PROPOSING AND INVESTIGATING THE EFFICIENCY OF VERTICAL PERFORATED BREAKWATER

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Abstract- This study aims to propose two types of an innovate breakwater with an economic feasibility. The first type is consists of two vertical perforated walls, the first wall is permeable in lower part (porosity ε =50%) and is impermeable in the upper part. The second wall is permeable in the upper part (porosity ε =50%) and the lower part is impermeable. Between the two walls there is a horizontal slotted wall. The second type is the same construction as on the first type but without horizontal slotted wall. The results indicates that the hydrodynamic performance of the first type is better than that of the second type in the percentage of (10-15%) because of the presence of the horizontal slotted wall. The effect of wave force on the first model bigger than the second model in the rang (10-15%). The wave force on the proposed models increases with increasing the relative depth (d/L). The transmission coefficient (kt) decreases with increasing the relative depth (d/L). The reflection coefficient (kr) increases with increasing the relative depth (d/L).

Index Terms-coastal structures - Permeable breakwater - perforated wall - numerical model - refraction - transmission - energy dissipation.

1 INTRODUCTION

Traditional breakwaters (i.e. rubble-mound, vertical caissons and gravity wall) are widely used to provide a protected calm water area to accommodate vessels and to allow loading and unloading processes. Such types possess a large width according to the water depth. Consequently, great amounts of construction material are required. Moreover; such breakwaters block the littoral drift leading to the occurrence of severe erosion or accretion. In addition, they dampen the water circulation leading to a deteriorated the water quality and achieving an unbalance to the ecosystem. Furthermore; traditional structures need skilled labor for their construction and certain foundation requirements. All the above leads to an uneconomic construction cost.

On the contrary, permeable breakwaters avoid the occurrence of the above side-effects, at the same time they provide reasonable protection with economic construction cost. This research was thus initiated with the objective of proposing and investigating the hydrodynamic performance of an innovative economic breakwater, numerically. This was achieved by undergoing the following research points.

- Reviewing the literature.
- Proposing an innovative breakwater.
- Investigating the proposed breakwater numerically.
- Analyzing and discussing the results.
- Comparing present study with previous study.

Many journals, periodicals and researches in the field of breakwaters were assembled, reviewed and comprehended from which it was clear that many researchers were occupied with finding out innovative types of economic breakwaters. Among these researchers were the following:-

Wiegel (1960) and Hayashi et al. (1966) investigated breakwaters in the form of a row of close piles. Herbich (1998) investigated double rows of close piles. Suh el at. (2006); K. Laju el at. (2007) stated that breakwaters in the form of thin, rigid, pile-supported vertical barriers or many rows of piles which is placed below the water surface would provide relatively greater protection. Rageh and Koraim (2010) examined the hydraulic performance of a vertical wall with horizontal slots. The upper part was impermeable but the lower part of model was horizontal slots. Ahmed et al. (2011) investigated the hydrodynamic characteristics of a vertical slotted wall breakwater. They further stated that for more protection and more dissipation of energy a pair of permeable barriers might be desired. Isaacson et al. (1999) examined a pair of thin vertical barriers placed below the water surface. Koraim et al. (2011) and (2014) investigated the hydrodynamic characteristics

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of double permeable breakwater under regular waves. The model consisted of double walls with horizontal slots. Suh et al. (1995) established an analytical model for predicting wave reflection from a perforated-wall caisson breakwater. They implemented the Galerkin-Eigen-function method to predict the reflection coefficient of a perforated wall caisson mounted on a rubble mound foundation. Hsu and Wu (1999) developed a numerical solution based on the boundary element method and boundary value problem for linear and second-order wave. Isaacson et al. (1998), Ahmed et al. (2011) and Suh el at. (2006) established the Eigen-function expansion method for linear waves, second-order waves and nonlinear waves. Lin et al. (2007), Huang et al. (2003), Lara et al. (2006), (2008) and Karim et al. (2009) executed numerical solutions for wave interaction with structures. They provided interesting examples for wave interaction with porous and submerged structure are reported. Ahmed (2014) investigated regular wave interaction using a numerical model of (FLOW-3D, VOF) with a single vertical perforated wall. From the reviewed literature, it was obvious that permeable breakwaters were meticulously investigated, even though; other types are required to be investigated. This research aims to proposing and investigating the hydrodynamic performance of an innovative economic breakwater, numerically. Moreover; a comparison between the hydraulic performance for the first and the second model. also comparison the present study with the previous study.

