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HYDRAULIC PERFORMANCE OF A 3-D COMPOSITE SEAWALL SYSTEM FORRENEWABLEWAVE ENERGY CONVERSION

A.B.M. Khan-Mozahedy

Abstract—Detrimental impacts of fossil fuels and their foreseen scarcity are encouraging research for the development of the renewable energy as alternatives. A newly developed composite seawall concept could be a vitally important technique for wave energy conversion. Composite seawall is a dual-purpose overtopping type of coastal shoreline device; wave energy conversion is considered as its by-product. A 3-D model of composite seawall has been simulated at University of Southampton to figure out its hydraulic performance. Overtopping water has generated hydraulic head convertible to electricity by means of low head hydro-power generator. Fraude scale laws (geometric scale 1:50) were followed in the model scaling. Total 72 simulations were conducted; overtopping and hydraulic powers that were generated at the crest of the ramp of the composite seawall for each simulated wave parameters were recorded. Hydraulic performances were measured based on the input wave parameters and simulation outputs. Results have demonstrated that maximum achievable hydraulic efficiency of the composite seawall is about 33.6 % and average hydraulic efficiencies are about 26.6%, 18.6%, 15.9% and 11.1% for the freeboard of 0.5 m, 1.0 m, 1.5 m and 2.0 m respectively. Hydraulic performance decreases for oblique wave approaches.

Index Terms—Composite seawall, Wave energy conversion, Hydraulic head, Hydrostatic pressure wheel, Hydraulic performance, Overtopping, Oblique wave approach

1 INTRODUCTION

Carbon emissions to the atmosphere are causing global warming phenomena. Uses of fossil fuel have brought detrimental changes to the global climate system during last decades. Moreover, world reserve of the fossil fuel (mainly oil and gas) is depleting at an accelerating rate due to tremendous dependence on them. Hence, development of renewable energies is a growing demand as an alternative source of energy. Among the renewable energies, wave energy is vitally important, although considerable progress is not achieved yet. Sometimes, composite sea walls consist of obstacles in front of it to enhance wave energy dissipation and reduce wave loadings and related damages. This type of composite seawall has been constructed recently at Mori port in Japan and research has shown that 15% construction cost has been reduced compared to the conventional Japanese seawall (Mori et al., 2008). A composite seawall with narrow reservoir along its length has been developed at Southampton University to convert wave energy into potential energy. Muller et al. (2009) has pointed out that advantage of this seawall is its cost-effectiveness, as it is a dual purpose structure; providing protection and power generation together. Hydraulic head difference has created by collecting water in the reservoir through wave overtopping. A Wave Energy Converter (WEC) has been developed using Hydrostatic Pressure Wheel (HPW), which has been found potentially very effective for low head differences. Muller et al. (2009) has described the HPW as a very simple and cost-effective hydropower converter which can tolerate large variations in flow rate. The combination of a low head (less than 1 meter) hydropower source with this converter could result in an overall effective system. Maravelakis (2009) has measured efficiencies of the composite seawall for WEC in 2-D physical model tests and found that maximum hydraulic efficiencies are about 32% in 1/50 scale model and about 28% in 1/23 scale model. Sea states are random and obviously it is three dimensional (3-D) in nature. Hydraulic performance mainly depends on the collection of water in the reservoir. Water collection depends on the wave height and its consistency with time. 3-D physical model testing might lead to determine hydraulic performance of the system. Parallel waves and oblique waves might have different results in collecting water, which has been compared in this article.

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2 WAVES AND COASTAL STRUCTURES

Ocean waves are caused by the wind stress while wind blows over the surface. Waves are sinusoidal fluctuations (ups and downs) of the water surface in the sea. Wave gets energy from wind stress and becomes a powerful source of renewable energy. The main purpose of the coastal structures (seawall, dike, revetment, breakwater etc) is to protect land and properties from the wave attack, still storm waves cause overtopping of these structures and endanger lives and coastal infrastructures. Lots of research and experimental investigation has been done during the last 50 years to understand overtopping phenomena and guidelines has been derived to design coastal structures in order to minimise overtopping damages. Overtopping discharges vary up to several orders of magnitude from one wave to another under random wave conditions, meaning that it is a non-linear function of wave height and wave period. Overtopping depends not only on wave parameters such as wave height, wave period, wave length, water level but also on geometric layout and material properties of the structure (Soliman, 2003). However many researchers have been trying to develop methods and formulas in order to predict overtopping discharges of coastal structures on certain possible conditions. These prediction formulas are widely varying and results in wide variations. Most of the prediction methods and formulas for mean overtopping rate are derived from numerical and physical modelling in the laboratory facilities, which leads to develop empirical relationship of overtopping discharge rate with wave parameters, basin geometry and material properties of the structures. Renowned prediction methods and formals are derived by *Owen* (1980, 1982), *Brudbury and Allsop* (1988), *Pedersen and Burchartch* (1992), *Van der Meer and Janssen* (1995) and *Goda* (2000). All of the prediction formulas are derived for sloping coastal structures (with or without rocked armour and crown wall) of more or less generally impermeable, smooth or rough, straight or bermed sloped seabed geometry (Soliman, 2003). Moravelakis (2009) has found *Owen* (1980) to be unsuitable for the predictions of wave overtopping over a composite seawall in 2-D wave basin and the prediction results by *Van der Meer and Janssen* (1995) and *Goda* (2000) are more ordered than that of *Owen* (1980). Among the above overtopping prediction formulas, *Owen* (1980) and *Van der Meer and Janssen* (1995) has been used to predict overtopping rates of the 3-D physical model in this study.

