

$$I\frac{d^2\theta}{d^2t} = M_l - M_r - M_u - K_{s3}\theta \tag{19}$$

where I is the rotational inertia of the structure, M_l is the moment of the pressure force on the left side of the structure, M_r the moment of the pressure force on the right side of the structure, M_u the moment of the buoyancy force and K_{s3} the equivalent rotational spring coefficient of the mooring device.

Numerical Solution

The set of differential equations (2), (3) describe the wave field around the floating breakwater and they are combined with equation (6) (modified momentum equation in the floating breakwater region) in order to provide equations (8)-(14) (2DV model) with the necessary information. The numerical solution of the wave model is based on the fourth-order time predictor-corrector scheme proposed by Wei and Kirby (1995), which uses a fourth-order predictor-corrector finite differences method to advance in time, and discretizes first-order spatial derivatives to fourth-order accuracy.

In the next step Laplace equation is solved. The domain beneath the floating structure is discretized into grids of size dx and dz. The discretisation step dz is time and space variable in order to adjust the computational 2DV field to the physical domain which is restructured in time due to the motion of the floating structure. If $\Phi_{i,j}$ is used to denote the grid-point value of the potential, standard discretization of equation (8) using second-order finite differences yields:

$$\frac{\Phi_{i+1,j}-2\Phi_{i,j}+\Phi_{i-1,j}}{dx^2}+\frac{\Phi_{i,j+1}-2\Phi_{i,j}+\Phi_{i,j-1}}{dz^2}=0 \tag{20}$$

The resulting system of equations is solved using an iterative method.

Finally, after the estimation of the pressure field beneath the structure (using equation (11)), the equations of motion (15), (17) and (19) are solved, which in return modify the continuity equation in the floating breakwater region (equations (4) and (5)), and adjust the 2DV computational field.

Model Verification

The basic assumptions of linear (i.e. excluding non linear terms) dispersive waves propagation and of the closed conduit flow under the floating breakwater are collectively tested against experimental results for shallow and intermediate water depth (Williams, 1988).

Williams (1988) carried out experiments in a sunken flume of total length 75 m, nominal width 1.8 m and total depth of 2 m in order to investigate the interaction between regular waves and a partially immersed , rectangular obstacle. Two basics sets of experiments were conducted. In the first set the immersed body was fixed (figures 2, 3) while in the second the structure was in floating mode (figures 4, 5) subject to a limited horizontal restraint in order to avoid any non-linear motion due to the presence of higher order drift forces. The stiffness of the horizontal spring that was used in order to ensure sinusoidal motion about a constant null-point changed according to the model that was used in the experiments. The transmission and the reflection coefficients of the specific experimental results which cover the area of shallow and intermediate water depth, for the same $\frac{B}{dr}$ ratio and the same range of $\frac{B}{L}$ are very well comparable to the ones given by the present model (Cm=1.5-2). As we can see in figures 2 and 3 (restraint model) the numerical model is able to describe satisfactorily the transmission and the reflection of the propagating waves. In these graphs we can also see the significant influence of certain geometric characteristics of the structure on wave attenuation. For the floating mode case we can see in figures 4 and 5 that the model is able to describe resonance phenomena that occur due to the dynamic behavior of the floating structure.

Conclusions

A finite difference numerical model is developed and tested for the investigation of the efficiency of elastically moored floating breakwaters. The model is tested against experimental results. The comparisons show good agreement between the numerical results provided by the present model and the experimental results of Williams (1988). The analysis reveals the significant influence of certain geometric characteristics of the floating structure in wave attenuation. In figures 2 and 3 (fixed model) we can see that the transmission coefficient decreases proportionally with the increase of the B/L ratio revealing the significant influence of the width of the structure B. Similar conclusions can be derived for the effect of the structure draught dr. Resonance phenomena are clearly seen when the structure is in floating mode subject only to a limited horizontal restraint, figures 4 and 5. We can clearly see in these graphs that transmission coefficients are minimized in points in which resonance is obtained between the floating structure and the incoming wave. The form of these graphs is completely different from the corresponding ones for the fixed breakwater case. Resonance is obtained in higher frequencies, lower periods that are close to the period of the floating structure.

