R&D Towards Commercialization of Sea Wave Slot Cone Generator (SSG) Overtopping Wave Energy Converter

selected topics in the field of wave energy

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R&D towards commercialization of the Sea wave Slot cone Generator (SSG) overtopping wave energy converter

PhD Thesis defended in public at Aalborg University (041209)

by

Lucia Margheritini

November 2009

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“Be the change you want to see in the world.” Mahatma Gandhi

Preface

This Thesis is submitted as one of the requirements set out in the Ministerial Order No. 1368 of December 7th, 2007 regarding PhD studies. The Thesis will be defended by a public lecture on the 4th of December 2009 at Aalborg University. This Thesis is presented as a collection of works published by the author on her research on the feasibility of full scale Sea wave Slot cone Generator wave energy converter. These include 1 accepted and 2 submitted journal papers; 7 peer-reviewed conference papers.

The author has been first employed at the Department of Civil Engineering at Aalborg University as Research Assistant on scour around monopole foundations for offshore wind turbines. It was in this occasion that by chance she first came into contact with wave energy.

Successively, the author has been employed as Research Assistant on wave energy at the Department of Civil Engineering at Aalborg University from 1st of June 2006 until the 29th of May 2009 for a total duration of three consecutive years. During this period, around 80% of the author research has been focused on one wave energy device, namely the Sea wave Slot cone Generator (SSG) collaborating with the developer WAVEenergy AS; the work included involvement in EU FP6 program, titled "Full scale demonstration of robust and high-efficiency wave energy converter" (WAVESSG). Being involved at different degrees on R&D of other wave energy devices, namely AquaBuOY, Wave Plane and Dexa, in the overall 95% of author´s research was focused on wave energy issues. Moreover the author has been able to participate to Wave Train seminars and in September 2007 spent one month at the Wave Energy Center, Lisbon under the Coordinated Action on Ocean Energy (CA-OE) mobility program for young researchers. The topics discussed in this Thesis delineated themselves during ongoing research on issues related to pre-commercial development of WECs. The present Thesis collects the author´s relevant work on the SSG WEC realized during these three years. This Thesis wouldn´t have been possible without the kind collaboration of WAVEenergy AS.

The author is deeply grateful to all the Wave Energy Group at Aalborg University, first of all Jens Peter Kofoed, for his great advices and commitment. She is also amazingly cheerful for all the people encountered through INORE network and the "wave energy community" as they do have special souls! The author would like to thank all the laboratory technicians for their help and kindness during the cold winters and fresh summers in the Lab, and the professor Diego Vicinanza from the Second University of Naples, for his assistance and enthusiasm. The author would like to thank particularly Professor Peter Frigaard and Monika Bakke for their friendship, as well as for their advices and collaborations.

Finally the author would like express her happiness for having such friends and companions to share her life with, and for having a family that is trying to understand and embrace her choices. Last but never least, the author would like to express her joy for the love, encouragement and motivation that Alex gave her: nothing would mean as much if she could not share it with him.

This is the beginning of something new.

Lucia Margheritini

Aalborg, November 2009

Would the first sin of science be to be certain of anything?

…Not that I consider myself a scientist at all, but this thought helped me a lot during my PhD.

The Author.

Abstract

Global energy needs are likely to continue to grow steadily for the next two and a half decades (International Energy Agency, 2006). If governments continue with current policies the world's energy needs would be more than 50% higher in 2030 than today. Over 60% of that increase would be covered in the form of oil and natural gas. Climate destabilizing carbon-dioxide emissions would continue to rise, calling into question the long-term sustainability of the global energy system. More vigorous government policies in consuming countries are steering the world onto an energy path oriented to reduce the consumption of fossil fuels and related greenhouse-gas emissions and to the development of Renewable Energy Sources (RES). Diversification of RES is fundamental in such a path to ensure sustainability. In this contest wave energy can provide great contribution, having its worldwide resource been estimated to be up to 10 TW (Engineering Committee on Oceanic Resources 2003; Cruz et al. 2008); depending on what is to be considered useful, this may cover from 15% to 60% of the World energy demand calculated for 2006. Indeed, together with the overall trend of all renewable energies, wave energy has enjoyed a fruitful decade. Improvement of technologies together with financial support at different levels gave space to new ideas, bringing the research to gamble on different concepts. While innumerable projects went through an initial testing phase that lasts 5-10 years, only few of them reached the sea prototype testing and eventually commercialization. After the phase of R&D developers had spent at least 15 mill Euro in average (Kofoed et al. 2008).

Good ideas can fail between the R&D and market stage. This event is described by Tom Delay, Head of the Carbon Trust, as "falling into the valley of death". This is the stage where the wave energy sector is. The limited ability of many ventures to attract private financing is certainly one of the major barriers. However, it is also very often a symptom of other underlying, and more fundamental issues. In reality, ventures fail to obtain funding because there are significant gaps between what the ventures are offering to investors and what the potential investors are seeking (Murphy and Edwards 2003). When risks associated to the investment is high, simply the deals often don't look very attractive. It is indeed necessary to reduce information gaps or asymmetries between ventures and private investors, and to promote an accelerated shift from a technology to a market focus.

This Thesis is presented as a collection of works published by the author on her research on the Sea wave Slot cone Generator wave energy converter. These include 1 accepted and 2 submitted journal papers; 7 peer-reviewed conference papers. The results are based on laboratory tests, numerical simulations and feasibility studies. Research presented in this Thesis contributes to reduce the technical and non-technical risks associated to the wave energy sector and promotes accelerated shift from technology to market focus. This has been done by using the R&D steps for a specific wave energy converter as an example of best practice for wave energy development towards commercialization.

The Sea wave Slot cone Generator (SSG) is a multilevel wave energy converter. Incoming waves overtop the structure and the water is temporarily stored in reservoirs at a higher level than sea water level. This water is returned through specially designed low head hydro turbines powering electrical generators. The device has been subject to 6 years of R&D at the Department of Civil Engineering of Aalborg University, involving the hydraulic performance such as geometric optimization for power capture and feasibility of the SSG-breakwater application. The issues under research led to close collaboration with Technical University of Munich (DE), for the turbine control and strategy; IKM Elektro for the operating procedures and generators (NO); WAVEenergy AS for the commercialization of the concept (NO); DNV for the insurance of the structure (DK); and Delta marine Consultants for the SSG-breakwater design (NL).

At the present stage of development of the SSG device, economical feasibility and reliability are at the first places on the ranking issues. The efficiency optimization is linked with the cost of the produced electricity. In the SSG device most of the optimization is done on the geometry as this has the biggest impact in the captured power and has the larger uncertainties. At the same time, the largest cost for

the device is the structure itself and therefore the amount of concrete utilized for its construction. Prediction of wave loading is indeed influencing both the reliability of the device and the final cost of electricity.

The most promising application for the SSG device is into breakwaters for harbor protection. Aspects related to the construction have also being reviewed in this work. The research carried out on this application demonstrated the device is economically feasible and competitive to OWC devices with the same application, offering moreover additional improvements to the protection.

Finally it must be noticed that due to the relative young stage of development of the entire sector (at least 10 years behind the offshore wind sector) frameworks and regulations for wave energy development are not fully ready. The majority of the Companies involved are small and unable to undertake time consuming consents processes. This may be the case also for the Environmental Impact Assessment (EIA) process. For this reason a study aimed at the simplification of the EIA of WECs, with particular reference to the scooping process, has been concluded. Based on the results, the potential environmental impact of the SSG device has been preliminary assessed.

Danske resume

Et element i kampen mod klimaændringer er udviklingen af alternative ikke-forurenende kilder til produktion af energi. SSG er netop en teknologi, som omdanner havets bølger til elektricitet. SSG konceptet fungerer ved at bølgerne løber op ad en rampe, over en kam og fanges i en række reservoirer. Det vand, som nu befinder sig på et højere niveau end havoverfladen, bringes tilbage gennem nogle lavt-tryks vandturbiner, og der udvindes elektricitet.

Nærværende afhandling indeholder en beskrivelse af forfatterens arbejde med tekniske og ikke-tekniske aspekter i forbindelse med udviklingen af SSG konceptet. Arbejdet er foregået i samarbejde med virksomhederne bag SSG teknologien. Her tænkes specielt på WAVEEnergy AS, hovedudvikleren og ejeren af SSG teknologien.Afhandlingen indeholder et udvalg af publikationer udført af forfatteren, og allerede offentliggjort. Publikationerne omfatter 3 tidsskrift papers, og 7 peer-reviewed konferencepapers.

Afhandlingen kan teknisk set opdelt i fire områder:

- Laboratorieundersøgelser af hydrauliske ydeevne af et SSG anlæg
- bølgebryder applikationer og sammenligninger med en OWC-teknologi
- økonomiske overvejelser
- og endelig miljøvurderinger.

Bølgeenergisektoren er i dag lang efter andre vedvarende energi teknologier som f.eks vindkraft. Bredden af forfatterens arbejde skal derfor ses i, og forstås i lyset af at SSG teknologien i de forløbne år er gået fra simpel ide til at være en af de seriøse bud på et kommende kommerciel vedvarende energiteknologi.

Contents

THESIS SUMMARY

Introduction

This summary is a stand-alone document which will introduce the reader to the topics presented in the author´s publications. The content can be divided in the following sections:

- background;
- objectives;
- general overview of the SSG concept;
- hydraulic characteristics of the device, including overtopping performance and efficiency, and wave loadings;
- application of the device into breakwaters, economical aspects related to this application and comparison with Oscillating Water Column (OWC) devices;
- environmental impact of SSG device based on new assessment methodology;
- recommendations for future research.

The background of the project shows the motivations behind the study while the objectives attempt to draw the attention to the general value of this work.

Each section is an extended summary and introduction to author´s publications and for each topic highlights the major conclusions. After providing the overview of the topics, the last section is dedicated to the author´s considerations on relevant future research related to the SSG device.

For the fullest details the reader is recommended to read the author´s selected publications re-printed in this Thesis.

Background

For different reasons our future energy supply should not depend on fossil fuels. The reasons that are acknowledged by concerned societies are: adverse impacts of climate change and local air pollution; volatility of world oil markets; wars and dependence of supply from political instable countries. Renewable energy (RE) is the alternative to fossil fuels. When shifting to a sustainable future diversification of renewable energy resources is the key to this change since it increases the share of indigenous energy and thus provides a more balanced and diversified energy mix. In this optic wave energy plays an important role both to supply energy to isolated communities but also to contribute to implementation of the resource as its potential is enormous: the useful worldwide resource has been estimated to be greater than 2TW *(*Committee on Oceanic Resources — Working Group on Wave Energy Conversion, 2003), corresponding to 15% of the total worldwide energy consumption and around 50% of the total electricity consumption (referred to 2005).

Cost of electricity from renewable energies, though, is roughly from 4 (wind power) to 60 (PV) times higher than the one from fossil fuels. It is difficult at the present time to estimate the cost for wave energy production; this is due to the enormous variety of devices (more than 90 different technologies worldwide) and to the lack of real sea data on power production. Nevertheless it appears clear that wave energy is no cheap technology. Devices have to face great challenges: prediction of wave resource, very high wave loading, demanding installation conditions and expensive grid connections infrastructures. For 10 years payback time, applications of OWC and SSG on breakwaters results in a cost of electricity < 0.33€/kWh (Margheritini&Frigaard 2009) meaning less than 10 times higher the cost from coal-fired power stations.

Exactly to fight the setbacks of the sector, studies presenting best practice for testing and comparing devices, real sea installations and cost of power production are particularly valuable. This is the case presented in this Thesis with the Sea wave Slot-cone Generator (SSG).

Objectives

The prime motivation of the study has been the development of the SSG device towards commercialization by reducing technical and non-technical risks. This through research based laboratory testing and numerical simulations and promoting accelerated shift from technology to market focus by collaborations with companies, other devices developers and investigating non-technical barriers. Beyond the prime motivation, a more general objective sits on the will of contributing positively to the development of the wave energy sector by drawing common conclusions form specific experience.

The specific objectives of this Thesis are:

1. **Demonstrate the best practice to the wave energy industry.**
   Denmark has a well acknowledged record of success promoting challenging technologies such as the offshore wind sector. The Danish government initiated in 1998 a special "Wave Energy Program" with focus on economical support of new inventions. In 1999 a commission `Bølgekraftudvalget` appointed by the Danish Government released a first protocol on testing and assessment of Ocean Energy Devices (Nielsen, 1999). Aalborg University has a long record on participating on the development of wave energy during the last 10 years.
   The developers of the SSG device accomplished a great level of development by mean of systematic research and collaborations with commercial firms.
2. **Advance research in wave energy**
   This Thesis contains technical advancements which are of interest of the wave energy and coastal engineering research: first, modeling the overtopping of low crested structures; second, by modeling wave loads on shoreline structure; third by featuring the first direct comparison between two wave energy technologies (SSG and OWC).
3. **Record the development of the SSG device**
   It is expected that common guidelines can be derived from the SSG experience and the value of recording both negative and positive results of the device will influence positively the wave energy sector in its totality.
4. **Demonstrate the feasibility of the SSG device**
   Feasibility of wave energy devices still needs to be demonstrated for the great majority of the concepts. Being the SSG mainly a shoreline or breakwater device, this limits the number of issues to face for its deployment. In the near future concrete contribution to the energy demand will come more from offshore installations rather than shoreline. Nevertheless it seems quite reasonable, considering the present status of technology, to turn to shoreline devices in order to address the first issues raised by the market such as feasibility and reliability.
5. **Address the EIA for the SSG device**
   EIA for wave energy devices is claimed by many not to be a relevant issue as the technologies have a small footprint on the natural environment. Nevertheless, due to the variety of technologies and the lack of knowledge of their interaction with the environment, EIA of WECs is nothing like an easy task. It is relevant to assess the impact of devices on the environment at an early stage also as a requirement for the commercialization phase.

In summary, this Thesis provides a record of the current situation of the SSG device, its performance in laboratory and it expected efficiency in full scale. Also, feasibility of the device on shore and on breakwaters is discussed. The author´s papers reprinted in this Thesis address these main topics. Finally, a list of future challengies from which the commercialization of the SSG device will benefit is presented.

This documentation will be of interest for wave energy developers and investors.

SSG wave energy converter

The Sea wave Slot cone Generator (SSG) multi-level overtopping device has been central to this Thesis. Intensive research, characterized by important laboratory testing (Fig. 1) and numerical modeling has been realized while the EU FP6 Pilot project was running. Once that this phase has been concluded, the development focused on feasibility and applications of the concept on breakwaters. Other devices working with the overtopping principle are, among others, Wave Dragon (Kofoed 2006a) and Wave Plane (Frigaard et al. 2008).

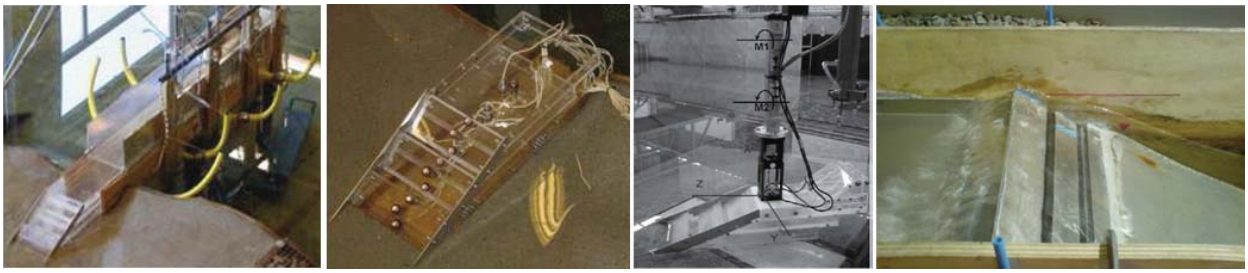


Figure 1. Images from laboratory tests on the SSG wave energy converter. From the left: 3D overtopping tests, 3D tests on wave loadings, 3D tests on overall forces and 2D tests on geometrical optimization of the fronts.

Concept description

The Sea-wave Slot-cone Generator (SSG) is a multi-level wave energy converter (WEC) of the overtopping type. It accumulates the water in a number of reservoirs at a higher level than sea water level optimizing the storage of potential energy of incoming waves. The stored water on its way back to the sea passes through specially designed low head hydro-turbines generating electricity. The energy extracted from a given volume of water in the reservoir is in direct proportion to its elevation above the mean sea level (turbine head). Different ventilation openings are included in the design of the structure in order to prevent air pressure to obstruct the water storage (Margheritini et al. 2006).

Part of the concept, but still under development, is the innovative concept of the Multi-Stage Turbine (MST) (Fig. 2). The design integrated in the structure consists of a number of turbines (depending on the number of reservoirs) staggered concentrically inside each other, driving a common generator through a common shaft. Each of the runners is connected to one of the reservoirs by concentric ducts. By taking advantage of different heights of water head, the MST technology is willing to minimize the start/stop sequences and operate even if only one reservoir is supplying water, resulting in a higher degree of efficiency.

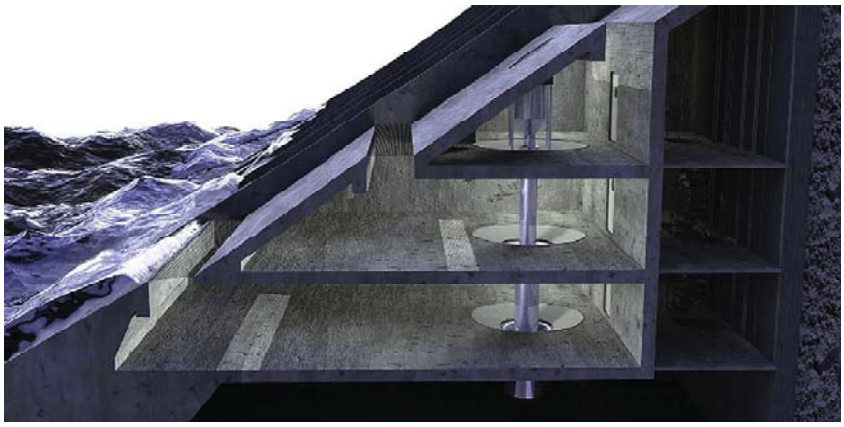


Figure 2. SSG wave energy converter with MST turbine.

Despite not denying a future possible offshore application of the concept, the developer focused on the shoreline and breakwater solutions as it is expected that fewer issues will be involved in the deployment: in particular easier installation, share of costs and infrastructures, lower forces on the structure are the advantages of these solutions. The main strength of the device consists on its robustness and low cost.

Pilot project

The Company WAVEenergy AS found in Stavanger, Norway, is developing the device (patented in 2003) since 2004 when the pilot project has been partially funded by the European Commission FP6-2004-Energy (WAVESSG project). The pilot project was meant to be built in the Island of Kvitsøy, Stavanger, Norway but at the very last phase, environmental concern was raised due to the required installation works that would see permanent alteration of the cliff at location. Consequently, public acceptance passed from being very positive to uncertain. In addition, wave climate at location resulted to be very challenging for a pilot device, featuring 19kW/m wave energy and 100 years return period waves of Hs = 12.5 m and Tp = 15.2 s, with almost prohibitive time window for installation. Based on these circumstances, decision was taken to close the EU project in early 2008. Since then, more than 4 different locations are under evaluation for the construction of the first full scale SSG device that is foreseen before 2011.

The optimization of the levels has been realized by mean of extensive laboratory tests and prediction on power production with numerical simulation with WOPSim 3.01 (Meinert 2008). 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 expected power production for the optimized device, 10 m wide and 3 reservoirs was 320 MWh/y.

An attempt of generalizing the specific results, during and after the EU project, converged in the implementation of the WOPSim overtopping simulation program Fig. 3. The latest version is a valid tool for the prediction of performance of overtopping wave energy converters.

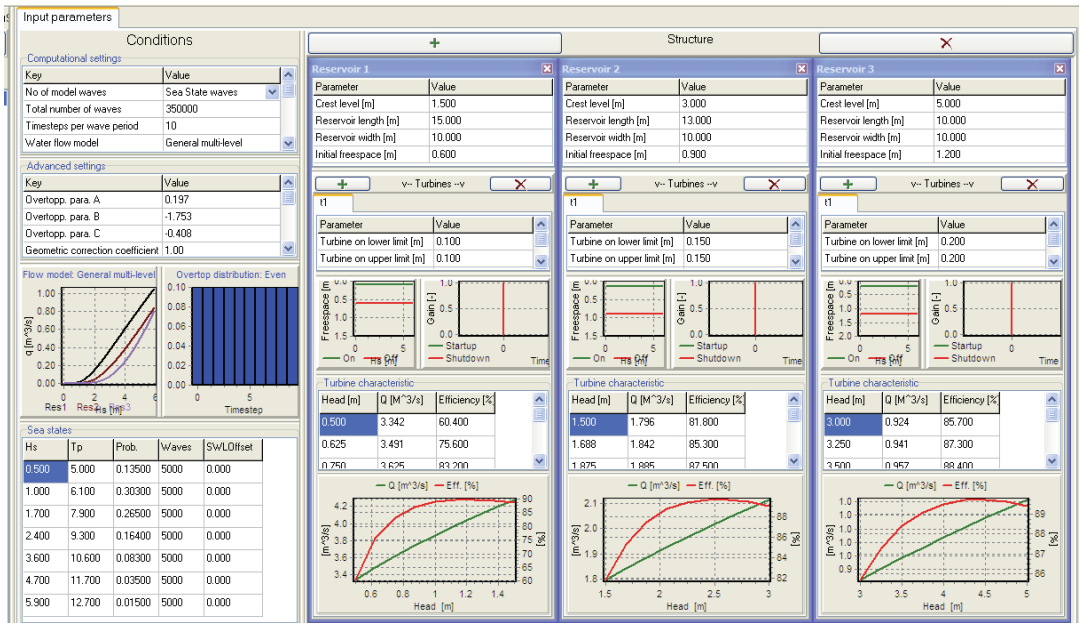


Figure 3. WOPSim 3.01, input page.

Conclusions

It has been demonstrated that the SSG device has the potential to considerably contribute to the generation of economical pollution free electricity.

Priceless experience has been gained toward the process meant to lead to the realization of the SSG Pilot plant in Kvitsøy and it can be said that all the steps previous installation have been successfully concluded: study of wave climate, optimization of the structure, turbine strategy and control, electrical equipment, instrumentation and data acquisition, operation procedure and installation method (Margheritini et al. 2009a, Margheritini et al. 2007, Margheritini&Kofoed 2006). If any further investigation should have been taken deeper into account, this regards the Environmental Impact Assessment of the device.

Hydraulic performance

The chapter on hydraulic performance of the SSG device includes overtopping performance and study on wave loadings. Both these aspects of the interaction wave-structure have direct influence on the cost of the power production. The influence on the overtopping has been examined from the geometrical parameters and environmental parameters such as directional wave spreading, wave directionality and tide. Hydraulic performance has been investigated by mean of laboratory tests and numerical modeling (WOPSim 3.01 overtopping simulation program, Meinert 2008).

Overtopping performance

Considerable increase on stored energy from the overtopping water can be obtained by using multilevel devices (Kofoed 2006); moreover this is the most effective solution for fix overtopping devices that can not adapt the crest free boards to the sea state by changing the buoyancy level like in Wave Dragon device.

The energy conversion steps in the SSG device (and more in general for overtopping devices) are as follow:

- wave to crests, where the different waves are captured at the crest heights of the reservoirs (Rcj, j=1, 2…n, n=number of reservoirs). It has been measured during different sets of laboratory tests on the SSG device that around 40% of the available energy is captured.
- Crests to reservoirs, where the potential energy relative to the specific crest heights is reduced by falling into the reservoir at a lower height. It is estimated that 75% of the energy from the previous step is maintained.
- Low head water turbines, where the water in the reservoirs is utilized by the hydraulic turbines with 98% efficiency.
- Electrical generator and electrical equipment, 95% efficiency.

The overall expected wave-to wire efficiency is 25%-35%.

Geometrical parameters

The geometrical parameters influencing the overtopping performance and then the power production for one single SSG module are (Fig. 4): length of the front run up ramp, length of the front of each reservoir, slope angle of the front run up ramp, slopes angles for the fronts of each reservoir, orthogonal distances between the fronts of 2 consecutive reservoirs, number of reservoirs and crest levels (Margheritini et al. 2009b) and reservoirs' length (Margheritini and Kofoed 2008a,b). Most of these parameters have been investigated for specific cases and limited conditions. Following the most solid result are summarized.

Identification of optimal crest levels for specific wave and tide climate is done through an iteration process target to the maximization of hydraulic efficiency defined as:

$$\eta Hyd. = {P_{crest}}/{P_{wave}} = \sum_{j=3}^{3} \rho g\, q_{ov,j} R_{c,j} \Big/ \frac{\rho g^2}{64\pi} H_S^2 T_E \qquad (1)$$

$$Q' = \frac{dq/dz}{\lambda_{dr}\sqrt{gH_s}} = Ae^{B\frac{z}{H_s}+C\frac{R_{c,1}}{H_s}} \qquad (2)$$

where Rc,j = crest height of the j-reservoir (j = counter of reservoirs, j=1,2…n, n= number of reservoirs) related to the MWL, ρ= density of the sea water ≈ 1025 Kg/m3 and g = gravity ≈ 9.82 m/s2. Hs is the significant wave height and Te is the energy period of incoming waves.

qov,j is total overtopping flow rate to the j-reservoir, calculated integrating expression (2) (Kofoed 2002), where Q´ is the dimensionless of the overtopping discharge with respect to the vertical distance z between Rc,j and Rc,1 is the crest freeboard of the lowest reservoir. λdr is a coefficient describing the dependency of the draught, in the current study set to unity, and coefficients A, B and C need to be fitted to experimental data for the specific case. The P_{crest} is the potential power in the overtopping water as it overtops the crest of the structure. The overtopping power is related to the crest height (Rc,j) relative to the MWL and the flow rate. The power in each reservoir is calculated as:

$$P_{\mathrm{res}} = \rho g H_j q_j \tag{3}$$

Where qj is the effective overtopping flow rate that stays in the reservoirs. It is important to notice that qov,j and qj differ because the first one includes also the volume of water that may actually spill out of the reservoir due to its limited capacity. Hj = water level in reservoirs relative to MWL.

Major downsides on this process regard the need for laboratory tests for the definition of A, B and C parameters in Eq. 2 as well as the optimum orthogonal distance between the fronts. Skipping the laboratory tests will result in an underestimation of around 30%.

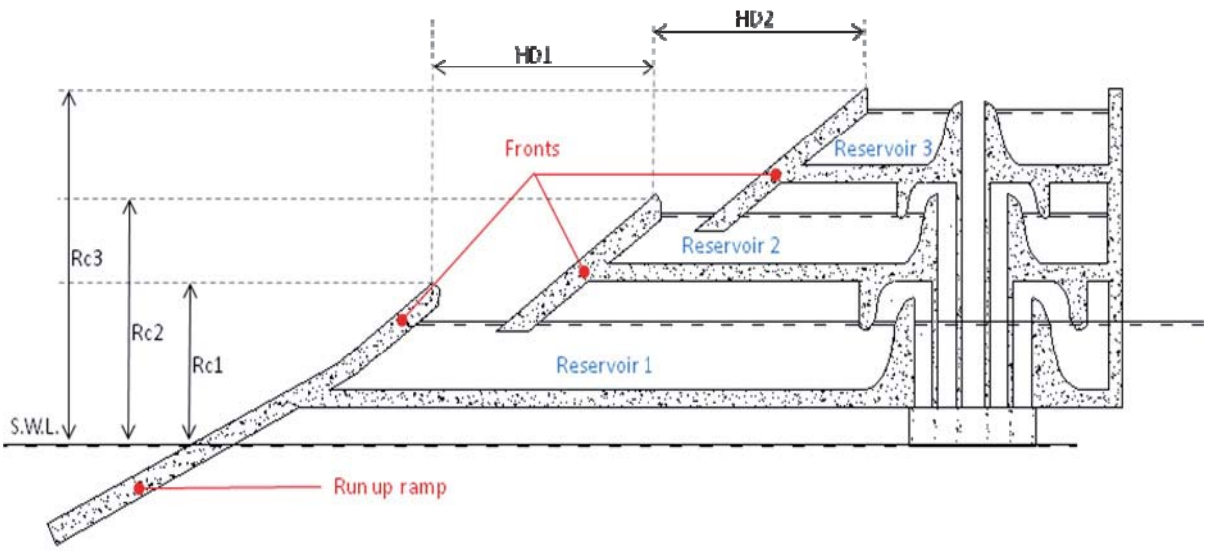


Figure 4. SSG definition sketch.

An effort to better understand the influence of orthogonal distances between the fronts of 2 consecutive reservoirs has been made by testing 13 different geometries in 2D irregular waves (Margheritini et al. 2009c). Analysis of results allowed finding a correction coefficient to be added in Eq. 2. Indeed, the structure geometry of multi-level WECs is such to require the introduction of new parameter to describe the overtopping into the reservoirs. From a comparison of the calculated and measured qj it emerged that Eq. 2 is imperfect in the description of the phenomena when varying the horizontal distance HD from the ranges in which Eq. 2 has been established.

In other words it seams necessary to introduce a new relation expressed by:

$$\frac{dq/dz}{\sqrt{gH_S}} = f_1\left(\frac{z}{H_S}, \frac{R_{c,1}}{H_S}, HD^*\right) \tag{4}$$

In Eq. 3 *HD** is the adimensionalized horizontal distance between the opening of two consecutive levels. Results indicate that *HD* influences the storage capacity and therefore the efficiency of the device (Fig. 5).

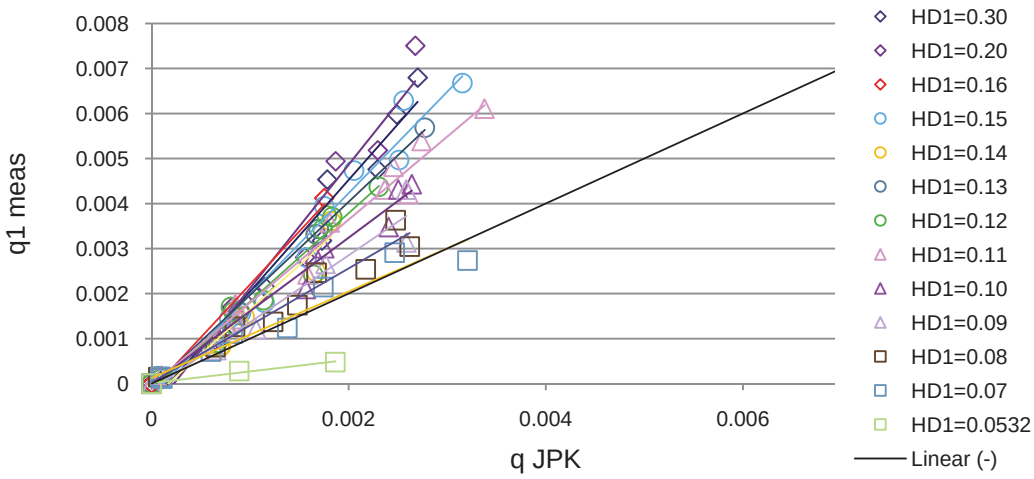


Figure 5. Comparison of measured and calculated values for different *HD1* for the lowest reservoir. *HD1* in meters.

Wave spreading, directionality and tide conditions

A specific study on the influence of oblique waves and directional spreading on the overtopping flow rates has been made for the SSG pilot (Margheritini et al. 2008). Both phenomena produce a reduction of overtopping proportional to their magnitude. It must be taken into account that both effects are more important if the device has a low width to depth ratio; in other worlds, because of the narrowness of the capture width (around 10 m full scale), the lateral walls are an obstacle to the storage of overtopping water from incoming waves. In this case a reduction of the hydraulic efficiency from 40% to 32% and from 40% to 35% in average respectively for wave spreading (Fig. 6) and directionality (Fig. 7) has been recorded.

For a fix shoreline device, another important issue is the tidal range. This issue has been study based on real case applications for different location (Sines, Portugal, Swakopmund Namibia and the North Sea, artificially over imposing to the wave climate the tide conditions of the two other locations), (Margheritini&Kofoed 2008a). It resulted that tide variation and distribution have an influence on the overtopping performance of a fix multi-level overtopping device. In particular, for a selected geometry, the overtopping decreases with increasing the tidal variation (Fig. 8). 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 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%), (Margheritini&Kofoed 2008b).

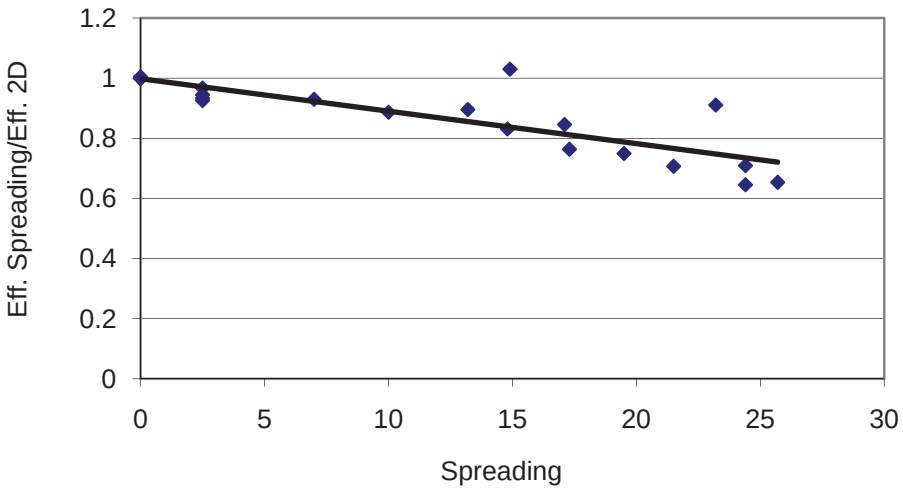


Figure 6. Influence of spreading on the hydraulic efficiency for different wave conditions.

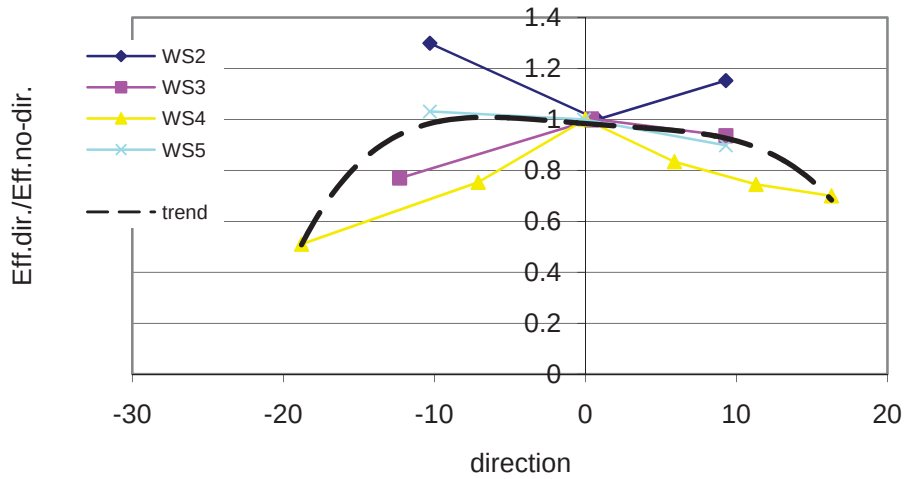


Figure 7. Influence of attack angle of incoming waves on the hydraulic efficiency for different wave conditions.

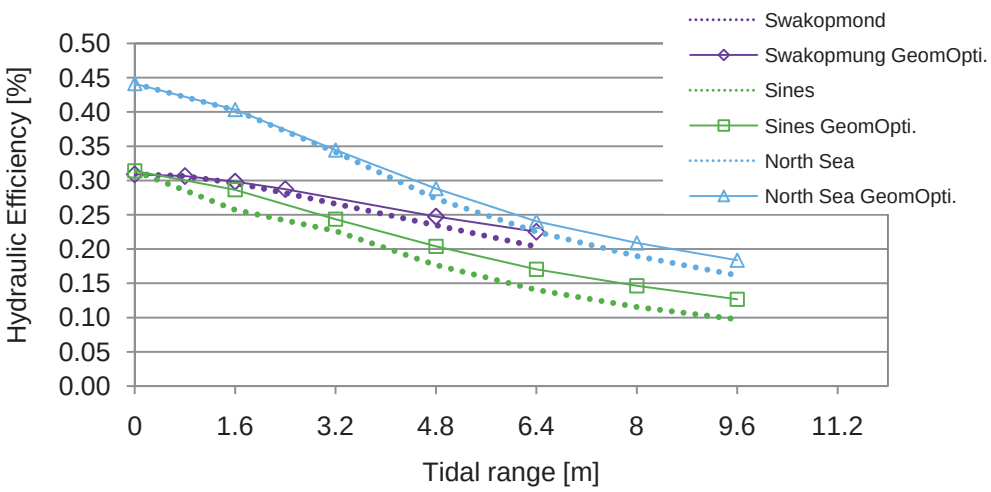


Figure 8. Decrease of hydraulic efficiency for different tidal ranges, for structures non optimized and optimized for tide in different wave and tide conditions.

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. In general, it seems more appropriate to optimize the device for a tidal range defined as the difference between mean high and mean lower water, but no common rule has been extrapolated yet.

Tidal range is taken into account in the design by:

1. Optimization of crest levels. In general, the efficiency of the device decreases when increasing the tidal range. Compare to a case where the geometry has not been optimized for tidal ranges, the losses are smaller. In average a tidal range of 1.6 m. gives a loss in hydraulic efficiency of 6.9%. For 4.8 m. tidal range the loss in efficiency is in average 29.9% (see again Fig. 8).
2. Adding an extra reservoir. For large tidal ranges it is advisable to check the feasibility of an extra reservoir to better optimize the power capture. From Fig. 9, where we have 4 different tidal ranges (no tide, T1=1.6m, T3= 4.8 m and T6= 9.6 m) emerges that in average the gain is 5 points % passing from 2 to 3 reservoirs, 3 points % passing from 3 to 4 reservoirs and 2 points % passing from 4 to 5 reservoirs. It is important to notice that for the same wave condition, the gain in percentage is bigger for bigger tidal ranges, meaning that it is beneficial to add a reservoir in case of tide. For example, passing from 2 to 3 reservoirs in the case of Sines gives 17.6% gain for no tide and 44.8% gain for 9.6 m tidal range.

The additional construction costs of adding one reservoir are around 4% (Oever 2008).

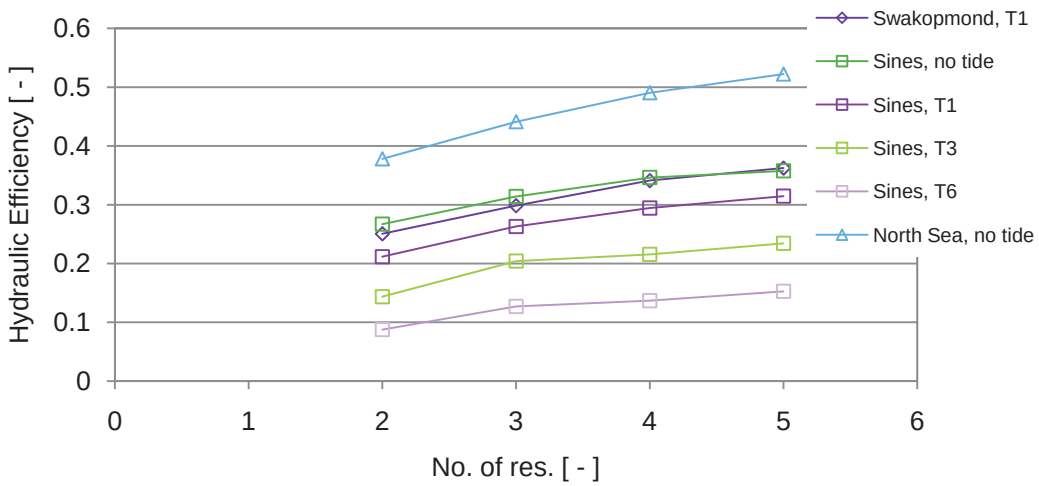


Figure 9. Performance of the SSG device for different number of reservoirs and wave and tide conditions.