2.1WaveEnergy Conversion

Fossil fuel causes detrimental changes in the environment due to CO_2 emissions. Moreover, extensive use of fossil fuel is depleting rapidly the reserves of oil, coal and gas. Wave energy conversion from renewable energy sources can be an alternative solution of the fossil fuel. Among the renewable energy resources, exploitation of wave energy has been studied during last decades, although considerable progress has not been achieved yet to compete with other energy sources in the market. The main hindrance is that it is hardly possible to harness this energy in economically viable way and convert it into electricity in large amounts. Kine (2005) has opined that there have been many attempts taken to harness the power of the ocean waves over the last century, some showing limited short-term success but ultimately failing due to technical or economic reasons. However, research continues to find out potential Wave Energy Converter (WEC). The already existed WECs can be distinguished into two categories: offshore WECs and shoreline WECs. Offshore WECs are floating or submerged devices in deep water and anchored to the seabed. Exploited energy is transferred to the shore by means of cables placed on the seabed. Shoreline WECs are generally placed along the shore in shallow water and sometimes, can be integrated with shoreline defenses (Maravelakis, 2009). Offshore WECs may exploit huge potential of high wave energy density environment but they suffer from extreme wave loadings, costly underwater cable connection for electricity transmission, and difficulty in maintenance works. Shoreline WECs has relatively low potential of wave energy as energy dissipated into the shallow waters. However, they may be cost-effective due to low initial and maintenance costs and greater accessibility. Shoreline WECs can be constructed in combination with shoreline defenses, which will eventually reduce production costs too (Marvelakis, 2009). Among the WECs, Oscillating Water Column (OWC), Pelamis, Wave dragon, Oyster etc. are prominent and recently developed devices.

2.2 Overtopping WECs: Wave Dragon and SSG

Overtopping devices are water reservoir, which collect water through wave overtopping and generate potential hydraulic head. This hydraulic head drives a turbine to produce electricity. Wave dragon is an overtopping device, which is usually installed in offshore. Waves run over the ramp of the device and the water is stored in a reservoir. As more water enters the reservoir, an equal amount of water is forced out through the turbine in the centre, causing it to rotate and generate electric power (Kine, 2005). Sea Slot-cone Generator (SSG) is another overtopping device, which is placed along the shore and integrated with shoreline defence systems like breakwater or rock cliff. There are multiple reservoirs placed on top of each other, in which water from incoming waves is stored and then runs multi-stage turbine to produce electricity. Multiple reservoirs utilize different heights of water head and hence result in a high overall efficiency (Kofoed, 2005).

2.3 Composite Seawall

Seawall is normally constructed along the coastline to protect land and property. However, extreme waves cause overtopping of the seawall and endanger lives and properties during storm and hence seawall has been gone through several modifications such as curved top etc. in order to minimise overtopping. Recently a high mound composite seawall has been developed and constructed at Mori port, Japan. Research shows that 15% construction cost has been reduced in the composite seawall compared to the conventional Japanese seawall (Mori, 2008). This composite seawall has an armoured slope and a curtain of vertical piles in the front of the actual seawall. The curtain dissipates wave energy and therefore, it reduces wave loads on the seawall. A sketch composite seawall is shown in the Figure 1.

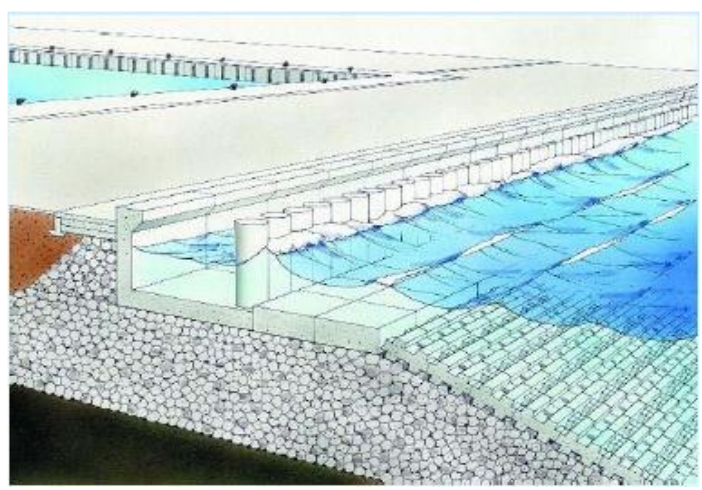

Figure 1: High mound composite seawall, Mori port, Japan (Mori, 2008)

Southampton University, UK has developed an overtopping type composite seawall for wave energy conversion. The curtain was replaced by an impermeable ramp to create a water reservoir, which is shown in the Figure 2. Overtopping water has been collected into the reservoir to create hydraulic head for energy conversion.

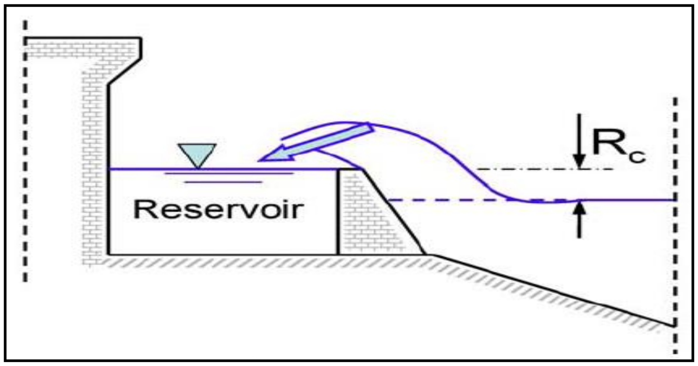

Figure 2: Composite seawall for energy conversion (Muller, 2009)

2.4 Hydrostatic Pressure Wheel

Composite seawall creates hydraulic head difference in the order of 1.0 meter. Low head hydropower converter is needed to exploit wave energy of overtopping type composite seawall in a cost-effective way. Wave dragon and SSG use low head water Kaplan turbines, but it is rarely considered as economically viable. Southampton University has developed a special type of Hydrostatic Pressure Wheel (HPW), which is very effective in conversion of low head differences (Muller, 2009). HPW is usually installed at the end (outlet of the reservoir water) of the composite seawall and the vanes rotate around its axis due to hydraulic head difference, when water passes through it. For average wave heights of 1 m and a head difference of 0.9 m, a hydraulic power of 1.2 to 2 KW/m wall can be expected, provided that the energy converter has an estimated efficiency of 65% (hydraulic to electric), giving an overall efficiency of 17 to 28 % (Muller, 2009).