2. PROPOSING AN INNOVATIVE BREAKWATER

The two innovative economic breakwaters were proposed as shown in Fig. (1) and (2). It is a permeable breakwater that comprises a pair of identical vertical perforated walls. first wall is impermeable in the upper and second wall is impermeable in the lower. A porosity of a permeable part =50%. The first and second perforated walls are placed apart by a distance of 0.5 of the water depth. Model (1) is vertical perforated wall breakwater with horizontal slotted wall and model (2) is vertical perforated wall breakwater without horizontal slotted wall.

3. INVESTIGATING THE PROPOSED BREAK WATER NUMERICALLY

This section presents the implemented model and its theory. It presents the validation process of the model together with the executed numerical simulations to the proposed breakwater.

3.1. THE IMPLEMENTED MODEL

The proposed breakwater was due to be investigated. This was achieved via achieving numerical simulations using the commercial "Computational Fluid Dynamics" (CFD) code FLOW-3D. This is attributed to the fact that from the assembled literature, it was clear that CFD applications are common practice in all sectors of engineering and they are increasingly becoming important in maritime and coastal engineering. Therefore, the commercial CFD code (i.e. FLOW-3D, Flow Science Inc.) was chosen to be implemented in this study.

3.2 THEORY OF FLOW-3D

Basically, FLOW-3D applies the finite volume theory to solve the three-dimensional Reynolds- Averaged Navier -Stokes (RANS) equations. The model is formed of a group of solid sub-components, Fig. (3) and (4). The numerical model within FLOW-3D represented the geometrical and hydraulic boundary conditions

3.3 NUMERICAL SIMULATIONS USING FLOW-3D

Confident with the validation process, the model was implemented, varying the different parameters. Numerical replications were achieved to simulate the proposed breakwater.

In order to get a good compromising between precision/accuracy and computation time, two independent meshes with different cell sizes were used. Mesh cells are sized by 1 cm in each direction for waves of small frequencies and mesh cells are sized by 0.5 cm for waves of large frequencies. The time window for analyzing the wave height is carefully selected according to the wave length and is adjusted to avoid any reflection from the flume end or the wave paddle.

The reflection coefficient was calculated by the three-probe method of Mansard and Funk (1980). The selected data are converted into frequency domain by Fast Fourier Transformation. Finally, the spectrum of the incident, transmitted and reflected wave height were calculated. Thereby, the reflection coefficient 'kr' is calculated from extracted wave profiles by:

$$kr = \frac{h_r}{h_i} \qquad (1)$$

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where:- h_r is the reflected wave height, h_i is incident wave height.

The transmission coefficient 'k_t' was calculated directly from the wave transmitted profile by:

$$kt = \frac{h_t}{h_i} \tag{2}$$

where:- h_t is the transmitted wave height.

The energy dissipation coefficient 'kl' is given

$$kl = 1 - kr^2 - kt^2 \tag{3}$$

The velocity potential "Φ" is assumed as periodic motion in time T and it can be expressed as follows:

$$\Phi(x,z,t) = \mathrm{Re}\,\frac{-igh_i}{2\omega}\phi(x,z)\frac{1}{\cosh Kd}e^{-i\omega t} \tag{4}$$

Where Re is real part of a complex value, ω is wave angular frequency, g is gravitational acceleration, $i = \sqrt{-1}$ and K is wave number (K=2π/L).

The dispersion relationship could be formulated as:

$$\omega = gk\tanh(Kd)$$

The permeable boundary condition along the structure has been developed on the basis of the formulation of Sollit and Cross (1972) and as adopted by Yu (1995) for a thin vertical barrier. This might be given by:

$$\frac{\partial\phi_1(x)}{\partial x} = \frac{\partial\phi_2(x)}{\partial x} = iG^-(\phi_1(x) - \phi_2(x))$$
$$\text{at } x = -\lambda \text{ for } -d \le z \le -du \tag{5}$$

$$\frac{\partial\phi_2(x)}{\partial x} = \frac{\partial\phi_3(x)}{\partial x} = iG^-(\phi_2(x) - \phi_3(x))$$
$$\text{at } x = \lambda \text{ for } -du \le z \le 0 \tag{6}$$

