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Overtopping performance of Sea wave Slot cone Generator

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Introduction

For a healthy growth of a renewable energy economy, the differentiation of resources is fundamental to achieve sustainability and reliability. The wave energy resource represents a huge potential for the future of renewable energy and different wave energy technologies are already competing in the market. It is obvious that nowadays the main challenges are component survivability and the cost per kWh of the produced electricity. The demand for reliable, effective and economically favourable concepts within wave energy is not yet fulfilled: energetic seas expose the structures to very high loads increasing costs to satisfy survivability.

The Sea-wave Slot-cone Generator (SSG) is a wave energy converter of the overtopping type: incoming waves overtop a multiple level structure and water is temporarily stored in reservoirs at a higher level than mean water level offering the chance to exploit the potential energy in the stored water by mean of specifically designed low head hydro turbines. Other overtopping devices are Wave Dragon and Wave Plane, both floating devices for offshore applications. The SSG can be suitable for onshore and breakwater applications, presenting particular advantages such as:

- Sharing of costs of the structure.
- Availability of grid connection and infrastructures.
- Recirculation of water inside the harbour as the outlet of the turbines is on the rear part of the device.

Part of the SSG concept is the Multi Stage Turbine, which is able to utilize several heights of water on one turbine wheel. It does only have one shaft and only require one generator and grid connection system for all reservoirs (Fig. 1).

Comprehensive studies for onshore and breakwater applications took place from 2004 to 2008. Results include knowledge of loads, optimal geometry for power capture, expected power production as well as construction and installation (Kofoed, 2006; Vicinanza and Frigaard, 2008; Margheritini et al., 2008; Oever, 2008). A simulation program WOPSim 3.01 for overtopping of WECs has been realized (Meinert, 2008) in an attempt of generalizing the performance results. The main inputs for the simulation program are geometry, wave and tide conditions and turbine strategy, characteristics and control. The outputs of the program are, among others, water flow into reservoirs, spill out water flow from reservoirs, flow through turbines, power production, efficiency of different steps and overall efficiency. The parameters influencing the efficiency and consequently the power production for one multi-level overtopping device of the SSG kind are both geometrical and related to the wave-tide climate. The present paper aims at explaining the influence on the overtopping of different parameters and draw conclusion on performance of the device. The results are derived both from laboratory tests in different rounds as well as numerical simulation with WOPSim3.01.

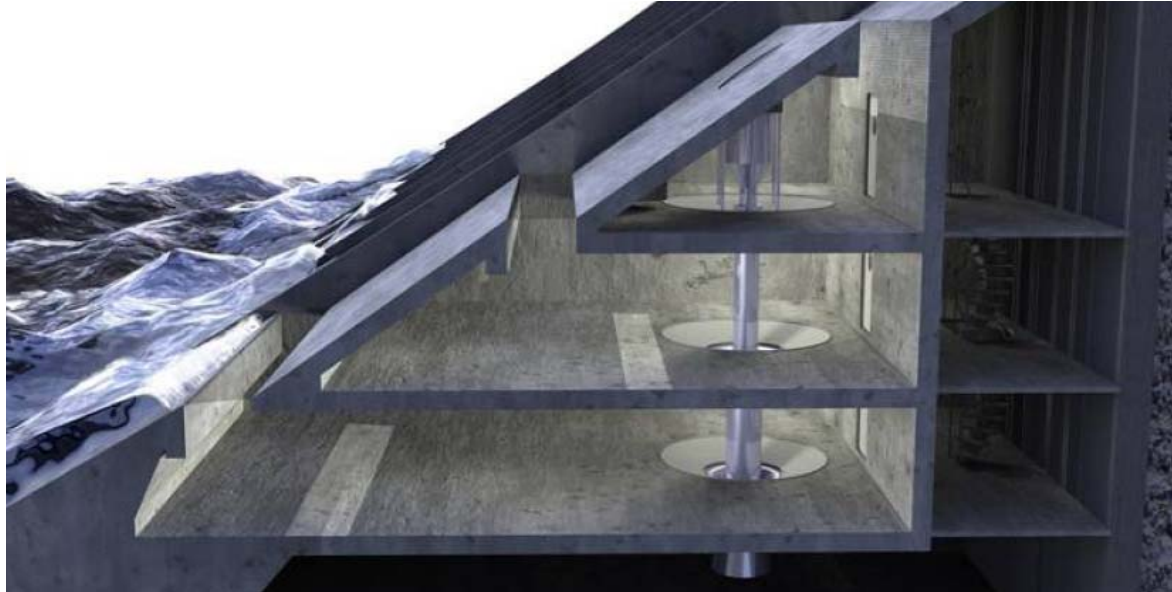


Figure 1. Artistic representation of one 3-level Sea Slot cone Generator mounted as a breakwater with multistage turbine.