2.5 Physical Modelling of Coastal Structures

Physical modeling is an important tool of testing and validation for coastal engineers. Physical models help to understand the complex hydrodynamic behavior of coastal structures, which provide reliable and economic engineering design solutions (Hughes, 1993). Hughes (1993) defines physical model as physical system reproduced at a reduced size so that the major dominant forces acting on the system are represented in the model in the correct proportion to the actual physical system. A prototype is the situation, which is being modelled, either in the same size or more often at reduced scale. Scaling is fundamental in order to predict the processes of the prototype under investigation. Scale ratio is the basis of correspondence tool of input parameters and results between prototype and model. Scaling of a model in coastal engineering can be done by dimensional analysis. Hughes (1993) has given details of methods of dimensional analysis in scaling of coastal physical models. Scale ratio (N_x) is the ratio of the value of a parameter in the prototype (X_p) to the value of the same parameter in the model (X_m). The reciprocal of this definition is also true.

$$N_x = X_p/X_m \tag{1}$$

Similarity and similitude requirements should be met to reproduce a good model to a prototype. Similitude is achieved when all the factors are in proportion between the prototype and the model and the factors that are not in proportion should be so small as to be insignificant in the process (Hughes, 1993).

Hughes (1993) defines scale effects in the physical modelling as the differences between prototype and model response that arise from the inevitably to simulate all the relevant forces in the model at the proper scale dictated by the scaling (similitude) criteria. Le Mehaute (1976) identifies scale effects as the error occurred due to unsatisfactory reproductions of some phenomena in the smaller scale of the model compared to the prototype. Hughes (1993) defines laboratory effects as the differences occurred in the physical modelling between prototype and model response due to limitations of the laboratory facilities such as wave and flow generation techniques, solid model boundaries etc. Incorrect reproduction of the prototype due to limitation of the model structure, geometry, model boundaries etc. leads to laboratory effects in the physical modelling study.

3 Experimental Setup

Physical model study of a composite seawall for wave energy conversion has been conducted at the Hydraulic Laboratory of the Southampton University, UK. A model of composite seawall (about 1.2 m long) has been developed in a geometric scale of 1:50 and placed at a rectangular wave tank (length 3.0 m, width 1.5 m and height 0.32 m). The wave tank was equipped with wave peddle and wave probes. The waves peddle and wave probes have been connected with the computer systems for wave generation and recording of wave parameters. Waves of specific parameters has been generated by the wave peddle systems. Software used in this system to generate waves was 'OWEL Drive and Collect', which was a batch data taker and collect program. The wave probes has been used to collect water level fluctuations with time, which was measured through the changes in the electric capacitance occurred due to water level fluctuations. The wave probes have been calibrated each day before going for model simulations. The software 'COLLECT32' was a John's 32-channel data collection program (1.01ASCII), which has been used to calibrate the wave probes to establish a relationship between water level fluctuations and changes in electric capacitance. The data recordings have processed in tables and graphs to find out the realized wave parameters with Microsoft Excel 2007. Wave parameters such as significant wave height, wave period etc. have been collected from the recording.Simulations have been conducted for three positions of the composite seawall with respect to the wave attack, which were normal wave attack (0^0angle) and oblique wave attacks (15^0 and 30^0 angles). The study was conducted for unbroken waves only. The volume of water entered into the water reservoir through overtopping has been measured by water level gauge (area of the reservoir was known) and mean overtopping rates have been calculated. All model input data and results have been converted into prototype input data and results for analysis.

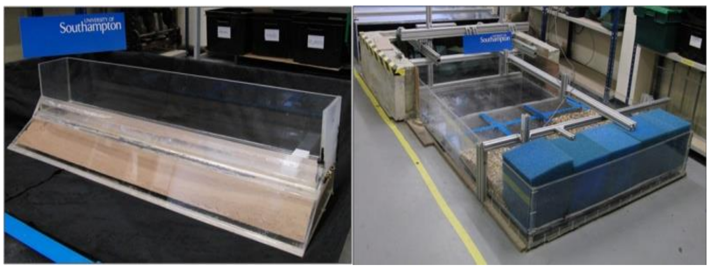

Figure 3: Model of Composite seawall (left) and Wave tank (right)

3.1 Scaling of Parameters

Scaling of the model dimensions, wave parameters (height, period and wave length) and overtopping rates were scaled following the Fraude scale laws. According to the Fraude scale laws, geometric scale of the model was chosen as 1:50 for an imaginary prototype. So, time scale and volumetric scale of the model became 1:7.07 and 1:125000 respectively. Prototype and scaled input of wave periods (time scaling) and freeboards (geometric scaling) are shown in the Table 1. Composite seawall concepts for wave energy conversion are suitable only for areas of low tidal ranges such as Mediterranean Sea and therefore, wave parameters and freeboards were chosen accordingly.

Table1: Prototype and scaled input of wave periods and freeboards

Wave period		Freeboard	
Prototype, sec	Model, sec	Prototype, m	Model, mm
4	0.57	0.5	10
6	0.85	1.0	20
8	1.13	1.5	30
		2.0	40

Frequency of the wave peddle movements has been calculated from the chosen wave period and corresponding amplitude was set from observations of the model simulation results by trial and error basis to set it for a specific significant wave height (H_s) generation. Three wave heights were chosen to generate in simulation of each wave period, ranged from 0.5 to 2.5 m (10 to 50 mm in the model scale). But realized wave periods and significant wave heights of the model simulations have been collected from recording after each simulation. Wave length has been calculated from the simulation results and water depth. All model results have been converted into full scale prototype results for analysis. In each simulation, model overtopping rate (q_m) was measured in $m^3/sec/m$ and then this has been converted into prototype overtopping rate (q_P) according to Fraude scale laws in the following way.

$$q_P = 50^{1.5} \times q_m \tag{2}$$

3.2 Wave Overtopping Prediction

Overtopping of the 3-D physical model has been predicted by *Owen* (1980) and *Van der Meer and Jassen* (1995) and compared with the measured overtopping rates. Aims of these formulas are to minimise the overtopping of the sloped coastal structures while aims of composite seawall are to maximise the overtopping in order to create hydraulic head as high as possible for energy conversion. Both of the formulas predict wave overtopping rates at slopping coastal structures, which are described below.