The proportional constant $G^- = \dfrac{G}{b}$, G is called the permeability parameter and is expressed by: $G = \dfrac{\varepsilon}{f - is}$ where ε is the porosity of the structure given by the dimension and spacing of the piles, f is the friction factor (empirical parameter) and s is the inertia coefficient and given by $s = 1 + cm\left(\dfrac{1-\varepsilon}{\varepsilon}\right)$ where cm is an added mass coefficient. Eigen function expansion solved the velocity potential in a series of infinite number of solutions as follow:

$$\phi_1(x) = \phi_i + \sum_{m=0}^{\infty} A_{1m}\cos[\mu_m(d+z)]\exp(\mu_m(x+\lambda))$$
$$\text{at } x \le -\lambda \tag{7}$$

$$\phi_2(x) = \sum_{m=0}^{\infty} A_{2m}\cos[\mu_m(d+z)]\exp(-\mu_m(x+\lambda)) + \sum_{m=0}^{\infty} A_{3m}\cos[\mu_m(d+z)]\exp(\mu_m(x-\lambda))$$
$$\text{At } -\lambda \le x \le \lambda \tag{8}$$

$$\text{and } \phi_3(x) = \sum_{m=0}^{\infty} A_{4m}\cos[\mu_m(d+z)]\exp(-\mu_m(x-\lambda))$$
$$\text{at } x \ge \lambda \tag{9}$$

Applying the matching conditions (i.e. combining (7), (8) and (9) with (4) and (5) at the breakwater; the coefficients A_{1m}, A_{2m}, A_{3m} and A_{4m} could be determined by the following matrix equation:

$$\begin{bmatrix} \sum_{m=0}^{\infty} C_{11}^{(mn)} & \sum_{m=0}^{\infty} C_{12}^{(mn)} & \sum_{m=0}^{\infty} C_{13}^{(mn)} & \sum_{m=0}^{\infty} C_{14}^{(mn)} \\ \sum_{m=0}^{\infty} C_{21}^{(mn)} & \sum_{m=0}^{\infty} C_{22}^{(mn)} & \sum_{m=0}^{\infty} C_{23}^{(mn)} & \sum_{m=0}^{\infty} C_{24}^{(mn)} \\ \sum_{m=0}^{\infty} C_{31}^{(mn)} & \sum_{m=0}^{\infty} C_{32}^{(mn)} & \sum_{m=0}^{\infty} C_{33}^{(mn)} & \sum_{m=0}^{\infty} C_{34}^{(mn)} \\ \sum_{m=0}^{\infty} C_{41}^{(mn)} & \sum_{m=0}^{\infty} C_{42}^{(mn)} & \sum_{m=0}^{\infty} C_{43}^{(mn)} & \sum_{m=0}^{\infty} C_{44}^{(mn)} \end{bmatrix} \begin{bmatrix} A_{1m} \\ A_{2m} \\ A_{3m} \\ A_{4m} \end{bmatrix} = \begin{bmatrix} b_{1n} \\ b_{2n} \\ b_{3n} \\ b_{4n} \end{bmatrix}$$
$$\text{For } n = 1, 2, 3, \ldots \infty \tag{10}$$

Equation (10) could be solved by numerical tools. Consequently, kr and kt could be obtained;

$$\text{kr} = |A_{10}|$$

$$\text{kt} = |A_{40}|$$

The energy dissipation coefficient can be determined using equation (3).

3.4. HYDRODYNAMIC FORCE

The hydrodynamic pressure (p) exerted on the surfaces of a body can be expressed by linearizing Bernoulli's equation as follows:

$$p = -\rho\left(\frac{\partial\phi}{\partial t}\right)_{x=-\lambda} = i\omega\rho(\phi_1 - \phi_2)_{x=-\lambda} \qquad x = -\lambda \tag{11}$$

$$p = -\rho\left(\frac{\partial\phi}{\partial t}\right)_{x=\lambda} = i\omega\rho(\phi_2 - \phi_3)_{x=\lambda} \qquad x = \lambda \tag{12}$$

$$p = \frac{-\rho g H_i}{2}\frac{\cosh k(z+d)}{\cosh kd}(1 + k_r - k_t) \tag{13}$$