The working principle of the SSG device is briefly presented in the following chapter. This will help to relate the overtopping to the efficiency of the device. Subsequently the parameters influencing the overtopping of a fixed multi-level overtopping WEC will be presented, following the order of investigation in time for the optimization of the device. Results will be presented both as average overtopping discharge and hydraulic efficiency.

Working principle

An overtopping device accumulates the water in a number of reservoirs at a higher level than sea water level optimizing the storage of potential energy in incoming waves. The design of the SSG device consists of a front ramp that leads the waves to different levels depending on the incoming wave height. Each level has a front ramp inclined of 30° allowing short term storage of water before turbine utilization. The ramp slope has been found to be optimal for maximisation of the overtopping (Le Mèhautè et al., 1968; Kofoed, 2002) (Fig. 2). The crest levels $R_{c,j}$ are optimized based on the wave and tide conditions at location. The idea is that waves run up the front ramp without losing much energy and reach the first reservoir where part of the overtopping water will be stored. If there is enough energy left, the water will run up the second front too and reach the second reservoir, being then stored at a higher level i.e. with a higher potential energy. The stored water on its way back to the sea passes through turbines (or possibly in Multi Stage Turbine in the SSG case) and the energy transformation to electricity is completed. It is clear, then, that hydraulic efficiency is directly proportional to the overtopping water flow in to the reservoirs:

$$\eta_{Hyd} = \frac{P_{crest}}{P_{wave}} = \frac{\sum_{j=1}^n \rho g q_{ov,j} R_{c,j}}{\frac{\rho g^2}{64\pi} H_s^2 T_e} \quad (1)$$

where $R_{c,j}$ = crest height of the j -reservoir (j = counter of reservoirs, $j=1,2\dots n$, n = number of reservoirs) related to the MWL , ρ = density of the sea water $\approx 1025 \text{ Kg/m}^3$, g = gravity $\approx 9.82 \text{ m/s}^2$. H_s is the significant wave height and T_E is the energy period of incoming waves, $q_{ov,j}$ is total overtopping flow rate to the j -reservoir.

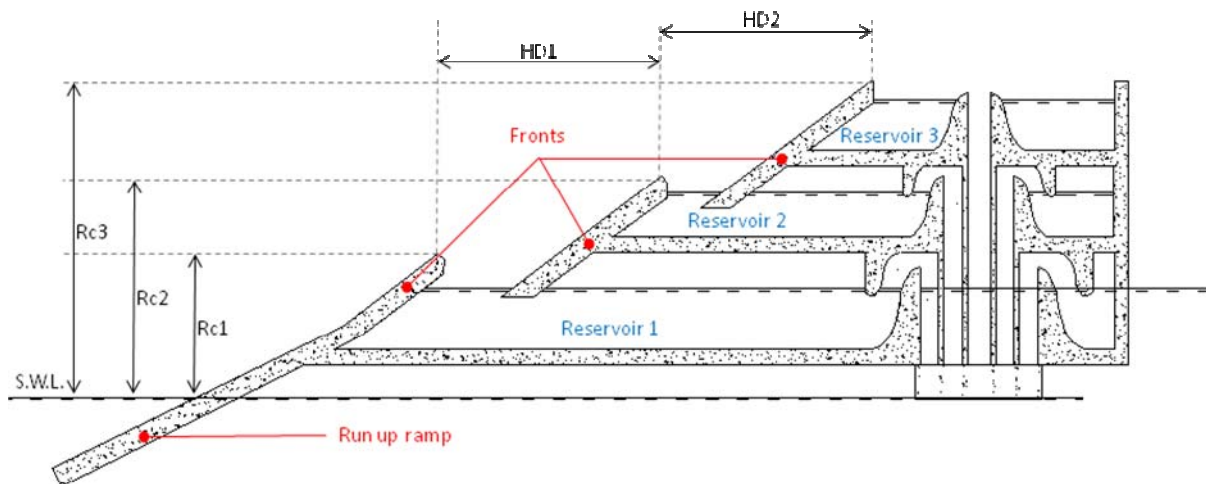


Figure 2. Definition sketch for a 3-level structure.

Influence of front ramp and crest levels

The front ramp leads the waves to the reservoirs. Its extension influences the overtopping flow to the upper reservoirs: it appears that the overtopping increases with increasing the front ramp length (for the same slope angle). The solution giving the highest overtopping flows is to extend the front ramp close to or to the bottom. From a construction/installation point of view it may be not convenient to realize such a slope that extends to the bottom, especially for breakwater SSGs which typically would be of the caisson type. In this case the front ramp could be cut off vertically to the bottom from a certain level d_r below swl. This has of course an influence in the overtopping discharge that it is possible to quantify.