Owen (1980)

Owen (1980) has proposed an overtopping formula of dimensionless overtopping rate (q) and dimensionless freeboard (R_c) for simple smooth impermeable and simply sloped seawall. This formula is originated based on extensive data set from model tests of sloped structures (Torch, 2004). *Owen* (1980) overtopping formula reads:

$$\frac{q}{gH_sT_m} = a_o \exp(-b_o \frac{R_c}{T_m\sqrt{(gH_s)}}\frac{1}{\gamma_r} \tag{3}$$

Valid only when $0.05 < \frac{R_c}{T_m\sqrt{(gH_s)}} < 0.30$

Where, q is mean overtopping discharge rate per meter of width of the structure

R_c is the freeboard of the structure

γ_r is surface roughness (γ_r= 1.0 for smooth surface)

T_m is the mean wave period at the toe of the structure

g is gravitational acceleration

H_s is the significant wave height at the toe of the structure

In *Owen* (1980) method, wave height is considered as post breaking wave height. *Besley* (1999) clarified the post breaking wave height as the significant wave height for correct overtopping based on physical model tests (Soliman, 2003). *Goda* (2000) has suggested that the wave height in the near shore should be considered as the wave height at the toe of the structure. *Franco et al.* (2009) have measured the wave height in the field 220 meters seaward of the structure and in the corresponding scaled distance in the model studies (Maravelakis, 2009).

a_o and b_o are two dimensionless empirically derived coefficient in this methods, whose values depend on the slope. *Owen* (1980) proposes values for different slopes, which are shown in the Table 2. Intermediate values were calculated by linear interpolation in this study.

Table 2: Values for empirical co-efficient a_o and b_o (Besley, 1999)

Slope of the seawall	Values of a_o	Values of b_o
1:1	0.00794	20.1
1:1.5	0.00884	19.9
1:2	0.00939	21.6
1:2.5	0.0103	24.5
1:3	0.0109	28.7
1:3.5	0.0112	34.1
1:4	0.0116	41.0

Besley (1999) (based on soliman, 2003) has recommended that *Owen* (1980) is applicable for smooth, simply slopping bermed seawall around UK coastline. Therefore he has proposed modification of the *Owen* (1980) formula for oblique waves, bermed slopes and surface roughness.

Eq. (3) has been used to calculate overtopping rate (q_{Owen}) as Owen prediction for a specific significant wave height (H_s), wave period (T_m) and freeboard (R_c) of the structure in the present study. The overtopping predictions have been calculated for smooth bed, straight slope and normal wave attack only.

Van der Meer and Janssen (1995)

*Van der meer*and *Janssen* (1995) has proposed different overtopping formulas for non-breaking and breaking waves on slopping structures, which has gone through minor changes from time to time. The new set of formulae relates to breaking waves and is valid up to a maximum which is in fact a non-breaking region. The rewritten overtopping formulas (Van der meer, 2002) for dikes are as follows.

$$\frac{q}{\sqrt{(gH_{mo}^3)}} = \frac{0.067}{\sqrt{tan\alpha}}.\gamma_b.\xi_o.\exp(-4.75\frac{R_c}{H_{mo}}.\frac{1}{\xi_o\gamma_b\gamma_f\gamma_\beta\gamma_v}) \text{ (for } \xi_0 > 2.0) \quad (4)$$

With a maximum (non-breaking condition)

$$\frac{q}{\sqrt{(gH_{mo}^3)}} = 0.2\exp(-2.6\frac{R_c}{H_{mo}}.\frac{1}{\gamma_f\gamma_\beta}) \quad \text{(for } \xi_0 > 2.0)$$

Where

q is overtopping rate per meter width of the structure

R_c is the freeboard of the structure

tanα is the average slope of the structure and the approaching seabed.

H_{m0} is significant wave height based on the spectrum $\sqrt[4]{(m_o)}$

g is the gravitational acceleration

γ_b, γ_f, γ_β and γ_v are correction factors for the presence of a berm, surface roughness, oblique wave attack and presence of a vertical wall on the slope respectively.

$\xi_o = tan\alpha/\sqrt{s_o}$is the breaker parameter, the above equations are valid for $\xi_o < 5.0$.

$s_o = H_{mo}/L_o$is the wave steepness, where L_o is the wave length in deep water.

Waves that have been generated in the present physical model study are non-breaking type; therefore equation (4) was used to calculate overtopping rates as *Van der Meer* and *Janssen* prediction for a specific significant wave height (H_s) and freeboard (R_c) of the structure. The values of the reduction factors in the eq. (4) were set as 1.0 as the model was smooth and impermeable and overtopping rates have been predicted for normal wave attack only.

3.3 Wave Overtopping Prediction

Overtopping Mean overtopping rates have been measured for each model simulation in $m^3/s/m$ and converted into full scale prototype mean overtopping rates using the eq. (2). Mean overtopping rates have been predicted by *Owen* (1980) formulas using eq. (3) and by *Van der Meer and Janssen* (1995) formula using eq. (4). Graphs are plotted for mean overtopping rates against significant wave heights (H_s). Characteristics of the mean overtopping against wave height and period has been evaluated and measured mean overtopping is compared with overtopping predictions. Mean overtopping has been measured for the composite seawall positions at 0^0, 15^0 and 30^0 angles with the incoming wave attack in the 3-D wave basin. Measured mean overtopping rates (q_M) are plotted against significant wave heights (H_s) for different positions of the composite seawall. Changes of the measured mean overtopping from normal wave approach to oblique wave approach have been evaluated carefully in the analysis.

3.4 Power Performance

It is assumed that the water reservoir of the composite seawall always remains full during operation. Properties of the wave including wave power (P_W) have been determined by the equations given in the following Table 3.