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Then, the hydrodynamic forces (F*) exerted on the breakwater can be evaluated by integrating the pressure around the body's wetted surface as follows:

$$F^{*} = \int_{-d}^{0} p(0,z)dz \qquad (14)$$

$$F^{*} = -\frac{\rho g H_i}{2k}(1 + k_r - k_t)\tanh kd \qquad (15)$$

$$F = \mathrm{Re}[F^{*} e^{-i\omega t}] \qquad (16)$$

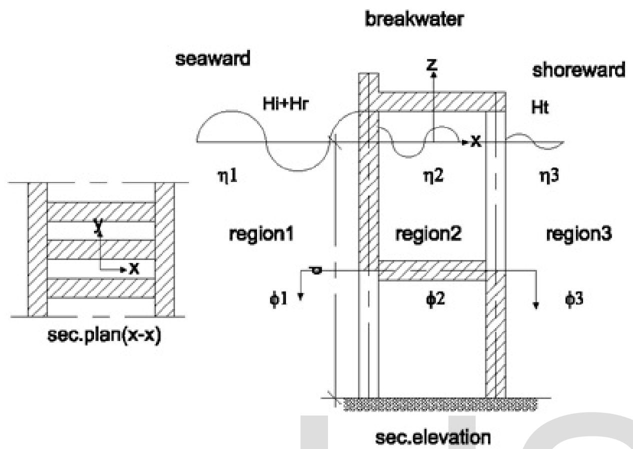


Fig. (1) Sketch for a vertical perforated wall breakwater with horizontal slot

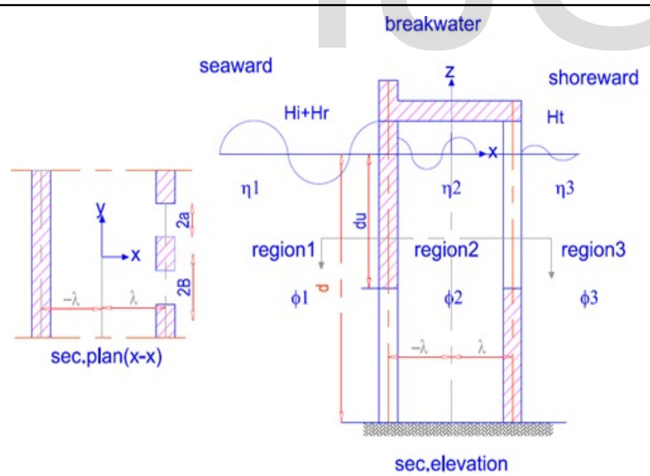
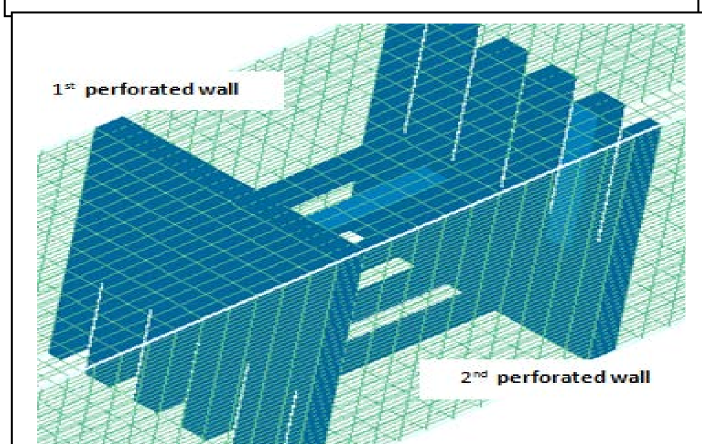


Fig. (2) Sketch for a vertical perforated wall breakwater without horizontal slot