The crest levels are the most important parameter to be defined in an overtopping device. Depending on the wave and tide climate at location, the crest levels must be worked out in order to harvest the most of the potential energy available. Of course, the final design will feature a fixed geometry that will result therefore optimal for a specific wave state (the most probable and energetic).

For a single level structure the overtopping is well described in literature among others by the Van der Meer and Janssen (1995) expression:

$$\frac{q}{\sqrt{gH_s^2}} = 0.2 \cdot e^{-2.6 \frac{R_c}{H_s} \gamma_r \gamma_b \gamma_n \gamma_\beta} \quad (2)$$

where R_c = crest height of related to the MWL, g = gravity acceleration, H_s is the significant wave height and the γ coefficients have been included to take care of the influence of the roughness, berm, shallow foreshore and angle of wave attack. According to Van der Meer and Janssen (1995) the expression is valid for $\xi_{p0} \geq 2$, with ξ_{p0} being the surfing similarity parameter defined as $\tan \alpha / (2\pi H_s / g T_p^2)^{0.5}$, T_p peak wave period.

Kofoed (2002) made tests for single level floating structure with varying front ramp angle α and length d_r and obtaining correction factors to be implemented in Eq. 2. Also a correction factor has been added to take into account of low relative crest freeboards (compare to Van der Meer and Janssen values).

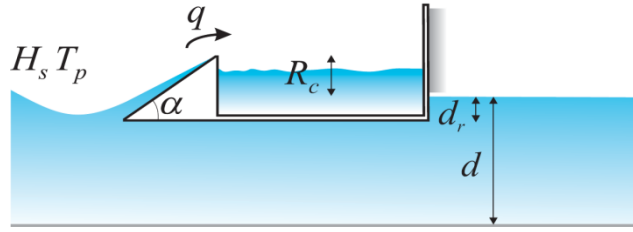


Figure 3. Definition sketch from tests performed by Kofoed (2002) to investigate the influence of ramp angle α , the length of the front ramp related to d_r and R_c on the overtopping discharge for single level floating structure.

The resulting expression obtained by Kofoed (2002) is presented as follow with indications of the corrections parameters mentioned above:

$$\frac{q}{\lambda_\alpha \lambda_{d_r} \lambda_s \sqrt{gH_s^2}} = 0.2 \cdot e^{-2.6 \frac{R_c}{H_s} \gamma_r \gamma_b \gamma_n \gamma_\beta} \quad (3)$$

Where

$$\lambda_\alpha = \cos^\beta(\alpha - \alpha_m) \quad (4)$$

Equation 4 is formulated so that λ_α is equal to 1 for optimal slope angle and decrease the more the slope angle differs from the optimal. With $\beta = 3$, α is the inclination of the front ramp and $\alpha_m = 30^\circ$ and is the optimal slope angle for maximization of the overtopping. In Fig. 4 the dependency of the overtopping discharge on the front ramp angle is presented. The dotted line is Eq. 2 by Van der Meer and Janssen (1995) and the solid line is the potential fit with all the data points shown.

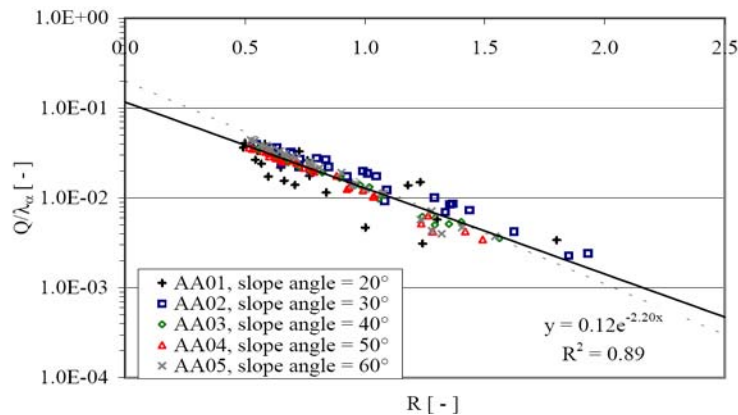


Figure 4. Tests results from Kofoed (2002) for single level structure with varying α , angle of the slope of the front ramp. The dimensionless average overtopping discharge Q is plotted as a function of the dimensionless crest free board $R=R_c/H_s$.

The λ_{d_r} coefficient that takes into account the length of the front ramp is expressed by Kofoed (2002) as follow:

$$\lambda_{dr} = 1 - k \cdot \frac{\sinh\left(2 \cdot k_p \cdot d \cdot \left(1 - \frac{d_r}{d}\right)\right) + 2 \cdot k_p \cdot d \cdot \left(1 - \frac{d_r}{d}\right)}{\sinh(2 \cdot k_p \cdot d) + 2 \cdot k_p \cdot d} \quad (5)$$

Where k_p is the wave number based on L_p and $k = 0.4$ is a coefficient controlling the degree of influence of the limited draft. The expression is based on the ratio between the average amount of energy flux integrated from the end of the ramp up to the surface and the average of energy flux integrated from the sea bed up to the surface.