Table 3: Key equations of linear wave theory

Wave parameter	Shallow water (d/L<1/20)	Intermediate water (1/20≤d/L≤1/2)	Deep water(d/L>1/2)
Wave length (L)	$L = T.(gd)^{1/2}$	$L = \frac{gT^2}{2\pi} tanh\frac{2\pi d}{L}$	$L = 1.56.T^2$
Celerity (C)	$C = (gd)^{1/2}$	$C = \sqrt{(\frac{gL}{2\pi} tanh\frac{2\pi d}{L})}$	$C = 1.56.T$
Group celerity (C_g)	$C_g = C/2$	$C_g = nC, where$ $n = \frac{1}{2}(1 + \frac{\frac{4\pi d}{L}}{\sinh\left(\frac{4\pi d}{L}\right)})$	$C_g = C$
Energy density (E)	$E = \rho g\frac{H^2}{8}$	$E = \rho g\frac{H^2}{8}$	$E = \rho g\frac{H^2}{8}$
Wave power (P)	$P = EC_g$	$P = EC_g$	$P = EC_g$

In Table 3, d= water depth, ρ= water density, L= wave length, g= gravitational acceleration, and H= wave height

Hydraulic power of the water collected over the crest (P_{cr}) of the composite seawall has been calculated by the following equation.

$$P_{cr}= q_M R_c \rho g \qquad (5)$$

Where, q_M= Measured mean overtopping rates in $m^3/s/m$
R_c= Freeboard in m
ρ= Density of water in Kg/m^3
g= Gravitational acceleration in m^2/s

Hydraulic efficiency of the composite seawall has been calculated by the following equation in percentage.

$$H_{hyd}= P_{cr}/P_W \times 100 \ \% \qquad (6)$$

Obtained hydraulic power and calculated hydraulic efficiencies are presented in the tables and graphs. Graphs are plotted to evaluate hydraulic power generated and hydraulic efficiency of the composite seawall against significant wave heights (H_s) for different freeboard conditions.

3.5Observations

A Composite seawall for WEC has been modeled physically in the Hydraulic Laboratory of the Southampton University. The physical model has been simulated to an imaginary prototype condition and coastal characteristics. The model behaviors were closely and carefully observed during simulations. The physical characteristics of the model simulations such as reflection, refraction and shoaling of waves in the wave basin, impacts of the wave period and wave height on the overtopping, types and patterns of overtopping, swelling of the water in the wave basin due to oblique wave approaches, effects of the sides of the wave basin and length of the seawall compared to the width of the basin, etc have been observed during simulations and evaluated based on the (imaginary) prototype situations.

4 Results and Discussions

The 3-D physical model of the composite seawall for wave energy conversion has been validated and tested in the hydraulic laboratory of the Southampton University, UK. Model results have been converted into full scale prototype results in the presentable form, which are discussed in this chapter.

4.1 Overtopping with Normal Waves

Figures 4 to 6 show general trends of wave overtopping and validate suitability of the prediction formulas. General trends of the Figure 3 give an idea that wave overtopping increases with increasing wave heights, while it decreases with increasing freeboard. It can also be that marked minimum wave height (threshold level) is required for wave overtopping, which increases as the freeboard increases.Figure 5 and 6 show the wide variation of performance of the prediction formulas. *Owen* (1980) prediction varies widely, which may be because of overemphasis that has been given on wave period in the eq. (3). *Van der Meer* (1995) prediction seems to be more controlled and realistic, although it has disregarded wave period in the prediction eq. (4) of non-breaking wave overtopping. It can be marked that *Owen* (1980) predicts relatively higher overtopping than the measured overtopping, whereas *Van der Meer* (1995) predicts relatively lower overtopping than the measured overtopping.

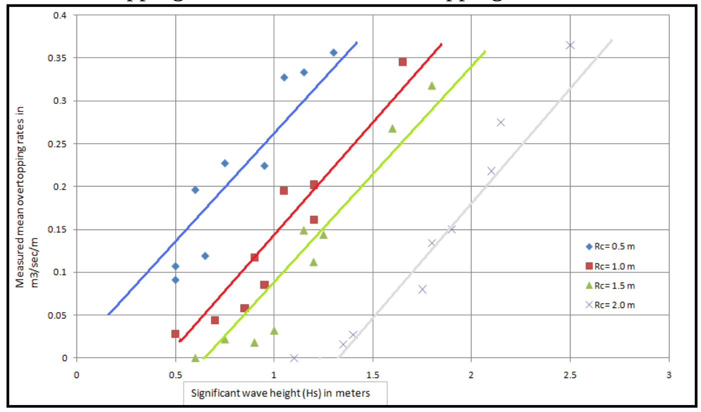

Figure 4: Measured overtopping at different freeboard conditions

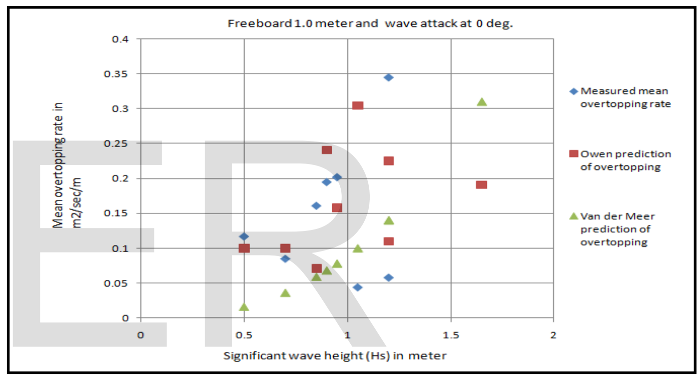

Figure 5: Comparisons of measured overtopping with predictions (Freeboard 1.0 m)

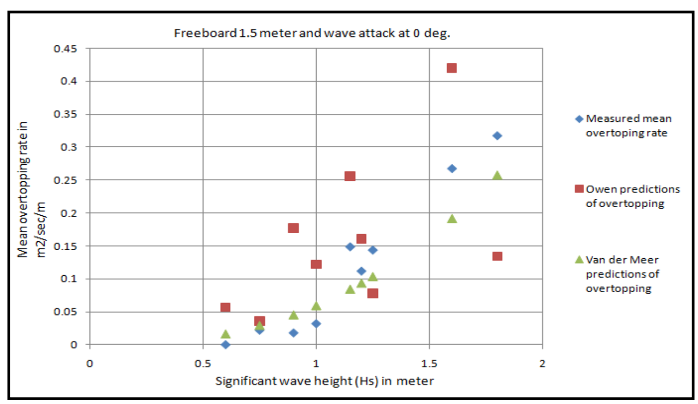

Figure 6: Comparisons of measured overtopping with predictions (Freeboard 1.5 m)

4.2 Hydraulic Perfomancewith Normal Waves

The measured maximum hydraulic efficiency of the composite seawall is 33.6 %. Average hydraulic efficiencies are about 26.6%, 18.6%, 15.9% and 11.1% for the freeboard of 0.5m, 1.0m, 1.5m and 2.0m respectively. Figure 7 shows the general trends of the realized hydraulic power at the crest. It shows that hydraulic power increases with increasing wave heights and decreases with increasing freeboard. Minimum wave height requirements for hydraulic power generation are about 0.4m, 0.6m, 0.8m and 1.2m for freeboard 0.5m, 1.0m, 1.5m and 2.0m respectively. Figure 8 shows that hydraulic efficiency decreases with increasing wave heights in case of freeboard 0.5m, while hydraulic efficiency increases with increasing wave heights in case of others freeboards. General trend of the hydraulic efficiency is that it decreases with increasing freeboards. Hydraulic efficiencies at 1.0 m and 1.5 m freeboards are more ordered and consistent than that at other freeboards.