In Table 1 results are presented comparing different optimized geometries with and without tide (Margheritini&Kofoed 2008b).

It must be noticed that it seems unluckily that structures with limited room between the floor and ceiling in the reservoirs will be realized as the access to the reservoirs should be garanted in case of need. Indeed, when adding a reservoir the resulting structure is characterized by a “denser” or busier vertical space and the minimum distance between the floor and the ceiling of the above reservoir must be seriously taken into account.

Table 1. Hydraulic efficiency for different optimized geometries (number of reservoirs)

Case study	Tidal range [m]	No. of Res.	Rc1 [m]	Rc2 [m]	Rc3 [m]	Rc4 [m]	Rc5 [m]	Hydraulic Eff.
Swakopmund	1.6	2	0.8	1.95	-	-	-	0.2505
	1.6	3	0.75	1.5	2.75	-	-	0.2984
	1.6	4	0.75	1.4	2.25	3.65		0.341
	1.6	5	0.75	1.25	1.8	2.65	4.05	0.3623
Sines	0	2	1	2.55	-	-	-	0.267
	0	3	0.75	1.5	3.1	-	-	0.314
	0	4	0.75	1.25	2.25	3.85	-	0.3463
	0	5	0.75	1.4	2.7	3.8	5.25	0.3576
	1.6	2	0.8	2.4	-	-	-	0.2117
	1.6	3	0.75	1.8	3.3	-	-	0.2634
	1.6	4	0.75	1.6	2.45	4	-	0.2943
	1.6	5	0.75	1.4	2.05	3	4.5	0.3145
	4.8	2	0.75	3.4	-	-	-	0.1435
	4.8	3	0.75	2.8	4.6	-	-	0.204
	4.8	4	0.75	2.2	3.4	5.05	-	0.2155
	4.8	5	0.75	1.75	2.8	4	5.3	0.2342
	9.6	2	0.9	4.6	-	-	-	0.0877
	9.6	3	0.75	3.4	5.5	-	-	0.127
	9.6	4	0.75	2.2	4.35	5.8	-	0.1368
	9.6	5	0.75	2.2	3.4	4.6	5.8	0.1527
North Sea	0	2	1	2.7	-	-	-	0.378
	0	3	1	2.05	3.75	-	-	0.4411
	0	4	0.75	1.3	2.35	4.05	-	0.4904
	0	5	0.75	1.25	2	3.05	4.65	0.5222

Wave loadings

Wave loads and pressures on the SSG structure have been analyzed by mean of laboratory tests in different set ups but always on the same geometry and for the case of the SSG Pilot in the island of Kvitsøy. In that occasion the best reproduction of the surrounding bathymetry has been realized and the model device has been positioned on top of it equipped with pressure transducers once and with load cells the other time. Results have been used both for calculation on concrete and for insurance of the structure to DNV.

The combined analysis of video-camera and pressures records made it possible to identify surging waves, characterized by a rapid rise of the wave along the three sloping front caisson plates (no breaking waves). A quasi-static loading time history is recognizable over all the front side plates and the pressure is almost hydrostatic: $p \approx \rho w g H m$, (Fig. 10). The pressure values for 1/250 corresponds to non-exceedance levels of about 99.7%. The analysis results indicate that Weibull is a more suitable CDF to describe the probability distribution of pressures (Vicinanza et al. 2007).

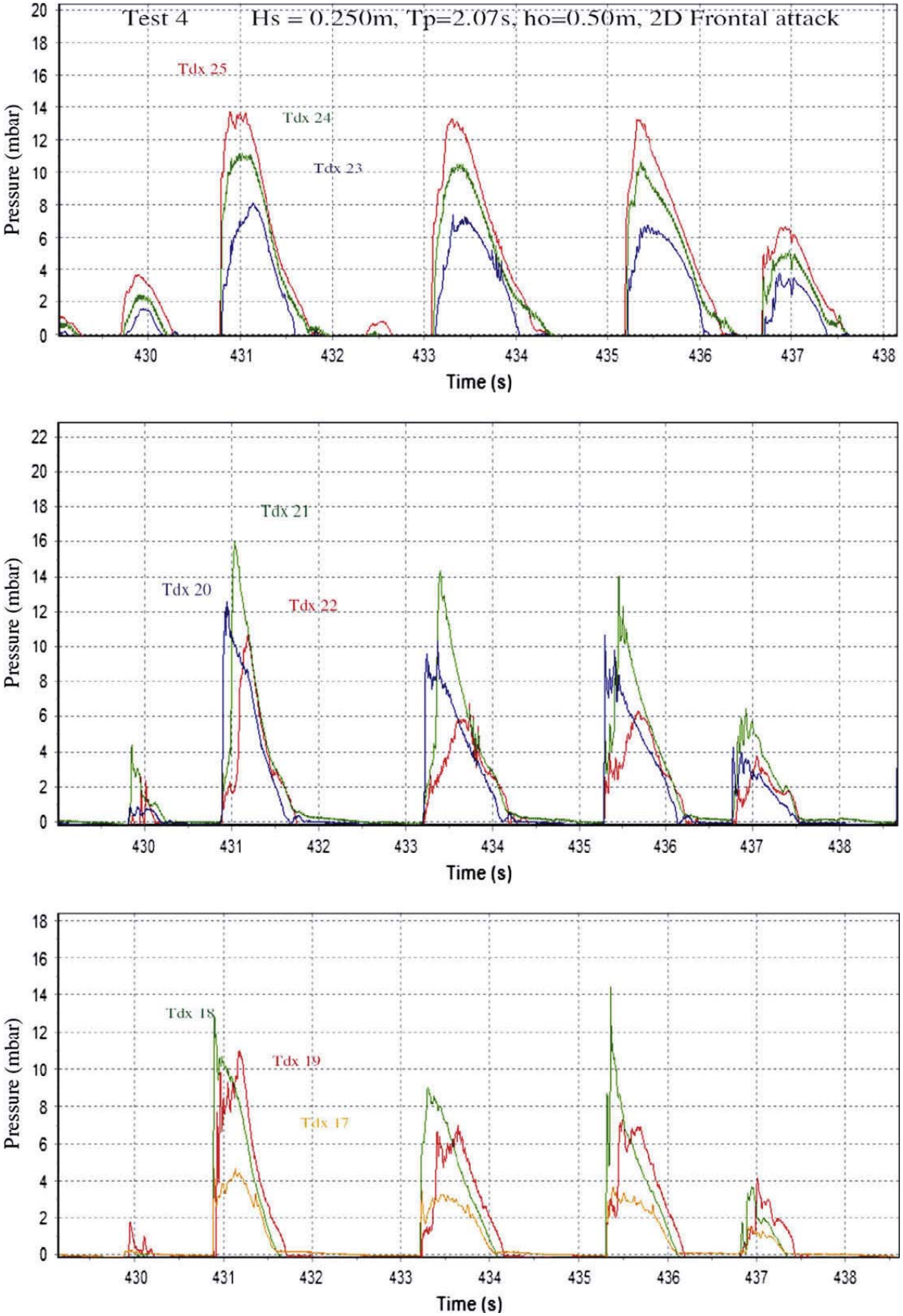


Figure 10. Pressure time history at the transducers on the fronts.

The SSG innovative structure cannot fit any standard design method, however in order to check the general tendency of these test results, the experimental pressures were compared with the design criteria suggested by the CEM (2000) for predicting pressure distribution on sloping top structures.

Pressure measurements compared with the prediction method made for caisson breakwaters with sloping top showed 20–50% higher wave pressures than the Takahashi et al. (1994) design equation (Vicinanza et al. 2009), (Fig. 11). One of the reasons is that the SSG model was fixed rigidly instead the design method was tested using sliding experiments. In fact the Takahashi et al. model caissons were fabricated from synthetic acrylic plates and had a bottom comprised of a concrete slab for simulating the friction factor. The fact that the tests show 50% higher wave pressures than the 'best' available design equation, suggests that design wave pressures is a topic needing careful attention, and not all experience from designing traditional maritime structures are usable. Laboratory tests are needed previous construction.

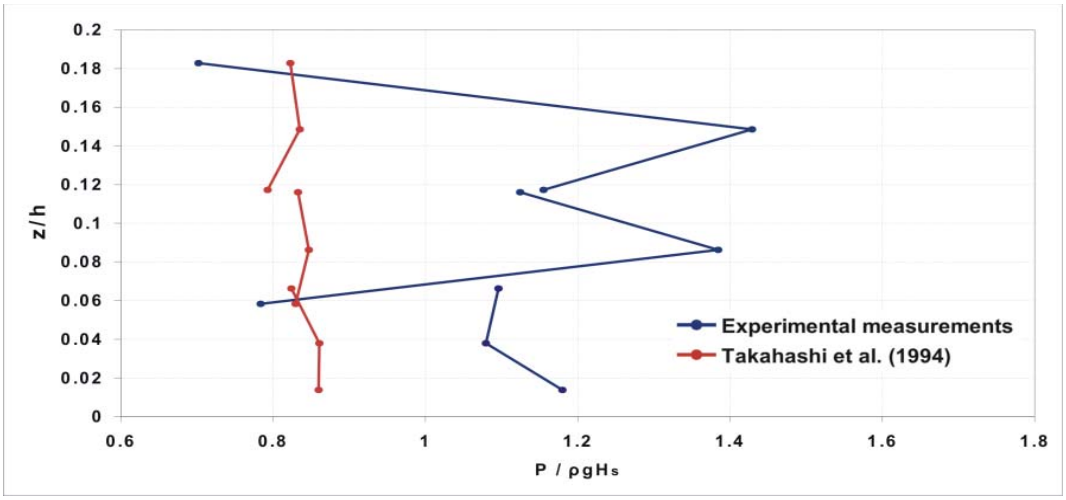


Figure 11. Takahashi et al. (1994) formula compared to measured data.

Conclusions

A variety of geometrical factors influences the hydraulic efficiency of the SSG device. Due to the possibly high number of combination and interactions of these parameters, laboratory tests for evaluation of hydraulic efficiency of the SSG device are fundamental for each application.

The existing overtopping formula used to identify the optimal crest levels related to wave conditions at location has been implemented with a parameter that takes into account the horizontal distance between the reservoirs.

Directional spreading and wave attack angle on the structure are decreasing the overtopping of a single module device, meaning in case of low width to depth ratio, from 40% to 32% and 35% respectively. However, for an array of devices mounted on a breakwater configuration the reduction on overtopping is not expected to be significant (Briganti et al. 2005) as side effects will be limited.

The presence of tide also penalizes the efficiency of the device compare to a case with no tide. Nevertheless it is possible to limit this effect by taking into account tidal variation on the design of the structure and even construct an extra reservoir to obtain a more flexible configuration. This last solution should be carefully evaluated by the construction and economical point of view for each specific situation.

Tests on wave loadings revealed a surging behavior on the structure. Quasi static loadings history over the front plates and almost hydrostatic pressures have been recorded. Obtained results have been used for the design of the SSG as well as for insuring the structure. None of the existing design

methods for coastal structures matches the results obtained for the SSG affirming that laboratory tests for each specific location are essential previous construction.

Economical aspects related to the application on breakwater of wave energy converters

Despite offshore wave energy installations can dispose of higher energy, it seems quite reasonable, if not necessary, to turn first to shoreline devices in order to address the first issues raised by the market such as feasibility and reliability.
In this chapter the first direct comparison between two energy technologies and relative economical aspects is presented. The technologies are the Oscillating Water Column (OWC) and the SSG, both applied on breakwaters for coastal protection. Considerations on additional costs for such a solution compare to traditional rubble mound or vertical breakwaters are also discussed.

The application of wave energy converters into breakwaters presents some advantages:

- sharing of construction costs.
- Access and therefore operation and maintenance are easier compare to an offshore situation.
- Sharing of infrastructures.

Three technologies have claimed their suitability for breakwater applications: ConvOn, OWC and SSG (Fig. 12).

Figure 12. Different WECs on Breakwaters, from the lf:ConvOn, OWC, SSG.

SSG application on breakwaters

Improvements of SSG-breakwater compared to a traditional solution are:

- Recirculation of the water inside the harbor i.e. improvement of water quality as the outlet of the turbines would be in the rear part of the breakwater.
- Potential lower visual impact as a consequence of a lower crest level.
- Clean electricity generation.

One issue that has been raided could be the fill in of the reservoirs with sediments especially for installations in water depth < 15 m. This issue could be solved by sloping floors in the reservoirs or programming adequate maintenance.

The most important parameters having an effect on the investment cost of the SSG are:

1. Local wave and tide climate (determines the number and size of reservoirs, in average passing from three to four reservoirs will see an increase of construction cost of 4%).
2. Design wave height (determines ballast and size of the structure).
3. Water depth (determines the construction method and overall size of the caisson).

The integration of the SSG on breakwater has been taken into account for the renovation of the harbor in Plentzia, Basque Country, Spain and for Hanstholm, North Jylland, Denmark. Also, implementation of the SSG in breakwaters in Swakopmund (Table 2 and 3), Namibia and Sines (Table 4), Portugal has been deeply analyzed in order to identify issues related to performance,

construction and installation (Margheritini&Kofoed 2008a,b). The further analysis will be based on these two last cases.

For Swakopmund, the installation at two different water depths is described for 3-reservoirs structure. For Sines, the installation of a 4-reservoirs structure is studied. A 4-reservoirs structure has been chosen due to the bigger tidal range at location. Tidal range is here taken as the difference between the highest high and the lowest low water.

Table 2. Swakopmund conditions at 6 m water depth.

Swakopmund 6	
Annual wave energy [kW/m]	15.5
Hs [m]	5.7
Hmax [m]	6.6
Tidal range [m]	1.6 (±0.8)
Water depth	6

Table 3. Swakopmund conditions at 11 m water depth.

Swakopmund 11	
Annual wave energy [kW/m]	15.7
Hs [m]	7.9
Hmax [m]	9.9
Tidal range [m]	1.6 (±0.8)
Water depth	11.3

Table 4. Sines conditions.

Sines	
Annual wave energy [kW/m]	14.41
Hs [m]	13.9
Hmax [m]	18.1
Tidal range [m]	3.37 (±1.68)
Water depth [m]	18

No data on the overtopping over the SSG-breakwater structure exist at the present time. Consequently the overtopping rates have been calculated according to EurOtop Manual 2007. The operability has been accessed with the 0.1% exceeded wave. Based on this the crest levels have been determined. Due to the geometry and the nature of the SSG, overtopping could be less than calculated with normal overtopping criteria based on actual crest level. This indicates that the structure can be potentially lower than a conventional structure which would reduce the costs.

For construction reasons the front ramp have been cut at -3.8 m below sea water level in the case of Swakopmund and at sea water level for Sines. This have an influence in the hydraulic efficiency of the device (i. e. its power production) as the captured power is lower than predicted. The decrease on hydraulic efficiency has been investigated by Kofoed 2005. Pressure relieve openings under the ceilings of the reservoirs are part of the design. These openings will facilitate the inflow of water and reduce the extreme wave pressure.

Construction costs have been analyzed by Oever (2008). They include: production of concrete elements, dry dock costs, float and transport elements, immerse and installed elements, gravel bed, sand fill, indirect costs, engineering costs, general costs, profit and risk. The cost of turbines and generators has been added to the total. Following the main characteristics of the devices are summarized by location.

Following the main characteristics of the studied installations are presented.

Swakopmund 6 m water depth
Capture crest levels: Rc1=1 m, Rc2=2.3 m, Rc3=3.6 m
Crest level: 6 m
Base width: 22.7 m (equal to conventional concrete caisson)
Installed capacity: 12.5 kW/m
Expected power production: 19000 kWh/y/m
Construction costs inclusive of turbines and generators: 76900€/m
Swakopmund 11 m water depth (Fig. 13)
Capture crest levels: Rc1=1 m, Rc2=2.5 m, Rc3=4 m
Crest level: 8m
Base width: 28 m (equal to conventional concrete caisson)
Installed capacity: 12.8 kW/m
Expected power production: 18000 kWh/y/m
Construction costs inclusive of turbines and generators: 150700€/m
Sines (Fig. 14)
Capture crest levels: Rc1=0.75 m, Rc2=2.05 m, Rc3=3.35 m, Rc4=4.65
Crest level: 15 m
Installed capacity: 12 kW/m
Expected power production: 12000 kWh/y/m
Construction costs inclusive of turbines and generators: 285800€/m

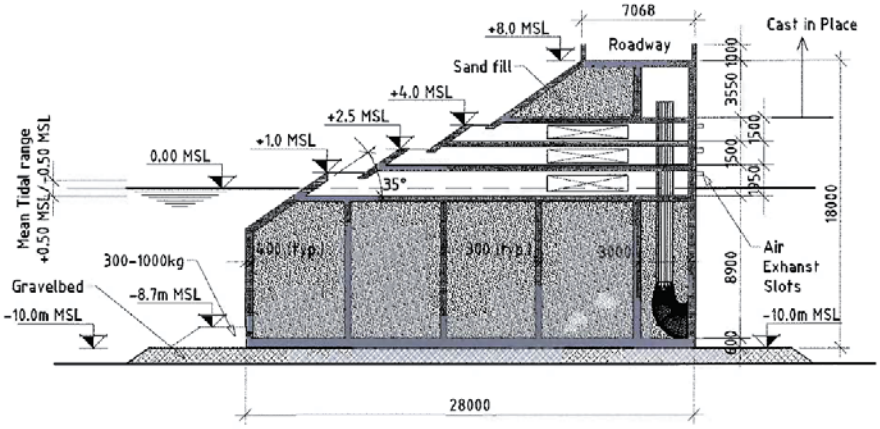


Figure 13. Section of the SSG-breakwater caisson at Swakopmund 11 m water depth (DMC 2008).

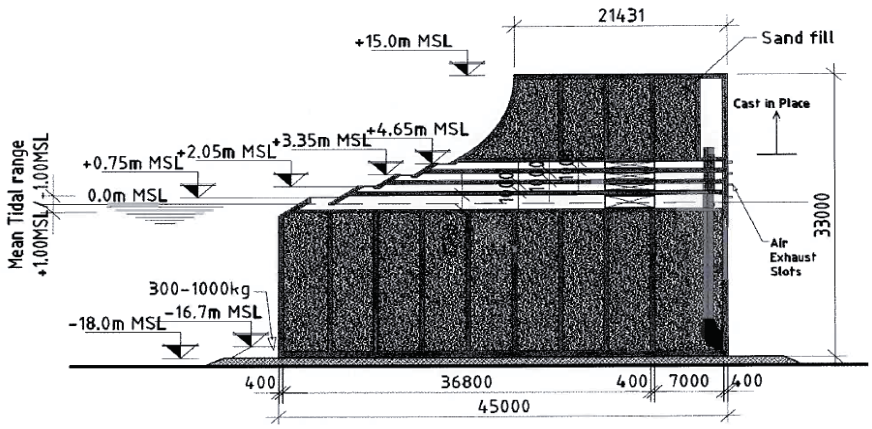


Figure 14. of the SSG-breakwater caisson at Sines 18 m water depth (DMC 2008).

The total concrete quantity for a concrete caisson is higher for the SSG-breakwater than for a conventional caisson (because of floor slabs). Other differences are the higher center of gravity and eccentric location of the center of gravity that have consequences on the draft of the floating caisson into position.

Indeed for Swakopmund the draft revealed to be critical for such shallow water so that the construction will perform two separated parts: the lower part reaches to the first slot and consists of elements of 5 m wide; the upper part is 10 m wide. The upper part fixed to the lower by the overhang of the upper part over the lower, to prevent uplift of the upper part it is secured by tension anchors in the walls. For Sines the structure is too heavy and can´t be lifted and therefore are constructed in a dry dock that will subsequently be flooded.

Additional cost for WEC into breakwaters is defined as the cost related to the construction and installation that would not occur in case of a traditional harbor protection. Examples of extra costs are: electrical equipment, turbines, extra concrete etc. By comparison with traditional breakwater solutions it is possible to deuce the additional costs related to the construction of an SSG-breakwater (Table 5).

Table 5. Summary of economics for SSG-breakwater application.

Location	Rubble mound	Traditional caisson	SSG-breakwater	Additional costs
Swakopmund 6 m water depth	28100€/m	-	76900€/m	48800€/m
Swakopmund 11 m water depth	67200€/m	124500€/m	150700€/m	83500€/m - 26200€/m
Sines 18 m water depth	-	231000€/m	285800€/m	54800€/m

The additional costs seem to be acceptable; if they are put in a payback scheme of 10 years, which is reasonable considering the lifetime of the protection, the cost of electricity is set around 0.25€/kWh.

Comparison between SSG and OWC on breakwaters

A comparison of the SSG-breakwater in Swakopmund 6 m water depth and OWC-breakwater in Mutriku showed that the two installations have comparable performance and issues related to installation (Table 6), (Margheritini&Frigaard 2009). The two concepts present more similarities than any other couple of random concepts among wave energy technologies:

- Massive reinforced concrete structure.
- Specially design turbines.
- Shore-line and breakwaters applications.

It is worth to remember that we are in front of two different stages of development: while several prototype scale OWCs have already been constructed and operated with varying degrees of success over the last 30 years, SSG prototypes haven´t yet been realized. Despite the two working principles being different as OWC are based on the oscillating water column principle, the two technologies have more in common than other WEC and the final cost of electricity considering 10 years payback time is the same. This suggests that one technology should be chosen over the other for extra benefits that it may bring to the protection.

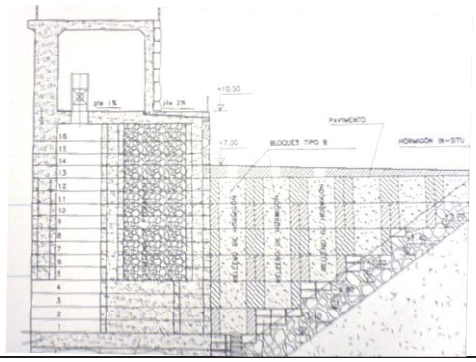
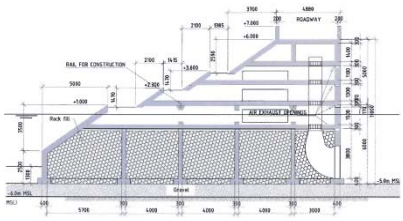
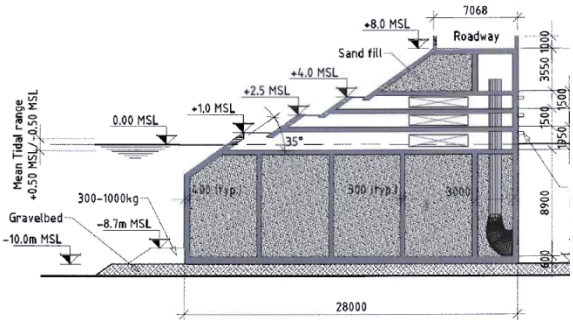
With regard to reflection performance of the integrated structure, preliminary comparison results from OWC and SSG laboratory tests show that in both cases we are in presence of highly reflective structures (Zanuttigh et al. 2009) with reflection coefficient never lower than 40% and that can rise up to 90%.

The characteristics of the OWC breakwater are presented as follow.

OWC Mutriku
Crest level: 16 m
Installed capacity: 2.96 kW/m
Expected power production: 6000 kWh/m/y
Construction costs: 60000 €/m.

The additional cost has been estimated by comparison to the construction of traditional rubble mound breakwater.

Table 6. Comparison of cost between OWC and SSG breakwaters.

Technology	Location	Cost of electricity
	Mutriku, Spain, 6 m water depth. 	0.33 €/kWh
	Swakopmund, Namibia, 6 m water depth. 	0.27 €/kWh
	Swakopmund, Namibia, 11 m water depth. 	0.16 €/kWh

With regard to reflection performance of the integrated structure, preliminary comparison results from OWC and SSG laboratory tests show that in both cases we are in presence of highly reflective structures with reflection coefficient never lower than 40% and that can rise up to 90% (Zanuttigh et al. 2009). It is then a design issue to construct a proper toe protection layer to avoid scour holes if not a berm to reduce the reflection.

Conclusions

It is concluded that:

- WECs built into breakwaters are economically feasible.
- Deep water applications (caisson type) have better payback time.
- Seen from an economical point of view OWC are comparable to SSG with respect to the present knowledge. The demonstrable differences are within the level of uncertainties.
- The breakwater mounted WECs may offer some additional performance to the protection structure: clean energy production; recirculation of water in the harbor and lower visual impact in the case of the SSG solution.

Nevertheless some relevant knowledge gaps have been spotted with regard to the SSG device and its application on breakwaters. Those regard the overtopping over the whole structure and influence of necessary mitigation measure for reflection on the hydraulic efficiency i.e. power production. Those gaps should be filled in, in the interest of commercialization of such a solution.

Environmental impact assessment, scooping

Frameworks and regulations for wave energy development are not fully ready, experiencing a setback caused, among others, by lack of understanding of the interaction of the technologies and marine environment, lack of coordination from the competent Authorities regulating devices deployment and conflicts of maritime areas utilization. The EIA is a necessary step toward WE deployment.

As wave energy devices are still relatively at an early stage, only few EIAs have been carried out. It is argued in the existing Environmental Impact Assessment reports that only minor impacts can be expected by deployment of ocean wave energy devices. Most impacts are associated with the establishment and decommissioning phase, like sediment spill, incidents, accidents, oil spill, waste handling, pile driving, laying of mooring blocks, sediment depositing, marine archaeology, navigation hazards and increased Shipping activity.

During the operational phase the following impacts on the marine mammals, fish, birds, and marine ecology has to be considered more deeply before any judgement can be given:

- Behaviour changes due to physical presence, noise, vibration
- Entanglement, entrapment, collision
- Loss / change of seabed habitat
- Change in distribution of prey items
- Change of wave climate / sediment transport leading to changes of habitats
- Colonisation of structure leading to increased biodiversity
- Navigation
- Coastal processes

In order to substantiate this statement further, it is recommended that the relevant baseline studies are conducted as projects reach the appropriate stage of development. These must be followed by an appropriate monitoring programme both during deployment and after decommissioning, in order to replace the proposed impacts by data reflecting the actual impacts, however minor (Sørensen, 2008).

According to the author, it is possible to clearly recognize five parameters relevant for the EIA of WECs. Based on these parameters a classification of the technologies can be derived and used to complete its preliminary EIA (Margheritini et al. 2009d). These parameters are:

1. *D* parameter, indicating the distance of the installation from shore.
2. *S* parameter, indicating the kind of element used for stabilizing the device.
3. *z/d* parameter, indicating the relative water column obstruction (vertical) caused by the presence of the device.
4. *w/a* parameter, indicating the relative horizontal obstruction of a wave energy farm.
5. *P* parameter, indicating the kind if power takeoff utilized in the installation.

This method could be used by developers, authorities and stakeholders as primary indication or fast consultation on relevant issues for the EIA of a specific technology once that basic information on the installation is known.

Following, the assessment table for the first thee parameters listed above are reported. With regard to the *P* parameter, some issues are related to specific power takeoffs: noise disturbance is relevant for air turbines in relation to local communities. Hydraulic ram and elastomeric hose pump system may generate oil spillage as consequence of malfunctioning if the system utilized hazardous substances. In general moving parts generate noise of possible concern for marine life and the existence of important gaps in knowledge suggests that the issues should be considered carefully. The *w/a* parameter will not be discussed here as impact assessment is made for one single onshore device.

Table 7. Assessment table based on Distance from shore.

D parameter	Local communities	Coastal processes and coastal spices	Navigation and fishery
Onshore devices	**Major**	**Major**	**Nil**
Intermediate water devices	**Moderate**	**Moderate**	**Major**
Offshore devices	**Negligible**	**Minor**	**Major**

Table 8. Assessment table based on the type of Stabilizing element.

S parameter	Benthonic habitats	Geology	Archeology	Water column spices
Simple moorings	**Minor**	**Negligible**	**Minor**	**Minor**
Complex moorings	**Minor**	**Negligible**	**Minor**	**Moderate**
Gravity foundations	**Moderate**	**Moderate**	**Moderate**	**Negligible**
Piles	**Major**	**Major**	**Major**	**Minor**

Table 9. Assessment table based on the obstruction parameter.

z/d parameter	Water column spices	Navigation and fishery	Coastal processes	Local communities
Little obstructive z>0	**Minor**	**Major ***	**Minor ****	**Negligible****
Little obstructive z<0	**Minor**	**Minor**	**Minor**	**Negligible**
Obstructive z>0	**Moderate**	**Major**	**Minor**	**Moderate**
Obstructive z<0	**Moderate**	**Moderate**	**Minor**	**Moderate**
Very obstructive z>0	**Major**	**Major**	**Moderate**	**Moderate**
Very obstructive z<0	**Major**	**Major**	**Moderate**	**Moderate**

*no interaction for onshore devices **Major for onshore devices

From the above tables, relevant issues related to the EIA of the SSG wave energy converter are summarized.

Impact on local community

Despite the possibility of the impact on local communities to be major due to the location, the SSG Pilot that was meant to be built in the Island of Kvitsøy, Stavanger, Norway (Margheritini et al. 2009a) has benefited of positive public acceptance. It was expected that the project would bring tourist to the island and to the demonstration center/museum that would also bad been built by the Municipality in Kvitsøy in occasion of the construction of the pilot device. In general islands and isolated communities will benefit of WE installations as the cost of electricity is competitive compared to the cost from diesel generators.

For the visual impact, it must be taken into account that the device is built in concrete; mitigations measures include the application of natural rocks on the side of the device to make it blend with the surroundings.

With regard to breakwater applications, water quality into harbours is an issue. The SSG-breakwater can bring considerable improvement to this situation as the outlet of the water turbines is on the leeside of the protection. Also, generation of pollution free energy for harbor facilities is to be considered as positive effect of this implementation.

Impact on coastal processes and coastal spices

Regarding breakwater installation, this is considered to have major impact on the close to shore sediment transport in the area; nonetheless this is an issue in coastal protection structures. Model for predictions of the effect are available on the market. Impact on coastal spices could be from moderate to major depending on to which extent the costal spices will re-colonize the interested area after installation. One concern that has been raised regards the numbers and the fate of the fish accidentally trapped in the reservoirs.

With regard to onshore applications, no impact or negligible impact on the sediment transport is expected.

Impact on geology and geomorphology and archeology

The device will be floated into position and anchored or ballasted to the bottom. If the seabed is sandy, a gravel bed must be positioned previous the installation of the device. The installation phase will consequently partially degrade (permanently in case of rocky seabed) the morphology of the interested area that can vary from 15m^2 in case of a single SSG module, to roughly 600000 m^2 in case of 2km breakwater application.

Impact on benthic habitats

It is easy to think that during installation phase the benthic habitats will suffer of different degrees of disturbance. The death of part of benthic communities is highly possible, both for operation and movements of materials and for changing in the natural conditions characteristic of pre-construction. Nevertheless, during operation the presence of the device is likely to offer new habitats as fish are known to congregate around objects rising from the seabed. Repopulation is expected within 2 years from installation.

Impact on water column spices

During installation phase it is expected that fish and mammals will be subject to lower distress than the benthonic fauna; in particular they are expected to leave the area and come back within short time. The impact in the overall is considered to be negligible, as the device will interest the coastline habitats more than the water column spices.

Noise disturbance

Dr Jeremy Nedwell is researching and validating the standards for rating the effects of underwater noise.
The operations that are sources of major concern are piling and drilling, during installation and decommission. It is possible that similar operation will be associated with the installation of the SSG device. In this the impact on marine life is spice dependent but could cause dead or permanent damage to the population.
For operation conditions, from Wave Dragon (Russell and Sorensen 2007) experience it is stated that the mechanical noise, which can be successfully controlled at the design stage of the turbine, is not considered significant for the Kaplan turbine due to the slow rotational speed of the blades.

Conclusions

Useful classification to spot relevant issues related to the EIA of a specific WECs has been proposed. The assessment tables provide a quick assessment method for the environmental impact of WECs.

In the case of the SSG device it can be argued that a limited number of issues are of concern; the most impacted receptors are: the coastal processes in case of breakwater applications, geology and coastal spices in relation to noise disturbance during installation.

Future challenges

Many challenges have been accepted, issues solved and innovative solutions found. The feeling is that there will always be new challenges, the difference is made by the choice on which challenge to pick. The work that led to the realization of the present PhD thesis contributed to spot relevant gaps in knowledge at the present stage of development of the device. This gaps need to be filled in also in the interest of near commercialization of the device.

In the author´s opinion, it is needed to focus on:

- Investigation on overtopping over the whole structure and its possible reduction compare to a typical breakwater due to the presence of reservoirs and energy absorption.
- With regard to the breakwater application of the device, research on the possibility of realizing a lower structure compared to typical protections. That would be more attractive for marinas and touristic harbours where the view is important but also from an economical point of view as lower structure means less construction material, i.e. lower costs.
- Research on the influence of the means for reduction of the reflection of the structure on the performance of the device.
- Design formula for wave loading in order to reduce the dependence on laboratory tests previous construction.
- Further implementation of the overtopping formula.
- Further implementation of the Overtopping Simulation tool WOPSim 3.01 especially in the turbine side and calibration, possibly with full scale data.
- Creation of a model to include together all the parameters influencing the design of the SSG device and their interactions, including economical parameters.
- Base line studies for EIA.
- Last but not least, the challenge personally I want to be involved into is the construction of a full scale device.

Conclusions

The research in this Thesis has advanced the development of the SSG device, while also contributing to furthering the research in the wave energy field. This has been accomplished by presenting best practice on testing and data analysis. Results on dependency of the overtopping on the horizontal distance parameter are of interest for the SSG and any other overtopping device developer. Great achievement of this work is the direct comparison between two wave energy technologies (SSG and OWC). The comparison is a central issue in the wave energy sector being difficult to relate one technology to another due to the wide variety of concepts and working principles, not to mention the lack of data. The comparison featured in this work is a result of intensive collaboration of the Author with colleagues of other Institutes involved on R&D of OWC technology. In particular the Wave Energy Center in Lisbon (PT) for the feasibility and laboratory testing on hydraulic performance; EVE in Bilbao (SP) for the economics and the construction on breakwaters and WaveGen for laboratory data on reflection. The work in this thesis also resulted in pricing the cost of electricity generated by SSG and OWC wave energy converters installed on breakwaters answering another big question of the sector. Finally, considering the process for installation to full scale devices to be full of obstacles, it is believed that the new classification of technologies suggested as a base for EIA of WECs is adding a wedge to the puzzle and reducing non-technical risks related to the wave energy sector. With specific regard to the breakwater application of the device, this links related but different areas such as wave energy and coastal structures with benefits from both sides.

The R&D of the SSG device towards commercialization provides a clear history useful for the SSG team but also for other developers. The comprehensive presentation of the development process of the SSG device, including case studies applications, led to relevant information on the expected performance of the SSG device narrowing related uncertainties. Cornerstone experience has been gained toward the process for construction of full scale device, useful in any further step of the SSG.

The author has learnt a lot during her studies and met many people at different level in the decision making system and must thank all her co-workers form Aalborg University, WaveEnergy AS and the “wave energy community”. The topics covered are broad, from overtopping of structures to environmental impact, from economics to wave loadings and so it is the Author´s view at the conclusion of these three years. During this period the SSG project has developed from laboratory tests to detailed planning for full scale installation. The Author´s research has aided this process and she hopes to continue to work on the development until the SSG technology proved the expectations.

References

Riccardo Briganti, Giorgio Bellotti, Leopoldo Franco, Julien De Rouck and Jimmy Geeraerts 2005. “Field measurements of wave overtopping at the rubble mound breakwater of Rome–Ostia yacht harbour”. Coastal Engineering, Volume 52, Issue 12, December 2005, Pages 1155-1174.

CEM (Coastal Engineering Manual), 2002. U.S. Army Corps of Engineers. 2002. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).

Cruz J.; Gunnar M., Barstow S., Mollison D. (2008), Joao Cruz, ed., Green Energy and Technology, Ocean Wave Energy, Springer Science+Business Media, pp. 93, ISBN 978-3-540-74894-6.

Engineering Committee on Oceanic Resources — Working Group on Wave Energy Conversion (2003), John Brooke, ed., Wave Energy Conversion, Elsevier, pp. 7, ISBN 0080442129;

Frigaard, Peter ; Trewers, Andrew ; Kofoed, Jens Peter ; Margheritini, Lucia. Aalborg 2008: “Conceptual Design of Wave Plane”. Aalborg University. Department of Civil Engineering, 2008. 233 s. (DCE Contract Reports; 38).

Kofoed J.P.(2005): “Experimental Hydraulic Optimization the Wave Energy Converter Seawave Slot-Cone Generator”. Hydraulics and Coastal Engineering No. 26, ISSN: 1603-9874, Department of Civil Engineering, Aalborg University.

Kofoed, Jens Peter ; Frigaard, Peter ; Friis-Madsen, Erik ; Sørensen, Hans Chr.. I 2006a: “Prototype Testing of the Wave Energy Converter Wave Dragon”. Renewable Energy; vol. 31, nr. 31, FEB. s. 181-189.

Kofoed, J. P. 2006b: “Vertical Distribution of Wave Overtopping for Design of Multi Level Overtopping Based Wave Energy Converters”, 30th International Conference on Coastal Engineering, ICCE San Diego.

Kofoed, Jens Peter ; Frigaard, Peter ; Hernandez, Lorenzo Banos. In 2008: “Development of Wave Energy Devices - Past, Present and Future : The Danish Case.” / ECOR Symposium. Engineering Committee on Oceanic Resources: Marine Renewable Energy: St. Johns, Canada, Oct. 23-24, 2008 : Specialists Panel. Institute for Ocean Technology, Canada, 2008. 12 p.

Margheritini L. and Frigaard P.. Technological and Commercial Comparison of OWC and SSG Wave Energy Converters Built into Breakwaters. Submitted to the Renewable Energy Journal, Elsevier 2009.

Margheritini, Lucia and Kofoed, Jens Peter. Specification for Instrumentation : Data Acquisition and Control System for SSG Pilot Plant at Kvitsøy : Norway. / Aalborg : Aalborg University. Department of Civil Engineering, 2006. 67 s. (DCE Technical Report; 19).

Margheritini, Lucia ; Vicinanza, Diego ; Frigaard, Peter. SSG Wave Energy Converter: Design, Reliability and Hydraulic Performance of an Innovative Overtopping Device. / I: Renewable Energy. 2009a ; vol. 34, nr. 5, 01.11.2008. s. 1371–1380

Margheritini, Lucia ; Vicinanza, Diego ; Kofoed, Jens Peter. Hydraulic Characteristics of Seawave Slot-cone Generator Pilot Plant at Kvitsøy (Norway). / I: Proceedings of the 7th European Wave and Tidal Energy Conference : Porto, Portugal, 11-14 September 2007. Porto: European Ocean Energy Association, 2007. 8 s.

Margheritini, Lucia ; Kofoed, Jens Peter ; Frigaard, Peter. Status and performance of the SSG Wave Energy Converter. / I: 4th Coordinated Action on Ocean Energy (CA-OE) Workshop : Performance Monitoring of Ocean Energy Systems. 2006. 10 s.

Margheritini, Lucia ; Vicinanza, Diego, Kofoed, J.P. Overtopping Performance of Sea Wave Slot Cone Generator. / I: Accepted at the conference Coasts, Marine Structures and Breakwater 2009b. 2 s.

Margheritini, Lucia ; Victor, Lander; Kofoed, Jens Peter ; Troch, Peter. Geometrical Optimization for Improved Power Capture of Multi-level Overtopping Based Wave Energy Converters. / I: Proceedings of ISOPE-2009, Osaka, Japan. 2009c. 2 s.

Margheritini Lucia, Merrild Anne, Frigaard Peter, 2009d: A method for EIA scoping of Wave Energy Converters - based on classification of the used technology. Submitted to the journal Environmental Impact Assessment Review, Elsevier.