Results are graphically presented in Fig. 5 and they show that when the ramp is extended to or very close to the bottom the overtopping is maximized ($dr/d \geq 0.75$), while it decreases for shorter front ramp length ($dr/d < 0.75$).

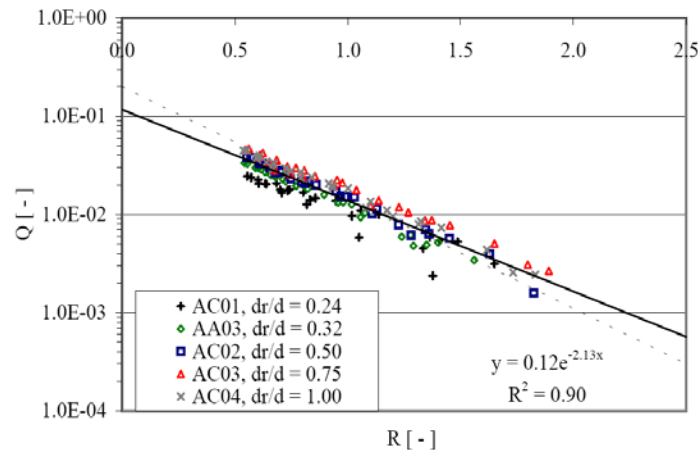


Figure 5. Tests results from Kofoed (2002) for single level structure with varying d_r , depth of the front ramp related to its length. The dimensionless average overtopping discharge Q is plotted as a function of the dimensionless crest free board $R = R_c/H_s$.

Modelling a floating structure, the set-up by Kofoed (2002) is different from a case of a front ramp of a fixed device where the energy instead of passing under the structure (transmitted energy) would be reflected back by a vertical wall (truncated ramp). This case is likely to bring less severe situation than the floating model, as the reflected energy will travel in the “wrong direction” (from the device instead of to the device) but will increase the local wave height at the structure and thereby also increase the overtopping, compare to the situation where the reflected energy is transmitted.

Specific physical model tests on the SSG optimization have been realized cutting vertically the front slope in 3 different points (Kofoed, 2005). Based on those results and averaging data from floating and structure and structure with truncated ramp, the loss on available power to the device has been calculated for 3 different locations for a breakwater SSG solution. The locations are of known wave power with different water depths (Sines 12 m, Swakopmund 11.3 and 6 m water depth). Results are presented as reference in Fig.6. The ratio of the overall hydraulic efficiency for truncated ramp and ramp extended to seabed is plotted for different truncation depths dr . Results are averaged over full range of wave and tidal conditions. It appears that there is a dependency more in the water depth than in the wave climate as the case of Swakopmund 11.3 m water depth and the case of Sines 12 m water depth present the same trend. This is mainly because we are in shallow waters and the energy in the waves is influenced by the interaction with the bottom. For water depth of 12 m. a truncation of the ramp at 8 m. results in a decrease of the available power in front of the structure of 10%, at 4

m. of 20% while not having a front ramp at all decreases the available power to 30%. For shallower waters the losses occur faster when decreasing the front ramp length.

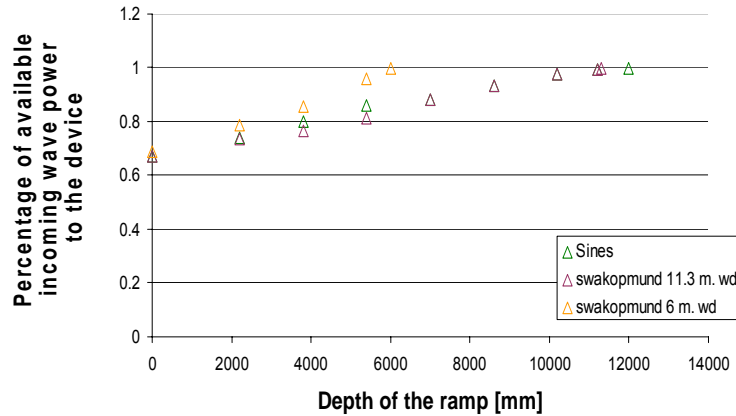


Figure 6. Reduction on available power to the device depending on the extension of the front ramp for the 3 locations under study.