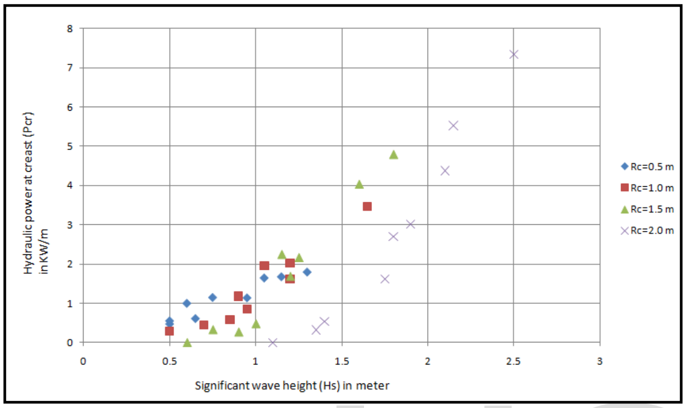

Figure 7: Hydraulic performance at significant wave heights

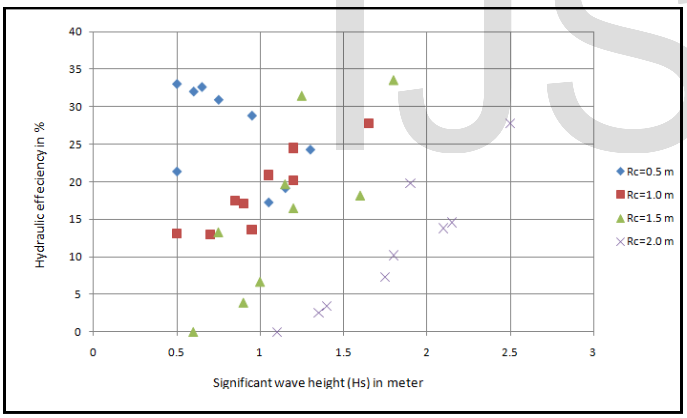

Figure 8: Hydraulic efficiency of the seawall at significant wave heights

4.3 Overtopping Performance at Oblique Waves

Mean overtopping at 0^0, 15^0 and 30^0 angles of wave approach to the composite seawall are measured for the freeboard of 1.0m and 1.5m. From the model results, overtopping clearly depends on freeboards, wave periods and significant wave heights. *Van der Meer* (1995) shows that overtopping decreases as the angle of wave approach increases and incorporates a reduction factor for oblique waves in the prediction formula. However, it is shown that effects of oblique waves are nominal up to 30^0 angle of approach and sets reduction factor as 1.0.Figure 9-10 show that overtopping decreases at 15^0 angles of wave approach, but increases again at 30^0 angles of wave approach.

Wave overtopping rate depends not only significant wave heights and angles of wave approach, but also on the wave periods (Figure 11).

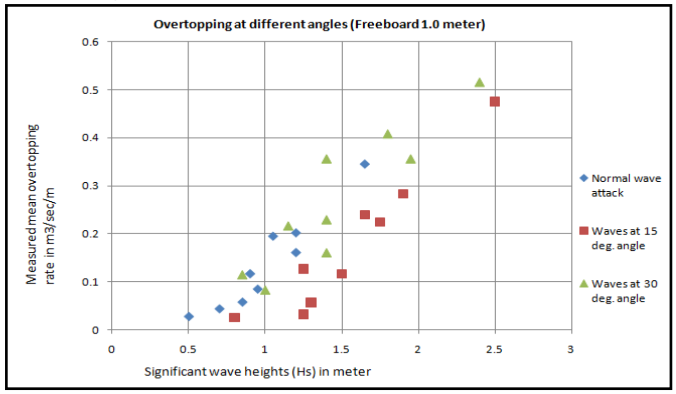

Figure 9: Wave overtopping at different angles (Freeboard 1.0 m)

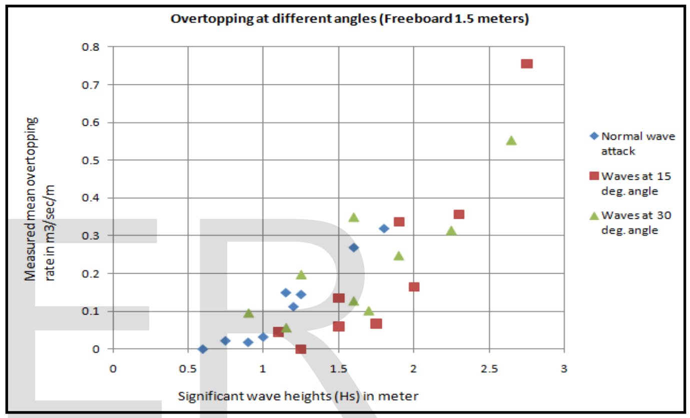

Figure 10: Wave overtopping at different angles (Freeboard 1.5 m)

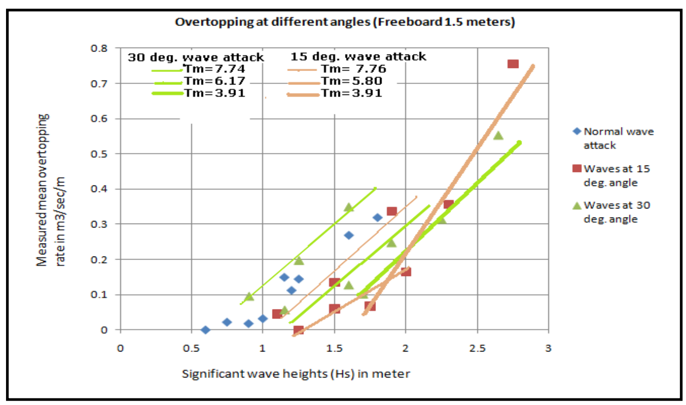

Figure 11: Wave period dependence of overtopping (Freeboard 1.5 m)

Higher wave period increases overtopping. *Owen* (1980) includes wave period in the prediction calculation (but seem to be over emphasized) and *Van der Meer* (1995) ignores wave period in non-breaking wave overtopping prediction. This is a reason of wide variations of performances of the overtopping prediction formulas (Figure 4-6).