Margheritini, Lucia ; Vicinanza, Diego ; Frigaard, Peter. Sea Slot Cone Generator Overtopping Performance in 3D Conditions. / 2008. 6 s. The International Offshore and Polar Engineering Conference (ISOPE), nr. 18, Vancouver, Canada, 6. juli 2008 - 11. juli 2008. ISOPE - Proceedings. 18.

Margheritini L. and Kofoed J.P.. Aalborg. Case study on Breakwater SSG application. Aalborg University. Department of Civil Engineering, 2008a. (DCE Contract Reports; 44).

Margheritini L. and Kofoed J.P. Aalborg . Parametrical numerical study on Breakwater SSG application. Aalborg University. Department of Civil Engineering, 2008b. (DCE Contract Reports; 47).

Meinert P. (2008), update of Meinert P. (2006). User Manual for SSG Power Simulation 2 DCE Technical Report No. 44, ISSN 1603-9874, Department of Civil Engineering, Aalborg University.

Murphy L. M. and Edwards P. L. (2003). Bridging the Valley of Death: Transitioning from Public to Private Sector Financing. NREL is a U.S. Department of Energy Laboratory. Operated by Midwest Research Institute - Battelle – Bechtel. Contract No. DE-AC36-99-GO10337, Prepared under Task No. (7200.2050).

Nielsen. K. Bølgekraftudvalgets Sekretariat. Bølgekraft – forslag til forsøg og rapportering. (61 pages, Danish). Pubblished by Bølgekraftudvalgets Sekretariat, Danish Energy Agency 1999.

Oever E. ten. The SSG- breakwater concept. Delta Marine Consultans. Report number 952350-rap-u-005.

Vicinanza, Diego ; Margheritini, Lucia ; Frigaard, Peter. Spatial Distribution of Wave Pressures on Seawave Slot-cone Generator. / I: COPRI-ASCE Coastal Structures 2007 International Conference : July 2-4 2007, Venice, Italy. 2009. 13 s.

Vicinanza, Diego ; Margheritini, Lucia ; Contestabile, Pasquale ; Kofoed, Jens Peter ; Frigaard, Peter. Seawave Slot-cone Generator : an Innovative Caisson Breakwaters for Energy Production. / 2009. 12 s. Konferencen: The International Conference on Coastal Engineering, nr. 31, Hamburg, Tyskland, 31. august 2008 - 5. september 2008.

Russell I. and Sorensen H. C.. Wave Dragon: Results From UK EIA and Consenting Process. Proceedings of the 7th European Wave Energy Conference, Porto, Portugal 2007.

Takahashi, S., S., Hosoyamada, and S. Yamamoto, 1994. Hydrodynamic characteristics of sloping top caissions. Proceedings of International Conference on Hydro-Technical Engineering for Port and Harbour Construction, Port and Harbour Research Institute, Japan, vol 1, pp. 733–746.

Zanuttigh, Barbara ; Margheritini, Lucia ; Martinelli, Luca. 2009. Analysis of Wave Reflection from Wave Energy Converters Installed as Breakwaters in Harbours. / Proceedings of the 8th European Wave and Tidal Energy Conference: Uppsala, Sweden l. European Ocean Energy Association, 2009.

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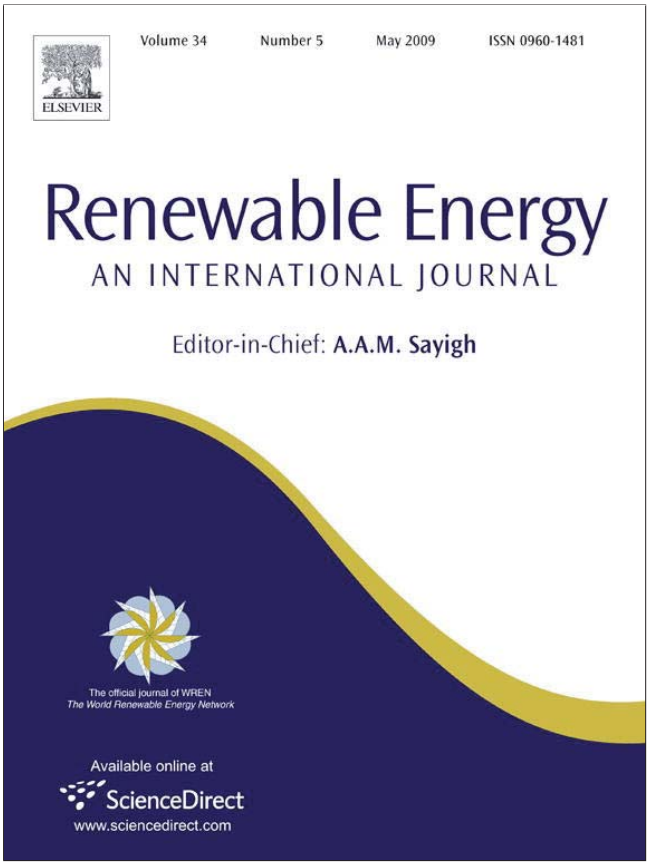
Marghertini L., Vicinanza D. and Frigaard P.

Thesis author´s contribution:

The Thesis author is the first author of this paper. She performed the laboratory tests on overtopping in 3D conditions for the SSG pilot and reviewed and implemented the information needed for the specification on instrumentation of the pilot device supervised by co-author P. Frigaard. She helped the Co-author D. Vicinanza in the presentation of his results of laboratory tests on wave loading on the SSG pilot of which he is responsible.



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SSG wave energy converter: Design, reliability and hydraulic performance of an innovative overtopping device

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ABSTRACT

The SSG (Sea Slot-cone Generator) is a wave energy converter of the overtopping type. The structure consists of a number of reservoirs one on the top of each other above the mean water level in which the water of incoming waves is stored temporary. In each reservoir, expressively designed low head hydroturbines are converting the potential energy of the stored water into power. A key to success for the SSG will be the low cost of the structure and its robustness. The construction of the pilot plant is scheduled and this paper aims to describe the concept of the SSG wave energy converter and the studies behind the process that leads to its construction. The pilot plant is an on-shore full-scale module in 3 levels with an expected power production of 320 MWh/y in the North Sea. Location, wave climate and laboratory tests' results will be used here to describe the pilot plant and its characteristics.

1. Introduction

Together with the overall trend of all renewable energies, wave energy has enjoyed a fruitful decade. Improvement of technologies and space for new ideas, together with financial support, led the research to gamble on different concepts and develop a number of new devices. While innumerable projects went through an initial simple testing phase, only few of them reached the sea prototype testing and even fewer have been commercialized. After many failures, it is obvious that much has been wasted on designs which could never be economic and serviced economically, or on designs which are unsuitable to survive storms.

The SSG is a Wave Energy Converter (WEC) of the overtopping type: the overtopping water of incoming waves is stored in different basins depending on the wave height. Turbines play an important and delicate role on the power takeoff of the device. They must work with very low head values (water levels in the reservoirs) and wide variations in a marine aggressive environment. In the following paragraph, the concept of the innovative Multi-Stage Turbine (MST) will be presented as integrant part of the SSG concept. The Company WAVEenergy AS found in Stavanger Norway, is developing the device (patented in 2003) since 2004 when the pilot project has been partially funded by the European Commission FP6-2004-Energy (WAVESSG project) and it can now benefit of 2.7 M€, the majority of which are from private investors. Partners from different countries in Europe collaborate for the realisation of the pilot project. The installation of the structure is foreseen for summer 2008 in the island of Kvitsøy, Norway (Fig. 1).

The main strength of the device consists on robustness, low cost and the possibility of being incorporated in breakwaters (layout of different modules installed side by side) or other coastal structures allowing sharing of costs and improving their performance while reducing reflection due to efficient absorption of energy. Even though, an offshore solution of the concept could be investigated to reach more energetic sea climates (Fig. 2).

In the following paragraphs the SSG concept and its optimizations will be presented, together with the work for the realisation of the prototype. Particularly the main results from power simulations, 3D model tests on overtopping and wave loadings used for the final design of the pilot plant will be reported. Moreover, other issues regarding funds, location of the pilot installation and instrumentation will be also discussed.

2. Concept description

Being an overtopping wave energy converter means that the structure must be overtopped by incoming waves; during these events, indeed, the overtopping water is captured in different basins above the mean sea level. The energy extracted from a given volume of water in the reservoir is in direct proportion to its elevation above the mean sea level (turbine head). Different ventilation openings must be included in the design of the structure in order to prevent air pressure to obstruct the water storage.

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Fig. 1. The SSG pilot plant in the island of Kvitsøy, Norway.

In the SSG the water in the reservoirs on its way back to the sea falls through a turbine spinning it and generating electricity (Fig. 3). For energy conversion, the innovative concept of the Multi-Stage Turbine (MST) is under development at WAVEenergy AS and its design integrated in the structure consists of a number of turbines (depending on the number of reservoirs) staggered concentrically inside each other, driving a common generator through a common shaft (Fig. 4). Each of the runners is connected to one of the reservoirs by concentric ducts. By taking advantage of different heights of water head, the MST technology is willing to minimize the start/stop sequences and operate even if only one reservoir is supplying water, resulting in a higher degree of efficiency. Preliminary 3D computational fluid dynamic analysis of the guide vane and the runner made by the Norwegian University of Science and Technology (NTNU) shows an efficiency of 90% for the individual stages with a quite flat efficiency curve. Further investigations are needed to test the behavior of the turbine under simultaneous varying conditions and in general to optimize the concept before manufacturing a full-scale machine. For this reason, the first devices that will be realized may not utilize this technology, but a set of Kaplan turbines instead. In any case, the flow to the turbine is regulated by gates that are virtually the only moving parts of the structure; this is an important characteristic for any device working on marine environment where loads on extreme events can be 100 times bigger than in operating normal conditions.

3. Optimization of the device

The optimization of the device regards particularly the geometry and the turbine strategy. These two aspects are tidily bonded one to the other as it will be explained.

With regard for the length and the inclination of the front plates leading to the different reservoirs, these are designed with the following purposes:

- Optimize the energy captured (waves overtopping and run-up).
- Reduce loads during design conditions.

Not only the wave climate but also the bathymetry of a specific location plays an important role on the design of the frontal plates as well as of the frontal "apron" at the toe of the structure that

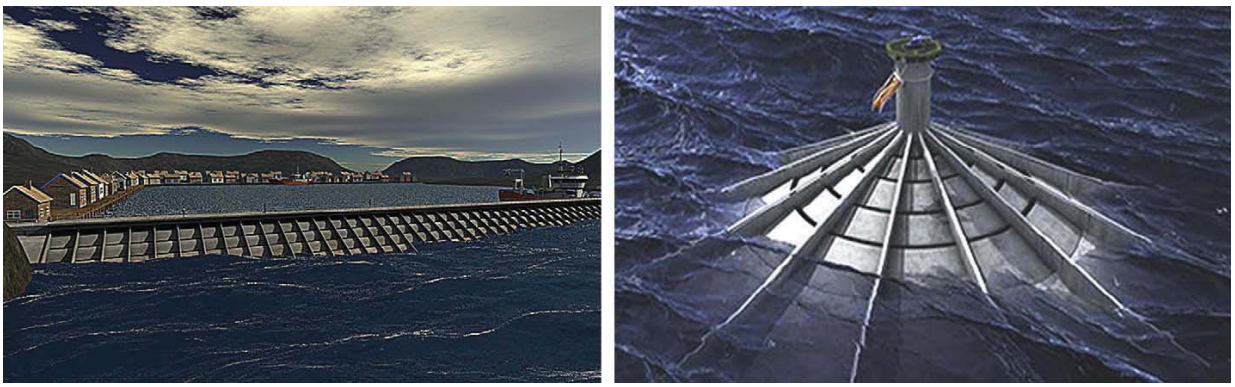

Fig. 2. Two applications of the SSG wave energy converter: on breakwaters (left) and offshore (right).

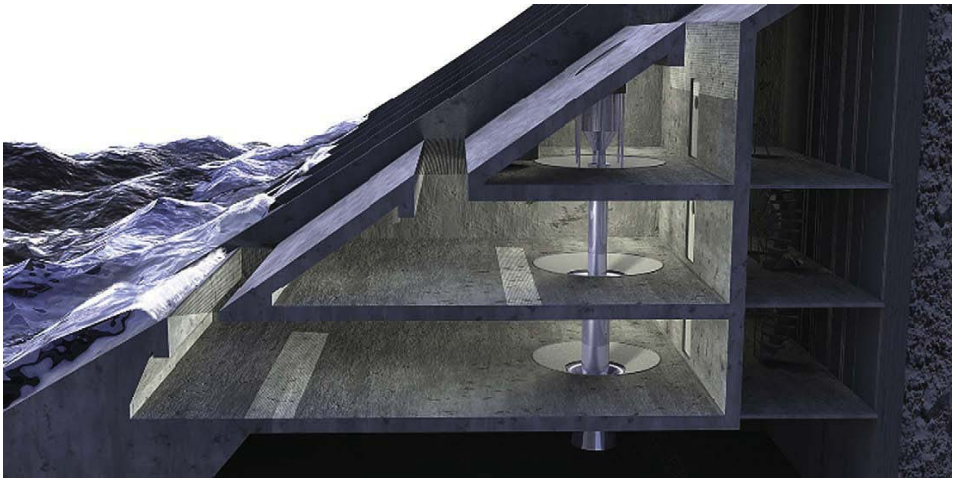

Fig. 3. Lateral section of a three-levels SSG device with Multi-stage Turbine (MST).

contributes to increase the run-up by offering a convenient slope to the incoming waves. The dimensions of the gaps between one reservoir and the above are controlled by the orthogonal distance between the reservoir and the fronts.

As mentioned, the dimensions of the reservoirs are affected by the strategy of the turbines: waves have a stochastic variation in height and period and it is impossible to predict how much water the next wave will bring, thus the dimensions of the reservoirs must be defined together with the operating strategy for the turbines. Using a multi level reservoirs results in a higher overall efficiency compared to single reservoir structure [1,2] and despite the similarity of the SSG structure to a breakwater caisson, the available formulas in literature to predict the overtopping in coastal protection projects are not sufficient in this case as they don't contain any information about the vertical distribution of it. Nevertheless this information is necessary with respect of the matter of maximizing the amount of stored potential energy at different heights: this is done through optimization of the crest levels $R_{c,n}$ (Fig. 5). Equation (1) describes the distribution of the overtopping rate with respect to the vertical distance [3,4]:

$$Q\prime = \frac{dq/dz}{\sqrt{gH_s}} = Ae^{B(z/H_s)+C(R_{c,1}/H_s)} \quad (1)$$

where Q' is the dimensionless derivate of the overtopping rate (q) with respect to the vertical distance z, $R_{c,1}$ is the crest freeboard of the lowest reservoir and H_s is the significant wave height. The coefficients A, B and C are empirical and need to be fitted with experimental data measured on a scale model of the SSG.

When a wave has just filled a reservoir, the turbine operates at a maximum head and power. It would then be ideal to stop the turbine when the reservoir is just empty enough for the next wave to fill it back to the brim without spillage. If increasing the installed turbine capacity or the storage volumes of the reservoirs has beneficial effects on the overall efficiency of the device, the cost may limit the enterprise. This is exactly the case when increasing the length of the structure ($L_{1,2,3,}$ in Fig. 5) and consequently the storage volume available per meter of crest; (increasing the width is considered as building a multi-module device, thus not affecting the overall efficiency). For a bigger storage volume the gradient of the water level in the reservoir is lower and the spillage losses are reduced, but the price of the construction is higher. In the other hands, when increasing the turbine capacity the consumption of energy utilized by the start/stop cycles of the turbines increases too, with a negative impact on the relative efficiency gain. The optimum turbine operating strategy minimizes the sum of head and spillage losses and finds a balance among all the above mentioned aspects. This can be done only by modelling the behavior of the whole system in the time domain. The SSG2 Power Simulation [5] is a program that has been realized in order to investigate the optimal geometry and the turbine strategy for the SSG wave energy converter. The power simulation program models the time distribution of the wave overtopping using a random process and the formulation for the overtopping flow rate of Equation (1). Every wave period is divided into a number of time steps for which Q (overtopping inflow), Q_{spill} (spilling discharge if above reservoir overflows), H_T (turbine head), Q_T

Fig. 4. Three-levels Multi-stage Turbine.

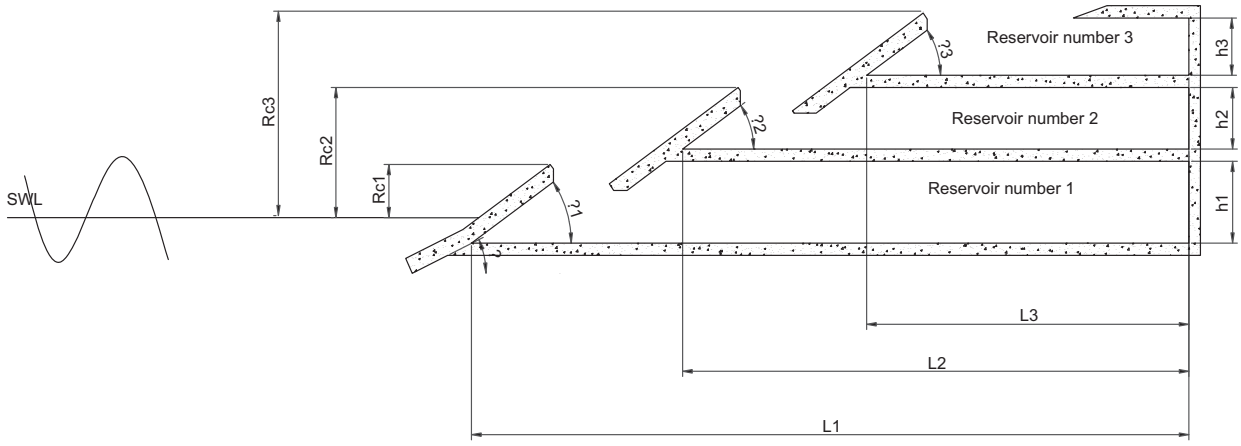

Fig. 5. Definition sketch of the SSG.

(turbine discharge), l_{res} (change of reservoir level) and P_{el} (generator output) are calculated. The used algorithm takes into account the spilling of the water from an above reservoir to a lower when the first is full and the head losses. In order to full fill its purpose, the program can elaborate various parameters that can be altered like geometry, sea state, turbine configuration (relationship between head, flow and efficiency) and turbine strategy (set points for switching turbines on and off). The power simulation program, supported by physical model tests, allows to predict the efficiency of the SSG in any wave situation and to estimate the annual power production.

4. Efficiency

Ideally, the power stored in the reservoirs (P_{res}) is the power related to the potential energy in the incoming waves (P_{wave}); in reality this storage incurs a limitation due to the dimensions of the structure and to the strategy of the turbines. For example, the power in the overtopping (P_{crest}) depends directly on the crest height, while the power in the reservoirs depends on their water level. In the same way, the power at the turbine (P_{tur}) is partially lost due to the hydraulic quality of the design and due to start-up and shutdown losses and so for the power at the generator (P_{gen}).

In Table 1, values for the partial efficiencies are presented; it has been estimated that overall efficiencies in the range of 10–26% can be obtained for different wave conditions.

Table 1
Relative efficiencies of the different energy conversion steps for the SSG device.

Formula	Definition	Efficiency %
$\eta_{crest} = \frac{P_{crest}}{P_{wave}}$	Hydraulic efficiency	30–40
$\eta_{res} = \frac{P_{res}}{P_{crest}}$	Reservoir efficiency	35–80
$\eta_{tur} = \frac{P_{tur}}{P_{res}}$	Turbines efficiency	80–90
$\eta_{net} = \frac{P_{gen}}{P_{tur}}$	Generator efficiency	95–97
$\eta_{tot} = \frac{P_{output}}{P_{wave}}$	Overall efficiency	10–26

5. Pilot project

The location for the pilot plant is the West part of the island of Kvitsøy in the Bokna fjord in Norway (Fig. 6). Kvitsøy municipality has 520 inhabitants and is one of 10,000 islands in Europe where wave energy can quickly be developed into a cost-effective energy production alternative to existing diesel generators. Preliminary estimates by WAVEnergy AS for first commercial shoreline SSG is that a full-scale plant of 500 m length will be able to produce 10–20 GWh/year for a price of electricity around 0.12 EUR/kWh. Even though indicative, such a price shows that the device will be cost effective and already competitive with the prices resulting from generating electricity on islands by means of diesel generators.

The SSG pilot project will be realized as a robust concrete structure built on the rocky shoreline and it is designed for a life

Fig. 6. The island of Kvitsøy, selected location for the SSG pilot plant.

Table 2
Summary of wave conditions at Kvitsøy location, with direction and probability of occurrence

T_p [s]	6.1	7.9	9.3	10.6	11.7	12.7	13.7
H_s [m] NW-315	1.2	1.7	2.2	2.6	3.2	4.4	6
Direction [deg.]	315	313	310	308	305	303	300
Prob. [%]	9.90	8.70	5.40	2.70	1.10	0.50	0.20
H_s [m] W-270	1.3	2.3	3.4	4.6	5.9	7.4	8.9
Direction [deg.]	270	273	275	278	280	283	285
Prob. [%]	4.80	4.20	2.60	1.30	0.60	0.20	0.10
H_s [m] SW-255	0.8	1.7	2.9	4.1	5.3	6.5	7.7
Direction [deg.]	225	230	235	240	245	250	255
Prob. [%]	7.50	6.50	4.00	2.00	0.90	0.40	0.10
H_s [m] S-180	0.6	1	1.2	3.2	4.2	5.5	7.1
Direction [deg.]	225	228	230	233	235	238	240
Prob. [%]	8.10	7.10	4.40	2.20	0.90	0.40	0.10

time of 25 years. The layout chosen consists of three reservoirs placed on the top of each others: total dimensions of the structure are approximately 17 m (length L_1) $\times$ 10 m (width) $\times$ 6 m (height). The crest levels ($R_{c1,2,3}$) are at 1.5 m, 3 m and 5 m and the inclination of the front plates resulted to be optimum at 35°. The structure is built in one piece and transported to the location via sea and installed on the foundation ensured by a number of tie beam anchors. The SSG pilot WEC will be connected to the grid for electricity production and the structure will be equipped with measuring devices in order to:

- monitor the efficiency and collect data;
- supervise the behaviours of the structure and for security reasons;
- validate model tests results.

The pilot project has been partially funded by the EU within the 6FW in 2004 with the objective of demonstrating at full scale the operation of a 150 kW module of the SSG wave energy converter, including turbine, generator, control system. Specific objectives of the 6th framework WAVESSG project are

- to design a full-scale technical prototype of the innovative MST turbine;
- to manufacture, test and install a full-scale technical prototype of the innovative MST turbine technology on the SSG;
- to design a full-scale 150 kW generator and control system;
- to measure the performance of the device including the structure in a period of up to six months for reliability and life time assessment;
- to manufacture, test and install a full-scale generator and control system equipped for grid connection;
- to obtain an hydraulic efficiency of at least 39% for the shoreline application;
- to obtain a wave to wire efficiency of more than 25% during the test period;
- to obtain a 96% availability of plant (with regard to operational hours);
- to obtain a 85% availability of production (with regard to wave climate).

The pilot project is meant to obtain reliability of the technology and contribute positively to wave energy; at the same time secondary issues like investigation of scale effects on pressures and overtopping flows will benefit of the high level of instrumentation of the SSG module. Field data within the pilot project will allow the correlation between real sea measurements, numerical model and tank testing.

The study of the wave climate at the selected location could benefit from three different offshore wave data sets: measurements at Utsira during the period 1961–1990 and from a buoy explicitly installed *m* offshore the selected location during the period 4/11/2004–11/3/2005 100; hindcast data from DNMI (Norwegian Meteorological Institute) during the period 1955–2005. The bathymetry of the area includes 100 m water depth on West direction, a plateau in front of the structure extending for 300 m at an average of 30 m depth and a steep slope leading to shore ($\approx 35°$). Such a bathymetry will allow higher waves to overtop the structure as waves of less then 15 m are not expected to break on the plateau. Transformation of waves from offshore to shore has been done by using the computer model MildSim developed at Aalborg University (AAU) [6,7]; the results are plotted in Table 2. For the device, West orientation was chosen being the best for capturing wave power. The near shore overall average power is estimated to be

Fig. 7. Physical setup of 3D laboratory tests on wave loadings. On the right the model in scale 1:60 equipped with pressure transducers.

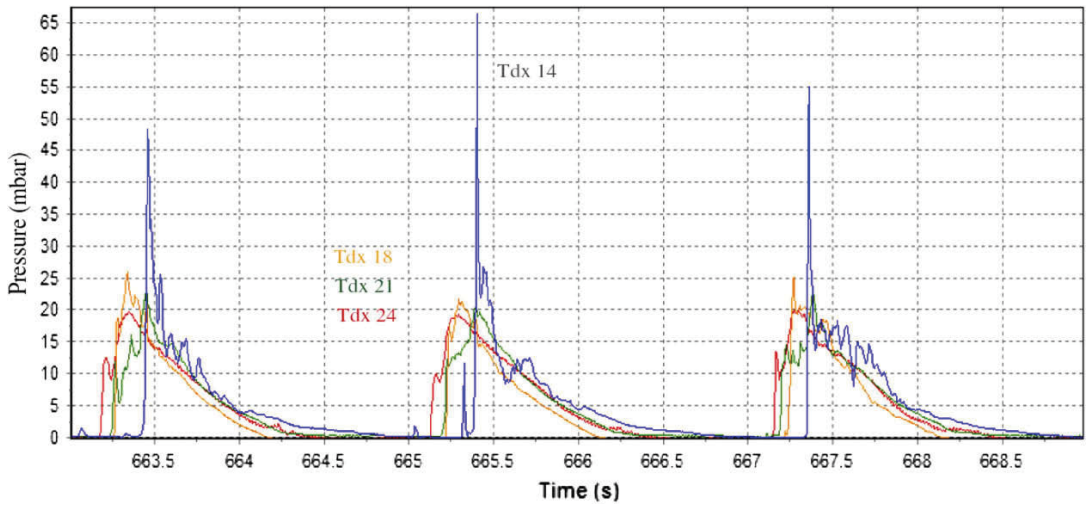

Fig. 8. Comparison of the pressures in the 3 front slopes (similar signals) and on the vertical real wall in the upper reservoir (peaked signal).

19.6 kW/m, (when neglecting wave conditions with H_s less then 1 m and more than 8 m, with a probability of occurrence, respectively, of 12.9% and 0.1%).

6. Wave loading

As mentioned during the description of the SSG concept, despite of the similarities of the structure to a breakwater caisson, the large differences existing prevented the utilization of literature formulas for design purpose without further investigations. This has been the case with overtopping and also with wave loadings. Thus, extensive 3D model tests on pressures under extreme conditions have been necessary in order to work out the design values for the construction of the structure. For those tests, the scanned bathymetry in the immediate proximity of the pilot plant was scaled down to model and used as the basis of the 3D laboratory tests (Fig. 7). SSG model in scale 1:60 to prototype, equipped with 14 pressure cells to measure 25 positions on the structure was tested under 32 different wave conditions, including waves of 100 years return period. In Fig. 8 difference between the pressures on the 3 front slops and on the rear vertical wall in the upper reservoir that was in the first design version of the SSG is presented: on this wall impact pressures under extreme waves (very peaked, short duration) of up to 580 kN/m^2 scaled to prototype were registered [8]. In order to avoid such loading on the structure vertical walls parallel to the attack wave crest will be avoided within the final design. In Fig. 9 a pressure history plot of 9 s for the three front plates is presented: the generated wave pressures do not vary substantially from one plate to the other, thus a quasi-static loading time history is recognizable. The order of magnitude of extreme peak pressure on front plates was up to 250 kN/m^2 scaled to prototype. Tests show an underestimation using prediction formula between 20 and 50% [9].

The results of these tests have also been used during the procedure to ensure the device as first full-scale wave energy converter.

7. Energy capture

The hydraulic efficiency has been defined as the ratio between the power in the overtopping water (Eq. (2)) and the potential power in incoming waves (Eq. (3)):

$$P_{\text{crest}} = \sum_{j=1}^{3} q_{\text{ov},j} R_{C,j} \rho g \tag{2}$$

$$P_{\text{wave}} = \frac{\rho g^2}{64\pi} H_S^2 T_E \tag{3}$$

where q_{ov} is the total overtopping in the single reservoirs and $T_E = m_{-1}/m_0$ is the energy period, where m_n is the n-th moment of the wave spectrum defined as

$$m_n = \int_0^\infty f^n S(f) df \tag{4}$$

Preliminary 2D tests with regular waves have been done in order to investigate a number of geometrical layouts and calculate a preliminary value of the stored potential energy in the reservoirs. A number of parameters influencing the capture of the overtopping water has been considered, those are the angle of inclination of the front plates, the horizontal distance between the front plates, the length of the frontal apron and the crest free boards $R_{c,n}$. During this phase measurements of overtopping flow rates for the individual reservoirs and incoming waves allowed the further calculation of the energy in the overtopping water and the hydraulic efficiency of the SSG pilot. At a second stage, 2D tests with irregular waves have been carried out in order to maximize the power capture of the SSG pilot and to estimate the efficiency of the device. This let to the present geometry of the wave energy converter that will be built in the island of Kvitsøy, Norway.

From the 2D physical tests on SSG model the hydraulic efficiency resulted to be 46%. This preliminary result was expected to change to a value of 40% for a 3D structure and to a value of around 30% in real operating conditions (effect of directionality and spreading of waves). This was verified by 3D laboratory tests where the effect of spreading and directionality was investigated separately and the effect of the combination of both was deduced [10]. In Fig. 10 on the left the 3D tests results are plotted against the efficiency with spreading divided the efficiency without spreading. The results are plotted for four typical wave conditions for West direction (frontal wave attack) at the selected pilot location for 2.3 m $\leq H_s \leq$ 5.9 m (Table 2). It is obvious that directionality is the primary responsible for decreasing the hydraulic efficiency of the SSG pilot module due to its low width-to-depth

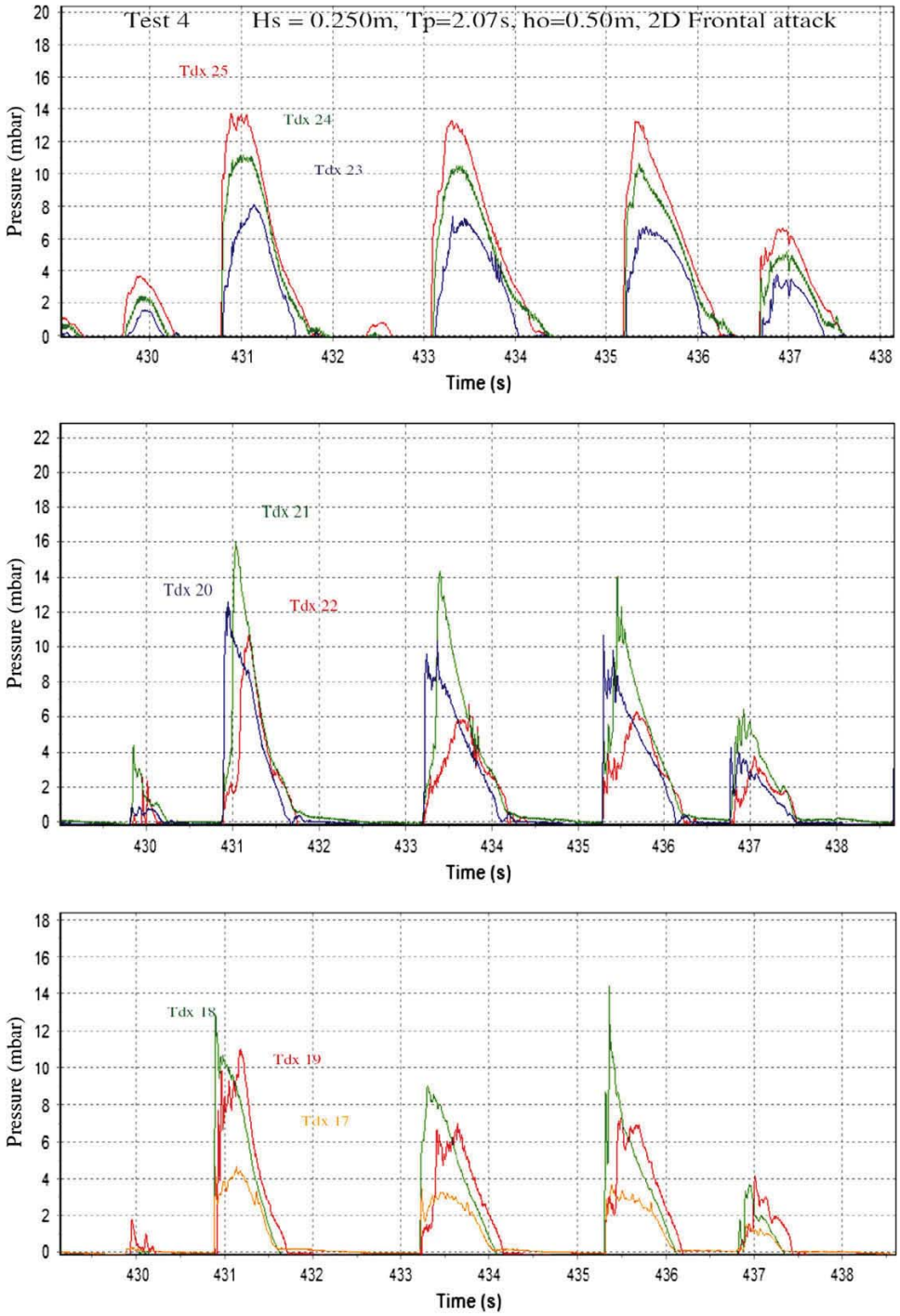

Fig. 9. Nine seconds history plot of the pressure in the 3 front plates (lower, middle and higher), each of them with signals from the pressure transducers.

ratio. In fact for side attack of waves, the side walls represent an obstacle to the storage of water. This "side effect" is not expected to be so dominant once that more that one module will be displaced one at the side of the other in a breakwater configuration.

8. Power takeoff

For the pilot plant of the SSG at island of Kvitsøy, 4 Kaplan turbines of identical size (0.6 m runner diameter) will be used: two in the lower reservoir and one in each of the middle and upper

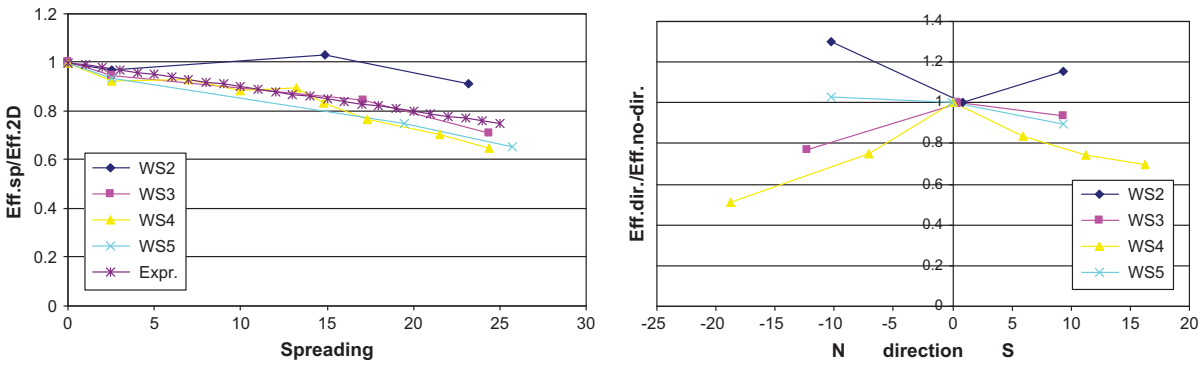

Fig. 10. Effect of wave spreading (left) and directionality (right) on the hydraulic efficiency of the SSG for different wave conditions (WS2,3,4,6 indicates increasing values of H_s).

reservoirs [11] (Fig. 11). The turbines will be manufactured using corrosion resistant steel. Due to economical considerations, the size of the reservoirs in the SSG pilot will be of the same order of magnitude as the overtopping resulting from a single big wave and this means that the turbines have a high frequency of start/stop cycles, approximately every 2 min. The cylinder gate has been chosen as mechanism to regulate the flow to the turbine; it consists of a cylinder directly combined to the turbine that when lifted allows the radial water flow to it. The cylinder gate seals by metal-to-metal contact to the outer turbine ring, closing by its own weight and offering a good reliability and transient time. The generators will be allocated at a higher level in order to avoid the risk of floods. They are driven by a tooth belt step-up drive which allows them to be matched with the optimal turbine speed. For the power levels of the pilot plant (15–100 kW), standard generators will be used.

9. Data acquisition and control system

The real time control of the pilot plant is one issue for the data acquisition and control system. This will mainly consist of generator control and emergency handling. Moreover, principal objective of the data acquisition and control system is to monitor the efficiency of the conversion of wave power into electrical power, thus the efficiency of the device, stage by stage. The structure will be monitored with pressure transducers for water levels measurements inside the reservoirs and run-up measurements on the slopes; these will record at 5 Hz in normal conditions. In storm and pre-storm conditions instruments on the front slopes will start acquiring at 20 Hz to investigate the occurrence of impact forces; this will be done by mean of a trigger criteria related to waves overcoming maximum heights. Moreover, significant wave heights, peak periods and energy periods will be processed by the whole spectrum of waves that will also be recorded in front of the SSG location. In Table 3 the measuring equipment for monitoring the hydraulic performance of the SSG in the island of Kvitsøy is presented. Moreover, the generator will be instrumented and the power production from the turbine measured directly on the generator. The data time series for each signal channel will be stored and elaborated in order to calculate statistical values. Once the pilot project will be built, it will be possible to carry out an

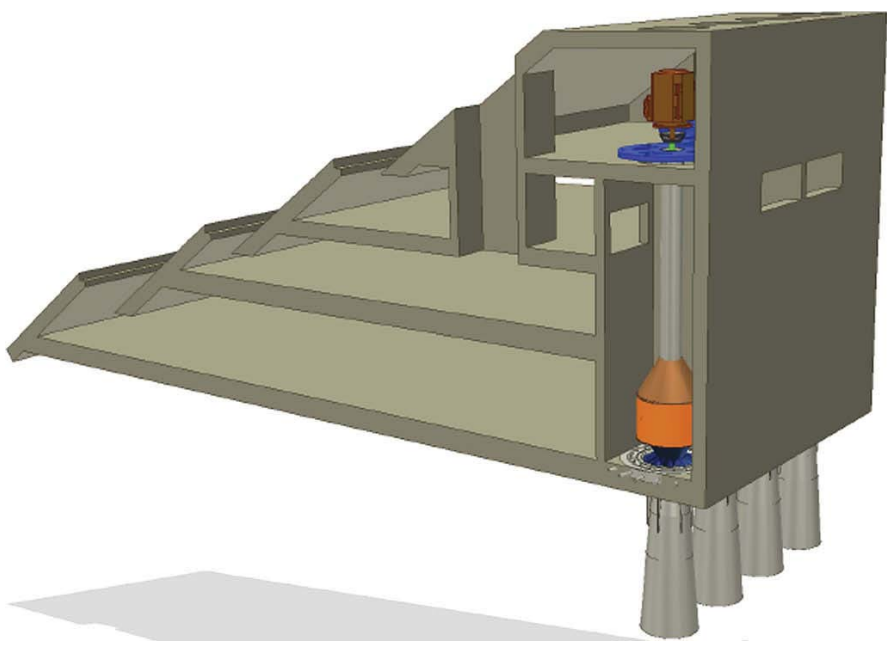

Fig. 11. The SSG pilot with 4 Kaplan turbines with cylinder gates, dry room for generators and outlets for the air trapped in the reservoirs.

Table 3
Instruments for monitoring the hydraulic behavior of the SSG pilot.