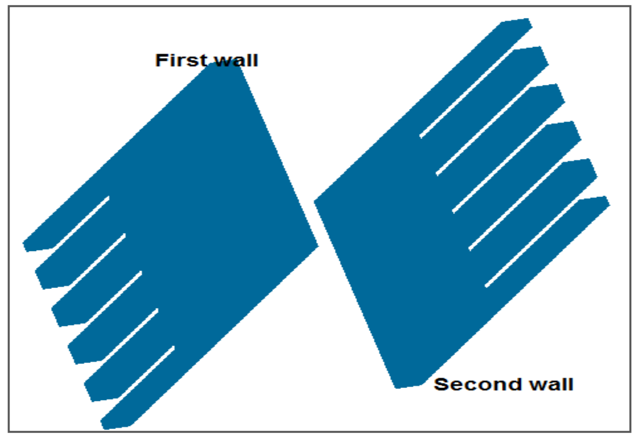


Fig. (4) Breakwater model without horizontal slot

4. Results and Desiccations

4.1 Validation of Hydro Dynamic Performance

In this paper, the hydrodynamic performance of a new type breakwaters are studied using numerical models. The breakwater consists of a double vertical perforated walls (with and without horizontal slots). The first wall impermeable upper part and permeable lower part. The second wall permeable upper part and impermeable lower part. The transmission, the reflection, and the wave energy dissipation coefficients are presented for different wave and structure parameters. It can be seen from Table (1) that's the results obtained from the experiments that was carried out in the Irrigation and Drainage Engineering laboratory of the Faculty of Engineering, Zagazig University. The experimental work was carried out without proposed breakwater to determine incident wave height and wave periods.

4.2 The Wave Flume

The wave flume used in this work has a rectangular cross section with 2 m bed width and 1.2 m depth. The overall length of the flume is 12 m. All sides of the flume are made of reinforced concrete of 0.25 m thickness. This flume divided into three parts (i.e the inlet, working section and the outlet parts). A gravel wave

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absorber with slope 3:1 is installed at outlet part of the flume to absorb the transmitted waves. A general view of the flume is photo (1).

TABLE 1

THE WAVE PERIODS, LENGTHS AND INCIDENT WAVE HEIGHTS FOR DIFFERENT WAVE GENERATOR FREQUENCIES WITHOUT MODELS

Parameters	units	Ranges						
Incident wave(hi)	cm	6.7	7.6	8.2	9	10.1	11.5	12.5
Wave period (T)	Sec.	1.4	1.35	1.3	1.2	1.1	1	0.9
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]4
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]1
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]3
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]2

Photo (1), A general view of the wave flume.

4.3 RESULTS ANALYSING

Fig. (5) shows the wave high resulted from the experimental test conducted on the laboratory without the using of the breakwater at different frequencies. Fig. (6) shows the relation between wave period and wave incident. It can be seen with the increasing of wave incident the wave period decreases. Fig. (7) shows the relation between wave period and inverter frequancy. Fig. (8) clear showes that the transion coefficant (kt) . Decresses with the increase of the relative depth (d/L) and with the comparasion of the first model and the second model it is cleared that the energy dispation coefficant is better for the first model than the second in the range betwwen 10-15%. Fig. (9) Comparison of predicted(double perforated walls without horizontal slot) hydrodynamic coefficients (kt, kr and kl) with results from Ji and Suh (2010) as a function of d/L, when D/d = 0.5, B/d = 1, ε = 0.5, f = 2. Fig. (10). Comparison of predicted hydrodynamic coefficients (kt, kr and kl) with results from Laju et al. (2011) as a function of d/L, when D/d = 0.35, B/d = 0.5, ε = 0.25 and f = 1.2 Fig. (11) Comparison of predicted and (Flolw-3D) hydrodynamic coefficients (kt and kr) with results from Laju et al. (2011) as a function of d/L, when D/d = 0.35, B/d = 0.5, ε = 0.25 and f = 1.2 From figure (12) the effect of the wave force on the first model is bigger than its effect on the second model. It also can be seen that with the increase of the relative depth the variation between the two models is increased.

Fig. (13) and (14) show the model in CED (FLOW-3D) and the location of measuring probes. Fig. (15) shows the free surface elevation (cm) after 2.0 meters from breakwater by using (FLOW -3D) at wave period (T) = 0.9sec and 1.1 sec. Fig. (16) shows surface elevation (cm) at wave period T =1.5 sec, wave translated at probe (1)and wave reflected at probe (2).