Finally, the λ_s correction coefficient to take into account low R_{c1} has been introduced. This was necessary as the discrepancy with Van der Meer and Janssen (1995) (Eq. 2) is increasing when $R = R_c/H_s$ decreases from 0.75 to 0. The expression for the correction factor is then:

$$\lambda_s = \begin{cases} 0.4 \cdot \sin\left(\frac{2\pi}{3}R\right) + 0.6 & \text{for } R < 0.75 \\ 1 & \text{for } R \geq 0.75 \end{cases} \quad (6)$$

Once the waves have been efficiently led to the structure, it is convenient to have more than one level in order to maximize the power capture. Indeed, with only one level, the energy of small waves would be lost as they would most likely not be able to enter the reservoir and then would be reflected while the energy of bigger waves would be also partially lost when they fall in a reservoir that is lower than they H_s .

To obtain a formulation for the overtopping of a multilevel structure, the vertical distribution of the overtopping has been investigated by Kofoed (2002). Consequently, the dimensionless derivate of the overtopping discharge with respect of the vertical distance z (Fig. 7) is described by:

$$\frac{\frac{dq}{dz}}{\sqrt{g \cdot H_s^2}} = A \cdot e^{B \cdot \frac{z}{H_s}} \cdot e^{C \cdot \frac{R_{c,1}}{H_s}} \quad (7)$$

where coefficients A, B and C are fitted to experimental data for the specific case.

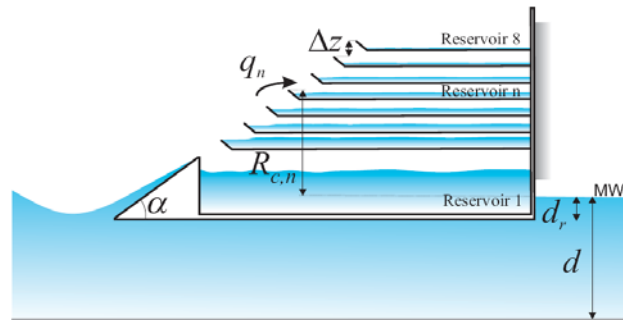


Figure 7. Definition sketch from tests performed by Kofoed 2002 to investigate of the overtopping for a multi-level wave energy device.

In the case of multi-level structure the overtopping performance is different for the different levels and is related to $R_{c,i}$ (Fig. 8) increasing while increasing H_s . Equation 7 predicts well the overtopping in the reservoirs despite giving an underestimation for the first level. In case of a fixed multi-level overtopping device the converter will result more efficient from a hydraulic point of view for sea states where the probability of occurrence of significant wave heights is spread more evenly along the sea states.

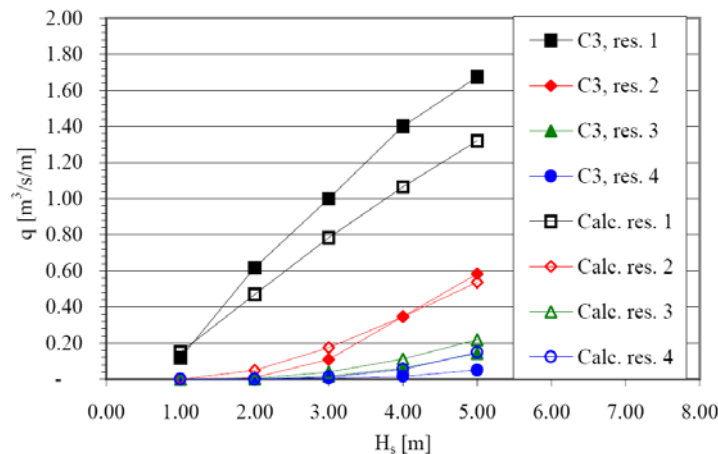


Figure 8. Comparison of measured and calculated values with Eq. 7 of the overtopping discharge for individual reservoirs q_n ($n=1, 2...4$) as a function of different sea states, (Kofoed 2002).

Influence of spreading and directionality

Further, laboratory tests have been carried out in a tank in order to investigate the effect spreading and directionality on the overtopping of a 3-level wave energy converter of the overtopping kind in scale 1:60 (Margheritini et al., 2008). The setup featured a model of the bathymetry of the area of installation, a part from the device itself. It has been found that spreading and directionality together decrease the overtopping and therefore the overall hydraulic efficiency from 40% in 2D conditions to 35% in average taking into consideration attack angles varying between -15° and 15° . In figures 9 and 10 the decrease of hydraulic efficiency when increasing wave spreading and attack angle for different wave conditions is shown.