4.4 Hydraulic Performance at Oblique Waves

Hydraulic performances of the composite seawall at 0^0, 15^0 and 30^0 angles of wave approach are presented here for 1.0m and 1.5m freeboards in order to look at the differences at different angles of wave approach. Maximum hydraulic efficiencies are measured about 36.5 %, 33.6% and 28.8% at 15^0, 0^0 and 30^0 angle of wave attack respectively. Figure 12-13 show the hydraulic power that are generated at the crest of the seawall at different angles of wave approach for freeboards 1.0 m and 1.5 m. Hydraulic power increases with the increasing wave heights in both freeboards. Hydraulic power generation decreases at 15^0 angle of wave approach, but increases at 30^0 angle of wave approach. The data of the hydraulic power at 30^0 angle of wave approach are more scattering and this scattering is increased at 1.5 m freeboard (Figure 13). Figures 14-15 are graphical presentations of the realized hydraulic efficiencies at various significant wave heights of 0^0, 15^0 and 30^0 angles of wave attack for freeboards 1.0m and 1.5m. These two graphs are rather more scattered, but the general trend is that hydraulic efficiency decreases with increasing angle of wave approach.

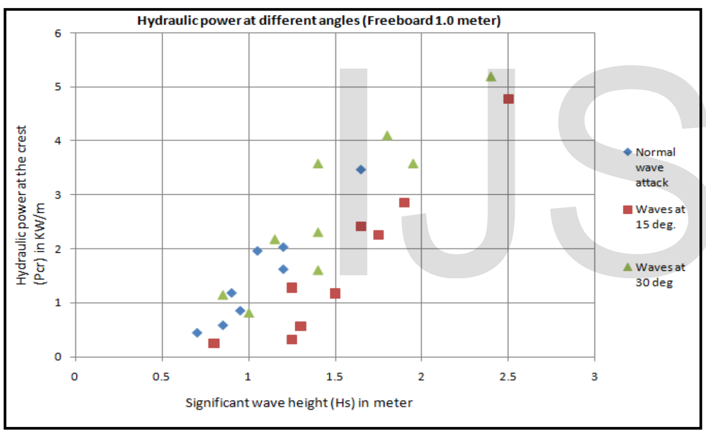

Figure 12: Hydraulic power at different angles (Freeboard 1.0 m)

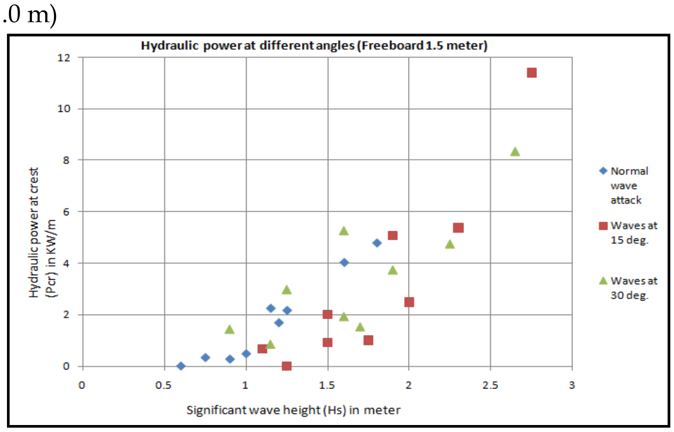

Figure 13: Hydraulic power at different angles (Freeboard 1.5 m)

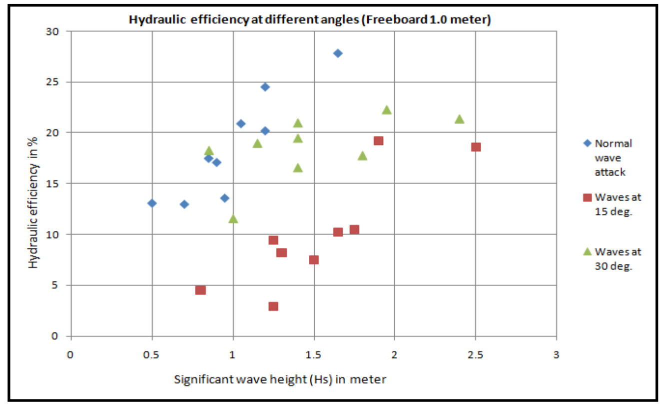

Figure 14: Hydraulic efficiency at different angles (Freeboard 1.0m)

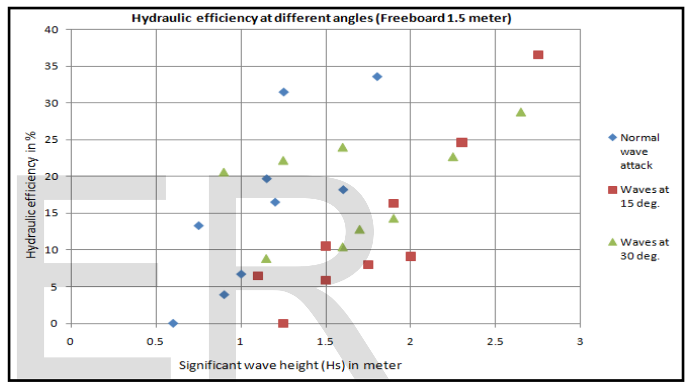

Figure 15: Hydraulic efficiency at different angles (Freeboard 1.5 m)

4.5 Experimental Observatons

The interactions between waves and the structure have been observed during the simulation experiments. The water depth in the wave basin was neither shallow nor deep in these tests considering wave parameters. Overtopping was non-breaking and it had not been uniformly distributed over the length of the ramp, especially in case of oblique wave approaches.