It is revealed that a modified Boussinesq wave model, in order to include a floating breakwater , coupled with a 2dv elliptic solver can be used in order to evaluate the efficiency of floating breakwaters in intermediate and shallow waters.

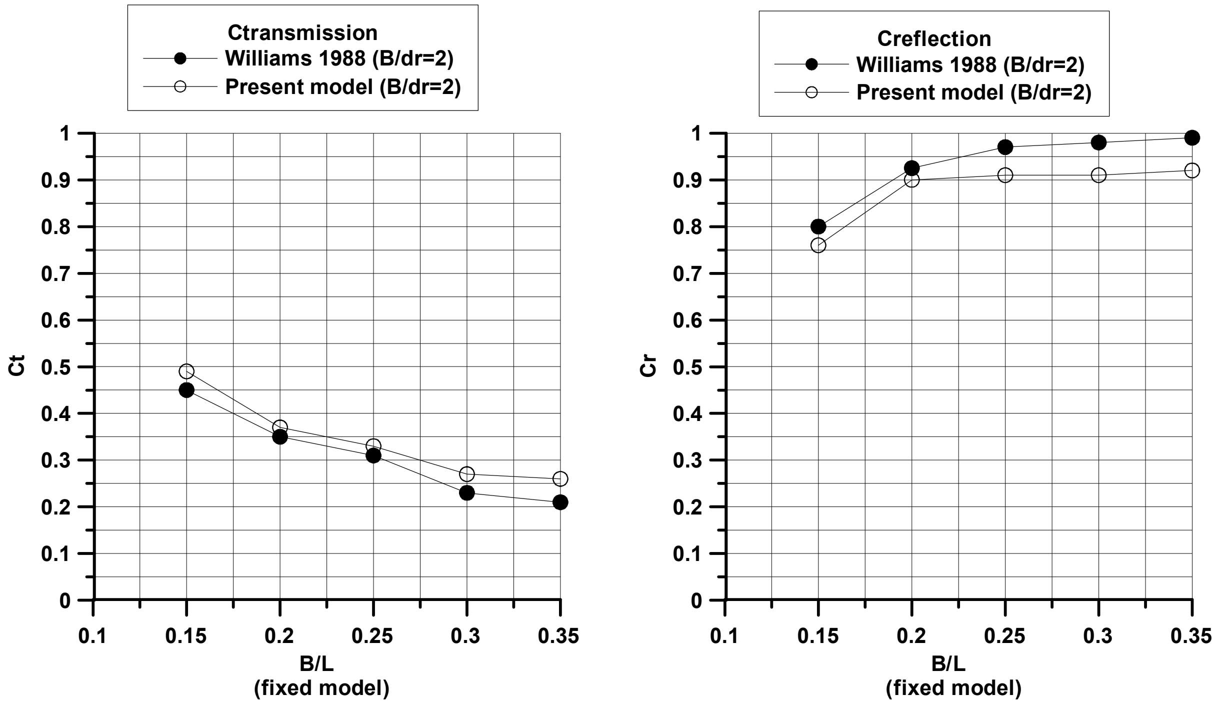

Figure 2. Comparison of the numerical results of the present model against the experimental results of Williams (1998) for Ct and Cr (restrained model).