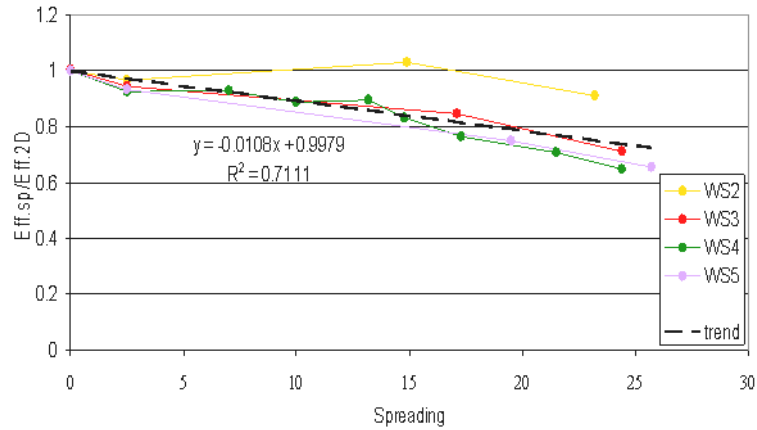


Figure 9. Influence of spreading on the hydraulic efficiency.

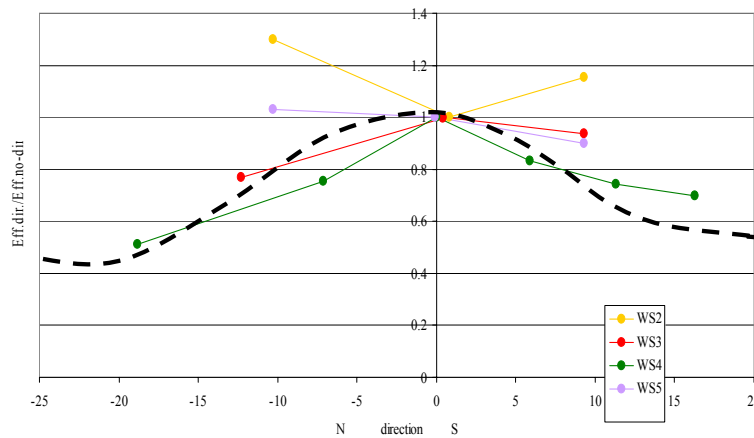


Figure 10. Influence of attack angle of incoming waves on the hydraulic efficiency. Dotted curve has been added to interpret the trend.

Influence of tide

The results of the present section are based on WOPSim 3.01 simulations (Meinert, 2008). Tide variation and distribution have an influence on the overtopping performance of a fix multi-level overtopping device. In particular the overtopping decreases with increasing the tidal variation for a selected geometry (Fig.11). Also, the higher the probability of occurrence of the water levels is spread evenly among the different conditions, the more the hydraulic efficiency is penalized. This is clear as it translate on a longer time that the device has to perform far away from its optimum. In figures 11 and 12, the lower curve represents the case of probability of occurrence evenly spread over 80% of the water levels, while for the higher curve only 30% of the water levels are covered by high probability of occurrence.

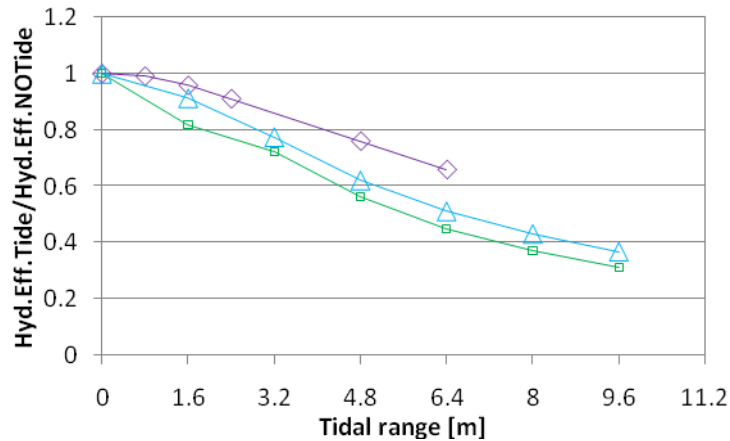


Figure 11. Decrease of hydraulic efficiency for different tidal ranges, for structures optimized for no tide with different tide distributions.

In average a tidal range of 3.2 m (± 1.6 m from s.w.l.) gives a loss in hydraulic efficiency of 21% (minimum 16%, maximum 27%) with little dependency on the sea conditions. For 4.8 m tidal range the loss in efficiency is in average 35% (minimum 24%, maximum 37.7%). It is possible to take into account the tide variations into the design of the device and therefore occur in minor losses especially for bigger tidal ranges (Fig. 12).

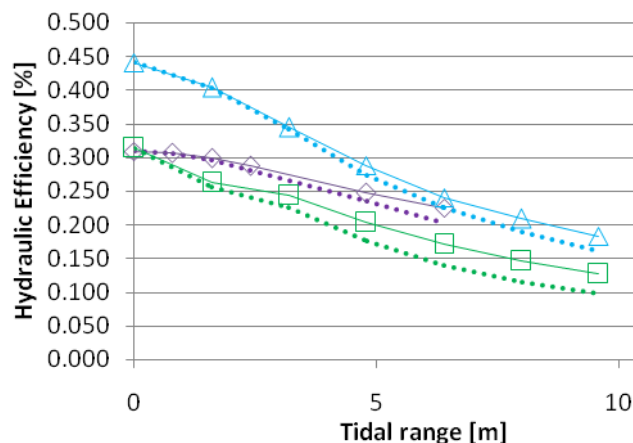


Figure 12. Comparison between hydraulic efficiency for geometries optimized (full line) and non-optimized (dotted line) for tide.