Sensor	Position	Sample freq.	Output	Range limits
Press. transducer	Lower reservoir, front position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Lower reservoir, middle position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Lower reservoir, rear position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Middle reservoir, front position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Middle reservoir, middle position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Middle reservoir, rear position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Upper reservoir, front position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Upper reservoir, front position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Upper reservoir, front position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Lower ramp, upper position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Lower ramp, middle position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Lower ramp, lower position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Middle ramp, upper position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Middle ramp, middle position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Middle ramp, lower position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Upper ramp, upper position	5 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Upper ramp, middle position	5–20 Hz	4–20 mA	0–100 mH_2O
Press. transducer	Upper ramp, lower position	5–20 Hz	4–20 mA	0–100 mH_2O
Press. transducer	In tail water	5–20 Hz	4–20 mA	0–100 mH_2O
Wave-rider buoy	100 m in front of the structure	2 Hz		

Table 4
Power matrix for the SSG pilot in the island of Kvitsøy, in a 19 kW/m wave climate.

H_s [m]	T_p [s]						
	5	6.1	7.9	9.3	10.6	11.7	12.7
0.5	0	0	0	0	0	0	0
1	3	3	3	3	3	3	3
1.7	21	21	21	20	20	20	20
2.4	67	65	62	60	59	57	56
3.6	144	140	134	130	127	124	122
4.7	154	152	150	148	146	144	143
5.9	155	154	152	151	149	148	147

important work regarding scale effects on pressures and overtopping by comparing the measurements at the pilot location to laboratory results.

10. Power production

The power matrix on Table 4 gives the power (kW) in different sea states for a structure with the characteristics of the pilot project. It should be noticed that while the main diagonal of the matrix corresponds to results from physical tests, the others are esteems obtained with the SSG2 simulation program. The formulas of the above-said program do not express a dependence of the power output on the period. This is because the period has not relevant influence on the average power output in long term. Moreover, the sides of the power matrix are very low probabilities combinations of significant wave heights and peak periods that induce breaking. By combining the power matrix with the probability of occurrence of the events, we obtain an expected annual production of approximately 320 MWh/y.

11. Conclusions

A new wave energy converter for electricity production based on overtopping principle has been tested and is now ready to be installed in the island of Kvitsøy, Norway. The device will be fully instrumented and will give real time data about energy production, wave loading and performance. Moreover the pilot project will contribute to the verification of results obtain in physical model tests and identification of scale effects.

The extensive preliminary studies led to the optimization of the design and even if some compromises have been done in order to realize the first full-scale wave energy converter (Kaplan turbines, low width-to-depth ratio…) their influence on the power production has been estimated. The pilot project will have

- three reservoirs one on the top of the other;
- an installed capacity of 163 kW;
- 4 hydro-turbines turbines;
- an annual production of 320 MWh/y.

Other devices are in the final stage of their R&D phase approaching the real sea testing with prospects for successful implementation. Nevertheless, extensive R&D work is continuously required, at both fundamental and application level, in order to improve steadily the performance of wave power conversion technologies and to establish their competitiveness in the global energy market.

Acknowledgements

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References

[1] Kofoed JP, Osaland E. Crest level optimization of the multilevel reservoir overtopping based wave energy converter seawave slot-cone generator. In: Sixth European wave and tidal conference, Glasgow.
[2] Kofoed JP. Experimental hydraulic optimization the wave energy converter seawave slot-cone generator. Hydraulics and Coastal Engineering 2005;26. ISSN: 1603-9874. Department of Civil Engineering, Aalborg University.
[3] Kofoed JP. Vertical distribution of wave overtopping for design of multi level overtopping based wave energy converters. In: 30th International conference on coastal engineering, San Diego; 2006.
[4] Kofoed JP. Wave overtopping of marine structures – utilization of wave energy. PhD thesis, Aalborg University. ISSN 0909-4296. Series paper no. 24; 2002.
[5] Meinert P, Gilling L, Kofoed JP. User manual for SSG power simulation 2. Technical report, Aalborg University. ISSN 1603-9874. Series paper no. 44; 2006.

[6] Andersen S, Klindt T. Random wave propagation–based on the mild-slope equation. Master thesis, Department of Civil Engineering, Aalborg University; 1994.
[7] Brorsen M, Helm-Petersen J. On the reflection of short-crested waves in numerical models. In: Proceedings from the 26th international conference on coastal engineering, Copenhagen; 1998. p. 394–407.
[8] Vicinanza D, Kofoed JP, Frigaard P. Wave loadings on seawave slot-cone generator (SSG) at Kvitsøy island (Stavanger, Norway). Hydraulics and Coastal Engineering 2006;35. ISSN: 1603-9874. Dep. of Civil Eng., Aalborg University.
[9] Vicinanza D, Frigaard P. Wave pressure acting on a seawave slot-cone generator. Coastal Engineering 2008;55. ISSN: 0378-3839:553–68.
[10] Margheritini L, Kofoed JP. Hydraulic characteristics of the seawave slot-cone generator pilot plant at Kvitsøy (Norway). In: Seventh European wave and tidal energy conference, Porto; 2007.
[11] Knapp W, Mayr B, Faust M, Schilling R. Optimal power takeoff system layout for the seawave slot-cone generator Kvitsøy pilot plant. In: Seventh European wave and tidal energy conference, Porto; 2007.

Sea Slot Cone Generator Overtopping Performance in 3D Conditions

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Margheritini, Lucia; Vicinanza, Diego; Frigaard, Peter.

Thesis author´s contribution:

The Thesis author is the first author of this paper. She performed the 3D laboratory tests on overtopping supervised by co-author P. Frigaard and received ideas for analysis from co-author D. Vicinanza.

Sea Slot Cone Generator overtopping performance in 3D conditions

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ABSTRACT This note describes the influence of wave spreading, directionality and local bathymetry on the efficiency of the Sea Slot Cone Generator (SSG) wave energy converter pilot plant in Kvitsøy, Norway. This is an overtopping device i.e. its efficiency is directly proportional to the overtopping flows into the three reservoirs the device has one on top of each other. The overtopping flow rates have been measured separately for each one of them, together with incoming waves during physical model tests at Aalborg University. The influence of the significant wave height H_s and of the wave length L on the captured overtopping water is also described. It has been found that the performance of the SSG pilot plant will be negatively affected by spreading and directionality of the incoming waves as direct consequence of reduction on the overtopping flow rates of 10% - 35% compared to 2D conditions.

KEY WORDS: Wave energy; overtopping; breakwater; directional wave spectrum.

INTRODUCTION

Different Wave Energy technologies are competing in the Renewable Energy market after the huge energy potential they can benefit from has been proved. Developers´ efforts are lately concentrated on demonstrating the reliability of the devices and on lowering the price per kW of produced power.
The SSG is a wave energy converter of the overtopping type. It has a number of reservoirs one on the top of each other specially designed to optimize the storage of potential energy of incoming waves from a specific wave spectrum. Efficiency is then directly proportional to the overtopping water temporarily stored in the reservoirs. The SSG pilot plant is a 10 m wide (capturing width) structure with three reservoirs one on the top of each other and installed capacity of 190 kW. The water temporarily stored in the reservoirs on its natural way back to the sea passes through turbines spinning them up and generating electricity. The pilot project at the island of Kvitsøy in Norway has been partially funded by the European Union FP6 and has the purpose of demonstrating the functioning of one full scale module of the SSG wave energy converter, including turbines and generators in 19 kW/m wave climate (Margheritini et al. 2008). In this case the reliability issue has been initially solved by realizing an "on shore" device where loads on the structure (Vicinanza et al. 2006) are considerably smaller than offshore, while the cost per kW compares prices of electricity for remote areas supplied by diesel generators. Nevertheless, when going from offshore to shore the bathymetry can influence the overtopping flow rates i.e. the overall efficiency of the converter. Another promising application of the SSG concept is on breakwaters; but while the design of such structures is made to minimize overtopping and run up, the SSG design focuses on a combination of maximization of both these events. The purpose of the paper is to investigate the influence of 3D waves, irregular bathymetry and spreading on the overtopping flow rates for the 3 reservoirs of the SSG pilot plant at Kvitsøy location. The effect of H_s and L has also been investigated. The research has been done by mean of physical model tests in the deep wave tank of the hydraulic and coastal engineering laboratories at Aalborg University AAU equipped with 3D wave generator.

BACKGROUND OF THE STUDY

The overall efficiency of the device is the ratio between power output and the available wave power, given by the formula:

$$P_{wave} = \frac{\rho g^2}{64\pi} H_s^2 T_E \tag{1}$$

Where ρ=1020 kg/m^3, g = 9.82 m/s^2 and T_E is the energy period = m-1/m0, where mn is the n-th moment of the wave spectrum defined as:

$$m_n = \int_0^\infty f^n \Phi(f) df \tag{2}$$

Φ, is the frequency spectrum. It is possible to consider the efficiency of the SSG overtopping device as a combination of partial efficiencies for every one of which it is necessary an optimization of parameters. The hydraulic efficiency is defined as:

$$\eta_{hy} = \frac{P_{crest}}{P_{wave}} \tag{3}$$

Where:

$$P_{crest} = \sum_{j=1}^{3} q_{ov,j} R_{c,j} \rho G \tag{4}$$

$q_{ov,j}$ is the total overtopping flow rate for the j-reservoir and $R_{c,j}$ is the crest level of the respective reservoir (figure 1).
The overall efficiency of the device is then the combination of the hydraulic efficiency, storage efficiency (dependent on the reservoirs´ volumes), turbines and grid connection efficiency. The design of the front of the SSG deals with the optimization of hydraulic performance.

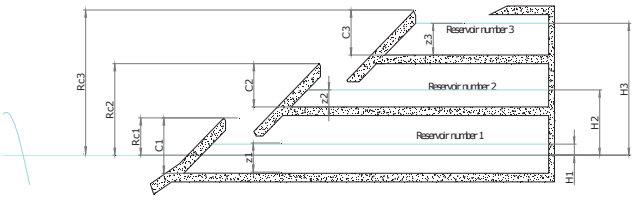

Figure 1. Definition sketch.

An expression for prediction of vertical distribution of overtopping has been suggested by Kofoed (2002) in the form:

$$Q' = \frac{dq/dz}{\lambda_{dr}\sqrt{gH_s}} = Ae^{B\frac{z}{H_s}+C\frac{R_{c,1}}{H_s}} \qquad (5)$$

Where Q' is the dimensionless of the overtopping discharge with respect to the vertical distance z and $R_{c,1}$ is the crest freeboard of the lowest reservoir. λ_{dr} is a coefficient describing the dependency of the draught and coefficients A, B and C need to be fitted to experimental data for the specific case. The eq.(5) is for long crested waves and horizontal bottom (2D model).
For the SSG pilot plant a number of 3 reservoirs has been chosen as adding extra reservoirs would only increase the hydraulic efficiency of 2 points percentage) (Kofoed 2006). 2D physical model tests have been carried out in order to optimize the geometry of the SSG pilot device (Kofoed 2005a). More then 30 geometries were tested under 2D irregular waves changing angles of the fronts, distances of the fronts, length of the fronts and crest levels. The analysis of overtopping flow rates in the 3 reservoirs from the best performing geometries lead to a set of coefficients A, B and C = 0.197, -1.753 and -0.408 respectively. As a result the final geometry has been defined with $R_{c1,\,2,\,3}$ = 1.5 m, 3m and 5m above SWL, angles of front = 35° and frontal front extended 5 meters under water level. An hydraulic efficiency of 50% has been estimated. 2D tests as such did not take into consideration the effect of bathymetry, directional wave spectrum and spreading, all phenomena that can influence the overtopping flow rates in the reservoirs.

EXPERIMENTAL SETUP

The model of the SSG used in laboratory was built at 1:60 scale and it was fixed rigidly on a 3D concrete model of the cliff located in the middle of the basin at 5 m from the paddles. The cliff is the best reproduction of the scanned bathymetry of the pilot plant location. The cliff has a very steep angle leading quickly to the sea bottom at –30 m CD. The pilot plant is more sheltered from waves coming from the left side as the cliff emerges from water (figure 2). The geometry of the model was realized according to the optimizations done by Kofoed, (2005a). The rear part of model was modified and equipped with four slopes leading to different small tank containers: one for each reservoir plus one for the overtopping over the whole structure. In this way infinite reservoir capacity was simulated. The captured overtopping water was temporally stored and then pumped out again in the basin by small pumps of known performance; the pumps were automatically activated when the water inside each container reached a pre-established level (figure 3). By the total utilization of the pumps and the records of water levels inside the rear tanks, the overtopping volumes and flow rates have been derived for the single reservoirs.
The measuring equipment included:

- 4 wave gauges installed to measure time series of water levels in the reservoirs tanks.
- 7 resistive wave probes on a pentangle array placed on the plateau in front of the model, enabling the collection of data for 3D wave analysis.

Tested sea states

Tested wave conditions refer to operating conditions of the SSG pilot plant at Kvitsøy (Kofoed and Guinot, 2005b). The wave generation is controlled by the software AWASYS5, developed by laboratory research staff (http://hydrosof.civl.aah.dk/AwaSys/). Generation of waves aimed to reproduce the following four offshore wave conditions: H_s =0.077 m and T_p= 1.37 s; H_s = 0.038 m and T_p= 1.02 s; H_s = 0.057 m and T_p= 1.20 s; H_s = 0.098 m and T_p= 1.51 s; with constant water depth of 0.51 m. Tests have been carried out generating waves with head on attack angle (D = 0) and with an attack angle varying between -15° and 15°for each of the wave condition; no spreading condition was added to wave directions. Further, 9 spreading conditions were tested for each wave condition; these have been run with head on attack angle. A narrower directional spectrum corresponds to higher input spreading parameters (S = 1000 ≈ 2D ≈ no spreading) as the directional spreading function adopted is expressed by a cosine power form.
2D conditions were also simulated in order to separate the effect of the 3D-ness of the structure from the effect of 3D wave spectrum. Each test comprised approximately 1500 waves.

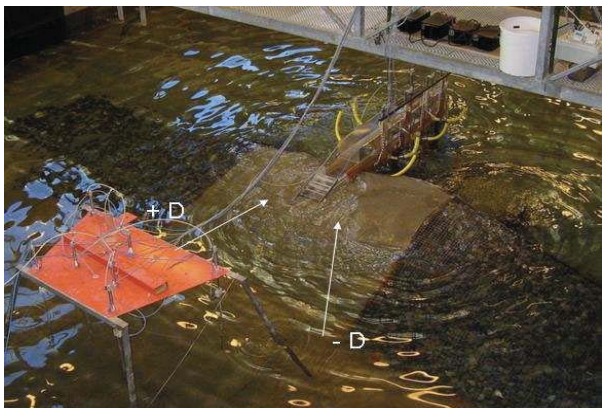

Figure 2. Tests setup. In evidence generated the wave directions.

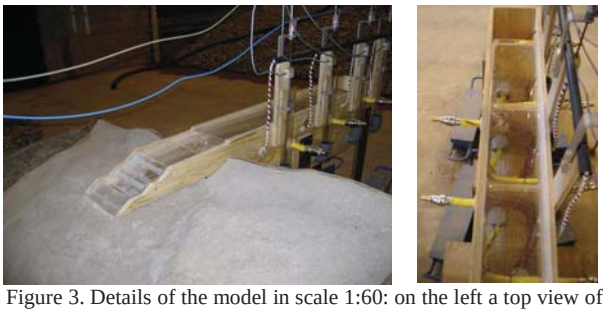
Figure 3. Details of the model in scale 1:60: on the left a top view of the rear tank containers equipped with pumps.

TEST RESULTS

Tests were carried out simulating spreading and different attack angles separated for each wave condition.

Dependency on the wave conditions

In figure 4 flow rates of the tests for the 3 reservoirs (q1, q2 and q3) are plotted for different spreading conditions. The results appear grouped in the graphics depending on the wave height (increasing with H_s). While little difference can be noticed comparing the 2D and the different spreading conditions for the same H_s in reservoir one and two, the difference between tests with low spreading (≈ 2D conditions) and high spreading are relevant in reservoir three for higher H_s; in this case higher spreading is limiting the amount of overtopping. In average an overall decrease by 10% of overtopping for the lower reservoirs and by 35% for the third reservoir is noticeable for situation with high spreading (S < 100) compared to situations without or with low spreading.

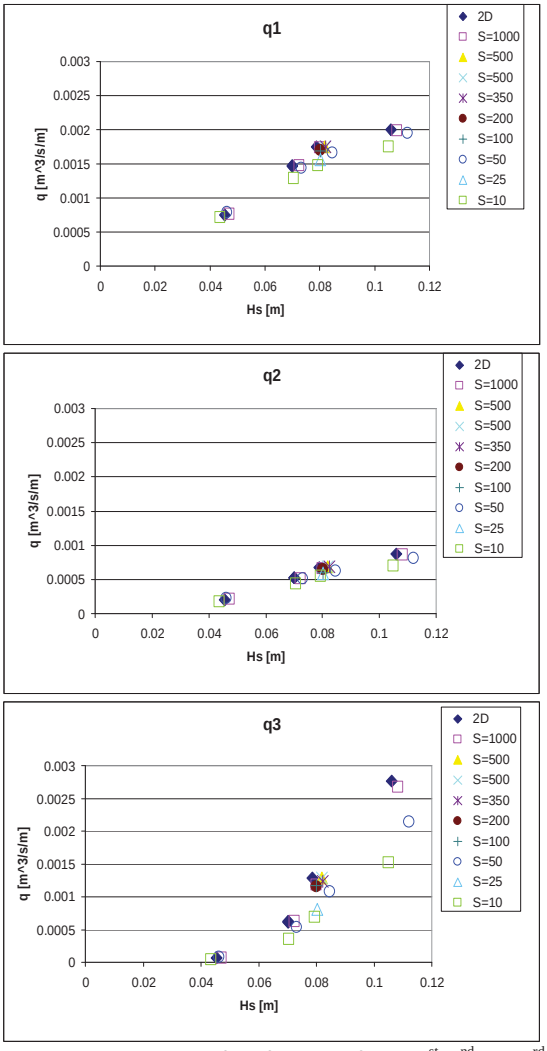

Figure 4. Dependency of the flow rates for the 1st, 2nd, and 3rd reservoir on H_s for different S and D=0. Scale model results.

In figure 5 the flow rates for the three reservoirs are plotted for different attack angles (D = 0= head on attack). Again little difference can be noticed in reservoir 1 and 2 when increasing D for the same H_s, while in reservoir number three the flow rates (q3) are very influence by the directionality (attack angle and directional spreading): for waves higher then 0.08 m directionality of incidents waves decreases the overtopping. When increasing D we can see the same reduction on overtopping rates that occurred when increasing S.

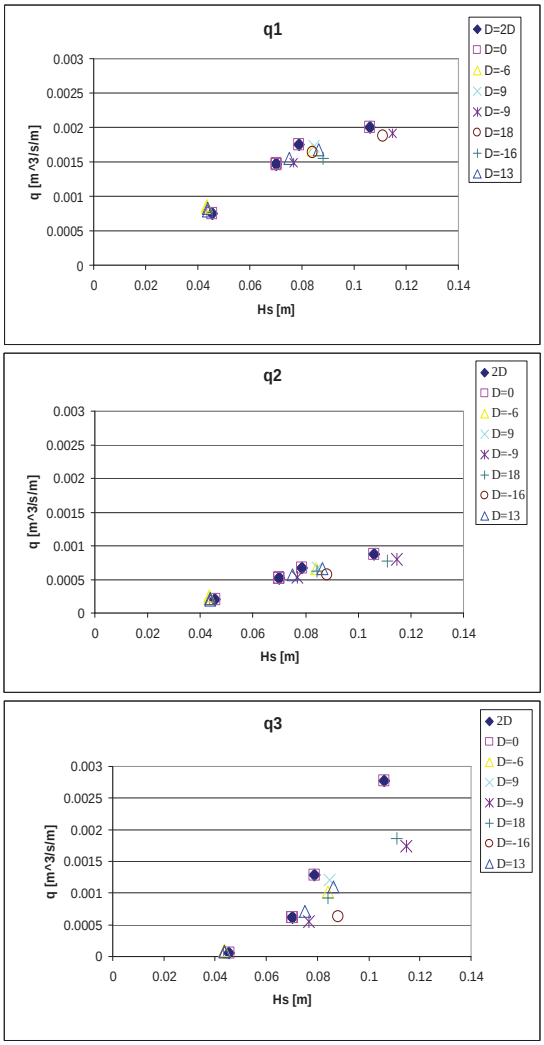

Figure 5. Dependency of the flow rates for the 1st, 2nd, and 3rd reservoir on H_s for different measured directions (attack angles in degree), no spreading. Scale model results.

The different trend that characterizes the dependency of the flow rates on H_s between reservoir one or two and reservoir three is due to the fact that the lowest ones have a roof while the highest one does not have any geometrical obstruction to the incoming flow. In other words it can be said that while the water to access the two lowest reservoirs has to enter in an opening, for the third reservoir the water needs to overtop a crest.

In general, the flow rates are higher in the third reservoir then in the lowest ones for bigger waves, according to expectations.

ANALISY OF RESULTS

Comparison with 2D test results

There is reasonable accordance when comparing the measured flow rates in the present set of tests in 2D conditions and in the 2D set of tests of Kofoed (2005a), figure 6. Nevertheless, it is possible to notice that there is better accordance to calculated results (integration of eq. 5) than to the measured results. At that time it was found that the formula was underestimating the flow rates for reservoirs number 1 and number 3. In this case the better fitting of the measured results to the calculated data could be explained by the occurrence of a scale effect as the model used here is at scale 1:60 instead of 1:25 as in the compared tests and so boundary effects are more relevant. Indeed, apart from the scale and the presence of bathymetry, the sample of results compared here was produced for the same conditions (waves and geometry).

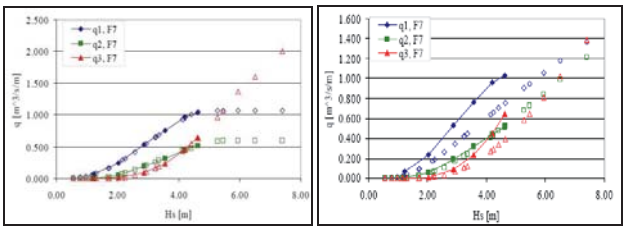

Figure 6. Model test results from Kofoed (2005a) for the same geometry of the present set of tests with inter and extrapolated data (open marks left) and with calculated data (open marks right).

Dependency on the wave length

By plotting the normalized overtopping flow rates against the R_C/L_{P0} (L_{P0} = offshore wave length referring to peak period T_P) for the different spreading conditions for reservoir 1 it can be noticed that before reaching a constant value around 0.02, the overtopping flow increases when decreasing L_{P0}., figure 7. For reservoir number 2 the constant value of 0.0075 is reached immediately, as shown in the same figure.

A completely different trend is found for flow rates in reservoir number 3: when the ratio R_C/L_{P0} increases the captured overtopping water degreases linearly for all the different spreading conditions (figure 8). This can be explained considering that the higher reservoir needs longer, bigger waves to be overtopped. By comparing the flow rates in the case with high spreading (S=10, dashed trend line) to the case with no spreading (2D, light continuous trend line) it is possible to estimate the losses of captured water in the higher reservoir.

The same behaviour can be found for the different reservoirs when plotting the normalized overtopping flow rates against R_C/L_{P0} for different attack angles of incoming waves: a longer wave "pumps" less water in first reservoir (figure 9) while increases the overtopping in the third reservoir (figure 10). This can be explained considering that steepest waves have a higher frequency and for the same time window more waves occur with shorter periods i.e. more water enters the first reservoir, but the height may be not enough to reach the higher reservoir. By comparison between the trend lines for the flow rates in the third reservoir for different directions, it seams obvious that the frontal attack (D = 0) brings more overtopping water than the cases when waves approach the structure with a certain angle. This is no longer evident when R_C/L_{P0} increases.

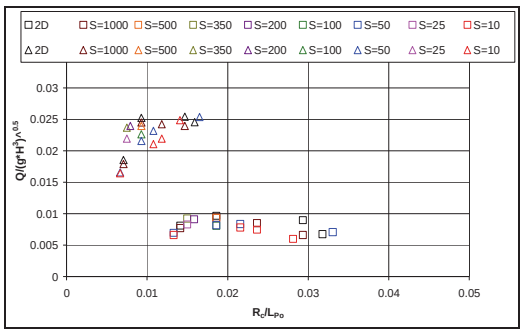

Figure 7. Dependency of the flow rates to reservoir no. 1 (triangles) and 2 (squares) on R_C/L_{P0} for different S.

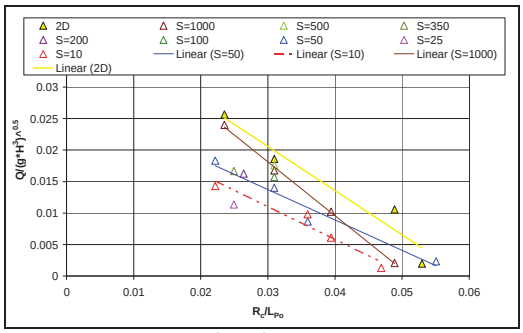

Figure 8. Dependency of the flow rates to reservoir no. 3 on R_C/L_{P0} for different S.

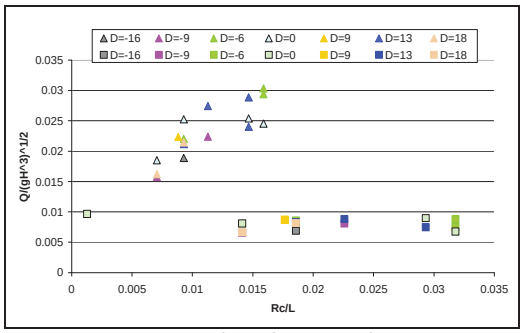

Figure 9. Dependency of the flow rates for reservoir no. 1 (triangles) and 2 (squares) on R_C/L_{P0} for different D.

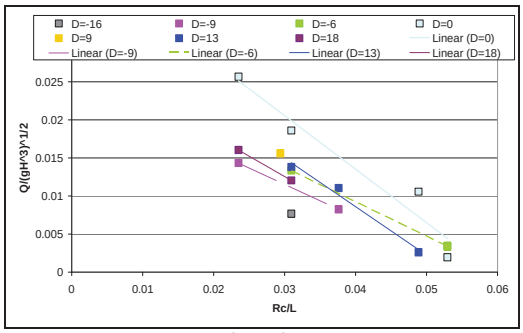

Figure 10. Dependency of the flow rates to reservoir no. 3 on R_C/L_{P0} for different D.

INFLUENCE OF 3D CONDITIONS

From the results presented in the previous section it is already possible to distinguish the effect of directional spectrum and spreading as reduction of overtopping water in the third reservoir (figures 8 and 10). It is also clear that the maximum overtopping in the different reservoirs occurs for different wave conditions. At the same time maximization of overtopping and optimization of the hydraulic efficiency are not the same thing as the last one aims to store bigger volumes selectively in the higher reservoirs.

In Figure 11 the calculated efficiency of laboratory tests with and without spreading is plotted against the efficiency with spreading divided by the efficiency without spreading (2D) for different wave conditions. W2, 3, 4 and 5 refer to the different tested wave conditions: W2: Hs = 0.038 m; Tp= 1.02 s. W3: Hs = 0.057 m and Tp= 1.20 s. W4: Hs =0.077 m and Tp= 1.37 s. W5: Hs = 0.098 m and Tp= 1.51 s

In black the overall trend of the results depending on spreading. A local effect regards the wave condition number 2 (W2) and it could be imputable to the different interaction of the specific short period of the waves with the bathymetry.

In Figure 12 the calculated efficiency of laboratory tests with and without directionality is plotted against the efficiency with directionality divided the by efficiency without directionality (2D) for different wave conditions. Again the W2 condition behaves weirdly when adding attack angle ≈ ±9°. What all the tests present is an asymmetry of the graphic. This is in line with the differences in the bathymetry at the location objective of this study: when waves approach the structure with +D attack angles they do not meet the same small mound then they do with – D attack angles (figure 2), but a favorable slope. In this way waves coming from the right side of the dive face smaller dissipation of energy and reach the reservoirs easily.

It is assumed that the efficiency will not go to zero while increasing the attack angle of incoming waves from 0 to ±90°. It is instead foreseen that the efficiency will stabilize around a certain value, also due to local effects caused by the wave-bathymetry interaction. The black line tries to represent this trend.

It is clear that directionality and spreading act on the same way on the overtopping for the three reservoirs of SSG pilot plant resulting in an overall reduction of the stored water up to 40%. This is specifically a problem for the SSG pilot plant as the device has a low width to depth ratio; in other worlds, because of the narrowness of the capture width, the lateral walls are an obstacle to the storage of overtopping water from incoming waves with an attack angle ≠ 0. Because for overtopping of breakwaters an attack angle -20 ≤ D ≤20 is not considered to have significant effects on the overtopping flow rates (Wave Overtopping of Sea Defences and Related Structures: Assessment Manual 2007), it is reasonable to think that implementing more modules of the SSG device close to the others forming a line along a section of the coast or on a breakwater, this effect would be reduced. The phenomena that appear to have relevance when passing from 2D to 3D conditions are listed bin table 1 with the evaluation of reduction of hydraulic efficiency from 50% realized in 2D conditions by Kofoed (2005a) for the specific case of the SSG pilot plant in the island of Kvitsøy.

Table 1.Reduction of the hydraulic efficiency from 2D to 3D conditions for the SSG pilot.

Reason of reduction of the η_{hy}	Average η_{hy}
- (2D conditions)	50%
Bathymetry	40%
Wave directionality	32%
Wave spreadind	35%
Bathymetry+wave direction+wave sprding	≈25%

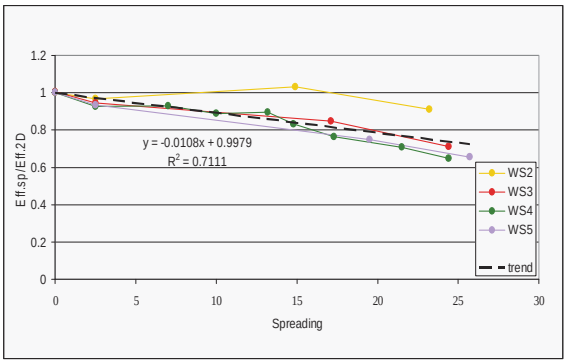

Figure 11. Influence of spreading on the hydraulic efficiency.

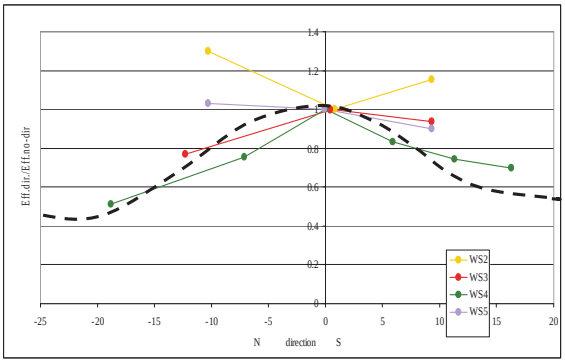

Figure 12. Influence of attack angle of incoming waves on the hydraulic efficiency.

CONCLUSIONS

By mean of 3D physical model tests in scale 1:60 to the SSG pilot plant in Kvitsøy it has been found that for bigger wave heights (and longer periods) the overtopping is higher in the third reservoir instead then in the lower ones; this is because the roof or the gaps between reservoirs one and two or two and three are setting an upper limit to the overtopping rates in reservoir one and two.

In general, to higher waves corresponds a higher volume of storage water in each of the three reservoirs.

It has also been found the effect of bathymetry: because of a non-symmetric, non-strait bottom, the overtopping flow rates are different for same |D| but different directions.

By mean of 3D physical model tests and comparison with previous 2D set of tests with the same model geometry and wave conditions but different scale and no reproduction of local bathymetry, the influence of boundary conditions, wave directionality and wave spreading on the hydraulic efficiency of the SSG pilot plant has been found.

It is clear that the phenomena listed above act reducing the amount of overtopping water in the reservoirs and then the hydraulic efficiency of 50% in average. This reduction is strait forward the reduction on energy capture. The main limitation of the structure are related to its low width to depth ratio, as incoming waves with attack angles different from head on will be reflected by the side walls of the device and not enter the reservoirs. This explains while, even with as small attack angle as ones tested in the present discussion, a considerable reduction of water storage occurs.

REFERENCES

Kofoed J. P. (2002): “Wave Overtopping of Marine Structures – Utilization of Wave Energy”. Ph. D. Thesis, defended January 17, 2003 at Aalborg University. Hydraulics & Coastal Engineering Laboratory, Department of Civil Engineering, Aalborg University.

Kofoed, J. P. (2005a): "Model testing of the wave energy converter Seawave Slot-Cone Generator", Hydraulics and Coastal Engineering No. 18, ISSN: 1603-9874, Dept. of Civil Eng., Aalborg University.

Kofoed, J. P. and Guinot, F. (2005b): “Study of Wave Conditions at Kvitsøy Prototype Location of Seawave Slot-Cone Generator”. Hydraulics and Coastal Engineering No. 25, ISSN: 1603-9874, Dept. of Civil Eng., Aalborg University, June, 2005.

Kofoed, J. P. (2006): "Vertical Distribution of Wave Overtopping for Design of Multi Level Overtopping Based Wave Energy Converters", 30th International Conference on Coastal Engineering, ICCE San Diego.

Margheritini, L., Vicinanza D., Frigaard, P. (2008): “SSG wave energy converter: design, reliability and hydraulic performance of an innovative overtopping device”, submitted to *Journal of Renewable Energy, Elsevier*.

Vicinanza, D., Kofoed, J. P. and Frigaard P. (2006), " Wave loadings on Seawave Slot-cone Generator (SSG) at Kvitsøy island (Stavanger, Norway)", Hydraulics and Coastal Engineering No. 35, ISSN: 1603-9874, Dep. Of Civil Eng., Aalborg University.

Wave Overtopping of Sea Defences and Related Structures: Assessment Manual (2007), EurOtop.

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Margheritini, Lucia ; Victor, Lander; Kofoed, Jens Peter ; Troch, Peter.

Thesis author´s contribution:

The Thesis author is the first author of this paper. First author is responsible for the results presented on this paper. She performed the laboratory tests on overtopping supervised by co-author J.P. Kofoed and with great help of co-author Lander Victor and assistance from co-author P. Troch. The first author is also responsible for the design of the setup and for analysis of the data with help of co-author Lander Victor.

Geometrical Optimization for Improved Power Capture of Multi-level Overtopping Based Wave Energy Converters

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ABSTRACT

In multi-level wave energy converters the water from incoming waves is stored in reservoirs one on top of the other. Prevision formula for the overtopping flow rates in the individual reservoirs is fundamental for dimensioning correctly the turbines and optimizing the device. Having a number of reservoirs one on top of each other means that the representative overtopping formulae for coastal structures are not sufficient to describe the phenomena. This paper proposes to describe the dependence of the overtopping on a new parameter which is the horizontal distance between the reservoirs or, in other words, the opening between two consecutive reservoirs. 13 different geometries have been tested in 2D irregular waves and a new formulation for prediction of overtopping in multilevel structures is presented.

KEY WORDS: Overtopping; wave energy converters, 2D testing, irregular waves, crest levels; optimization; power capture.

INTRODUCTION

Wave energy converters (WECs) based on the overtopping principle utilize the potential energy of incoming waves by storing the overtopping water in reservoirs where specifically design low head hydro turbines convert the available potential power. Some advantages have been foreseen for such a kind of devices over different WE technologies: in first place the derived electricity is characterized by small fluctuations because the conversion from wave energy to potential energy can benefit of a relatively calm water in the reservoirs. In second place the economics as well as the environmental impact of the device can be shared with other coastal facilities such as breakwaters. Despite the application on breakwater not being a prerogative of overtopping wave energy converters, these devices are the only ones that can contribute to improve the water quality in closed harbours with the seawater ejected from the turbine outlet. WECs based on the overtopping principle are Wave Dragon (Kofoed 2006a), Wave Plane (Frigaard 2008) and SSG (Margheritini 2008a). Considerable increase in stored energy from the overtopping water can be obtained by using multilevel devices (Kofoed 2006b); moreover this is the most effective solution for fix overtopping devices that can not adapt the crest free boards to the sea state by changing the buoyancy level like in Wave Dragon device. In the design of an overtopping device the main parameter to be defined are the crest free boards R_{cj}, j=1,2...n, n=number of reservoirs. These levels are defined through an iteration process that leads to maximization of hydraulic efficiency defined as:

$$\eta_{hyd} = \frac{\sum_{j=1}^{n} \rho g q_j R_{cj}}{\frac{\rho g^2}{64\pi} H_s^2 T_E} = \frac{P_{crest}}{P_{wave}} \tag{1}$$

Where ρ=1020 kg/m^3, g = 9.82 m/s^2, H_S is the significant wave height and T_E is the energy period = $m\text{-}1/m_0$, where m_n is the n-th moment of the wave spectrum. $R_{c,j}$ is the crest level of the respective reservoir and $q_{ov,j}$ is the total overtopping flow rate for the j-reservoir.
The parameters influencing the overtopping are well known and they include both effects of the wave climate as well as of the structure geometry (Van der Meer and Janssen 1995; Franco et al. 1995). A specific study on the influence of oblique waves and directional spreading has been done for fix geometry of specific multi-level WEC with good accordance of results with literature (Margheritini 2008b). The expression available now to calculate q_j in Eq. 1 is the integration of the derivative overtopping discharge with respect to the vertical distance z (Kofoed 2002):

$$\frac{dq/dz}{\sqrt{gH_S}} = Ae^{B\frac{z}{H_S}} e^{C\frac{R_{c,1}}{H_S}} \tag{2}$$

The coefficients A, B and C are fitted from laboratory tests, q is the average overtopping discharge per width [m^3/s/m], z is the vertical distance from the s.w.l., g is the gravity acceleration, H_s is the significant wave height, and R_{c1} the crest free board of the lower reservoir.
Nevertheless the structure geometry of multi-level WECs is such to require the introduction of new parameter to describe the overtopping into the reservoirs. From a comparison of the calculated and measured q_j during 2D physical model tests it emerged that Eq. 2 is imperfect in the description of the phenomena when varying the horizontal distance HD from the ranges in which Eq. 2 has been established. In other words it seams necessary to introduce a new relation expressed by:

$$\frac{dq/dz}{\sqrt{gH_S}} = f_1\left(\frac{z}{H_S}, \frac{R_{c,1}}{H_S}, HD^*\right) \tag{3}$$

In Eq. 3 *HD** is the adimensionalized horizontal distance between the opening of two consecutive levels (Fig 1).
The purpose of this paper is to define the effect of the horizontal distance *HD* on the overtopping discharge and find a parameter to be added in the existing formula that can include this effect.

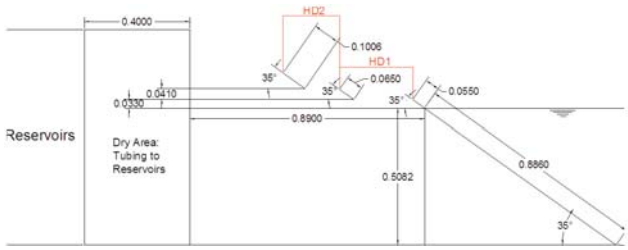

Figure 1. Definition sketch. measures in meters, side view.