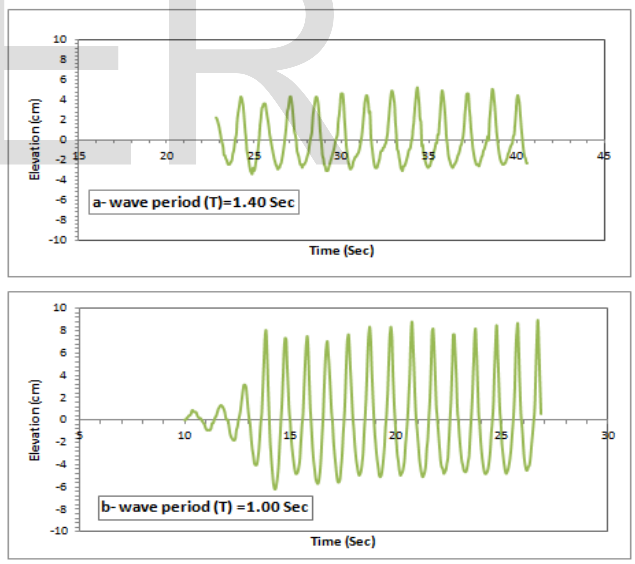


Fig. (5) Waves profiles for wave periods: (a) T=1.4, (b) T=1.0 sec.

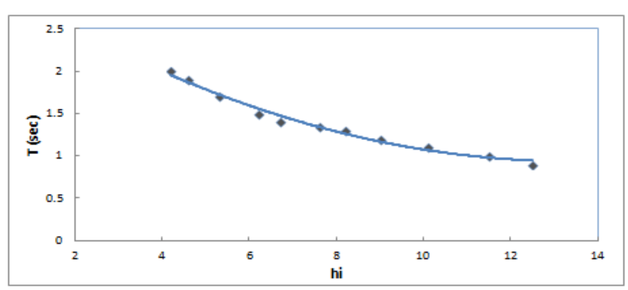


Figure (6) Relation between wave period and wave incident.

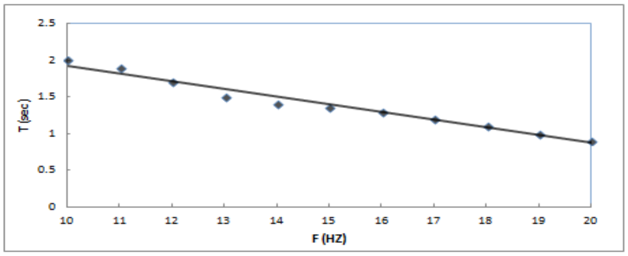


Fig. (7) Relation between wave period and inverter frequancy.

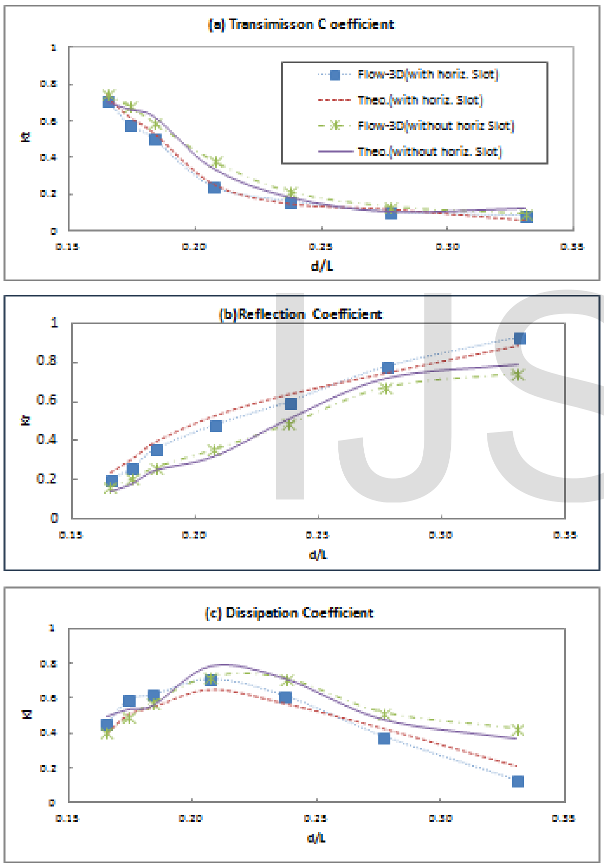


Fig. (8) Comparison between CFD (FLOW-3D) and predicted results for a double perforated walls with horizontal wall slot and without horizontal wall slot as function of (d/L) at $2\lambda = 0.5d$, (a) kt, (b) kr and (c) kl.