Influence of number of reservoirs

The results of the present section are based on WOPSim 3.01 simulations (Meinert, 2008) for the different locations (Swakopmund and Sines). The benefit of adding an extra reservoir has been earlier investigated by Kofoed 2002. It is obvious that adding an extra reservoir does not mean adding an extra level for specific wave climate but better optimizing the vertical space for power capture, i.e. the $R_{c,j}$. Kofoed results have been confirmed by a latest study on specific locations by Margheritini and Kofoed (2008a) and Margheritini and Kofoed (2008b): a structure with 2 reservoirs instead of 3 loses 15% in hydraulic efficiency in a no tide situation. A structure with 4 reservoirs instead of 3 gains 12% in hydraulic efficiency in a no tide situation while with 5 reservoirs instead of 4 gains 5% in hydraulic efficiency in a no tide situation (Fig. 12). These are definitely considerations that must be taken further for economical feasibility of the extra reservoirs. Adding a large number of reservoirs can increase the efficiency of the device but also has cost. The presence of tide is something that

must be taken into account in the design of a fixed overtopping wave energy converter. The contribution of tidal variations can be seen as a widening parameter for the distribution of sea states. For this reason the structure should be more flexible than in case of no tide for the same wave conditions. This can be achieved by adding a reservoir in case of tide so that the device is able to better optimized the power capture. The larger tide variations are the ones that have more gain when adding an extra reservoir. The gain for Sines in a simulated 9.6 m. tidal range is 17% compared to 12% of the case with no tide.

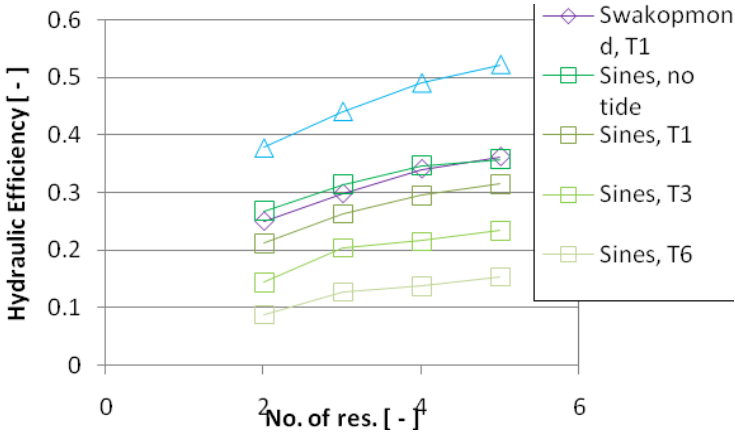


Figure 13. Comparison between hydraulic efficiency for different number of reservoirs. T1=0.8 m tidal range, T3=4.8m. tidal range, T6=9.6m .tidal range.

Influence of horizontal distances

The horizontal distances (*HD* in definition sketch Fig.2) between one reservoir and the other also influence the amount of overtopping in each of them. In the simple case of a 2-level structure, the overtopping into the lower reservoir increases when increases *HDI* while for the above level is obviously true the opposite (Fig. 14). The overtopping in reservoirs 1 will increase when increasing *HDI* while will decrease in reservoir 2. This variations occur within a limit that is set up by the equation of overtopping for single level structure (Eq. 3) considering $R_c = R_{c1}$ and R_{c2} respectively.

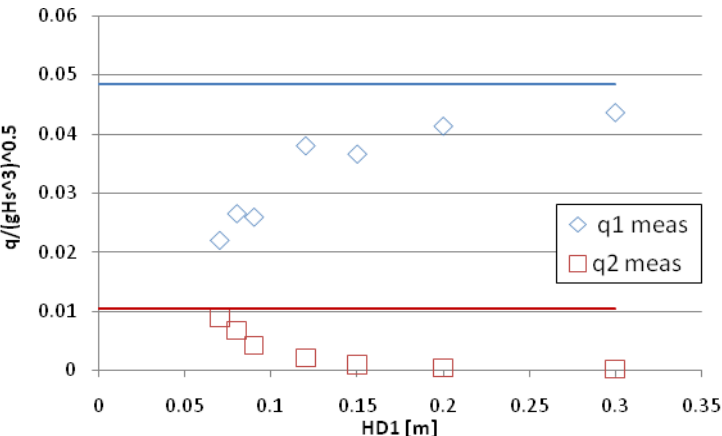


Figure 14. Overtopping discharges measured in low (q1) and top (q2) reservoirs 2-level structure, under wave conditions characterized by $H_s=0.067$ m and $T_p=2.9$ s. Upper limits calculated for R_{c1} (upper line) and R_{c2} (lower line) from Eq.3, for single level structure.