TESTS SET UP

Tests in scale 1:30 of the North Sea conditions have been carried out in the shallow water wave flume at the Hydraulics and Coastal Engineering laboratory of the Department of Civil Engineering of Aalborg University. The flume is 25 m long, 1.5 m wide and 1 m deep. The flume is equipped with a piston type wave generator with a stroke length of approximately 70 cm. The wave generator is controlled by a PC-controlled DHU Servo Amplifier. The standard generation software is AWASYS, which is an active absorption system that can be used to generate both regular and irregular waves. AWASYS is developed in the laboratory.
The model was placed at the end of the flume, in center position occupying 0.514 m of in width (Fig. 2). Approximately 2 m long leading walls to the model assured pure 2D waves. At the sides of the model outside the leading walls an artificial dissipating beach was realized.
The multi-level model structure consisting of 3 horizontal metal plates with fronts inclined of 35° as it has been demonstrated that this is the optimal value for maximization of overtopping (Le Mèhautè et al. 1968, and Kofoed 2002). The plates can be dismounted to vary the number of reservoirs from 1 to 3 and can slide one respect to the others in order to change the horizontal distances *HD1* and *HD2* (Definition sketch, Fig. 1). The majority of the tests have been carried out with 2 levels and results will be presented only for this configuration. 13 different geometries with 0.30 m. <HD1<0.053 m. have been testes.
The crest levels R_{c1} and R_{c2} are fixed respectively at 0.0333 m., 0.0716 m. from mean water level (m.w.l.). A wooden run-up ramp 0.886 m. long inclined of 35° leads the waves to the model.

Wave conditions

2D irregular waves from the Jonswap spectrum (3.3 peak enhancement constant) have been generated during the tests with water depth = 0.51 m. in front of the structure. Tested wave conditions are presented in Table 1. Each tests lasted 30 minutes. Wave conditions have been selected among the most common in the North Sea (probability of occurrence > 5%), (W1, 2, 3, 4 in Table 1). Additionally, different conditions have been investigated by changing T_p for the same H_S (W1a, W2a,b; W3b,c; W4b,c in Table 1) in order to investigate the effect of the wave steepness:

$$s_0 = \frac{2\pi H_S}{g T_P^2} \qquad (4)$$

Table 1. Tested conditions, model scale.

Name	Hs [m]	Tp [s]	s0
W1	0.033	1.022	0.020403
W1a	0.033	2.066	0.004997
W2	0.067	1.278	0.026116
W2a	0.067	2.922	0.004996
W2b	0.067	0.924	0.049961
W3	0.100	1.534	0.027204
W3b	0.100	1.132	0.049932
W3c	0.100	2.531	0.009988
W4	0.133	1.789	0.026649
W4b	0.133	1.307	0.049941
W4c	0.133	2.922	0.009992

Wave measurements

Generated waves have been measured with 3 wave gauges in front of the structure, the closest one distanced 1.96 m. from the model, allowing the separation of incident and reflected waves according to the Mansard & Funke´s method (1980). The data acquisition was at 50 Hz.
For the wave analysis the software WaveLab 3 has been used developed at Aalborg University.

Overtopping measurements

The water overtopping the crest levels was temporary stored in rear tanks after passing the tubing section (refer to Fig. 2). To each level corresponds one rear tank equipped with pumps of known performance and wave gauges for measuring the water level inside the tanks. The pumps were automatically emptying the tanks by pumping back in the basin when the water reached a pre-established level. By the total utilization of the pumps and the records of water levels inside the rear tanks, the overtopping volumes and flow rates q_j have been derived for the single reservoirs during each test. Final data were the average overtopping discharges over the generated sea state for each test.

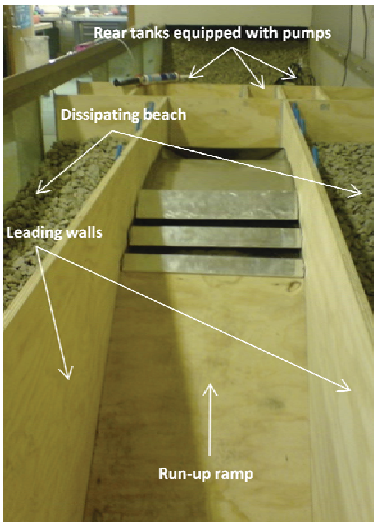

Figure 2. Front view of the model with 3 levels mounted.

RESULTS

When waves overtop the structure they showed surging behavior. The water first runs up the slope leading to the structure then the front of the first reservoir and if the wave is big enough, also the second front resulting in a cascade into the reservoir (Fig 3).
Following results are presented in terms of average overtopping discharge for each level with comparison with existing formulae.

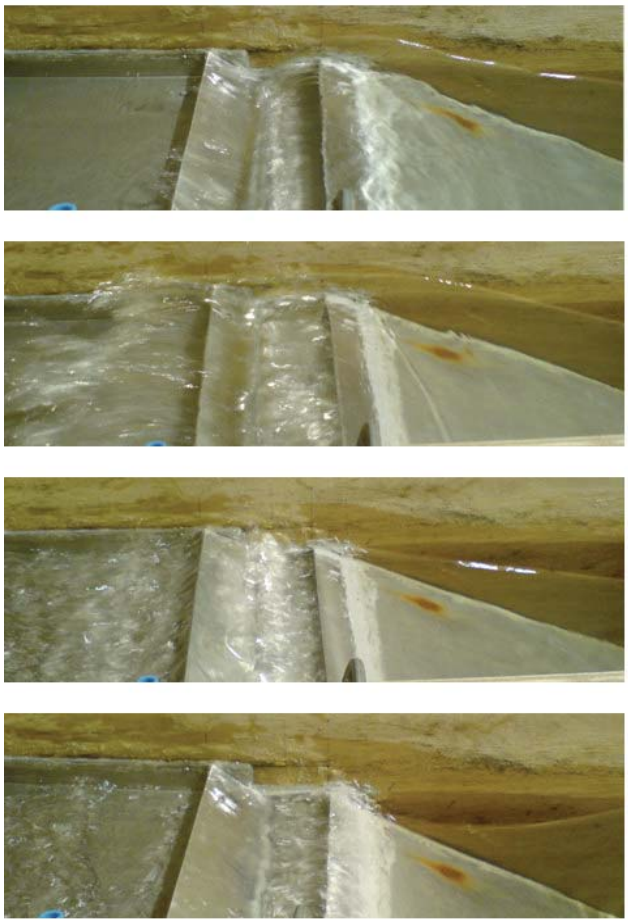

Figure 1. Overtopping event for wave condition W3, 2 levels model with HD1=0.11 m.

Influence of wave climate

The overtopping in the first reservoir appears to be slightly dependent on the wave steepness S_0 when plotting the results excluding wave condition W1 (Fig. 4). This wave condition has been excluded from Fig. 4 because responsible of very low overtopping rates and therefore misleading the overall results. It seams as if there is a parabolic trend for all the different geometries, with lower overtopping for smaller *HDs*. Trend lines have been added for HD1=0.07, 0.10, 0.15 and 0.20 m. in Fig. 4. The higher overtopping occurs for values of $S_0 \approx 0.022$.

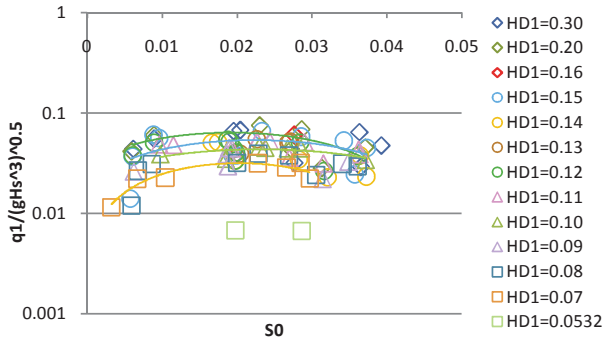

Figure 2. Adimensionalized overtopping in the lower reservoir for different geometries, plotted against the wave steepness S_0, logarithmic scale. *HD1* in meters.

Distinct behavior can be recongnized for all the tested geometries when plotting the adimensionalized discharge against the adimensionalized *HD1* (Fig. 5). Trend lines have been added for HD1=0.07, 0.11, 0.15 and 0.30 m. in Fig. 5. This emphasizes that there is an effect of the parameter *HD1* on the overtopping on the first reservoir, with a linear trend for all the geometries. Obviously it is common for all *HDs* that the higher overtopping occurs for higher waves (small HD/H_S).
For the same values of *HD1* the higher overtopping will occur for the geometry that features the less obstruction to the reservoir, i.e. for the larger *HD1*. For the specific data set, the higher overtopping in the first reservoir occurs for $HD/H_S \approx 3$ corresponding to *HD1*>0.15 m.. Those represent the conditions when the top level is not interfering with the water capture in the level below. For $HD/H_S<2$ the upper level has an influence on the water storage capturing indeed part of the water that would have instead been stored in the lower reservoir.

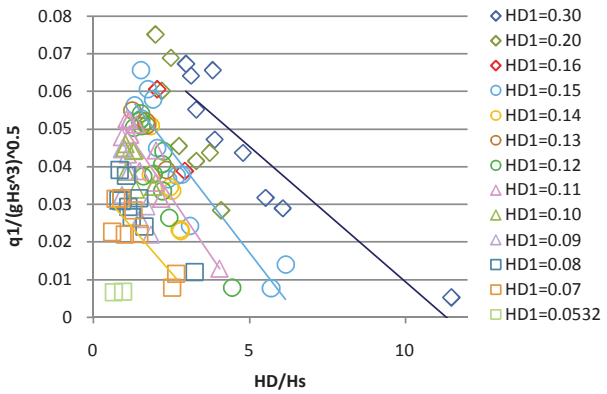

Figure 3. Adimensionalized overtopping in the lower reservoir for different values of *HD*, plotted against the adimensionalized *HD*. *HD1* in meters.

For the higher reservoir all the geometries follow the same trend (Fig. 6) with opposite behavior from the lower reservoir: higher overtopping occurs for *HD*< 0.12m. but the overtopping goes quickly to 0 for $HD/H_S>2$. Those are the cases where the upper level is caching part of the overtopping that would instead end entirely in the lower reservoir id *HD1* was bigger. It is anyway difficult to delineate a threshold with regard to the geometry that indicates when the upper level starts having

an effect on the water capture and not much difference can be appreciated for *HD1*=0.13 and 0.14 m.
These differences on the overtopping trends for the two reservoirs must be explained considering that the overtopping in the lower reservoir is not completely open but partially close by the above level.

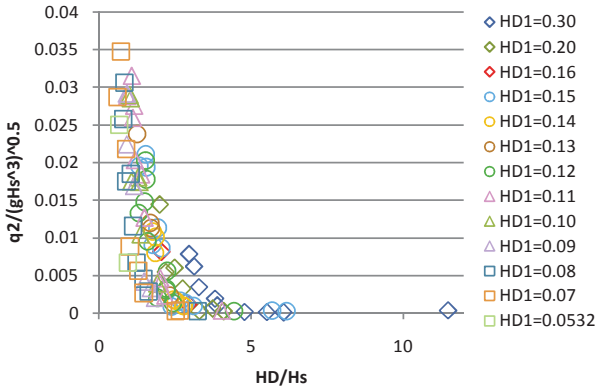

Figure 4. Adimensionalized overtopping in the upper reservoir for different values of *HD*, plotted against the adimensionalized *HD*. *HD1* in meters.

Comparison with prediction formulae

At first the overtopping of the two reservoirs have been summed up and related to the crest freeboard of the lowest reservoir. Results are shown as a comparison between measured and calculated data from Kofoed 2002, Eq. 2 , with *A*=0.197, *B*=-1.753 and *C*=-0.408 (Fig 7).

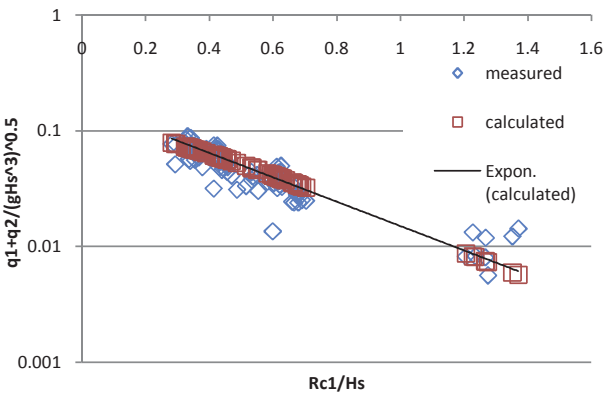

Figure 5. Adimensionalized total overtopping discharge as a function of dimensionless freeboard: comparison between measured and calculated data.

In general a good accordance can be noticed apart for few points: the prediction formulae overestimate the overtopping for those tests with the smallest *HD* while it underestimate the overtopping for some tests with high R_{c1}/H_s value. From a closer look the reasons behind the overestimation are clear as when *HD1* is very small it means the structure behaves like a single level but with a crest freeboard equal to $R_{c,2}$. The underestimated points instead correspond to wave condition W1a, where the period is particularly high despite waves being small. It is indeed possible to notice that when plotting the adimensionalized overtopping discharges from tests of different representative wave condition (Fig. 8), the overtopping measured in the lower reservoir increases depending on the *HD* while the overtopping in the upper reservoir decreases.
Both the overtopping rates have an upper limit that is given by the equation from Van der Meer and Janssen (1995) with coefficient λ_s added by Kofoed 2002 to take care of small Rc/Hs values, calculated for R_{c1} and R_{c2}:

$$\frac{q}{\lambda_S\sqrt{gH_S^3}} = 0.2e^{-2.6\frac{R_c}{H_S}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}} \tag{5}$$

Where the γ coefficients have been introduced to take into account the influence of geometric parameters and angle of wave attack.

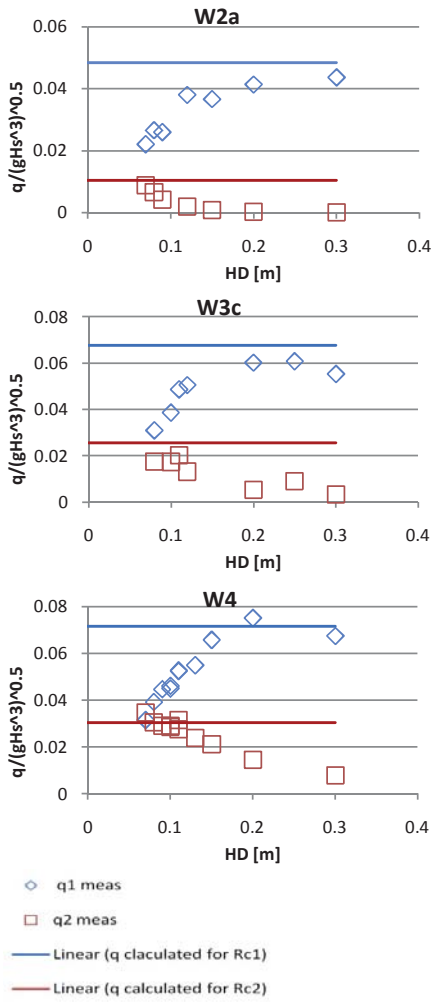

Figure 6. Overtopping discharges measured in low (q1) and top (q2) reservoirs for selected representative wave conditions, for different *HDs* with upper limits calculated for R_{c1} (upper line) and R_{c2} (lower line) from Eq.6.

ANALYSIS OF RESULTS

The results from each reservoir are presented separately and compared to prediction formulae derived from Eq. 2. Discordance have been underlined and it appeared clear that by taking into consideration the new parameter *HD* two different formulations of the overtopping prediction formula are needed, one for each reservoir, as the phenomena behind overtopping events in the first and second reservoir is different due to the presence or not of a roof i.e. the above reservoir. For this reason 2 coefficients to be implemented in Eq. 2 have been derived, one for each reservoir.

For the lower reservoir the prediction formula overestimates the overtopping for the tests with smallest *HD1* (Fig. 9). For the majority of the other data points the prediction formula underestimate the overtopping into the reservoir the more as the distance *HD1* increases. The only points that are correctly predicted are the ones with *HD1*=0.07 m which is indeed the value for which the formulation has been established.

For the second reservoir we don´t have good accordance and measured values are drawn from the prediction formula for all the cases (Fig. 10). In particular we have overestimation of the formula for all the *HD*>0.15 m. and underestimation for *HD*<0.11 m.

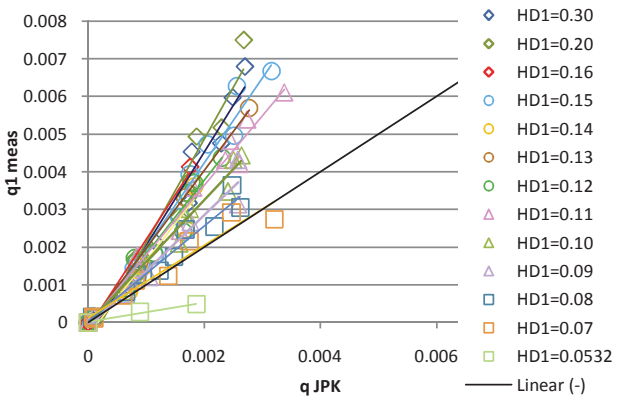

Figure 7. Comparison of measured and calculated values for different *HD1* for the lowest reservoir. *HD1* in meters.

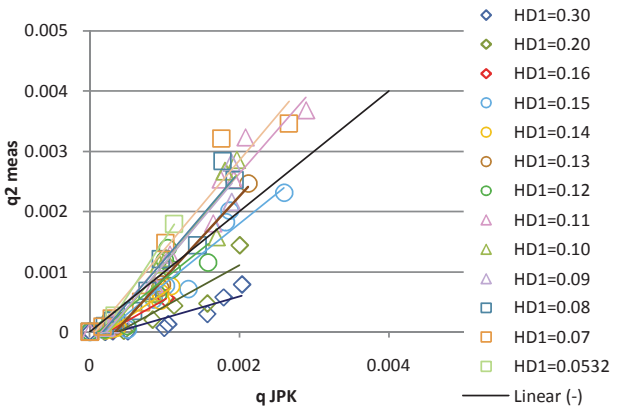

Figure 8. Comparison of measured and calculated values for different *HD1* for the higher reservoir. *HD1* in meters.

Linear curves with angular coefficients varying depending on *HD* show good agreement with the trend of the data with values of R-squared between 0.94 and 1 for the lower reservoir and between 0.81 and 1 for the second reservoir; in this last case the bigger discrepancies are for higher values of *HD* while for *HD*<0.15 m the agreement is satisfactory. A coefficient to be added at Eq. 2 has been defined and its equation found by mean of regression analysis (Fig. 11) in the form of second order logarithm that gives a value of R-squared = 0.99:

$$\lambda 1_{HD} = a + b\ln\left(\frac{HD}{R_{C1}}\right) + c\ln\left(\frac{HD}{R_{C1}}\right)^2 \qquad (6)$$

Where a=-1.15191, b=3.39915 and c = -0.76366.

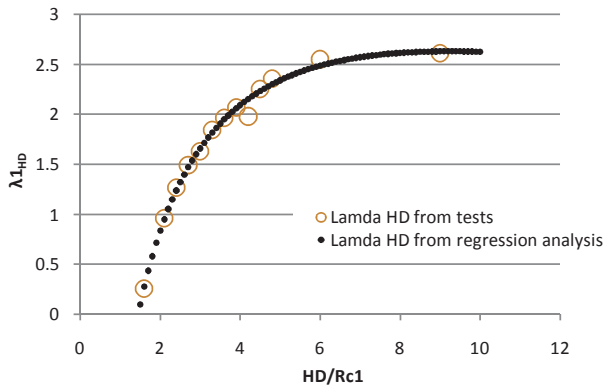

Figure 9. Dependency of the new coefficient $\lambda 1_{HD}$ on the HD/R_{c1}. The black curve have been calculated after Eq. 6. While the empty marks are derived from the tests.

For the second reservoir in the same way we have:

$$\lambda 2_{HD} = a\left(\frac{HD}{R_{C1}}\right)^2 + b\left(\frac{HD}{R_{C1}}\right) + c \qquad (7)$$

Where a=0.024061, b=-0.45563 and c = 2.50675 that gives a agreement expressed by R-squared =0.89.

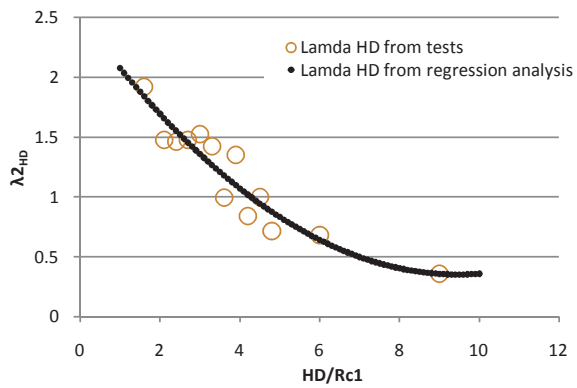

Figure 12. Dependency of the new coefficient $\lambda 2_{HD}$ on the HD/R_{c1}. The black curve have been calculated after Eq. 7. While the empty marks are derived from the tests.

We have now two different formulae for the overtopping in the first and in the second reservoir; rewriting Eq. 2 results in:

$$\frac{dq_n / dz}{\lambda n_{HD} \sqrt{gH_S}} = Ae^{B\frac{z}{H_S}} e^{C\frac{R_{c,1}}{H_S}} \quad (7)$$

CONCLUSIONS

For multi-level overtopping devices the phenomena of the overtopping is quite peculiar, having different behavior depending on the reservoirs´ position.

The present study demonstrated the influence that the horizontal distance *HD* has on the stored overtopping water:

- The overtopping in the lower reservoir increases while increasing *HD*.
- The overtopping in the upper reservoir decreases while increasing *HD*.
- For the specific set of tests for $HD/H_S<2$ the upper level has a big influence on the water storage on the level below. This indicates that there is a threshold after which the parameter *HD1* may increase or decrease the overall efficiency of the device.

Two different formulations (one for each reservoir) of the overtopping prediction formula by Kofoed 2002 have been derived that can take into account the *HD* parameter.
Further work will be done to analyze the reliability of the new formula and investigate the case of overtopping dependence on the parameter *HD* for 3 level overtopping device.

REFERENCES

Franco, L., de Gerloni, and L. Van der , Meer (1995): "wave overtopping on vertical and composite breakwaters". Proc. 24th International Conference of Coastal Engineering, Kobe, Japan 1, 1030-1044.

Kofoed J. P. (2002): "Wave Overtopping of Marine Structures – Utilization of Wave Energy". Ph. D. Thesis, defended January 17, 2003 at Aalborg University. Hydraulics & Coastal Engineering Laboratory, Department of Civil Engineering, Aalborg University.

Kofoed, J. P.; Frigaard, P.; Friis-Madsen, E. ; Sørensen, H. C. (2006a): "Prototype Testing of the Wave Energy Converter Wave Dragon". I: Renewable Energy. 2006 ; vol. 31, nr. 31, FEB. s. 181-189.

Kofoed, J. P. (2006b): "Vertical Distribution of Wave Overtopping for Design of Multi Level Overtopping Based Wave Energy Converters", 30th International Conference on Coastal Engineering, ICCE San Diego.

Le Méhauté, B., R. C. Y. Koh, and L. Hwang (1968) : "A systhesis of wave run-up.˝ Journal of waterways and coastal engineering division, ASCE Vol. 94 (No. WW1, proc. Paper 5807), pp. 77-92.

Mansard and Funke, 1980. E. Mansard and E. Funke, The measurement of incident and reflected spectra using a least squares method. In: Proceedings, 17th International Conference on Coastal Engineering, Sydney Vol. 1, pp. 154–172.

Frigaard, P. ; Trewers, A.; Kofoed, J. P. ; Margheritini, L. (2008):"Conceptual Design of Wave Plane". Aalborg : Aalborg University : Department of Civil Engineering, 2008. 233 s. (DCE Contract Reports; 38)

Margheritini, L. ; Vicinanza, D. ; Frigaard, P. (2008a): "SSG Wave Energy Converter : Design, Reliability and Hydraulic Performance of an Innovative Overtopping Device". I: Renewable Energy. 2009 ; vol. 34, nr. 5, 01.11.2008. s. 1371–1380

Margheritini L., Vicinanza D. And Frigaard P. (2008b): "Sea Slot Cone Generator Overtopping Performance in 3D Conditions". International Offshore and Polar Engineering Conference, ISOPE Vancouver, Canada, No 18, ISSN 1098-6189.

Van der Meer, J.W. and J. P. F. M. Janssen (1995): "wave run up and wave overtopping at dikes". Technical report, Task Committee Reports, ASCE.

Overtopping performance of Sea wave Slot-cone Generator

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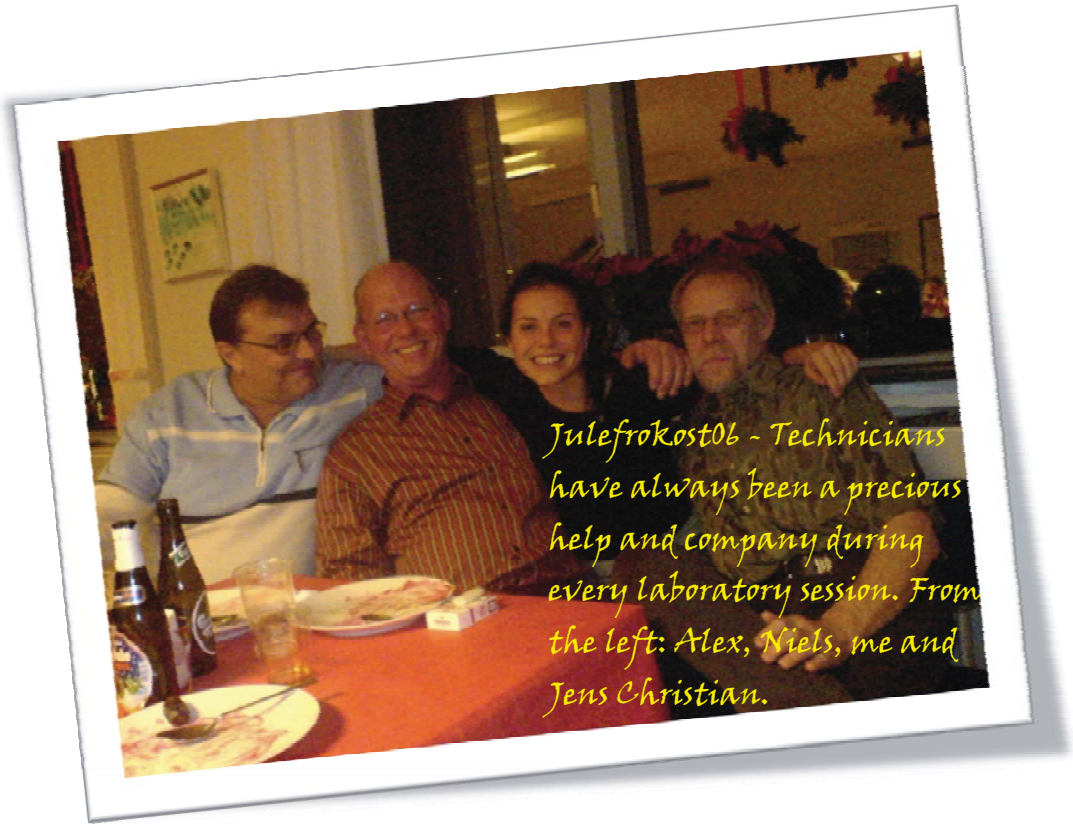

Thesis author´s contribution:

The Thesis author is the main author of this paper. She performed the laboratory tests and the numerical simulations on overtopping of SSG device. The first author is also responsible for the analysis of results derived from numerical simulations and recent laboratory tests. Co-author J.P. Kofoed is instead responsible of literature review and results. Co-author Diego Vicinanza helped in the revision of the paper with great enthusiasm.

Overtopping performance of Sea wave Slot cone Generator

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Introduction

For a healthy grow 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 of the 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 then mean water level offering the chance to exploit the potential energy by mean of specifically design low head hydro turbines. Other overtopping devices are Wave Dragon and Wave Plane, both floating devices with offshore applications. The SSG can be suitable for onshore and breakwaters 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 Multistage turbine 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 (Vicinanza 2008a, b, Kofoed 2006, Margheritini 2008a, 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. The outputs of the program are, among others, water into reservoirs, spill out water from reservoirs, power production, efficiency of different steps and overall efficiency.

The parameters influencing the efficiency and then 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 to explain 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.

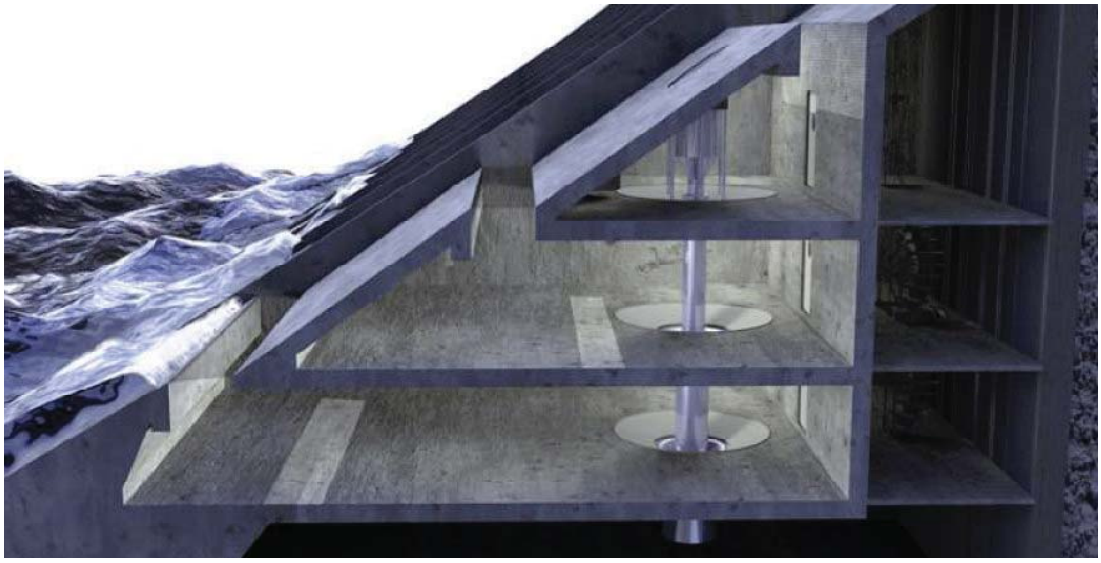

Figure 1. Artistic representation of one 3-level Sea Slot cone Generator mounted as a breakwater with multistage turbine.

Following, the working principle of the SSG device is briefly presented. This will help to relate the overtopping to the efficiency of the device. Subsequently the parameters influencing the overtopping of a fix multi-level overtopping WEC will be presented following an order that has been the natural 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 then sea water level optimizing the storage of potential energy in incoming waves. The design of the SSG device consists of a front ramp inclined of ≈ 30° that leads the waves to different levels depending on the incoming wave height. Each level has a front, also inclined of ≈ 30° allowing short term storage of water before turbine utilization. 30° has been found to be optimal for maximisation of the overtopping (Le Mèhautè et al. 1968) (Fig. 2). The crest levels $R_{c,j}$ are worked out after 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 Multi Stage turbine and the energy transformation is completed. It is clear, then, that hydraulic efficiency is then directly proportional to the overtopping water temporarily stored in the reservoirs:

$$\eta\mathrm{Hyd.} = {P_{crest}}/{P_{wave}} = {\sum_{j=1}^{n} \rho g\, q_{ov,j} R_{c,j}}\Big/{\frac{\rho g^2}{64\pi} H_S^2 T_E} \qquad (1)$$

where Rc,j = crest height of the j-reservoir (j = counter of reservoirs, $j=1,2\ldots n$, n= number of reservoirs) related to the MWL, ρ= density of the sea water ≈ 1025 Kg/m3 and g = gravity ≈ 9.82 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.

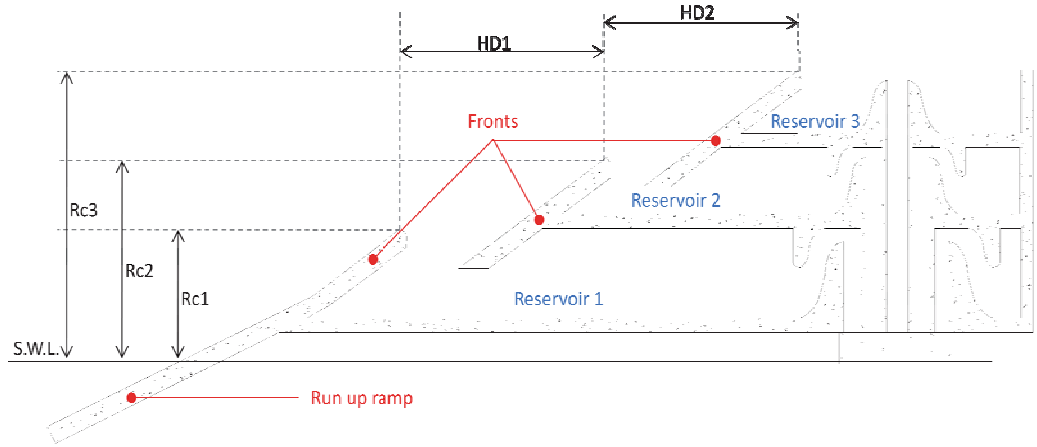

Figure 2. Definition sketch for a 3-level structure.

Influence of front run up ramp and crest levels

The front run up ramp leads the waves to the reservoirs. Its lengths and inclination can therefore influence the overtopping water to the upper levels: it appears that the overtopping increases with increasing the run up ramp length. The best solution is to extend the front slope close to or to the bottom as the orbital motion of waves is persisting till the sea bed in shallow waters. From a construction/installation point of view it may be not convenient to realize such a slope that extends to the bottom, especially for breakwaters-SSG. 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 result will be a fix 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 (1995) expression:

(2)

where R_c = crest height of related to the MWL, g = gravity ≈ 9.82 m/s2, 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 the expression is valid for $\xi_{p0} \geq 2$, with ξ_{p0} being the surfing similarity parameter defined as $tan\alpha/(2\pi H_s/gT_p^2)^{0.5}$, T_p peak wave period.
Kofoed (2002) made tests for 1-level floating structure with varying front run up 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 an extraordinary low R_{c1} compare to Van der Meer values.

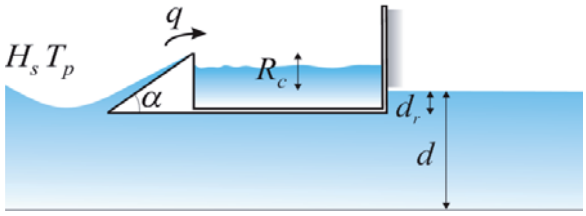

Figure 3. Definition sketch from tests performed by Kofoed 2002 to investigate the influence of run up angle *α*, the length of the run up ramp related to d_r and R_c on the overtopping discharge for 1-level floating structure.

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

$$\frac{q}{\lambda_\alpha \lambda_{dr} \lambda_s \sqrt{g H_S^3}} = 0.2 e^{-2.6 \frac{R_c}{H_S} \frac{1}{\gamma_r \gamma_b \gamma_h \gamma_\beta}} \tag{3}$$

Where

$$\lambda_\alpha = cos^\beta (\alpha - \alpha_m) \tag{4}$$

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

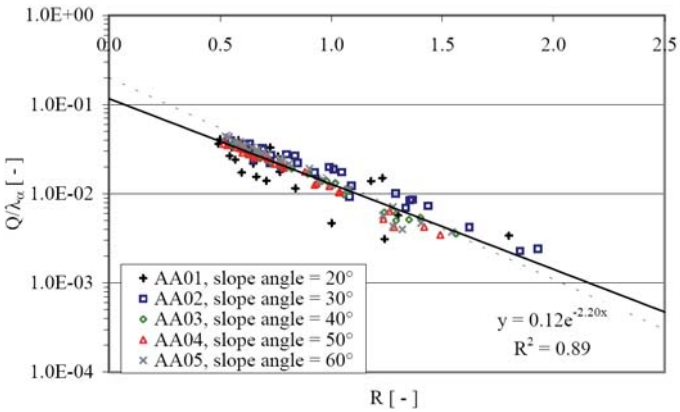

Figure 4. Tests results from Kofoed 2002 for 1-level structure with varying α, angle of the slope of the run up ramp. The dimensionless average overtopping discharge *Q* is plotted as a function of the dimensionless crest free board $R=R_c/H_s$.

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

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

Where k_p is the wave number based on L_p anf k = 0.4 is a coefficient controlling the degree of influence of the limited draft. The expression is based on the ration 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 run up ramp length ($dr/d < 0.75$).

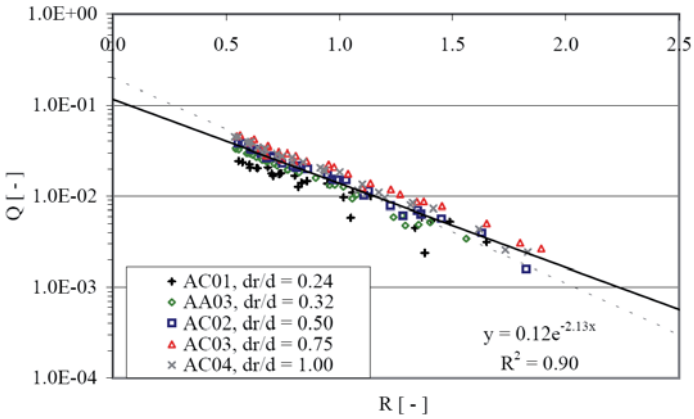

Figure 5. Tests results from Kofoed 2002 for 1-level structure with varying d_r, depth of the run up ramp related to its length. The dimensionless average overtopping discharge Q is plotted as a function of the dimensionless crest free board $R=R_c/H_s$.

Modelling a floating structure, the set-up by Kofoed (2002) is different from a case of a run up ramp of a fix device where the energy instead of passing under the structure would be reflected back by a vertical wall (trunked ramp). This case is most likely to brings less severe situation then the floating model as the waves will be reflected backwards and so their energy; part of it will then be "cached" (summed to) by the next wave but part will travel in the wrong direction and be lost as well. Moreover, in the same way as reflected waves can be summed up to the incoming wave, they could also be subtracted, depending on the frequencies. The phenomena behind this is obviously a dissipation of energy.

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 trunked 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 percentage of available incoming wave power over the case with run up ramp extended to the bottom (100%) is plotted for different dr. It appears that there is a dependency more in the water depth then 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 a 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 run up ramp at all decreases the available power to 30%. For shallower waters the losses occur faster when decreasing the run up ramp length.

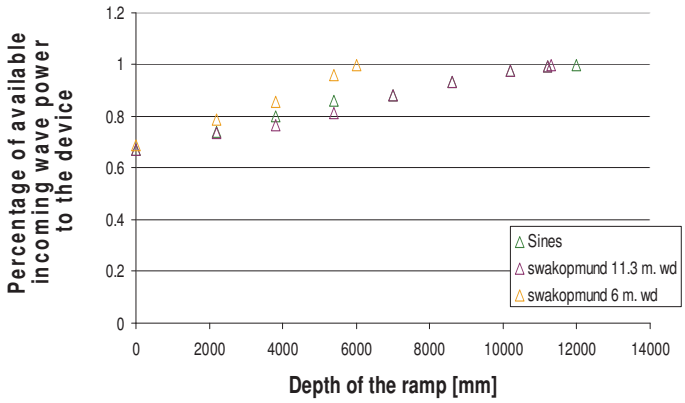

Figure 6. Reduction on available power to the device depending on the extension of the front ramp for the 3 locations under study.

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 (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:

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

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:

$$\frac{\frac{dq}{dz}}{\sqrt{gH_S^3}} = Ae^{B\frac{z}{H_S}}e^{C\frac{R_{c,1}}{H_S}} \tag{7}$$

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

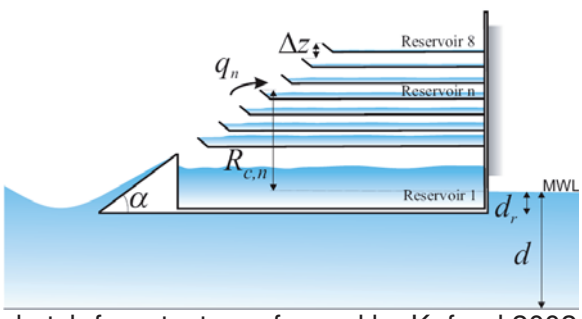

Figure 7. Definition sketch from tests performed by Kofoed 2002 to investigate of the overtopping for a multi-level wave energy device.