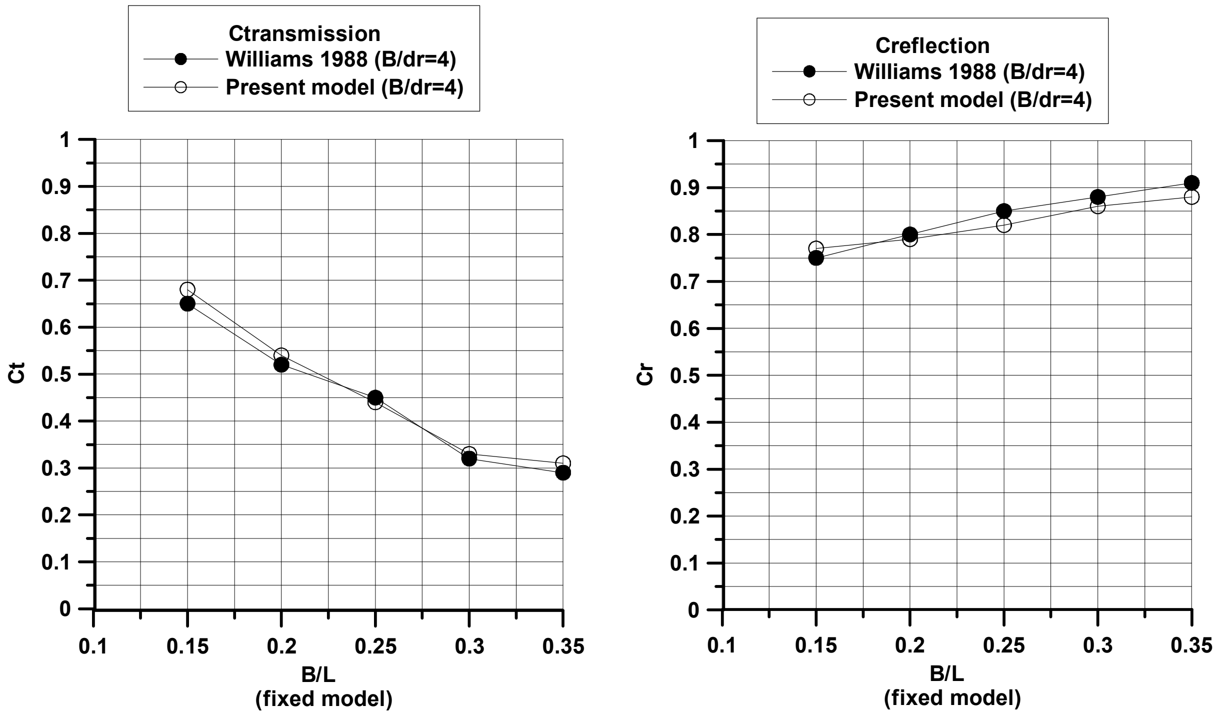

Figure 3. Comparison of the numerical results of the present model against the experimental results of Williams (1998) for Ct and Cr (restrained model).