The total overtopping discharge decreases linearly for increasing *HDI*/ H_s . This is expectable

as it means that for smaller waves there is less overtopping (Fig. 15). Eq. 7 has been based on a single HD value equal to 7 cm and does not take into account the influence of the HD parameter HD despite having an influence on the overtopping flow rates. A new formulation of the overtopping expression is needed.

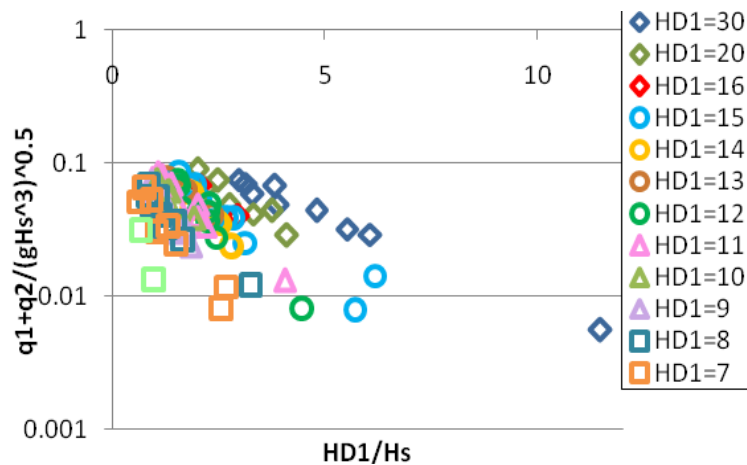


Figure 15. Sum of the measured overtopping discharges in the first and second reservoirs as a function of $HD1/H_s$ for different tested geometries varying $HD1$ in cm.

Influence of front angles

As well as the angle of the front ramp, also the front angles have an influence on the stored water in the different reservoirs. Kofoed (2002) investigated the influence of different angles on the overtopping discharge for a 4-level structure with angles varying between 20° and 50° . Not much difference emerges from the different tested geometries with hydraulic efficiencies varying from 26% to 31%. The higher calculated hydraulic efficiency corresponds to a milder inclination for the higher front than for the ones below, probably because the waves are facilitated to enter the higher reservoir that is having a milder front slope.

Conclusions

In order to have a complete knowledge on the overtopping performance of the SSG WEC, many parameters have been investigated by mean of laboratory tests and numerical simulations. A number of such results have been collected in this paper and indicate that:

- The angle of inclination of the front ramp influences the overtopping over the next levels. In particular it has been found that the angle that maximized the overtopping is $\alpha=30^\circ$, with little change between 30° and 40° .
- For an application of the SSG WEC on breakwaters it may not be possible to extend the front ramp to the bottom. A truncation of the front ramp at a certain depth with a vertical wall reflects energy resulting in a decrease of available power in front of the device i.e. decrease of the overtopping discharge into the reservoirs.
- Increasing H_s there is a direct increase on the overtopping discharge of the reservoirs.
- Spreading and wave directionality decrease the overtopping into the reservoirs from 40% in 2D conditions (with 3dimensional bathymetry) to 35% in average. To higher spreading and angle of wave attack corresponds decreasing in overtopping.
- For a fixed device, tidal ranges can decrease the overtopping into the reservoirs significantly, compared to a situation with the same wave climate and no tidal variations. Nevertheless tidal variation can be taken into account in the design of the device. This is particular efficient for tidal ranges larger than 2 m. (± 1 m.). For crest

levels design taking into consideration tidal variations, 6.9% loss on hydraulic efficiency for 1.6 m tidal range and 29.9% loss on hydraulic efficiency for 4.8 m. tidal range has been observed.

- Adding more reservoirs optimizes the power capture. Ideally, the device should have as many reservoirs as the different wave heights reaching the structure. This is obviously not possible. The cost effective improvement is when passing from 1 to 2 reservoirs and from 2 to 3 reservoirs, gaining respectively 20% and 15% on hydraulic efficiency, while passing from 3 to 4 and 4 to 5 reservoirs there is only a gain of 12% and 5%, respectively.
- Adding an extra reservoir can mitigate the downside of the effect of tide.
- The horizontal distance into reservoirs influences the overtopping in the two consecutive reservoirs in opposite ways.
- Front angles have little influence on the overtopping performance of the device despite the case with smaller inclination of the higher front shows slightly higher overtopping rates.

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