Influence of sea states, spreading and directionality

In the case of multi-level structure the overtopping performance is different for the different levels and is related to R_{c1} (Fig. 8) increasing when increasing H_s. Expression 3 seams to well predict the overtopping in the reservoirs despite giving an underestimation for the first level. In case of a fix multi-level overtopping device the converter will result more efficient from a hydraulic point of view for flatter wave spectrum, i.e. for sea states where the probability of occurrence of significant wave heights is spread more evenly along the sea states.

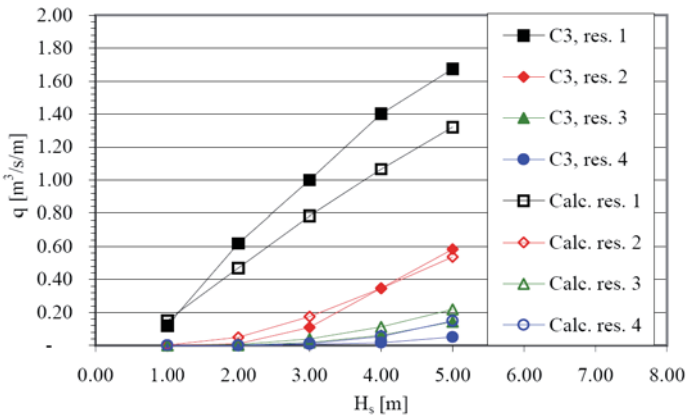

Figure 8. Comparison of measured and calculated values with Eq. 4 of the overtopping discharge for individual reservoirs q_n ($n=1, 2...4$) as a function of different sea states.

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 2008). It has been found that spreading and directionality together decrease the overtopping and therefore the hydraulic efficiency from 50% in 2D conditions (Kofoed 2002) to 35% in average taking into consideration attack angles varying between -15° and 15°. Figures 9 and 10 show the decrease of hydraulic efficiency when increasing wave spreading and attack angle for different wave conditions.

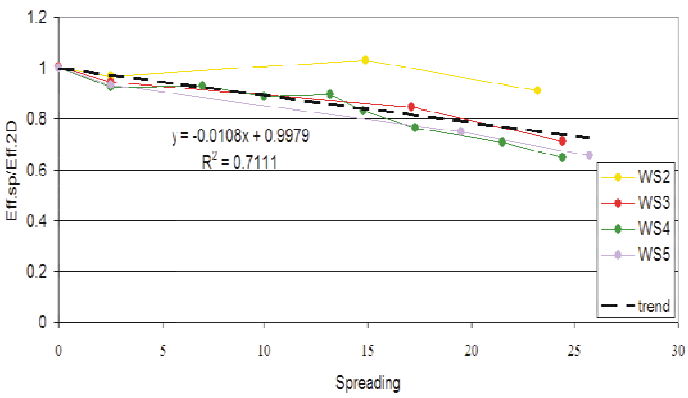

Figure 9. Influence of spreading on the hydraulic efficiency.

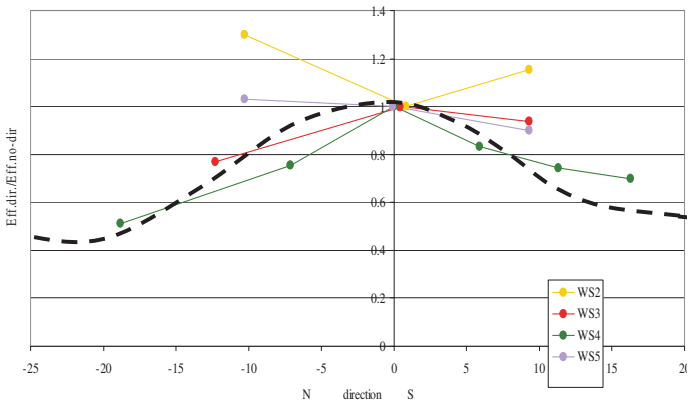

Figure 10. Influence of attack angle of incoming waves on the hydraulic efficiency.

Influence of tide

Being the tide a wave with a very long period, it seams clear that 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 more 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.

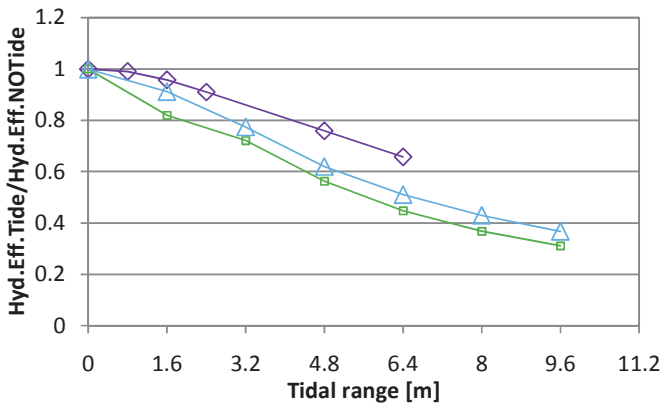

Figure 11. Decrease of hydraulic efficiency for different tidal ranges, for structures optimized for no tide with different tide distributions.

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).

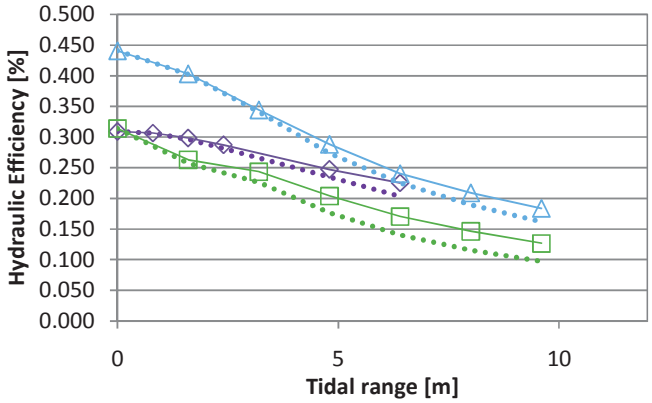

Figure 12. Comparison between hydraulic efficiency for geometries optimized and non-optimized for tide.

Influence of number of reservoirs

The benefit of adding an extra reservoir has bees 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 (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 big number of reservoirs can increase the efficiency of the device but has an added value when considering installation in locations characterized by tidal variations. The presence of tide is something that must be taken into account in the design of a fix overtopping wave energy converter. The contribution of tidal variations can be seen as a widening parameter for the wave spectrum as the wave heights are influenced by the water level. For this reason the structure should be more flexible then 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.

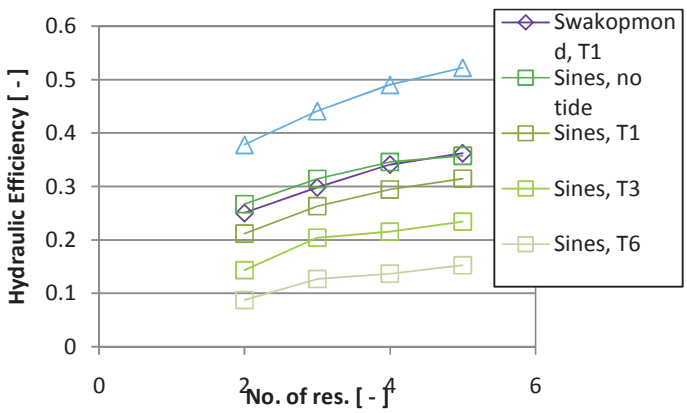

Figure 13. Comparison between hydraulic efficiency for different number of reservoirs. T1=0.8 m tidal range, T3=4.8m. tidal range, T6=9.6m .tidal range.

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 *HD1* while for the above level is obviously true the opposite (Fig. 14). The overtopping in reservoirs 1 and 2 will increase when increasing and decreasing *HD1* up to 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.

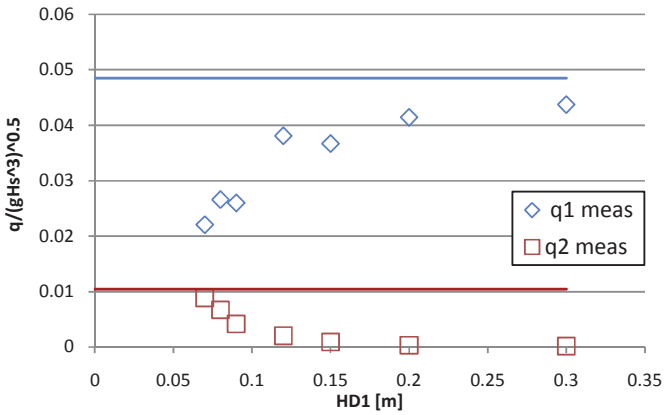

Figure 14. Overtopping discharges measured in low (q1) and top (q2) reservoirs 2-level structure, under wave conditions characterized by H_s=0.067 m and T_p=2.922 s. Upper limits calculated for R_{c1} (upper line) and R_{c2} (lower line) from Eq.3, for single level structure.

The total overtopping discharge decreases linearly when increases *HD1/Hs*. This is normal as it means that for smaller waves there is less overtopping (Fig. 15). Eq. 7 does not take into account the influence of the *HD* parameter despite it having an influence on the overtopping flow rates. A new formulation of the overtopping expression is needed.

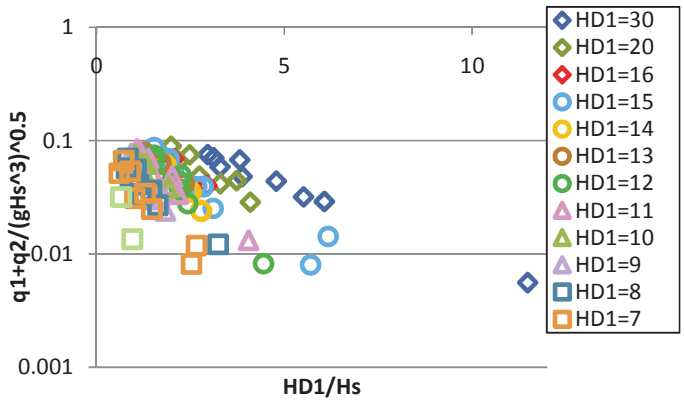

Figure 15. Sum of the measured overtopping discharges in the first and second reservoirs as a function of $HD1/H_s$ for different tested geometries varying *HD1*.

Influence of front angles

As well as the angle of the front run up ramp, also the front angles has 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 then 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. The results have been collected in this paper and indicate that:

- The angle of inclination of the run up ramp influences the overtopping over the next levels. In particular it has been found that the angle that maximized the overtopping is α=30˚, with little change between 30˚ and 40˚.
- For an application of the SSG WEC on breakwaters it may be not possible to extend the front run up ramp to the bottom. A truncation of the run up ramp at a certain depth with a vertical wall generate dissipation phenomena that result 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 50% in 2D conditions to 35% in average. To higher spreading and angle of wave attack corresponds bigger losses on stored water.
- For a fix device, tidal ranges can decrease significantly the overtopping into the reservoirs 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 bigger then 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.
- Adding more reservoirs optimized 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 better 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 consecutives 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.

References

Kofoed J. P. (2002): “Wave Overtopping of Marine Structures – Utilization of Wave Energy”. Ph. D. Thesis, defended January 17, 2003 at Aalborg University. Hydraulics & Coastal Engineering Laboratory, Department of Civil Engineering, Aalborg University.

Kofoed J.P.(2005): “Experimental Hydraulic Optimization the Wave Energy Converter Seawave Slot-Cone Generator”. Hydraulics and Coastal Engineering No. 26, ISSN: 1603-9874, Department of Civil Engineering, Aalborg University.

Kofoed, J. P. (2006): "Vertical Distribution of Wave Overtopping for Design of Multi Level Overtopping Based Wave Energy Converters", 30th International Conference on Coastal Engineering, ICCE San Diego.

Le Méhauté, B., R. C. Y. Koh, and L. Hwang (1968) : "A systhesis of wave run-up.˝ Journal of waterways and coastal engineering division, ASCE Vol. 94 (No. WW1, proc. Paper 5807), pp. 77-92.

Margheritini L., Vicinanza D. And Frigaard P. (2008a): "Sea Slot Cone Generator Overtopping Performance in 3D Conditions". International Offshore and Polar Engineering Conference, ISOPE Vancouver, Canada, No 18, ISSN 1098-6189.

Margheritini L., Kofoed J.P. (2008b): "Parametrical numerical study on Breakwater SSG application."Aalborg : Aalborg University : Department of Civil Engineering, (DCE Contract Reports; 47).

Meinert P. (2008), update of Meinert P. (2006): "User Manual for SSG Power Simulation 2" DCE Technical Report No. 44, ISSN 1603-9874, Department of Civil Engineering, Aalborg University.

Oever ten E. (2008): "The SSG breakwater concept" Report No952350-rap-u-0005, Delta Marine Consultants.

Kofoed J. P. (2002): "Wave Overtopping of Marine Structures – Utilization of Wave Energy". Ph. D. Thesis, defended January 17, 2003 at Aalborg University. Hydraulics & Coastal Engineering Laboratory, Department of Civil Engineering, Aalborg University.

Margheritini L., Kofoed J.P. (2008): "Case study for breakwater SSG application" DCE Contract Report No. 44

Van der Meer, J.W. and J. P. F. M. Janssen (1995): "Wave run up and wave overtopping at dikes". Technical report, Task Committee Reports, ASCE.

Vicinanza D., Margheritini L., Contestabile P., Kofoed J.P., Frigaard P. (2008a): "Wave loadings acting on an innovative caisson breakwater for energy production". International conference of Coastal Engineering, ICCE No. 31, Hamburg, Germany.

Vicinanza, D., and P. Frigaard. (2008b). "Wave pressure acting on a seawave slotcone Generator", Coastal Engineering, 55, 553-568.

Hydraulic characteristics of Sea Wave Slot Cone Generator pilot plant at Kvitsøy (Norway)

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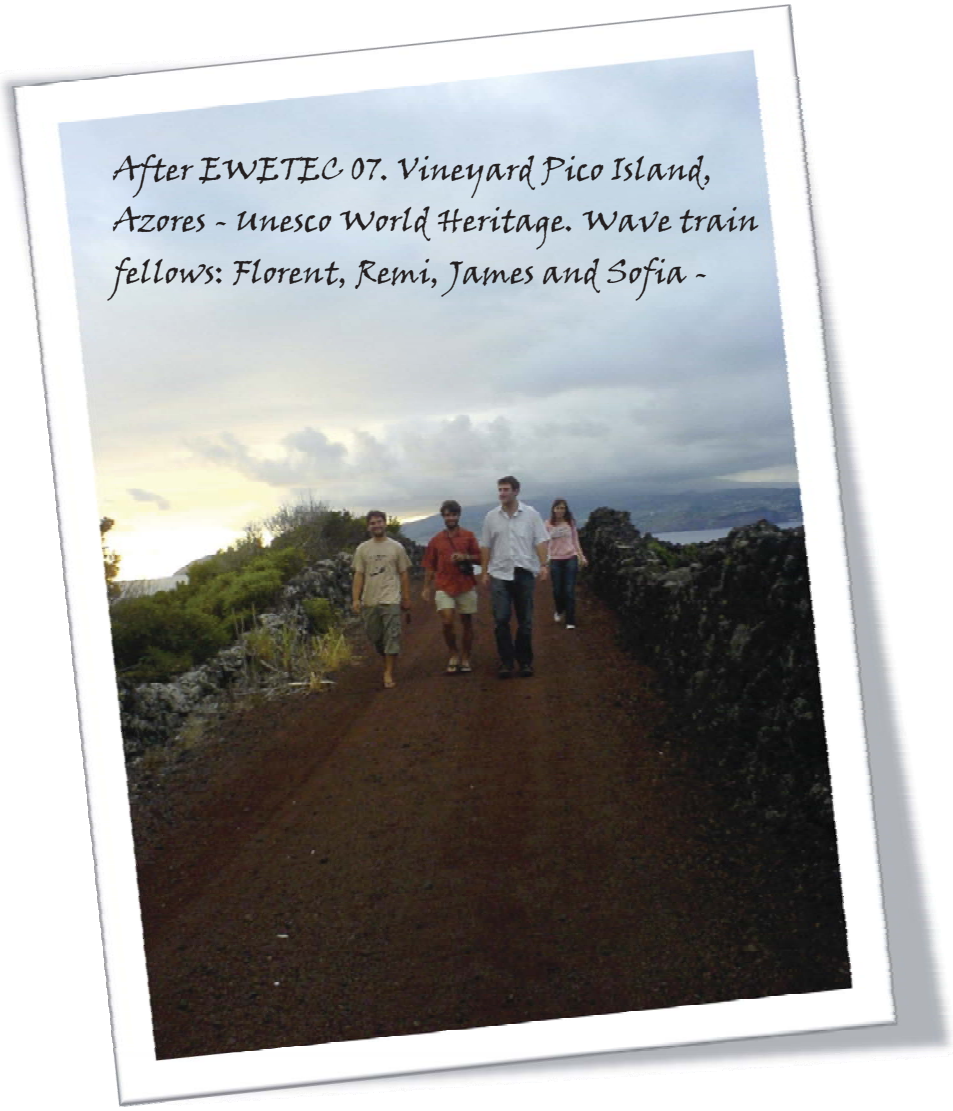
Margheritini, Lucia ; Vicinanza, Diego; Kofoed, Jens Peter.



Thesis author´s contribution:

The Thesis author is the first author of this paper. She performed the laboratory tests on overtopping of SSG device assisted by co-author J.P. Kofoed. She is responsible for the analysis of results of 3D overtopping data. Co-author Diego Vicinanza is responsible for test and results on wave loading that the author helped to present.

Hydraulic characteristics of seawave slot-cone generator pilot plant at Kvitsøy (Norway)

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Abstract

This paper presents results on wave overtopping and loading on an innovative caisson breakwater for electricity production. The work reported here contributes to the European Union FP6 priority 6.1 (Sustainable Energy System). The design of the structure consists of three reservoirs one on the top of each other to optimize the storage of potential energy in the overtopping water. The wave loadings on the main structure can be estimated using experiences from breakwater design, but the differences between the structures is so large that more reliable knowledge is needed. Model tests were carried out to measure wave loadings and overtopping rates using realistic random 2D and 3D wave conditions; the model scale used was 1:60 of the SSG pilot at the selected location in the island of Kvitsøy, Norway. Pressure transducers were placed in order to achieve information on impact/pulsating loadings while in a second phase the model has been adapted and equipped with pumps to measure the overtopping flow rates in the single reservoirs. The results of the tests highlight differences between 2D and 3D conditions in terms of pressures and hydraulic efficiency.

Keywords: breakwaters, loadings, overtopping, SSG, 3D model tests.

Introduction

The Sea Slot-cone Generator (SSG) is a wave energy converter (WEC) of the overtopping kind; it has a number of reservoirs one on the top of each others to optimize the capture on the potential energy in the overtopping water. It is a patented and certificated device developed by WAVEEnergy, Stavanger, Norway. In 2008 the SSG pilot will be the first full scale wave energy converter producing electricity for the 520 inhabitants of Kvitsøy; the project regards a 150 kW onshore installation with approximately dimensions of 17 m (length) x 10 m (width) x 6 m (height) and three reservoirs one on the top of each others (Figure 1) to optimize the storage of power in the overtopping waves (Kofoed, 2006; Vicinanza et al., 2006). The works for the construction of the structure will start in summer 2007 at the selected location in the island of Kvitsøy, Norway. The objective of the pilot project is to demonstrate at full-scale, the operation of one module of the SSG wave energy converter in a 19 kW/m wave climate, including turbine, generator and control system, and to connect the system to the public grid for electricity production. At this stage of development the wave energy sector needs reliable devices with a proved technology at a low cost. The SSG device will be built as a robust concrete structure and one of its future applications will be on breakwaters enabling sharing of costs. The purpose of the work described in this paper is to derive information on wave pressures/forces acting on sloping and vertical walls as well as on overtopping flow rates in 3D conditions. The overtopping results are used for geometrical optimization while the ones on loadings have been used for structural design as well as stability evaluation and have been presented at international level and for the certification of the pilot plant under construction.

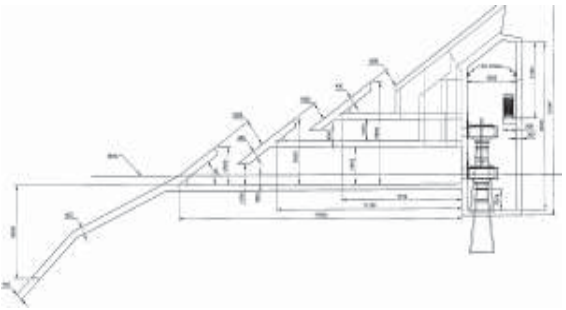

Figure 1: Section of the SSG pilot; on the rear part there is a dry room that will contain the turbines and generators; on the front, the apron and the 3 slopes designed to optimize the storage of the overtopping water to the three reservoirs. Reservoirs number: one the lower, two the middle and three the higher.

1 Tests set up

Model tests have been performed in a wave tank at Aalborg University, in 1:60 length scale compared to the prototype. This wave basin is a steel bar reinforced concrete tank with the dimensions 15.7 x 8.5 x 1.5 m. The paddle system is a snake-front piston type with a total of ten actuators, enabling generation of short-crested waves. The waves are absorbed by a rubble beach slope in the back of the basin to minimize reflection.

The wave generation software used for controlling the wave paddles is AWASYS5, developed by the laboratory research staff. The bathymetry in the immediate proximity of the pilot plant has been surveyed and the results have been used as the basis for the laboratory model. The SSG caisson model was built in plexiglas with dimension of 0.471 x 0.179 m (Figure 2).

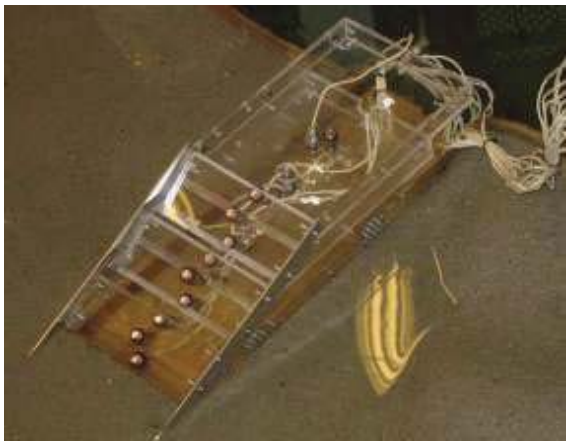

Figure 2: SSG caisson model.

Figure 3: Plexiglas model on the reproduction of the cliff at Kvitsøy; in front the seven resistive probes.

The three front plates were positioned with a slope of $\alpha = 35°$. The model was fixed rigidly on a 3D concrete model of the cliff located in the middle of the basin at 5 m from the paddles. Seven resistive wave probes were located on a pentangle array placed on the plateau (Figure 3). Fourteen Kulite Semiconductor pressure cells were used to measure the pressure in a total of 25 positions on the structure plates. Two different transducer configurations were needed because of the very limited space inside the model combined with the physical dimensions of the pressure transducers (Fig. 4).

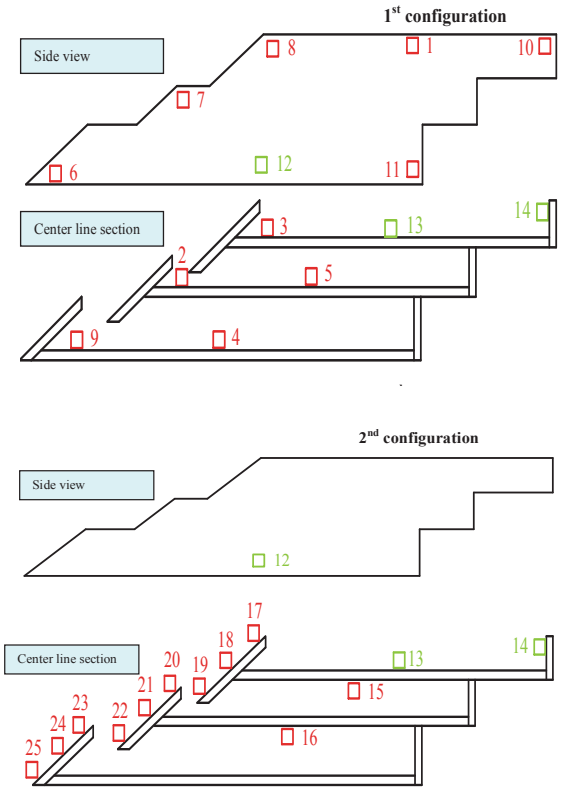

Figure 4: Test configuration and pressure cells locations (green identify transducers locations used in both configurations).

Table 1 shows the JONSWAP sea states selected for the tests. Each test comprised approximately 1000 waves. Tests were carried out with frontal and oblique waves (45°, denoted "Side"), with various levels of directional spreading (n).

The experimental procedure has been designed to ensure that data are available to allow a good estimation of the surface loads corresponding to the design 100 years return period wave event at the plateau, given by wave condition Hs = 12.5 m and Tp = 15.2 s. Not only the 100 years return period wave event were simulated in order to allow comparisons between laboratory data and field measured from the pilot plant once built. The wave signals were stored and reused from transducer configuration number one to configuration number two. Each of the 32 tests was thereby performed twice.

Test	H_s [m]	T_p [s]	swl [m]	Direction	Wave field	n
1	0.125	1.55	0.50	Front	2D	-
2	0.167	1.81	0.50	Front	2D	-
3	0.208	1.94	0.50	Front	2D	-
4	0.250	2.07	0.50	Front	2D	-
5	0.042	1.03	0.50	Side	2D	-
6	0.083	1.29	0.50	Side	2D	-
7	0.125	1.55	0.50	Side	2D	-
8	0.167	1.81	0.50	Side	2D	-
9	0.125	1.55	0.53	Front	2D	-
10	0.167	1.81	0.53	Front	2D	-
11	0.208	1.94	0.53	Front	2D	-
12	0.250	2.07	0.53	Front	2D	-
13	0.042	1.03	0.53	Side	2D	-
14	0.083	1.29	0.53	Side	2D	-
15	0.125	1.55	0.53	Side	2D	-
16	0.167	1.81	0.53	Side	2D	-
17	0.125	1.55	0.53	Front	3D	4
18	0.167	1.81	0.53	Front	3D	4
19	0.208	1.94	0.53	Front	3D	4
20	0.250	2.07	0.53	Front	3D	4
21	0.042	1.03	0.53	Side	3D	4
22	0.083	1.29	0.53	Side	3D	4
23	0.125	1.55	0.53	Side	3D	4
24	0.167	1.81	0.53	Side	3D	4
25	0.125	1.55	0.53	Front	3D	10
26	0.167	1.81	0.53	Front	3D	10
27	0.208	1.94	0.53	Front	3D	10
28	0.250	2.07	0.53	Front	3D	10
29	0.042	1.03	0.53	Side	3D	10
30	0.083	1.29	0.53	Side	3D	10
31	0.125	1.55	0.53	Side	3D	10
32	0.167	1.81	0.53	Side	3D	10

Table 1. Summary of model wave conditions.

2 Loading conditions

Previous works by Allsop et al. (1996), Calabrese and Vicinanza (1999), Vicinanza (1999) show how the forms and magnitudes of wave pressures acting upon caisson breakwaters under random wave conditions are highly variable and they are divided into "pulsating", when they are slowly-varying in time and the pressure spatial gradients are mild, and "impact", when they are rapidly-varying in time and the pressure spatial gradients are extremely high

Two principal quasi-static loadings may be considered here. First, a wave crest impinges directly against the structure applying a hydro-static pressure difference. The obstruction of the momentum of the wave causes the wave surface to rise up the wall, increasing the pressure difference across the plates; the net force is approximately proportional to the wave height, and can be estimated using relatively simple methods. Wave impacts occurs when the waves break directly on the structure with almost vertical front surface at the moment of impact or as a plunging breaker with cushion of air inducing loads of much greater intensity and shorter duration than the quasi-static loads (Figure 4).

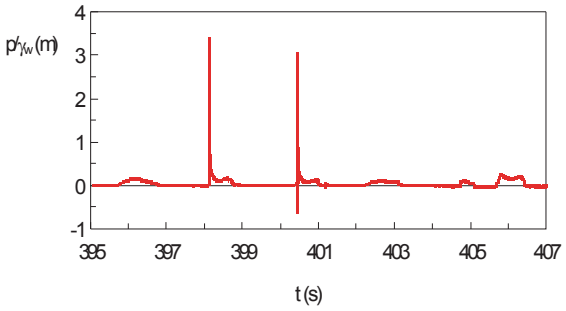

Figure 4: Quasi-static and impact pressure time history (after Vicinanza, 1999).

A preliminary visual test analysis (Fig. 5) permitted to identify two different behaviors of waves acting on the structure:

- surging waves, characterized by a rapid rise of the wave along the three sloping front caisson plates – no breaking waves;
- impact of water jet, resulting from massive wave overtopping directly hitting the vertical rear wall in upper reservoir, characterized by evident wave slamming.

Because of this different wave-structure interactions two different pressure sampling rate were set up. Each test was run twice. On the first run pressure data were acquired at a rate of 200 Hz. A second run was carried out at sampling rate of 1200 Hz.

4 Results of pressures on the structure

The first part of the experimental data analysis was finalized to identify the loading regime on different structure locations. In Figure 6 an example of pressure time history recorded by transducers mounted on the front sloping walls under normal extreme wave attack is shown. It should be noted that the generated wave pressures shows higher values on the central plate 2. A quasi-static loading time history is recognizable over all the front side plates and the pressure is almost hydrostatic ($p \approx \rho w\ g\ Hm$).

It should be noted that the generated wave pressures do not vary substantially from one plate to another. Thus, a quasi-static loading time history is recognizable.

The shape of the spatial pressure distribution on the front plates is shown in Figure 7. The non-dimensional pressure is plotted against the transducer position at time of the maximum pressure on plate 2. The pressure distribution assumes a typical trapezoidal shape (Goda, 1974; Goda, 1985). A completely different behaviour was recognized from time history analysis of the pressure transducer at the rear wall in the upper reservoir (Fig.8). Comparison with front plate transducer signal show evident rapidly-varying in time and high pressure peaks typically described as "impact" ($p \approx 4\ \rho w\ g\ Hm$). This pressure example exhibits a relative small impact pressure due to the damped breaking waves (impacts pressures can be up to $p \approx 50$ - $100\ \rho w\ g\ Hm$).

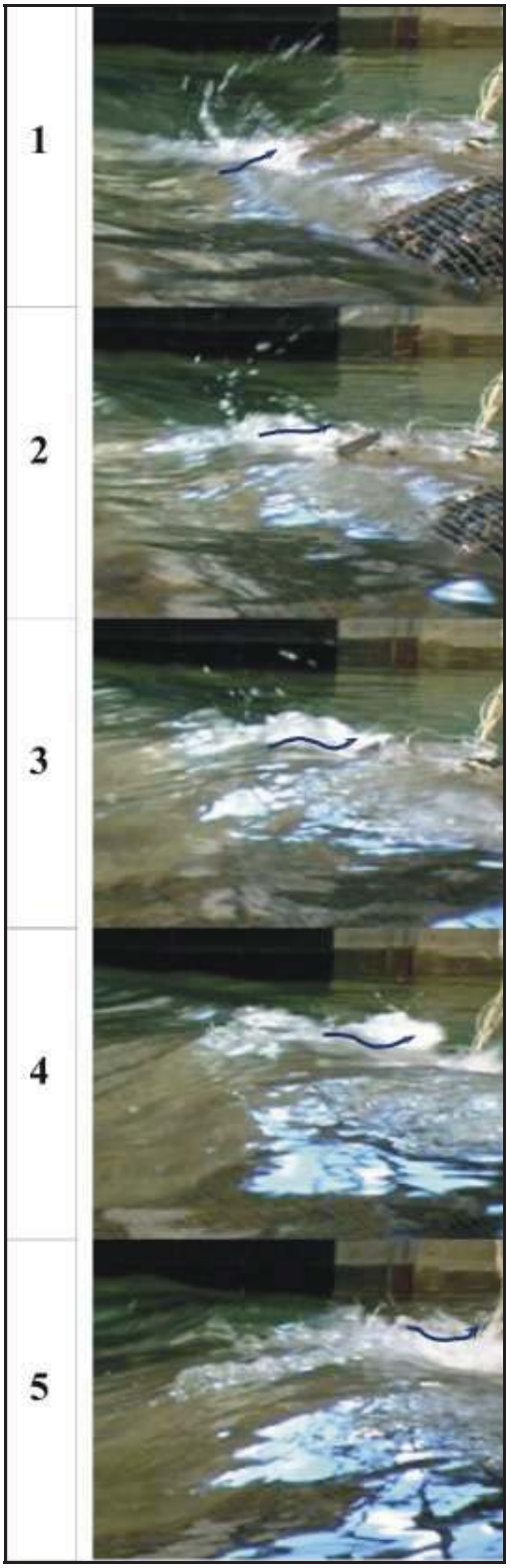

Figure 5: Sequence of video frames from test 4 (time between frames: 0.2 s).

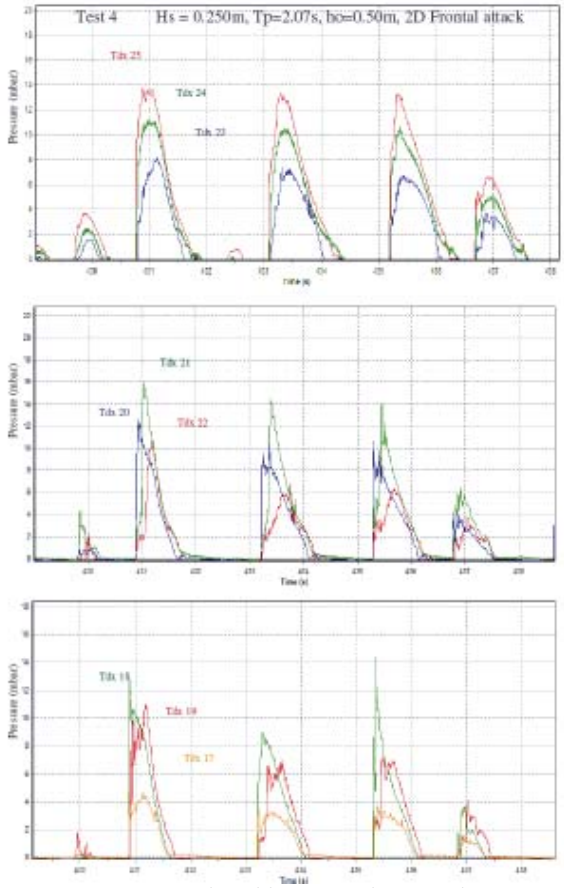

Figure 6: Pressure time history at the transducers on the front plates.

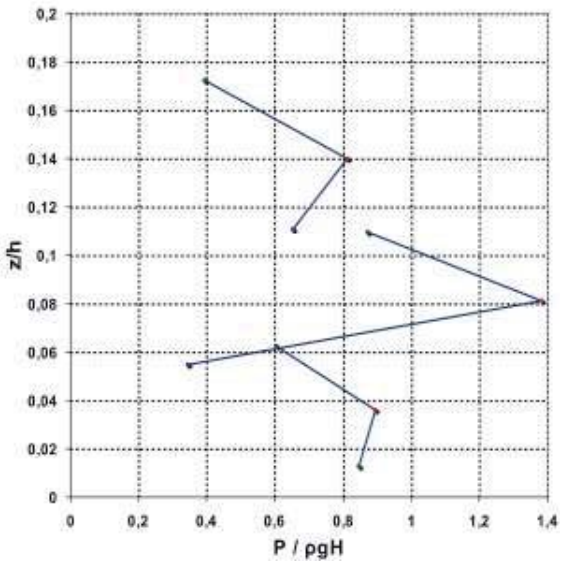

Figure 7: Maximum pressure spatial distribution at the transducers on the front plates.

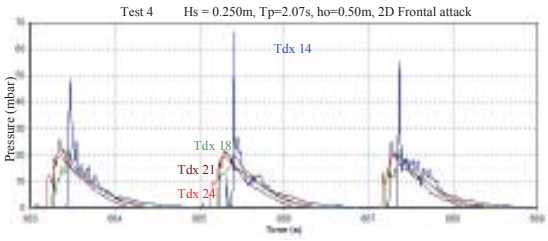

Figure 8: Comparison between transducer on the front plate and on the rear wall.

The major emphasis in any study on wave loadings is on the overall or average level of pressures, which is needed to determine the overall stability of the structure. Data on local pressures and pressure gradients are also needed in any analysis of conditions leading to local damage. The results appear to indicate that pressures on front plates are quasi static (p1/250 ~ ρw g Hmax) or pulsating loads generated by non-breaking waves. The wave loading on the rear vertical wall are varying over 2 - 3 ρw g Hmax. In this case the wave is collapsing in the upper reservoir in front of the wall. This loading case exhibits a relative small impact pressure due to the damped breaking waves.

The analysis of these pressure measurements made at laboratory scale using fresh water has explicitly assumed a Froude scale conversion to prototype values. In the case of pulsating wave pressures the assumption of Froude scaling is realistic while for wave impact pressure scaling is less simple. It has long been argued in the EU project on caisson breakwaters, PROVERBS (Oumeraci et al., 1999), that wave impact in small scale hydraulic model tests will be greater in magnitude, but shorter in duration than their equivalents at full scale in (invariably aerated) sea water. It is very probable that the higher peak pressures measured in these model tests can be scaled to lower values, but probably each will attend by longer impulse durations. The argument on scaling these peak pressures requires information not presently available on the relationships between the statistics of the pressure time gradients and the magnitude of the pressure impulses. It can be argued that the magnitude of the pressure impulse, given perhaps by (p Δt) will not be changed between model and prototype, other than by the normal scaling relationships. Measurements of wave pressures planned at pilot SSG in Kvitsøy will be useful to estimate model-prototype scaling discrepancies.

5 Overtopping tests

The present section investigates the phenomena responsible of the reduction of efficiency passing from 2D laboratory conditions to 3D conditions.

These are:

- Directionality.
- Spreading.
- 3D-ness of the structure (boundary effects, not optimal slope leading to the model…).

The objective was to estimate the hydraulic efficiency of the SSG pilot.

The last point has been investigated with a comparison between 2D waves in the described setup and 2D waves in a 2D setup of earlier tests not described in this work (Kofoed 2005); the result of that study indicates an hydraulic efficiency for the SSG pilot of 50%.

Each test was of approximately 1500 waves in normal operational conditions (H_s<7.4 m and 6.1 s <T_p < 12.7 s). Tests have been carried out with attack angles varying between -15° and 15° (directions between 255° and 285° at the pilot location), 8 spreading conditions and 3 water levels. Spreading and directionality were investigated separately. The directional spreading (n) function adopted is expressed by the following form:

$$\cos^{2n}\left|\left(\beta-\beta_0\right)/2\right|$$

The rear part of model was modified and equipped with four slopes leading to different small tank containers: one for each reservoir plus one for the overtopping of the whole structure (Figure 9). In this way the front part was the same as the loading tests (Section 1). The captured overtopping water was then temporally stored and then pumped out again in the basin by small pumps of known capacity; the pumps were automatically activated when the water inside the single containers was reaching a certain pre-established level. By the total utilization of the pumps and the records of water levels inside the rear tanks, the overtopping flow rates have been derived for the single reservoirs. The hydraulic efficiency has been defined and calculated as the ratio between the power in the overtopping water (*Pcrest*) and the power in incoming waves (*Pwave*):

$$P_{crest}=\sum_{j=1}^{3} q_{ov,j} R_{C,j} \rho g$$

$$P_{wave}=\frac{\rho g^2}{64\pi} H_S^2 T_E$$

The measuring equipment included:

1. 4 wave gauges installed to measure time series of water levels in the reservoirs tanks.
2. 7 resistive wave probes on a pentangle array placed on the plateau in front of the model, enabling the collection of data for 3D wave analysis.

Figure 9: Model for 3D overtopping tests.