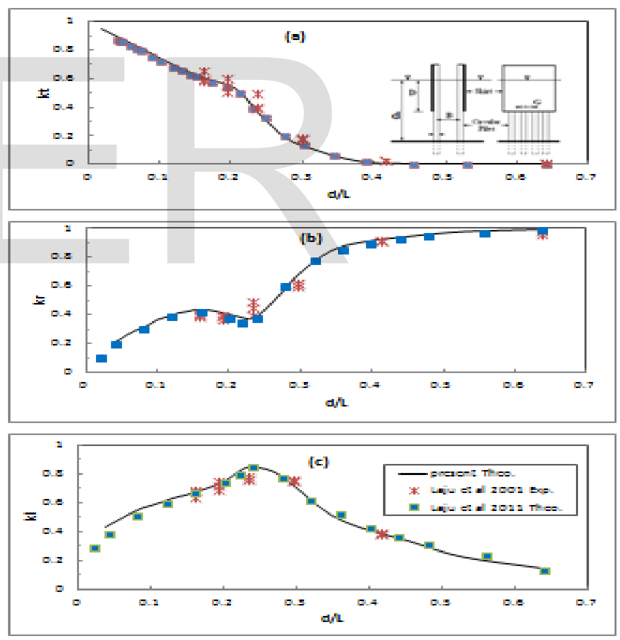
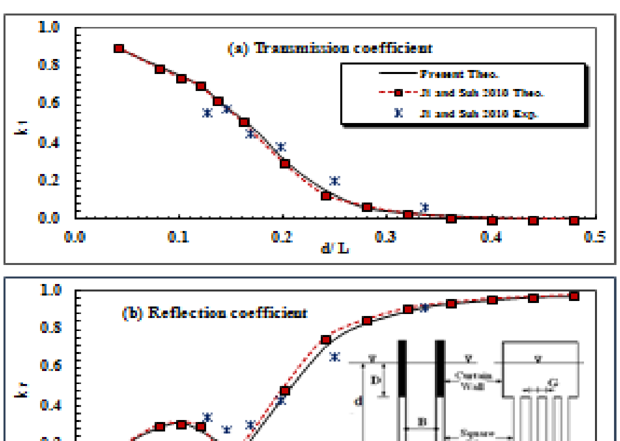
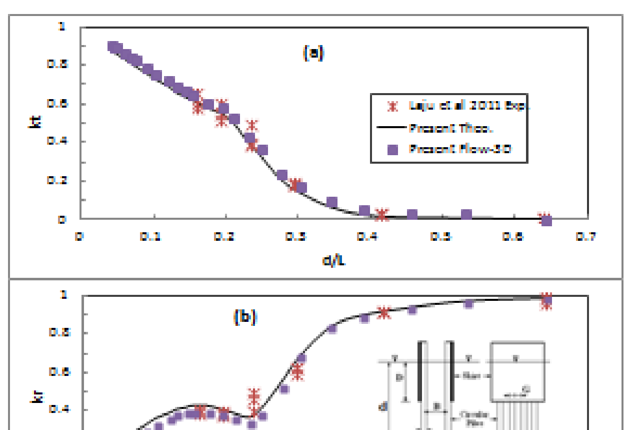


Fig. (10) Comparison of predicted (double perforated walls without horizontal slot) hydrodynamic coefficients (a)kt, (b)kr and (c)kl with results from Laju et al. (2011) as a function of d/L, when D/d = 0.35, B/d = 0.5, ε = 0.25 and f= 1.2





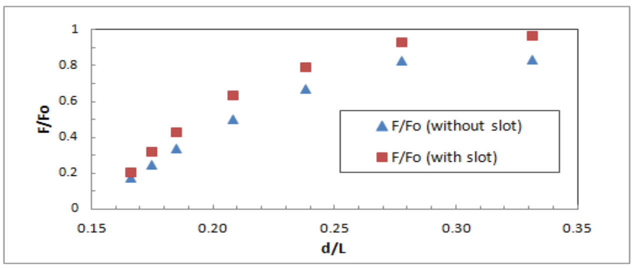


Fig.(12) Comparison of dimensionless wave forces between a double perforated walls with horizontal slot and a double perforated walls without horizontal slot as function of (d/L) for 2λ= 0.5d and ε = 0.5

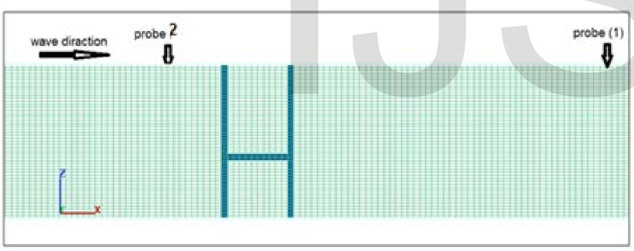


Fig. (13) shows wave direction, location wave reflection and wave translation.