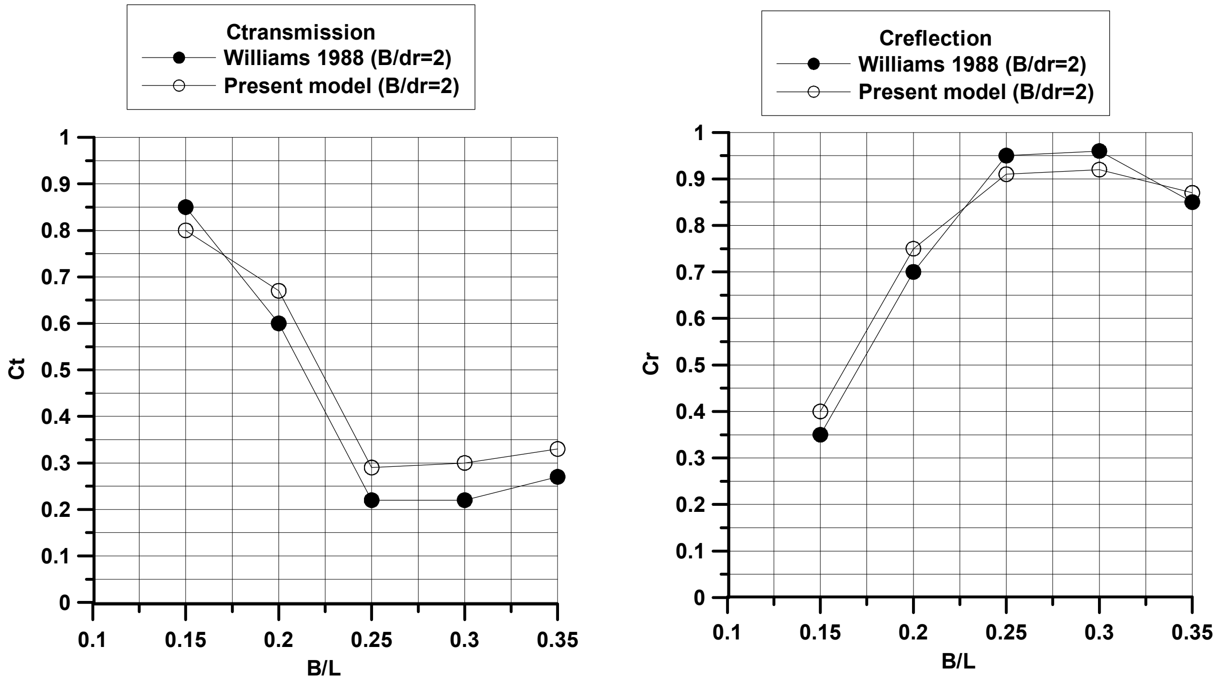

Figure 4. Comparison of the numerical results of the present model against the experimental results of Williams (1998) for Ct and Cr (floating model).

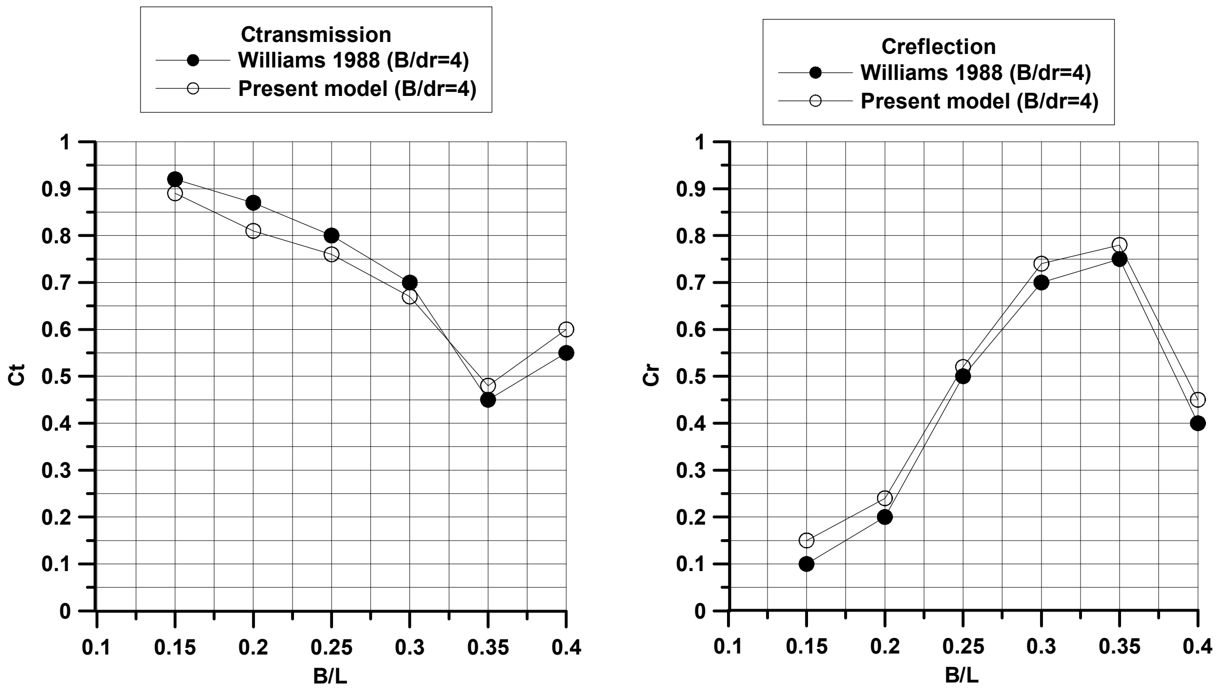

Figure 5. Comparison of the numerical results of the present model against the experimental results of Williams (1998) for Ct and Cr (floating model).