3 Overtopping results

In Figure 10 flow rates of the tests for the 3 reservoirs (q1, q2 and q3) are plotted for different spreading conditions; reservoir number 1 is the lower, while nr. 2 and 3 are the middle and higher ones. The results appear grouped in the graphics depending on the wave high (increasing with Hs). While little difference can be noticed comparing the 2D and the different spreading conditions in reservoir one and two, in reservoir number three for higher Hs the difference between tests with low spreading (≈ 2D conditions) and high spreading are relevant. In Figure 11 the calculated efficiency of laboratory tests with and without spreading is plotted against the efficiency with spreading divided the efficiency without spreading. In black the overall trend of the results depending on spreading. A local effect regards the W2 condition and it could be imputable to the different interaction of the specific short period of the waves with the bathymetry.

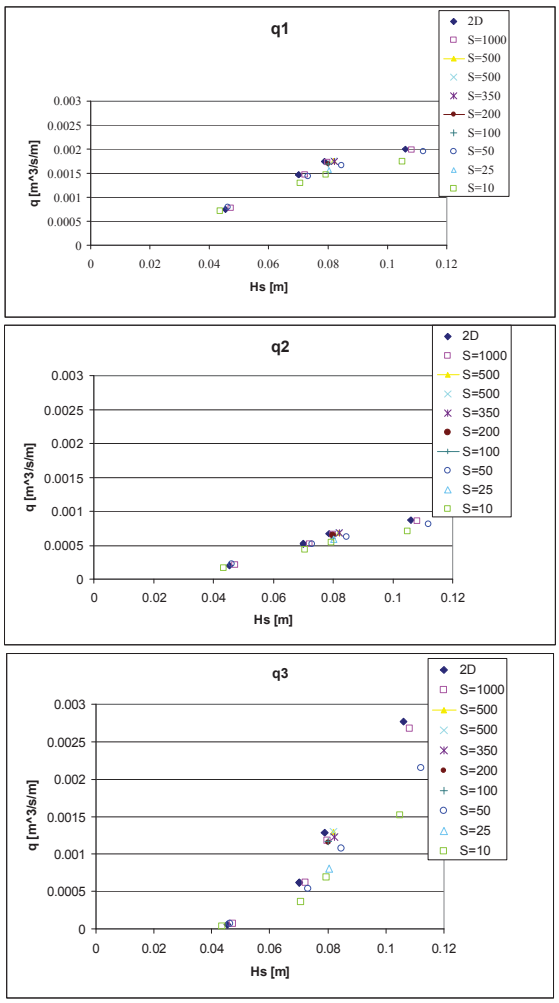

Figure 10: Flow rates into the 1st, 2nd and 3rd reservoir for different wave heights and input spreading coefficients.

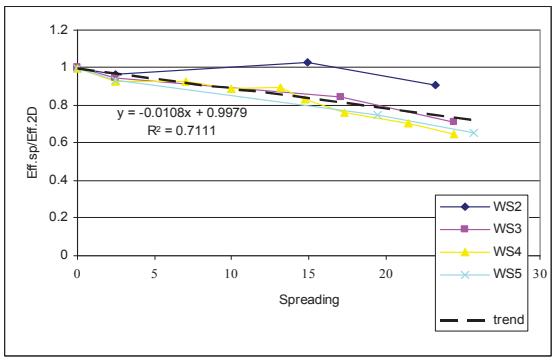

Figure 11: Tests results from laboratory plotted against the efficiency with spreading (2D conditions) divided by the efficiency without spreading. The results are plotted for 4 different wave conditions.

In figure 12 the flow rates for the three reservoirs are plotted for different attack angles (θ = 0 = direct attack). Again little difference can be noticed in reservoir 1 and 2 by increasing θ for the same Hs, while in reservoir number three the flow rates (q3) are very influence by the directionality. A local effect can also be distinguished by a closer look to the graphic: for the same wave highs waves with a positive attack angle (+ θ) give a bigger flow rate in the 3rd reservoir than the ones with a negative attack angle (- θ) and comparable absolute value. This is probably due to the influence of the bathymetry which has a steep slope or focusing characteristics on the left part of the structure (facing the sea). This asymmetry of results is even more evident when plotting the efficiency.

In Figure 13 the calculated efficiency of laboratory tests with and without directionality is plotted against the efficiency with directionality divided the efficiency without directionality. Again the W2 condition behaves weirdly when adding an attack angle but all the tests present is an asymmetry of the results. Not all the range of attack angles has been tested; in reality at the selected SSG pilot location the attack angle can be ± 40° while in the laboratory only an attack angles up to ± 15°have been tested. It is anyway suggested that the efficiency can not decrease to 0 while increasing the attack angle in a range between ± 40°. What it is expected to happened in that case, is that local phenomena will convert the waves to the structure and the efficiency will converge to a low threshold. For this reason the trend of the red line in Figure 13 is suggested for the location tested in laboratory. The asymmetric effect still present but a limit has been set up for the lowest decrease of efficiency from the 2D conditions; the reduction of efficiency has been estimated to be of 0.6 for the NW directions while 0.45 for the SW directions.

The results from the laboratory tests indicate a decrease of efficiency from 50% in 2D conditions:

- to 40% due to 3D characteristic of the structure (as expected).
- to 30% (severe spreading). Spreading coming with waves can not be avoided and depending on its magnitude, it can decrease the hydraulic efficiency of the pilot project. In average it can be

said that spreading decreases efficiency up to 32%.

- to 25% for unfavorable attack angle on the structure. The influence of directionality is difficult to classify as strictly dependent on the bathymetry of the area and different wave conditions interact differently with the bottom; in average it can be said that directionality decreases efficiency up to 35%.

The combination of 3D-ness, spreading and directionality in the most severe condition decreases the efficiency of the SSG pilot from 50% to 15%. In average the overall decrease would be from 50% to 25%. These results are valid for the SSG pilot that has a very low width to depth ratio and it is therefore extremely sensitive to spreading and directionality. On a different configuration (more modules on a breakwater) those negative effects are milder.

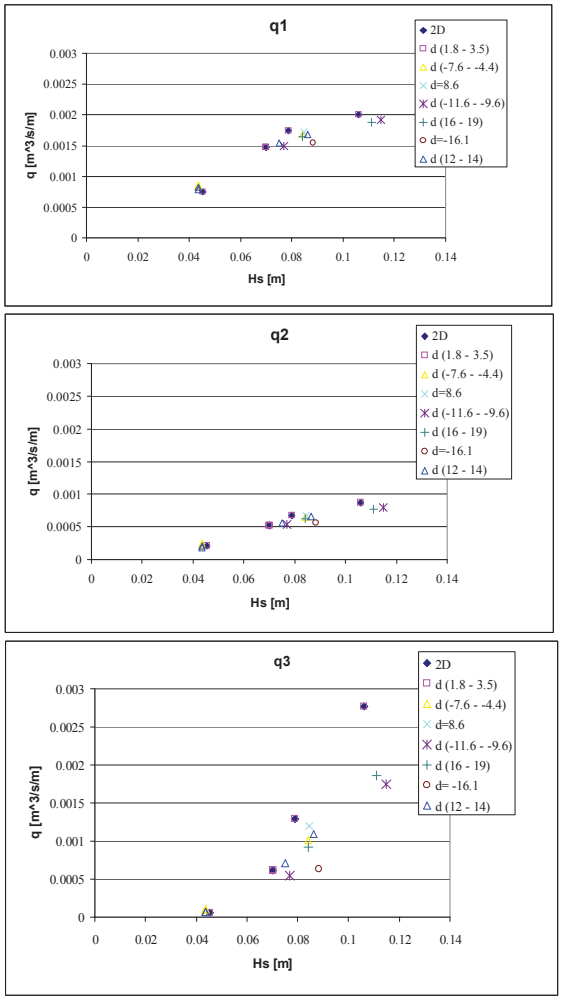

Figure 12: Flow rates into the 1st, 2nd and 3rd reservoir for different wave heights and input spreading coefficients.

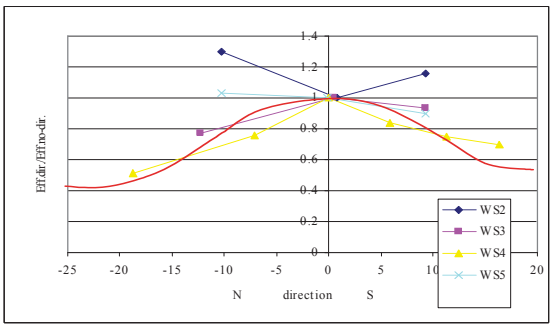

Figure 13: Flow rates into the 1st, 2nd and 3rd reservoir for different wave heights and input spreading coefficients and suggested trend of normalized efficiency depending on attack angle at the SSG structure (selected location).

3 Conclusions

The results of some recent research regarding a new type of structure for wave energy conversion Seawave Slot-Cone Generator (SSG) have been reviewed and discussed. For the first time at the Aalborg University the SSG concept has been modelled and tested with the main aim to give advice on expected overtopping rates and power production and on the structure designers on wave loading acting on different parts of the structure. Mainly two different behaviours were identified: surging waves on the front sloping plates and damped impact water jet on the vertical rear wall in upper reservoir. The order of magnitude of the extreme peak pressure on the front plates scaled to prototype were up to 250 kN/m2. On the vertical rear wall in the upper reservoir impact pressures (very peaked, short duration) of up to 580 kN/m2 were registered. For wave impact pressure scaling (vertical rear wall) some prototype measurements are needed. Wave pressures measurements planned at pilot SSG in Kvitsoy will be useful to estimate model-prototype scaling discrepancies.

Additionally has been shown and discussed how 3D phenomena are expected to reduce the hydraulic efficiency estimated to be around 50% in 2D studies, to 25%. This is manly due to the spreading and the directionality that are reducing the overtopping flow rates inside the reservoirs and so the stored potential energy. This is a result valid for the SSG pilot that is a module with a low width to depth ratio: when an attack angle is present, it has been noticed that waves hit the side walls and part of the water finds an obstacle to enter the reservoirs.

From these results the following conclusions have been reached:

1. a reduction of efficiency from 50% (2D conditions) to 40% due to the 3D-ness of the structure has been calculated.

2. A reduction of efficiency from 50% (2D conditions) to 32% due to spreading has been calculated.
3. A reduction of efficiency 50% (2D conditions) to 35% due to directionality has been calculated.
4. A reduction of efficiency from 50% (2D conditions) to 25% due to the combination of 3d-ness, spreading and directionality has been calculated.
5. The negative spreading effect on the efficiency increases with the increase of the spreading.
6. The negative directionality effect on the efficiency increases with the increase of the attack angle.
7. Prove of the influence of the bathymetry has been highlighted: waves with a positive attack angle (SW) have less negative influence on the flow rates and on the efficiency than the corresponding waves with a negative attack angle (NW).

References

[1] J. P. Kofoed. *Vertical Distribution of wave overtopping for design of multilevel overtopping based wave energy converter.* International Conference on Coastal Engineering, San Diego, US, 2006.

[2] D. Vicinanza, J. P. Kofoed, and P. Frigaard. *Wave loadings on Seawave Slot-cone Generator (SSG) at Kvitsøy island (Stavanger, Norway).* Hydraulics and Coastal Engineering No. 35, ISSN: 1603-9874, Dep. of Civil Eng., Aalborg University, 2006.

[3]N. W. H. Allsop, D. Vicinanza &J.E. Mckenna. *Wave forces on vertical and composite breakwaters.* Strategic Research Report, Hydraulic Research Wallingford, Sr 443, Wallingford 1996.

[4] M. Calabrese, D. Vicinanza. *Prediction of wave impact occurrence on vertical and composite breakwaters.* Excerpta of Italian contributions to the field of hydraulic engineering, Vol.XIII, 25pp., ISSN 0394-526X, 1999.

[5] D. Vicinanza. *Breaking wave kinematics in front of composite breakwaters.* Proceedings of the International Conference Coastal Structures, Santander, Spain, 1999.

[6] Y. Goda. *New Wave Pressure Formulae for Composite Breakwaters.* Proceedings of the 14th International Coastal Engineering Conference, Vol 3, pp 1702-1720, 1974.

[7] Y. Goda. *Random Seas and Design of Maritime Structures.* University of Tokyo Press, Tokyo, Japan, 1985.

[8] H. Oumeraci, P. Klammer & H. W. Partenscky. *Classification of breaking wave loads on vertical structures.* Proc. ASCE, J. Waterw. Port Coastal Ocean Eng. Div., 119 (4), 1993.

[9] J. P. Kofoed. *Model testing of the wave energy converter Seawave Slot-Cone Generator.* Hydraulic and Coastal Engineering No. 18, ISSN 1603-9874, Dep. Of Civil Eng., Aalborg University, 2005.

Spatial distribution of wave pressures on Sea Wave Slot Cone Generator

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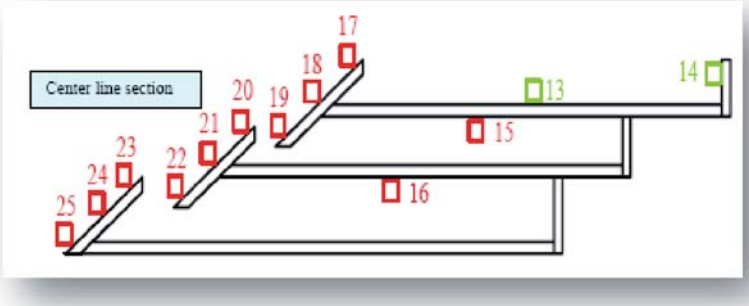
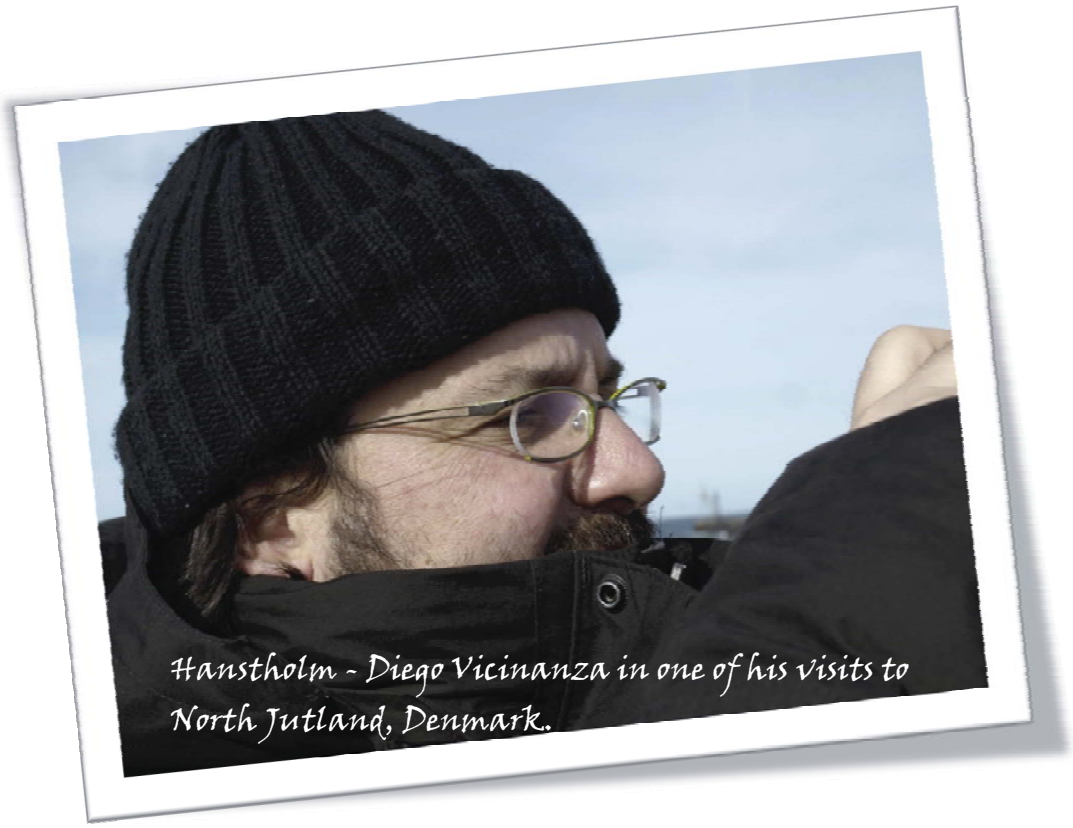
Vicinanza, Diego; Margheritini, Lucia; Peter Frigaard.





Thesis author´s contribution:

The Thesis author is the second author of this paper. She helped in the presentation of the results together with co-author Peter Frigaard. Diego Vicinanza is responsible for tests and analysis.

SPATIAL DISTRIBUTION OF WAVE PRESSURES ON SEAWAVE SLOT-CONE GENERATOR

Diego Vicinanza[1], Lucia Margheritini[2], Peter Frigaard[3],

This paper presents results on loading acting on an innovative caisson breakwater for electricity production. The work reported here is part of the European Union Sixth Framework programme priority 6.1 (Sustainable Energy System), contract 019831, titled "Full-scale demonstration of robust and high-efficiency wave energy converter" (WAVESSG). Information on wave loadings acting on Wave Energy Convert (WEC) Seawave Slot-Cone Generator (SSG) exposed to extreme wave conditions are reported. The SSG concept is based on the principle of overtopping and storing the wave energy in several reservoirs placed one above the other. Comprehensive 2D and 3D wave tank model tests were carried out at the Department of Civil Engineering, Aalborg University (Denmark) in the 3D deep water wave tank. The model scale used was 1:60 of the SSG prototype at the planned location of a pilot plant at the west coast of the island Kvitsøy near Stavanger, Norway. The research study is intended to be of direct use to engineers analyzing design and stability of the pilot plant.

INTRODUCTION

Global energy needs are likely to continue to grow steadily for at least the next two-and-a-half decades (International Energy Agency, 2006). If governments stick with current policies the world's energy needs would be more than 50% higher in 2030 than today. Over 60% of that increase would be in the form of oil and natural gas. Climate destabilising carbon-dioxide emissions would continue to rise, calling into question the long-term sustainability of the global energy system. More vigorous government policies in consuming countries are steering the world onto an energy path oriented to reduce the consumption of fossil fuels and related greenhouse-gas emissions and to the development of Renewable Energy Sources (RES).

No source of energy would be such without an effective, efficient and economic way to capture it. For millenniums oil has not been a font of energy, until the invention of the burst motor. To meet the need to integrate energy and environmental policies, researchers will be challenged to develop devices able to economically generate power from renewable energy sources as waves.

Wave energy is a renewable and pollution-free energy source that has the potential world-wide contribution in the electricity market estimated in the order of 2,000 TWh/year, that represent about 10% of the world electricity consumption with an investment cost of EUR 820 billion (Thorpe, 1999).

Today, the largest problem in harvesting wave energy is obtaining reliability of the technology and bringing the cost down.

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WAVEenergy AS company (Stavanger, Norway) was founded in April 2004 to develop the Seawave Slot-cone Generator (SSG) concept. The SSG is a wave energy converter based on the wave overtopping principle utilizing a total of three reservoirs placed on top of each other, in which the potential energy of the incoming wave will be stored (Fig. 1). The water captured in the reservoirs will then run through the multi-stage turbine for electricity production.

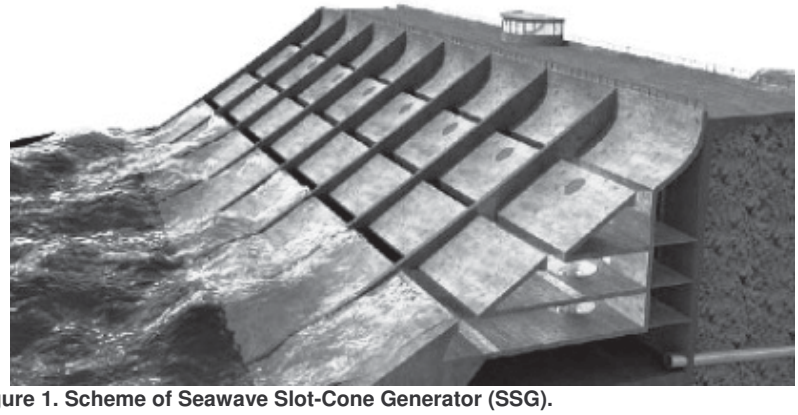

Figure 1. Scheme of Seawave Slot-Cone Generator (SSG).

WAVEenergy AS is currently carrying out a pilot project of the SSG wave converter at the island of Kvitsøy – Norway, partly founded by the European Commission (WAVESSG project). The Kvitsøy municipality has 520 inhabitants and is one of 10,000 islands in Europe where wave energy can quickly be developed into a cost effective energy production alternative to existing diesel generators.

The full-scale technical prototype of the SSG includes three reservoirs for capturing the ocean energy and is constructed as a robust shoreline device.

Preliminary estimates by WAVEenergy AS for the first commercial shoreline SSG is that a full scale SSG shoreline plant of 500 m length will be able to produce 10-20 GWh/year for a price of electricity of around 0,12 EUR/kWh in 2008. Such a price is already competitive with generation of electricity on islands by means of diesel-generators and in-line with payment schemes set up for wave energy in Portugal and Scotland. With further technical development and utilization of economies of scale, the forecasted ultimate price will be 0,04-0,06 EUR/kWh.

The main objective of the pilot project is to demonstrate at full-scale, the operation of one module of the SSG wave energy converter in a 19 kW/m wave climate, including turbine, generator and control system, and to connect the system to the public grid for electricity production. The pilot project regards a 10 m wide civil structure module of the SSG and will be installed within 2008.

In order to set-up and evaluate the optimal control strategy for the turbine, the SSG will be instrumented. The monitoring program will include measurements

of the wave characteristics, water levels in the 3 reservoirs and measurements of power production from the turbine. The water levels in the 3 reservoirs will not be still. Due to the wave disturbance in the reservoirs multipoint measurements of the water levels are needed. Consequently, a high number (9-12 plus spares) water level transducers will be installed. Attention will be given to positions of the water level transducers and to the reliability of the transducers. In addition at least one water level sensor (or other type) will be needed for wave measurements in front of the SSG to enable evaluation of the incoming waves.

The generator will be instrumented and power production from the turbine will be measured directly on the generator. Nevertheless, for evaluation of the SSG concept knowledge about the power productions coming from each of the 3 reservoirs are wanted. To achieve this knowledge the flow out of each of the reservoirs will be measured.

A key to success for the SSG will be low cost of the structure. The wave loadings on the main structure can be estimated using experiences from coastal protection structures, but the differences between SSG and such structures are so large that more reliable knowledge on the wave pressures is desired.

The aim is to optimize the structural design and geometrical layout of the SSG under extreme wave conditions (Vicinanza et al., 2006).

Measurements of wave pressures planned at pilot SSG in Kvitsoy will be useful to estimate model-prototype scaling discrepancies.

DESIGN CONDITIONS

The design sea states used in the model tests are found through a study of the wave climate in the area since 1955 (Larsen and Kofoed, 2005).

According to NORSOK (1999) the following sea-state parameters has an annual exceedance probability of 0.01 for sea-states of 3 hours duration at the Kvitsøy test site: H_{m0} = 14.5 m and T_p = 16 s. The maximum single wave height H_{100} is assumed to be 1.8 times H_{m0}. Statoil has gathered material on waves from 1955 to 2001 (Nygaard and Kenneth, 2002). In Table 1 the 100 years extreme events of the offshore environment near the test site are shown to the left. Due to refraction and diffraction in the near shore environment those offshore conditions gives the conditions on the plateau in front of the structure that are listed to the right in Table 1.

Table 1. 100 years extreme events.

	Offshore			Plateau	
	Θ [°]	Hs [m]	Tp [s]	Θ [°]	Hs [m]
	150	10.3	14.0	185	2.5
S	180	11.7	14.8	195	4.5
	210	10.8	14.3	225	5.5
	240	10.8	14.3	240	10.5
W	270	12.5	15.2	270	12.5
	300	13.2	15.6	285	9.5
	330	14.3	16.2	300	5.5
N	0	14.3	16.2	315	2.5

The waves from West (270°) are head-on waves. Hindcast wave data, DNMI, has been analyzed with a P.O.T. analysis (Goda, 1985).
From November 4th 2004 to March 11th 2005 the waves approximately 400 meters west of the test site have been measured. So far the largest observed H_s over half an hour on the test site is 9.77 m (T_p=14.8 s) reached on the 12th of January 2005. Furthermore, it was found that the maximum height of a single wave during the storm was 17.78 m. This occurred at 11.30 where the half hour Hs was 9.29 m. If the maximum height is compared to the six hour H_s the ratio Hmax/Hs is 2.03, i.e. considerably higher than 1.80.
West of the considered location the water depth is +100 meters. The plateau in front of the structure is approximately 300 meters in stretch and the depth is roughly speaking 30 meters on the entire plateau (Fig. 2).

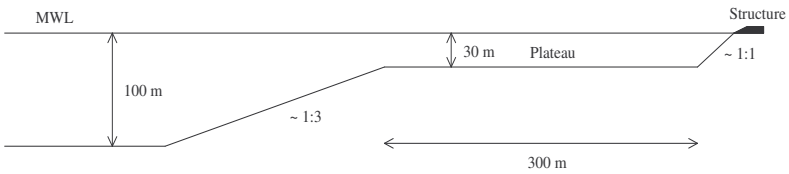

Figure 2. Rough sketch of the foreshore.

Therefore waves of less than 15 meters can not be expected to break on the plateau. If the waves are assumed no higher than 0.8 h in the near shore environment the largest possible wave height on the plateau would be 24 meters.
The variation of the water level in the region has been measured each 10 minutes all through the year 2000. The highest level above mean water level reached in one year was 1.54 m. For head-on waves the 100 year event at the plateau can be given by wave condition Hs = 12.5 m and Tp = 15.2 s, based on the study by Nygaard and Kenneth (2002). According to Table 1 it would be on the safe side to test waves in an angle of 315° with Hs up to 5.5 m. Based on the available tide information the extreme wave condition should be considered with a water level at least 1.54 m above normal. However the data referred only covers one year. Therefore it will be performed tests with a conservatively estimated high water level of 1.75 m.

Table 2. Summary wave sea state.

	Hs [m]	Tp [s]	H100 [m]	T100 [s]
NORSOK	14.5	16.0	27.6	13.4 – 17.4
Statoil				
Offshore	14.3	16.2	26.6	13.1 – 17.1
Plateau	12.5	15.2	23.3	12.3 – 16.0
Hindcast				
1261	11.9	15.2	22.1	12.0 – 15.6
1262	9.6	12.7	17.9	10.8 – 14.0
Test site	8.8	14.8	16.4	10.3 – 13.4
Max on plateau			24.0	12.5 – 16.2

WAVE PRESSURES ON CAISSON BREAKWATERS

A key to success for SSG devices is the optimization of costs maintaining the stability, the hydraulic performances and the energetic efficiency. To date few data are available for the design of these devices.

The methods described in the following section are not directly applicable to the tested SSG structure because of its novel design. Anyway the prediction methods described are the engineering tools that come closest.

Loading conditions

The forms and magnitudes of wave loadings acting upon caisson breakwaters under random wave conditions are highly variable and they are conveniently divided into "pulsating", when they are slowly-varying in time and the pressure spatial gradients are relatively mild, and "impact", when they are rapidly-varying in time and the pressure spatial gradients are extremely high (Allsop et al., 1996b; Vicinanza, 1997a; Vicinanza, 1997b; Vicinanza, 1999; Calabrese and Vicinanza, 1999).

Quasi-static or pulsating wave pressures change relatively slowly, varying at rates of the same order of magnitude as the wave crest ($p_{max} \approx \rho_w\ g\ H_{max}$). Two principal quasi-static loadings may be considered here. In the first, a wave crest impinges directly against the structure applying a hydro-static pressure difference. The obstruction of the momentum of the wave causes the wave surface to rise up the wall, increasing the pressure difference across the plates. The net force is approximately proportional to the wave height, and can be estimated using relatively simple methods (Fig. 3). Wave impacts occurs when the waves break directly on the structure with almost vertical front surface at the moment of impact or as a plunging breaker with small or large cushion of air inducing loads of much greater intensity and shorter duration than the quasi-static loads. The pressure/force history generally exhibit an impulsive zone characterised by high pressures ($p_{max} \approx 50 - 100\ \rho_w\ g\ H_{max}$) with shorter duration followed by a longer-lasting quasi-static force (Fig. 3).

Previous studies by Vicinanza (1997a, b), Vicinanza (1999) Calabrese and Vicinanza (1999) have shown that it is possible to distinguish between impact and quasi-standing waves from the probability distributions of wave forces on the structure. In this approach, all forces are ranked and plotted on a Weibull paper. A reduce variate u = f (Fhi) was adopted to build the probability paper related to each distribution examined*. Any significant departure of forces above the Weibull line is taken as indication of wave impacts. The percentage of impacts is given by the probability level, P, at which forces start to depart from the Weibull line. Where they follow the Weibull line, it is deemed that quasi-static conditions had occurred (Fig. 4).

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* taking in account that is valid the condition P(u) = P(Fhi).

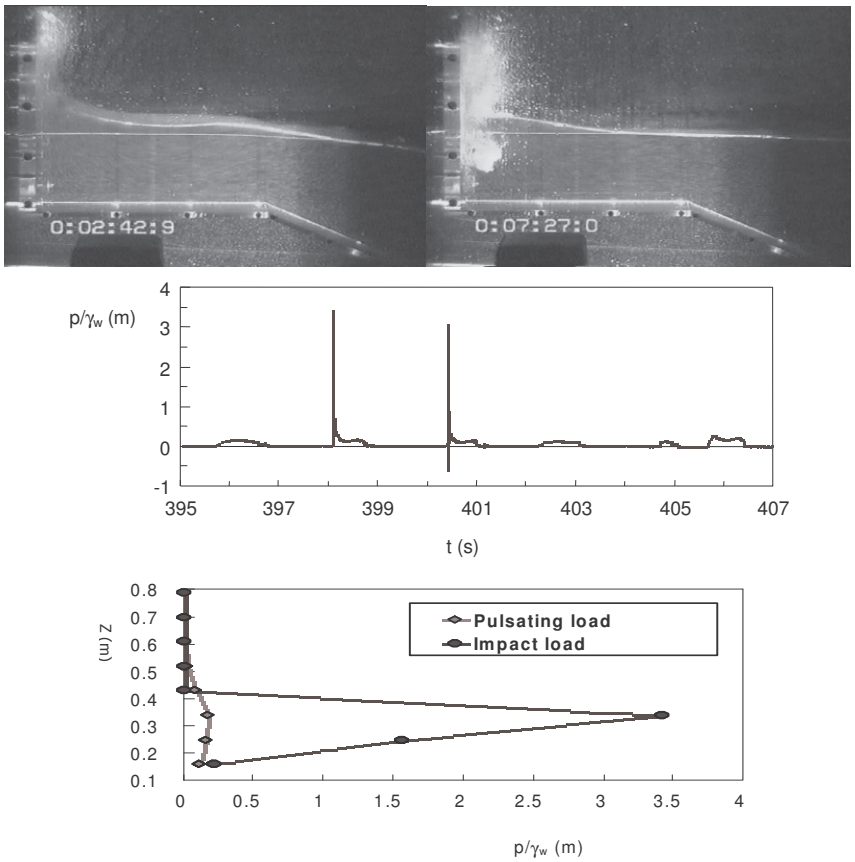

Figure 3. Quasi-static and impact time history and pressure spatial gradients (after Vicinanza, 1997a, b; Vicinanza, 1999).

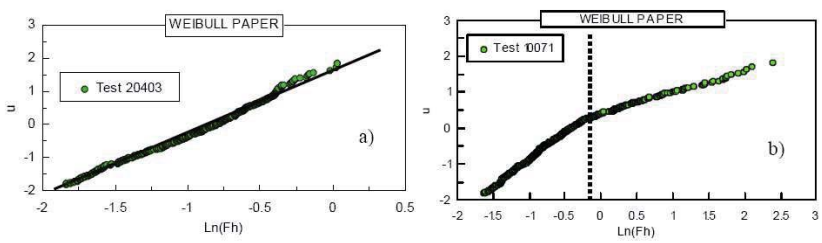

Figure 4. Weibull plot for conditions: a) quasi –static b) impact (after Vicinanza, 1997b).

Kortenhaus and Löffler (1998) use the analysis of force time series to characterise impact waves. In this method impacts occur when the maximum of a force event is higher than two well defined threshold values namely: the

maximum of the relative peak force $F_h / \rho_w \cdot g \cdot H_s^2$ has to exceed 2.5; furthermore the peak force $F_{h,max}$ has to be 2.5 times larger than the quasi-static maximum $F_{h,q}$ of the force event. The evaluation of relative frequencies of the breaker types at the structure may be also assessed from the analysis of wave pressure rise-time, t_r, at the s.w.l. (Martinelli, 1998).

Under the Research Project PROVERBS (PRObabilistic design tools for VERtical BreakwaterS) a parameter decision map has been developed to provide easy guidance to identifying the possible loading cases of waves attacking the front face of caisson breakwaters starting from dimensionless parameters based on structure geometry, water depth and wave conditions in the nearfield (Oumeraci et al., 1999). The parameter map for wave load classification has been set-up under PROVERBS to render decision of the expected design wave conditions at the structure. It allows to distinguish between impact loads, for which the load duration/time history is most relevant for the dynamic response of the structure, and the other wave loads for which the expected response of the structure is such that "quasi-static approaches" might apply. An initial version of the parameter map was suggested in 1996 by Allsop et al (1996a, b) analysing the HR94 data set. Subsequently some improvements of the map were performed by Kortenhaus & Oumeraci (1998). The Authors provided to feed same gaps persisting in the regions where only few data were available with supplementary data.

Design formulae

The most used method for pressure distribution on inclined wall is from Tanimoto and Kimura (1985). The Authors performed model tests and demonstrated that the Goda formula (1975) can be applied by projection of the Goda wave pressures calculated for a vertical wall with the same height* (crest level). The design method suggested by the CEM (2002) for prediction of pressure distribution on sloping top structures is Takahashi et al. (1994) formula. The sloping top caisson has been used for many years against very heavy wave conditions; the oldest caisson of this type being constructed in 1906 at Naples harbour (Italy). The Authors developed corrections to the well known Goda's p_1, p_2, p_3 (Goda, 1974; 1985) to take into account for a structure with a sloped portion beginning just below the waterline. The formula was based on the results of a series of laboratory experiments. The design method was tested using sliding experiments. The Authors found that the wave forces on the slope of the sloping top caissons are larger than those calculated by the previous design methods, while their formula overestimate the wave forces of the upright wall of the sloping top caissons. From this results the design method proposed by Takahashi et al. (1994) overestimate the minimum caisson weight for stability.

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* The T&K formula is valid for $\alpha \geq 70°$ and $l_d < 0.1$ L

LABORATORY STUDY

Model tests have been performed in a wave tank at Aalborg University, in 1:60 length scale compared to the prototype (Vicinanza et al., 2006). This wave basin (commonly called the deep 3-D wave basin) is a steel bar reinforced concrete tank with the dimensions 15.7 x 8.5 x 1.5 m. The paddle system is a snake-front piston type with a total of ten actuators, enabling generation of short-crested waves. The waves are absorbed by a rubble beach slope in the back of the basin to minimize reflection. The bathymetry in the immediate proximity of the pilot plant has been surveyed and the results have been used as the basis for the laboratory model. The SSG caisson model set up was designed following a specific study on hydraulic performances by Kofoed (2005, 2006) in which a total of 7 geometries have been tested. The overtopping rates for the individual reservoirs were measured and the power in the overtopping water was calculated. The geometry resulting in the highest overall average hydraulic efficiency was found. The model was built in plexiglass with dimension of 0.471 x 0.179 m. The three front plates were positioned with a slope of $\theta = 35°$. The model was fixed rigidly on a 3D concrete model of the cliff located in the middle of the basin at 5 m from the paddles. Fourteen Kulite Semiconductor pressure cells were used to measure the pressure in a total of 25 positions on the structure plates. Two different transducer configurations were needed because of the very limited space inside the model combined with the physical dimensions of the pressure transducers (Fig. 5).

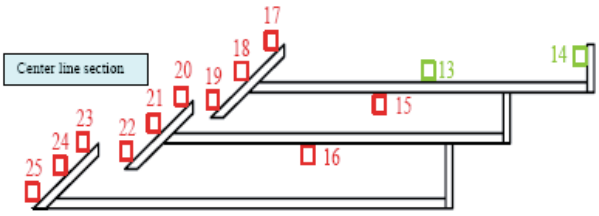

Figure 5. Tests configurations and pressure cells locations at center line section.

Video camera recordings of wave shapes at the structure were taken. JONSWAP sea states selected for the tests. Each test comprised approximately 1000 waves (1800 s). Tests were carried out with frontal and oblique waves (45°, denoted "Side" in Table 3), with various levels of directional spreading (n). Due to the extension of test setup, the oblique wave attack was realized by turning the complete model in the basin. A wave calibration method which takes into account the contribution of re-reflected waves from the wavemaker paddle has been used. The agreement with the target wave parameters was very good (within 2% for the considered tests). The experimental procedure has been designed to ensure that data are available to allow a good estimation of the surface loads corresponding to the design 100 years return period wave event at the plateau, given by wave condition Hs = 12.5 m and Tp = 15.2 s (Vicinanza et

al., 2006) corresponding to test 3 in Table 3. As reported in Table 1 not only the 100 years return period wave event were simulated in order to allow comparisons between laboratory data and field measured from the pilot plant once built. The wave signals were stored and reused from transducer configuration number one to configuration number two. The 32 tests were thus performed twice.

Table 3. Summary of model wave conditions

Test	Hs [m]	Tp [s]	swl [m]	Direction	Wave field	Test	Hs [m]	Tp [s]	swl [m]	Direction	Wave field	n
1	0.125	1.55	0.50	Front	2D	17	0.125	1.55	0.53	Front	3D	4
2	0.167	1.81	0.50	Front	2D	18	0.167	1.81	0.53	Front	3D	4
3	0.208	1.94	0.50	Front	2D	19	0.208	1.94	0.53	Front	3D	4
4	0.250	2.07	0.50	Front	2D	20	0.250	2.07	0.53	Front	3D	4
5	0.042	1.03	0.50	Side	2D	21	0.042	1.03	0.53	Side	3D	4
6	0.083	1.29	0.50	Side	2D	22	0.083	1.29	0.53	Side	3D	4
7	0.125	1.55	0.50	Side	2D	23	0.125	1.55	0.53	Side	3D	4
8	0.167	1.81	0.50	Side	2D	24	0.167	1.81	0.53	Side	3D	4
9	0.125	1.55	0.53	Front	2D	25	0.125	1.55	0.53	Front	3D	10
10	0.167	1.81	0.53	Front	2D	26	0.167	1.81	0.53	Front	3D	10
11	0.208	1.94	0.53	Front	2D	27	0.208	1.94	0.53	Front	3D	10
12	0.250	2.07	0.53	Front	2D	28	0.250	2.07	0.53	Front	3D	10
13	0.042	1.03	0.53	Side	2D	29	0.042	1.03	0.53	Side	3D	10
14	0.083	1.29	0.53	Side	2D	30	0.083	1.29	0.53	Side	3D	10
15	0.125	1.55	0.53	Side	2D	31	0.125	1.55	0.53	Side	3D	10
16	0.167	1.81	0.53	Side	2D	32	0.167	1.81	0.53	Side	3D	10

RESULTS

The major emphasis in any study on wave loadings is on the overall or average level of pressures, which is needed to determine the overall stability of the structure. Data on local pressures and pressure gradients are also needed in any analysis of conditions leading to local damage. The results for the most dangerous condition (normal attack and lower s.w.l.) reported in Table 4, indicate that pressures on front plates are quasi static ($p_{1/250} \sim \rho_w\ g\ H_s$) or pulsating loads generated by non-breaking waves.