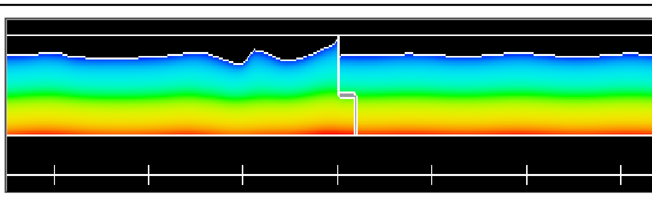


Fig. (14) Wave shape in CFD (FLOW-3D).

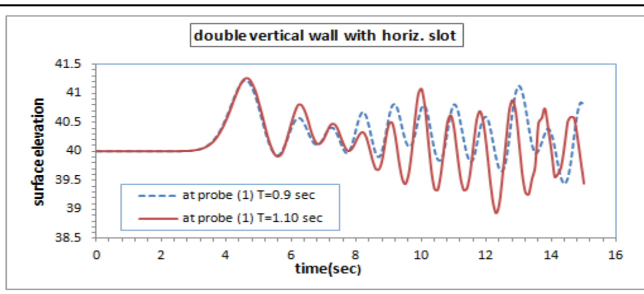


Fig. (15) Free surface elevation (cm) after 2.00 meters from breakwater by using (FLOW -3D).

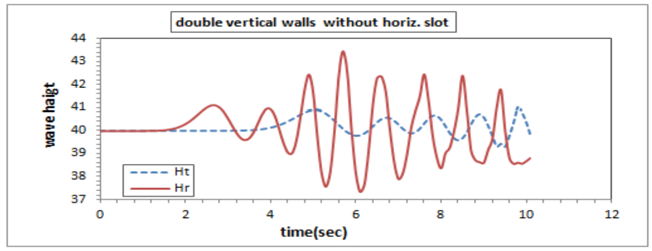


Fig. (16) Surface elevation (cm) at wave period T =1.5 sec, wave translated at probe 1and wave reflected at probe 2.

5. CONCLUSION:-

1- The reviewed literature revealed that the breakwaters were physically modeled and investigated meticulously but breakwater numerical modeling has some discrepancies. It was also clear that extra investigations are needed. Among the reviewed available models, model Flow 3-D was found to be capable of simulating the proposed breakwater.

2- Flow-3D was validated against extensive laboratory investigations and theoretical model.

3- Flow -3D is capable of describing the wave interaction of a linear wave with double vertical perforated walls. Flow -3D is capable of reproducing most of the important features of the experimental data and semi-analytical results. Flow -3D reproduced numerical results that are perfectly acceptable. The wave force on the proposed models increases with increasing the relative depth (d/L).

4- The effect of wave force on the first model bigger than the second model in the rang (10-15%).

5- The transmission coefficient (kt) decreases with increasing the relative depth (d/L) and the reflection coefficient (kr) increases with increasing the relative depth (d/L).

6- The comparasion of the first and the second model it is cleared that the energy dispation coefficant is better for the first model than that for the seconde in the range betwwen 10-15%.

7- The hydrodynamic performance of the seconed model is lower than that of the previous study in the rang of (3-7%).

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NOTATIONS:-

The following symbols have been adopted for use in this paper:

A10 = complex reflection coefficient;
A40 = complex transmission coefficient;
A1n = complex unknown coefficients;
λ = half distance between the two walls;
b = thickness of the vertical wall;
Cm =added mass coefficient;
f = friction coefficient;
G = permeability parameter;
g = acceleration of gravity;
hi = incident wave height;
hr = reflected wave height;
Ht = transmitted wave height;
d = water depth;
k = incident wave number;
kl = energy dissipation coefficient;
kr = reflection coefficient;
kt = transmission coefficient;
L = wave length;
T = wave period;
t = time;
x , z = two dimensional axis;
ε1 = porosity of the lower part of the first wall;
ϕp = total flow velocity potential;
ϕ1 = seaward velocity potential;
ϕ2 = velocity potential between the two walls;
ϕ3 = shoreward velocity potential;
ω = angular wave frequency.
F* = wave force

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