References

Drimer N., Agnon Y., Stiassnie M. (1992). "A simplified analytical model for a floating breakwater in water of finite depth. " *Applied Ocean Research*, 14, 33-41

Fugazza M. and Natale L. (1988). "Energy losses and floating breakwater response. " *Journal of Waterway Port Coastal and Ocean Engineering*, vol. 114, pp 191-205

Isaacson M., Bhat S. (1994). "Wave force on an horizontal plate" *International Symposium : waves - physical and numerical modelling*, pp. 1184 – 119

Karambas Th. V. (1999). ″A unified model for periodic non linear dispersive wave in intermediate and shallow water. ″ *Journal of Coastal Research*, Vol 15, No 1, pp. 128-139.

Kriezi E.E., Th. V. Karambas, P. Prinos and C. Koutitas , (2001) ″Interaction of floating breakwaters with waves in shallow waters″, *Int. Conf. IAHR 2001*, Theme E, pp 69-76, Beijing, China.

Losada I.J., M.D. Petterson and M.A. Losada, (1997) ″Harmonic generation past a submerged porous step, ″ *Coastal Engineering*, 31: 281-304.

Madsen, P.A, Murray R. and Sorensen O. R. (1991). ″A new form of the Boussinesq equations with improved linear dispersion characteristics. ″ *Coastal Engineering*, 15: 371-388.

Neelamani S., Sundaravadivelu R., Anbukumar S. (1997). ″Partially immersed oscillating water column type caisson breakwater. ″ *Journal of the Waterway Port Coastal and Ocean Engineering*.

Niwinski C.T. and Isaacson M. (1982). ″Non linear wave forces on floating breakwaters.″ *Proc. 18th Int. Conf. on Coastal Engineering*, ASCE, pp 2009-2025

Nossen J., Grue J. and Palm E. (1991). ″Wave forces on three-dimensional floating bodies with small forward speed.” *Journal of Fluid Mechanics*, vol.227, pp.153-160.

Peregrine, D.H., (1972). ″Equations for water waves and the approximation behind them″, Waves on Beaches and Resulting Sediment Transport. Ed. Meyet, Academic Press.

Sannasiraj S.A., Sundar V., Sundaravadivelu R. (1998). ″Mooring forces and motions responses of pontoon-type floating breakwaters. ″ *Ocean Engng*, vol. 25, No.1, pp 27-48.

Sutko A.A. , Haden E.L., (1974). "The Effect of Surge, Heave and Pitch on the Performance of a Floating Breakwater", *Proceedings of Floating Breakwater Conference*, Rhode Island, pp 41 -53.

Takayama T., Moroisi K. (1984). "Motion and mooring force of an axisymmetric floating bodies", *Coastal Engineering in Japan*, vol. 27, pp. 265-277

Tolba E.R.A.S., 1998. ″Behaviour of floating breakwaters under wave action. ″ Ph.D. thesis, Suez Canal University

Wei G. and Kirby T. (1995). ″Time-dependent numerical code for extended Boussinesq equations.″ *Journal of Waterway, Port, Coastal, and Ocean Engineering*, vol. 121, no 5, pp. 251-261.

Williams, K. J., (1988). ″An experimental study of wave obstacle interaction in a two dimensional domain″, *Journal of Hydraulic Research* , vol. 26, pp. 463-482.

Williams A. N. and McDougal W. G. (1991). ″Flexible floating breakwater, ″ *Journal of the Waterway Port Coastal and Ocean Engineering*, vol.117, No.5, pp.429-450.

Yamamoto T. (1982). ″Moored floating breakwater response to regular and irregular waves. ″ *Dynamic analysis of offshore structures*, CML publications Southampton ,vol 1,pp 114-123