Wave shoaling has occurred on the slope of the structure and wave reflection on the ramp of the composite seawall has produced reflected waves while overtopping over the ramp. Reflected waves have interacted with the incoming waves and have generated standing waves. Properties of the standing waves (either perfect or imperfect) have depended on the wave periods and position of the composite seawall with respect to incoming waves. After 4-5 cycles of wave (as distance between wave peddle and ramp of the seawall was about 2.0 m), the reflected waves again has reflected on the wave peddle and thus the whole wave basin has become fully random.

While overtopping of the waves over the ramp and water has entered into the reservoir, sloshing of the reservoir water was occurred. Relatively higher waves have reflected on the vertical seawall behind the ramp after overtopping, which has generated irregular movement of the reservoir water. This reflection has caused spilling of the reservoir water back into the wave basin, especially when reservoir was full.

Positions of the composite seawall in the wave basin were set at angles of 0^0, 15^0 and 30^0 with the incoming waves. Oblique position of the composite seawall along with the side of the wave basin was looked like a funnel shape. Waves have converged in this funnel shape portion and have created swelling of the water surface, which has increased wave height in the narrow portion of the wave basin. This funnelling effect has become very important in overtopping and it has caused bulk overtopping at the corner of the composite seawall. Oblique wave approach has normally reduced overtopping in the real sea state, while funnelling effect has increased overtopping in the 3-D physical model. These effects have become vivid at 30^0 angle of wave approach and were the main cause of increasing overtopping performance and hydraulic power performance of the composite seawall, while it should be decreasing (Figure 7-8 and Figure 14-15).

The funnelling effect has occurred because of errors in model set up; length of the composite seawall (1.2 m long) was very large compared to the width of the wave basin (1.5 m wide). Although the model was set for 3-D conditions, it has seemed acting as 2-D model and was producing funnelling effects.

To reflect the 3-D physical modelling of a real sea state condition, the length of the composite seawall should be smaller than the used one. Because of that there is no barrier like sides of the wave basin in real sea state and hence oblique waves do not create funnelling effects in the real sea state conditions.

5 Conclusions

From analysis of the model simulation results, the following conclusions can be made:

- Overtopping Prediction formulas (*Owen* (1980) and *Van der Meer* (1995)) are widely varying in predictions. However, *Van der Meer* (1995) is found more ordered and relatively good than *Owen* (1980) in predictions.
- Maximum hydraulic efficiency of the composite seawall is about 37 %. Average hydraulic efficiencies are about 26.6%, 18.6%, 15.9% and 11.1% for the freeboard of 0.5m, 1.0m, 1.5m and 2.0m respectively. Hydraulic power (generated at the crest of the ramp) increases with increasing wave heights and decreases with increasing freeboard.
- Minimum wave heights required for hydraulic power generation are about 0.4 m, 0.6 m, 0.8 m and 1.2 m for freeboard 0.5 m, 1.0 m, 1.5 m and 2.0 m respectively. Hydraulic efficiencies at 1.0 m and 1.5 m freeboards are more ordered and consistent than that at other freeboards for given Mediterranean Sea climatic conditions.
- Both overtopping and hydraulic power generation decrease at oblique waves. Maximum hydraulic efficiencies are measured about 33.6%, 36.5% and 28.8% at 0^0, 15^0 and 30^0 angle of wave attack respectively.

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References

[1] Besley, P., Wave overtopping of seawalls: design and assessment manual, Hydraulics Research Wallingford, R &D Technical Report, W178, ISBN 1 85705 069 X, 1999.

[2] Canning, A., Investigation into the use of composite seawalls for power generation and energy dissipation in extreme conditions, School of Civil Engineering and the Environment, University of Southampton, 2009.

[3] Dalrymple, R.A., Introduction to physical models in coastal engineering, in physical modeling in coastal engineering, Edited by Dalrymple R.A., pp.3-9, 1985.

[4] Hughes, S.A.,*Physical models and laboratory techniques in coastal engineering*. World Scientific, Singapore, 1993.

[5] Kamphuis, J.W.,*Introduction to Coastal Engineering and Management*. World Scientific, Singapore, 2002.

[6] Kane, M., California small hydropower and ocean wave energy resources, staff paper presented at California Energy Commission, California, 2005.

[7] Kofoed, J. P., Model testing of the wave energy converter Sea wave Slot-Cone Generator, Hydraulics and Coastal Engineering No. 18, ISSN: 1603-9874, Dept. of Civil Eng., Aalborg University, 2005.

[8] Kortenhaus, A., Van der Meer, J., Burcharth, H.F., Geeraerts, J., Pullen, T., Ingram, D. and Troch, P., CLASH workpakage 7: Quantification of measurement errors: model and scale effects related to wave overtopping,Leichtweib Institute for Hydraulics, Germany, 2005.

[9] Le Mehaute B., Similitude in coastal engineering. Journal of the waterways, harbors and coastal engineering division 102, WW3, pp.317-335, 1976.

[10] Mori, M., Yamamoto, Y. and Kimura, K., Wave force and stability of upright section of high mound composite seawall. ICCE, Hamburg, 2008.

[11] Maravelakis, N., Experimental investigation of a novel composite seawall for wave energy conversion. School of Civil Engineering and the Environment, University of Southampton, 2009.

[12] Muller, G., Canning, A., Magagna, D., Stagonas, D. and Warbrick, D., Composite seawalls for wave energy conversion. School of Civil Engineering and the Environment, University of Southampton, 2009.

[13] Svendsen, I.A., Physical modeling of water waves, in physical modeling in coastal engineering. Edited by Dalrymple R.A., pp.13-47, 1985.

[14] Soliman, A., Numerical study of irregular wave overtopping and overflow. PhD thesis, School of Civil Engineering, University of Nottingham, 2003.

[15] Troch, P., Geeraerts, J., Walle, B.V., Rouck, J.D., Damme, L.V., Allsop, W. and Franco, L., Full-scale overtopping measurements on the Zeebrugge rubble mound breakwater. Journal of Coastal Engineering 51, pp. 609-628, 2004.

[16] Van der Meer, J.W., Conceptual design of rubble mound breakwaters. The Netherlands, 1993.

[17] Van der Meer, J.W. and others, Technical report: wave run-up and wave overtopping at dikes. Technical advisory committee on flood defence, Delft, 2002.