Table 4. Summary of model tests pressure 1/250

Plate	Sampling rate (Hz)	Tdx	Test 1 p1/250 (kN/m2)	Test 2 p1/250 (kN/m2)	Test 3 p1/250 (kN/m2)	Test 4 p1/250 (kN/m2)
9	1200	14	5.16	5.51	7.84	9.74
3	200	17	0.86	1.37	1.48	2.15
	200	18	1.47	2.60	3.02	4.19
	200	19	1.44	2.30	2.44	2.90
2	200	20	1.49	2.08	2.37	3.03
	200	21	1.87	2.70	2.92	3.67
	200	22	1.12	1.55	1.65	1.89
1	200	23	1.05	1.56	2.31	2.53
	200	24	1.40	2.07	2.28	2.83
	200	25	1.61	2.43	2.49	3.31

Considering for comparison with Takahashi et al. (1994) formula only the no spreading tests (1-16), the results show an underestimation using the prediction formula between 20-50%. Pressure gradients analysis for test 3 (design condition) and 4 (extreme condition) highlights large discrepancies (Figure 6-7).

One of the reasons is that the SSG model was fixed rigidly instead the design method was tested using sliding experiments. In fact the Takahashi et al. model caissons were fabricated from synthetic acrylic plates and had a bottom comprised of a concrete slab for simulating the friction factor.

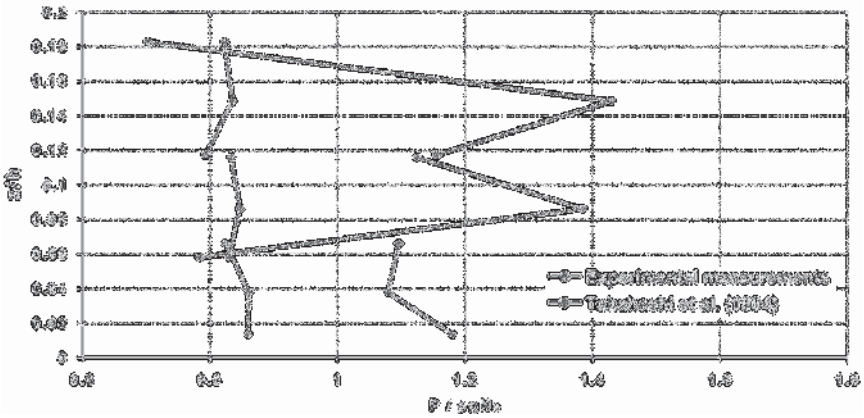

Figure 6. Takahashi et al. (1994) formula compared test 3.

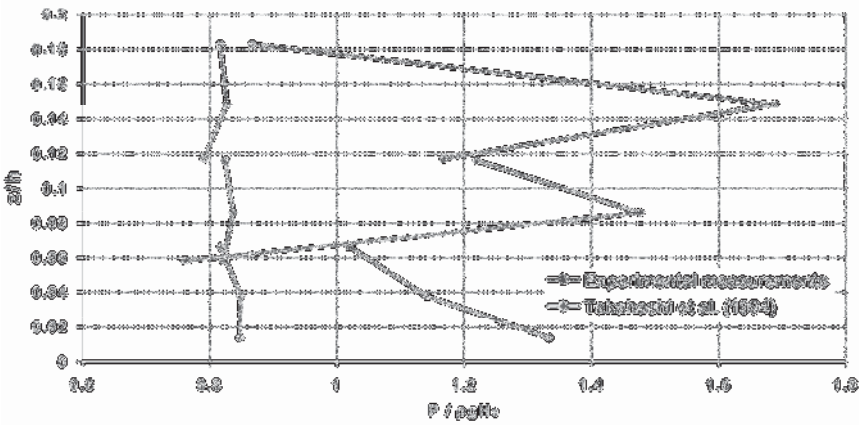

Figure 7. Takahashi et al. (1994) formula compared to test 4.

DISCUSSION AND CONCLUSIONS

Laboratory test with a Seawave Slot-Cone Generator show very high pressures from the design waves. Devices to capture wave energy are by nature very exposed to large wave forces. Opposite to traditional sea defence structures wave energy structures are designed in a way so they face and challenge the sea as much as possible. Never the less the fact that the tests show 50% higher wave pressures than the 'best' available design equation (Takahashi et al., 1994), suggests that design wave pressures is a topic needing careful attention, and not all experience from designing traditional maritime structures are usable.

Prediction method by Takahashi et al. (1994) gives an underestimation of pressures values acting on the front sloping plates between 20-50%.

The analysis of these pressure measurements made at laboratory scale using fresh water has explicitly assumed a Froude scale conversion to prototype values. In the case of pulsating wave pressures the assumption of Froude scaling is realistic while for wave impact pressure scaling is less simple. It has long been argued in PROVERBS, that wave impact in small scale hydraulic model tests will be greater in magnitude, but shorter in duration than their equivalents at full scale in (invariably aerated) sea water. It is very probable that the higher peak pressures measured in these model tests can be scaled to lower values, but probably each will attend by longer impulse durations. The argument on scaling these peak pressures requires information not presently available on the relationships between the statistics of the pressure time gradients and the magnitude of the pressure impulses. It can be argued that the magnitude of the pressure impulse, given perhaps by ($p\ \Delta t$) will not be changed between model and prototype, other than by the normal scaling relationships.

In order to follow up on model-prototype scaling discrepancies the full scale pilot device in Kvitsoy will be instrumented and measurements will be taken over the next years.

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REFERENCES

Allsop, N.W.H., Vicinanza, D., M. Calabrese & L. Centurioni, (1996°). Breaking wave impact loads on vertical faces, 6th Int. Conf. ISOPE, Los Angeles.

Allsop, N.W.H., D. Vicinanza & J.E. Mckenna, (1996b), "Wave Forces On Vertical And Composite Breakwaters", Strategic Research Report, Hydraulic Research Wallingford, Sr 443, Wallingford.

Calabrese, M., Vicinanza, D. (1999), "Prediction of wave impact occurrence on vertical and composite breakwaters", Excerpta of the Italian contributions to the field of hydraulic engineering, Vol.XIII, 25pp., ISSN 0394-526X.

CEM - Coastal Engineering Manual (2002), U.S. Army Corps of Engineers. 2002. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).

Goda, Y. (1975). "New Wave Pressure Formulae for Composite Breakwaters," Proceedings of the 14th International Coastal Engineering Conference, Vol 3, pp 1702-1720.

Goda, Y. (1985), Random Seas and Design of Maritime Structures, University of Tokyo Press, Tokyo, Japan.

Kofoed, J.P., 2005. Experimental Hydraulic Optimization the Wave Energy Converter Seawave Slot-Cone Generator. Hydraulics and Coastal Engineering No. 26, ISSN: 1603-9874, Department of Civil Engineering, Aalborg University.

Kofoed, J.P., 2006. Vertical Distribution of Wave Overtopping for Design of Multi Level Overtopping Based Wave Energy Converters. 30th International Conference on Coastal Engineering, San Diego.

Kortenhaus, A., & A. Löffler, 1998. Differences in estimating probability of impacts. Status Report 2nd Overall Project Workshop MAST III (PROVERBS). Naples.

Kortenhaus, A., & H. Oumeraci, 1998. Classification of waves loading on monolithic coastal

Larsen, B. J. and Kofoed, J. P. (2005), "Model Test Setup and Program for Experimental Estimation of Surface Loads of the SSG Kvitsøy Pilot Plant from Extreme Wave Conditions", Hydraulics and Coastal Engineering No. 32, ISSN: 1603-9874, Dep. of Civil Eng., Aalborg University.

Martinelli, L., 1998. Risk analysis of a vertical breakwater subjected to breaking waves: the case of Genoa Voltri. Ph.D. thesis, DISTART, Ingegneria Idraulica, Università di Bologna.

Nygaard E. and Kenneth J. E. (2002), “Refraction Analysis – Boknafjorden”, Sarepta Journal: PTT NKG NO 0044 - Statoil internal report.

Oumeraci H., N.W.H. Allsop, M.B. de Groot, R.S. Crouch & J.K. Vrijling, 1999. Probabilistic design tools for vertical breakwaters. Rotterdam the Netherlands: Balkema.

structures. 26th Int. Conf. on Coastal Eng., Copenaghen.

Takahashi, S., Hosoyamada, S., Yamamoto, S., 1994. Hydrodynamic Characteristics of Sloping Top Caissions. Proceedings of International Conference on Hydro-Technical Engineering for Port and Harbour Construction, Port and Harbour Research Institute, Japan, Vol 1, pp. 733-746.

Tanimoto, K., and Kimura, K. (1985), "A Hydraulic Experimental Study on Trapezoidal Caisson Breakwaters," Technical Note No. 528, Port and Harbour Research Institute, Yokosuka, Japan (in Japanese).

Thorpe, T.W., (1999),"An Overview of Wave Energy Technologies: Status, Performance and Costs","Wave Power - Moving Towards Commercial Viability", IMECHE Seminar, London, UK.

Vicinanza, D. (1997a), "Probabilistic analysis of horizontal wave forces on composite and vertical breakwaters", 2nd PROVERBS MAST III technical workshop, Las Palmas.

Vicinanza, D. (1997b), "Pressioni e forze di impatto di onde frangenti su dighe a paramento verticale e composte", PhD. thesis, University of Naples, Italy.

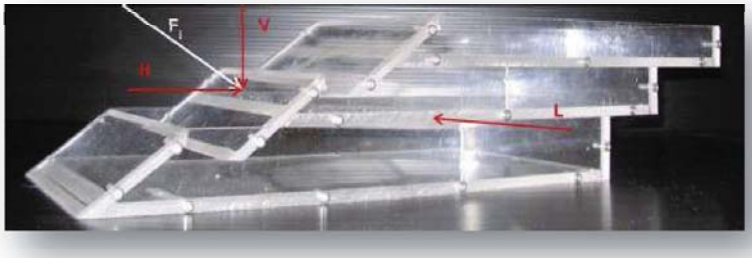
Vicinanza, D. (1999). "Breaking wave kinematics in front of composite breakwaters", Proceedings of the International Conference Coastal Structures ‘99, Santander, Spain.

Vicinanza, D., Kofoed, J. P. and Frigaard P. (2006), " Wave loadings on Seawave Slot-cone Generator (SSG) at Kvitsøy island (Stavanger, Norway)", Hydraulics and Coastal Engineering No. 35, ISSN: 1603-9874, Dep. Civil Eng., Aalborg University.

Sea Wave Slot Cone Generator: an innovative caisson breakwater for energy production.

The International Conference on Coastal Engineering, nr. 31, Hamburg, Tyskland, 31. august 2008 - 5. september 2008

Vicinanza, Diego; Margheritini, Lucia ;Pasquale Contestabile, Kofoed, Jens Peter and Peter Frigaard.



Thesis author´s contribution:

The Thesis author is the second author of this paper. She performed the laboratory tests on overall forces on the SSG device with advice from co-author J.P. Kofoed and made the comparison with the results from first author´s Diego Vicinanza on wave loading. Co-author Pasquale Contestabile assisted the first author on data analysis and co-author Peter Frigaard provided great motivation.

SEAWAVE SLOT-CONE GENERATOR: AN INNOVATIVE CAISSON BREAKWATERS FOR ENERGY PRODUCTION

Diego Vicinanza[1], Lucia Margheritini[2], Pasquale Contestabile[1], Jens Peter Kofoed[2] and Peter Frigaard[2]

This paper discusses a new type of Wave Energy Converter (WEC) named Seawave Slot-Cone Generator (SSG). The SSG is a WEC of the overtopping type. The structure consists of a number of reservoirs one on the top of each others above the mean water level in which the water of incoming waves is stored temporary. Using this method practically all waves, regardless of size and speed are captured for energy production. In each reservoir, expressively designed low head hydro-turbines are converting the potential energy of the stored water into electrical power. A key to success for the SSG will be low cost of the structure. The wave forces on the main structure can be estimated using experiences from coastal protection structures, but the differences between the structures are so large that more reliable knowledge on the wave forces is desired. The purpose of the work is to derive information on wave loadings acting on sloping walls constituting the structure. The research is intended to be of direct use to engineers analyzing design and stability of this peculiar kind of coastal structure.

INTRODUCTION

World energy needs are continuing to grow steadily for at least the next two-and-a-half decades. If governments stick with current policies the world's energy needs would be more than 50% higher in 2030 than today. Over 60% of that increase would be in the form of oil and natural gas. Climate destabilizing carbon-dioxide emissions would continue to rise, calling into question the long-term sustainability of the global energy system.

No source of energy would be such without an effective, efficient and economic way to capture it. For millenniums oil has not been a font of energy, until the invention of the burst motor. To meet the need to integrate energy and environmental policies, researchers will be challenged to develop devices able to economically generate power from renewable energy sources as ocean waves.

Likewise the other renewable energies, wave energy has experimented a prosperous moment during the last years. The first patented WEC device is from Girard and sons in 1799. The intensive research and development study of wave energy conversion began however after the dramatic increase in oil prices in 1973. Different European countries considered wave energy as a possible source of power supply and introduced support measures and related programmes for developing WECs. Denmark, Ireland, Norway, Portugal, Sweden and the United

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Kingdom have developed, nowadays, industrially exploitable wave power conversion technologies.
The amount of the work on wave energy is very large and extensive reviews have been made by Leishman and Scobie (1976), Salter (1989), Thorpe (1999), Clément et al. (2002), Cruz (2008) and many others.
However, whereas innumerable projects went through a simple initial testing phase, only few of them reached the sea prototype testing and even fewer have been commercialized. After many failures, it is obvious that much has been wasted on designs which could never be cost-effective, or capable to survive storms. The offshore devices, although more efficient in terms of energy recovery, are, in fact, too much exposed to severe ocean waves so they become expensive since they require more maintenance. Developers´ efforts are lately concentrated on demonstrating the reliability of the devices and on lowering the price per kW of produced power.
The Seawave Slot-cone Generator (SSG) concept is developed by WAVEenergy AS, Stavanger (Norway) since April 2004 (Kofoed and Osaland, 2005; Vicinanza et al. 2006; Vicinanza et al. 2007; Vicinanza and Frigaard, 2008). The SSG is based on storing the potential energy of the incoming waves in several reservoirs placed one above the other. Using this method practically all waves, regardless of size and speed are captured for energy production. The incoming wave will run uphill a slope and on its return it will flow into the reservoirs. After the wave is captured inside the reservoirs the water will run thru the turbine (Fig. 1).

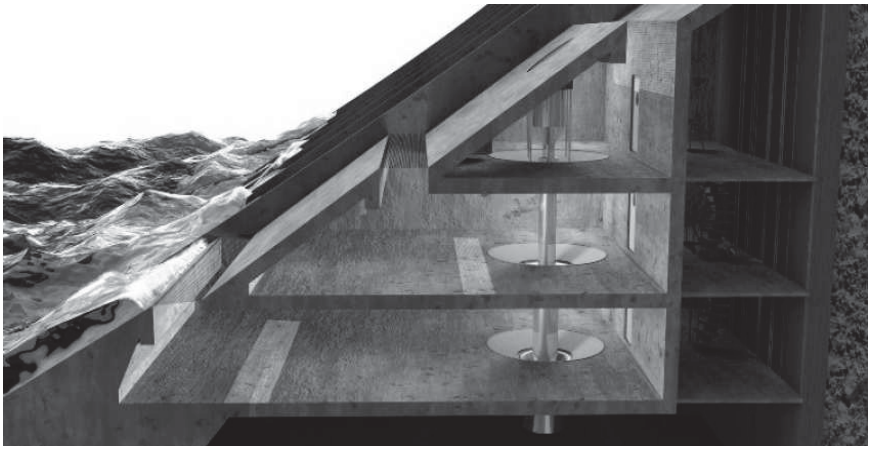

Figure 1. Lateral section of a three-levels SSG device with Multistage Stage Turbine.

In this case the reliability issue has been initially solved by realizing an “on shore” device where loads on the structure are considerably smaller than offshore, while the cost per kW compares prices of electricity for remote areas supplied by diesel generators. Nevertheless, when going from offshore to shore

the bathymetry can influence the overtopping flow rates i.e. the overall efficiency of the converter. A promising application of the SSG concept is as crown wall on a vertical breakwater; but while the design of such structures is made to minimize overtopping and run up, the SSG design focuses on a combination of maximization of both these events.
WAVEnergy AS is currently carrying out a pilot project of the SSG, partly founded by the European Commission (http://www.wavessg.com). The project can now benefit of 2.7M€, the majority of which are from private investors.
The purpose of the work described in this paper is to report the state of the art on wave loadings acting on SSG. The aim is to optimize the structural design and geometrical layout of the SSG under extreme wave conditions.

PREVIOUS STUDIES

Device efficiency

The SSG overall efficiency is the combination of the hydraulic efficiency, storage efficiency (dependent on the reservoirs volumes), turbines and grid connection efficiency. The design of the front face of the SSG deals with the optimization of hydraulic performance.
The overall efficiency of the device is the ratio between power output and the available wave power, given by the formula:

$$P_{wave} = \frac{\rho \cdot g^2}{64 \cdot \pi} \cdot H_s^2 \cdot T_E \tag{1}$$

where ρ =1020 kg/m^3, g = 9.81 m/s^2 and T_E is the energy period = m-1/m_0, where m_n is the n-th moment of the wave spectrum defined as:

$$m_n = \int_0^\infty f^n \cdot \Phi(f) \cdot df \tag{2}$$

where Φ is the frequency spectrum. It is possible to consider the efficiency of the SSG overtopping device as a combination of partial efficiencies for every one of which it is necessary an optimization of parameters. The hydraulic efficiency is defined as:

$$\eta_{hydr} = \frac{P_{hydr}}{P_{wave}} \tag{3}$$

where

$$P_{hydr} = \sum_{j=1}^{3} q_{ov,j} \cdot R_{c,j} \cdot \rho \cdot g \tag{4}$$

$q_{ov,j}$ is the total overtopping flow rate for the j-reservoir $R_{c,j}$ is the crest level of the respective reservoir, ρ the density of the sea water and g is the acceleration of gravity (Fig. 2). For the SSG pilot plant a number of 3 reservoirs has been chosen as adding extra reservoirs would only increase the hydraulic efficiency of 2% (Kofoed, 2006). 2D physical model tests have been carried out in order to optimize the geometry of the SSG pilot device (Kofoed, 2005). More then 30 geometries were tested under 2D irregular waves changing angles of the fronts, distances of the fronts, length of the fronts and crest levels. As a result the final geometry has been defined with front plates angles of θ = 35°. Hydraulic efficiency of 50% has been estimated. 2D tests as such did not take into consideration the effect of bathymetry, directional wave spectrum and spreading, all phenomena that can influence the overtopping flow rates in the reservoirs. Margheritini et al. (2008) found that directionality and spreading act on the overtopping for the three reservoirs of SSG pilot plant resulting in an overall reduction of the stored water up to 40%.

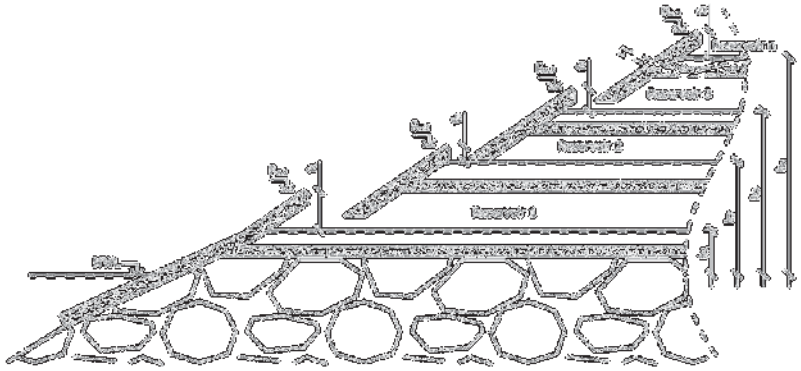

Figure 2. Definition sketch.

Wave loadings

Unlike traditional harbor defenses, WEC devices need by nature to be exposed to large wave forces and are generally designed to face and challenge the sea as much as possible. Vicinanza et al. (2006) suggest that the design criteria of traditional maritime structures may be not satisfactory for designing innovative breakwater as SSGs.

The physical experiments employed for the analyses presented below were conducted at the wave basin of Aalborg University (commonly called the deep 3-D wave basin), in 1:60 length scale compared to the prototype. The model was built in plexiglass with dimension of 0.471 x 0.179 m. The three front plates were positioned with a slope of θ = 35°. The model was fixed rigidly on a 3D concrete model of the cliff located in the middle of the basin. Pressure cells

were used to measure the pressure in a total of 25 positions on the structure plates. The tests are described in detail in Vicinanza and Frigaard (2008). The wave condition tested are reported in table 1.

Table 1. Summary of wave conditions.

Test	Hs [m]	Tp [s]	swl [m]	Direction	Wave field	n
1	0.125	1.55	0.50	Front	2D	-
2	0.167	1.81	0.50	Front	2D	-
3	0.208	1.94	0.50	Front	2D	-
4	0.250	2.07	0.50	Front	2D	-
5	0.042	1.03	0.50	Side	2D	-
6	0.083	1.29	0.50	Side	2D	-
7	0.125	1.55	0.50	Side	2D	-
8	0.167	1.81	0.50	Side	2D	-
9	0.125	1.55	0.53	Front	2D	-
10	0.167	1.81	0.53	Front	2D	-
11	0.208	1.94	0.53	Front	2D	-
12	0.250	2.07	0.53	Front	2D	-
13	0.042	1.03	0.53	Side	2D	-
14	0.083	1.29	0.53	Side	2D	-
15	0.125	1.55	0.53	Side	2D	-
16	0.167	1.81	0.53	Side	2D	-
17	0.125	1.55	0.53	Front	3D	4
18	0.167	1.81	0.53	Front	3D	4
19	0.208	1.94	0.53	Front	3D	4
20	0.250	2.07	0.53	Front	3D	4
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30	0.083	1.29	0.53	Side	3D	10
31	0.125	1.55	0.53	Side	3D	10
32	0.167	1.81	0.53	Side	3D	10

The SSG innovative structure cannot fit any standard design method, however in order to check the general tendency of these test results, the experimental pressures were compared with the design criteria suggested by the CEM (2000) for predicting pressure distribution on sloping top structures. Pressure measurements compared with the prediction method made for caisson breakwaters with sloping top showed 20–50% higher wave pressures than the Takahashi et al. (1994) design equation (Fig. 3).

Directionality and spreading effect highlights different behavior for each front sloping plate. Obliquity loading reduction is about 12-17%. Spreading loading reduction is about 10% (front attack) and 13% (side attack).

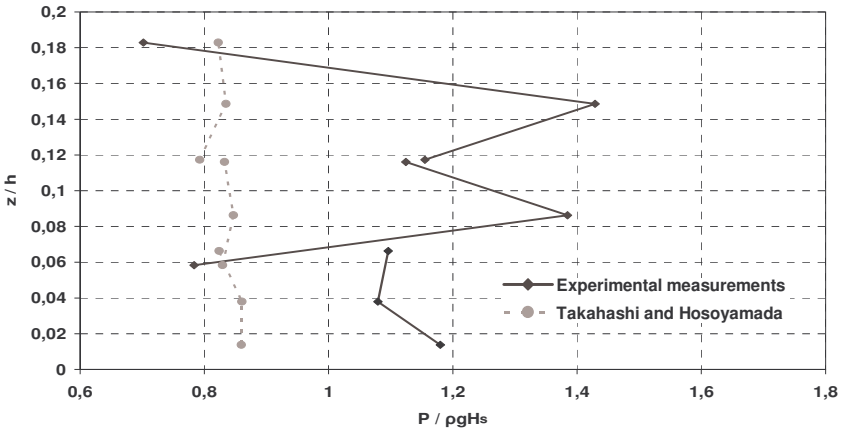

Figure 3. Takahashi et al. (1994) formula compared to measured data (after Vicinanza and Frigaard, 2008).

These results suggested using the experimental data as design pressures. Moreover, the combined analysis of video-camera and pressures records made it possible to identify two different behaviors of waves acting on the sloping front plates: surging waves (frontal attack) and partially damped plunging breaker (side attack). The vertical rear wall in upper reservoir is characterized by evident wave impact considerably damped by the preceding foamy mass.

The inspection by visual analysis of results obtained adopting this method has confirmed that forces Weibull distributed corresponded to quasi-standing waves, as shown in Figure 4 (Vicinanza, 1997). The evaluation of relative frequencies of the breaker types at the structure may be also assessed from the analysis of wave pressure rise-time, tr, at the s.w.l. (Martinelli, 1998). Figure 5 shows, for non breaking wave conditions, the frequency distribution of tr divided by the mean wave period Tm.

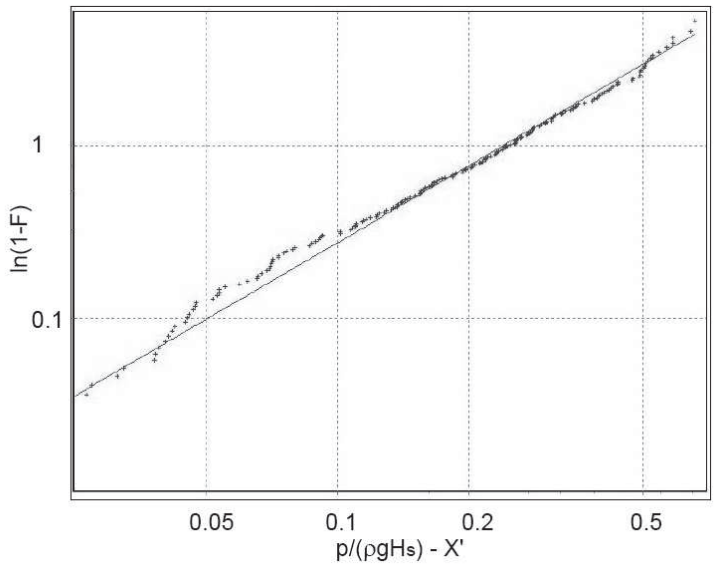

Figure 4. Weibull plot for quasi –static conditions.

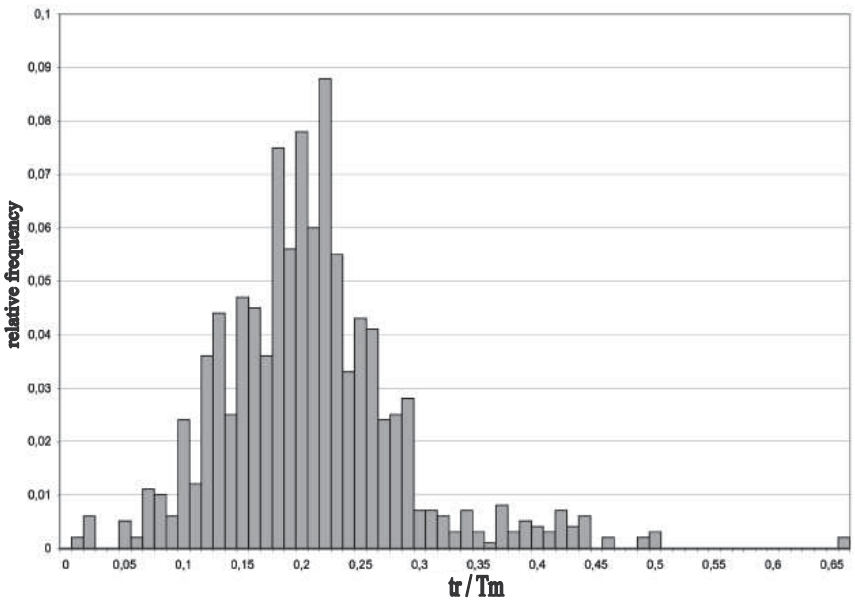

Figure 5. Relative frequency of t_r /T_m for quasi –static conditions.

LABORATORY STUDY

Force transducers have been used in order to understand the overall forces acting on the structure in the three main directions and their application points, as well as overturning moments for stability calculations. Being a substantially different method of measuring with respect to the one with pressure transducers, this set of tests also offers the chance to compare and validate with results from forces derived by pressures.

Two transducers have been used. One is a 'bone' transducer using pairs of strain gauges in a Wheatstone Bridge to measure the moments at two points (M1 and M2 in Fig. 6) and the other one is a tri-axial load cell (point O in Fig. 6) which measures the forces in all three axes. Both the instruments were recording at 50Hz. The 'bone' transducer measures moments in 2 planes. For this reason some tests were repeated with the moment transducer turned of 90 degrees to measure the sideways moments. The model is suspended a few millimetres above an artificial landscape by rigid connection to the bridge over the tank.

The wave conditions for the tests correspond to values around the design criteria of a 100 years event, as well as the wave condition reported in table 1. In particular the 45° attack angle on the structure has been tested as it represents the most demanding condition for the lateral vertical wall.

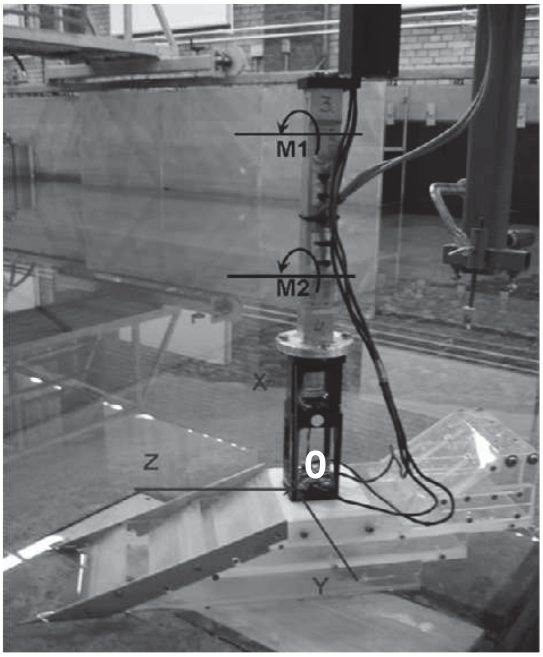

Figure 6. Force Transducers

RESULTS

The results are reported in terms of maximum forces ($F_{1/250}$) and presented scaled up to prototype. It is assumed that there is no friction between the model and the water and therefore the force due to the water acts normally to the surface of the model. This means that the waves striking the side wall of the model have only a horizontal component and this contributes positively to the overturning moment. Wave forces acting on the front of the structure have vertical and horizontal components and the downward vertical force (which would be acting when the waves strike the front of the model) contributes negatively to the overturning moment. The significant results on measured force for the 100 years return period event and for the 45° attack angle wave condition are summarized in Table 2.

Table 2. Summary of the most demanding overall forces on the structure in prototype scale.

Test description	Force in x positive direction	Force in z positive direction	Force in y positive direction	Force in y positive direction
Head on attack (Hs=12.5m,Tp=15s)	6.25 MN	7.31 MN	5.95 MN	8.41 MN
45° attack angle (Hs=7.5m, Tp=12s)	5.35 MN	3.55 MN	8.48 MN	-

It can be noticed that the force on the lateral vertical wall under 45° attack angle condition is bigger (8.48 MN) then the force acting on the frontal plates (7.31 MN). This is because the frontal plates have an inclination that improves run-up performances and decreases loads on the structure, while the side walls allow impact and so higher forces on them. Very high forces have also been recorded in the y positive direction under extreme wave conditions (8.41 MN).

The comparison between the force calculated from pressure measurements and the one directly measurement show that forces derived by integration are medially 1.7 times higher than forces directly measured for the same wave conditions. In Figure 7 is shown a direct comparison of a time series of forces on the sloping wall from direct measurements and from calculation from pressures on the model. In Table 3 are reported the most significant results in terms of forces measured and calculated from pressures.

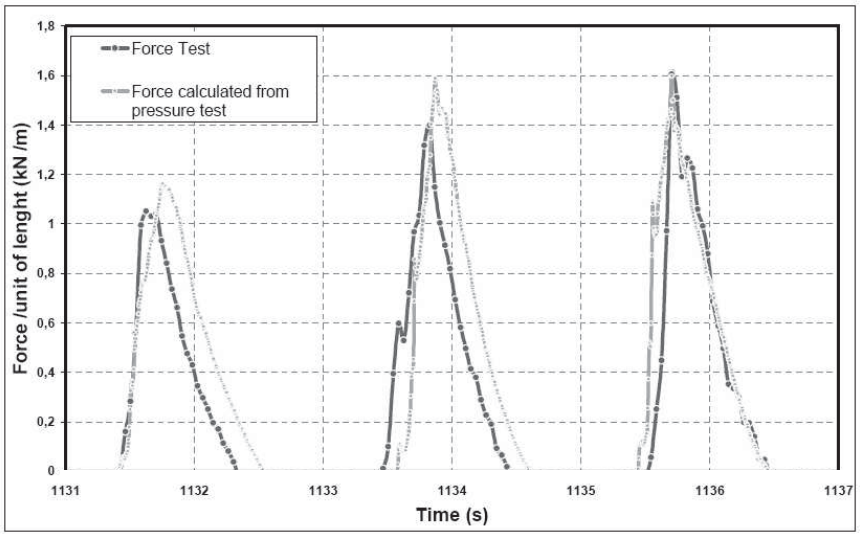

Figure 7. Comparison between frontal forces F measured with the load cell and calculated by integration from pressures files for the 100 years return event with head on attack.

Table 3. Comparison between maximum forces from direct model measurements and calculated by integration from pressures data files, prototype scale.

	Front attack. Hs=12.5m, Tp=15s			45° attack angle Hs=7.5m, Tp=12s		
	Pressure test	Force test	ratio	Pressure test	Force test	ratio
V (MN)	8.43	6.24	1.3	10.29	5.35	1.9
H (MN)	12.04	7.31	1.6	7.02	3.55	2.0
L (MN)	-11.99	-5.95	2.0	-12.21	-8.48	1.4

CONCLUSIONS

Comparison between maximum forces calculated from direct measurements on the model and worked out by integration from pressures (also measured on a similar model) showed a discrepancy of up to a factor of 1.7 in between them. This difference was expected because in the first model setup the structure was fixed rigidly on a 3D concrete cliff while the second setup is suspended a few millimetres above an artificial landscape by rigid connection to the bridge over the tank.

The test have showed that design equations developed in the coastal engineering field i.e. Takahashi's prediction tools for wave pressures acting on breakwaters caissons only predict half of the actual pressures/forces relatively to the first setup while are comparable whit the second setup.

Some consideration are needed about the application of SSG structure as sloping crown wall on a vertical breakwater. As suggested in design practices in Japan, the placement of the crest of caisson should be relatively low, allowing heavy overtopping and reducing wave forces and reflection. Using the SSG as a sloping top caisson (Fig.7) is also possible to produce pollution free energy.

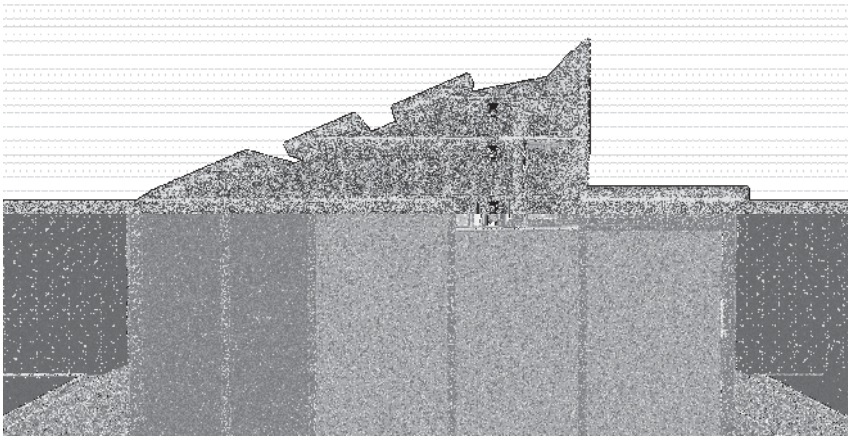

Figure 7. Lateral section of SSG structure as sloping crown wall on a vertical breakwater

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REFERNCES

CEM (Coastal Engineering Manual), 2002. U.S. Army Corps of Engineers. 2002. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).

Clément, A.H., P., McCullen, A., Falcão Fiorentino, F., Gardner, Hammarlund, K., Lemonis, G., Lewis, T., Nielsen, K., Petroncini, S., Pontes, M.-T., Schild, P., Sjöström, B.iO., Sørensen H. C., , T, Thorpe, 2002. Wave Energy in Europe - Current Status and Perspectives. Renewable and Sustainable Energy Reviews 6, pp. 405-431

Cruz, J., 2008. Ocean Wave Energy, Current Status and Future Perspectives. Series: Green Energy and Technology. ISBN: 978-3-540-74894-6.

Leishman, J. M., G., Scobie, 1976. The development of wave power - a techno economical study, Dept. of Industry, NEL Report, EAU M25

Kofoed, J.P., 2005. Experimental hydraulic optimization the wave energy converter seawave slot-cone generator. *Hydraulics and Coastal Engineering*, No. 26, ISSN: 1603-9874, Dep. of Civil Eng. Aalborg University.

Kofoed, J.P., 2006. Vertical Distribution of Wave Overtopping for Design of Multi Level Overtopping Based Wave Energy Converters. *30th International Conference on Coastal Engineering*, San Diego, USA.

Margheritini, L., D., Vicinanza, P., Frigaard. 2008. Seawave Slot-cone Generator Overtopping Performance in 3D Conditions, 1*8th International Conference ISOPE*, Vancouver, BC, Canada.

Martinelli, L., 1998. Risk analysis of a vertical breakwater subjected to breaking waves: the case of Genoa Voltri. *Ph.D. thesis*, DISTART, Ingegneria Idraulica, Università di Bologna.

Salter, S.H., 1989. World progress in wave energy – 1988, The international Journal of Ambient Energy, Vol. 10 (1).

Takahashi, S., S., Hosoyamada, and S. Yamamoto, 1994. Hydrodynamic characteristics of sloping top caissions. *Proceedings of International Conference on Hydro-Technical Engineering for Port and Harbour Construction*, Port and Harbour Research Institute, Japan, vol 1, pp. 733–746.

Thorpe, T.W., (1999),"An Overview of Wave Energy Technologies: Status, Performance and Costs","Wave Power - Moving Towards Commercial Viability", IMECHE Seminar, London, UK

Vicinanza, D., 1997. Pressioni e forze di impatto di onde frangenti su dighe a paramento verticale e composte, *PhD. thesis*, University of Naples (in Italiana).

Vicinanza, D., J.P., Kofoed, and P. Frigaard. 2006. Wave loadings on Seawave Slot-cone Generator (SSG) at Kvitsøy island (Stavanger, Norway). *Hydraulics and Coastal Engineering*, No. 35, ISSN: 1603-9874, Dep. of Civil Eng. Aalborg University.

Vicinanza, D., and P. Frigaard. 2008. Wave pressure acting on a seawave slot-cone generator, *Coastal Engineering*, 55, 553-568.

Technological and Commercial Comparison of OWC and SSG Wave Energy Converters Built into Breakwaters

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Margheritini, Lucia and Peter Frigaard.

Thesis author´s contribution:

The Thesis author is the first author of this paper. She is responsible for the data on SSG performance presented in this work (economical data on SSG are from WaveEnergy AS). She collected and analyzed material and data on OWC and together with co-author Peter Frigaard collaborated with EVE on exchange of information of WECs breakwater applications. Co-author Peter Frigaard helped analyzing economical data.

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Abstract: The paper presents a comparison between two different wave energy technologies built into breakwaters: the Oscillating Water Column (OWC) and the Sea wave Slot cone Generator (SSG). The applications resulted to be equivalent and economically feasible with a cost of <0.33 €/kWh considering 10 years payback time. The comparison is based on two installations, one for each solution: OWC breakwater in Mutriku, Basque Country, Spain and SSG breakwater in Swakopmund, Namibia. The concepts differ from working principles: the first is based on wave to pneumatic energy conversion where air trapped in a chamber is compressed by the incoming waves to an air turbine, while the second is based on the overtopping principle, where potential energy of incoming waves is stored in a number of reservoirs at higher level than sea water level and converted into electricity by mean of low head hydro-turbines.

Cover letter

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27 June 2009

To whom it may concern:

Please find attached the revised paper titled **"Technological and Commercial Comparison of OWC and SSG Wave Energy Converters Built into Breakwaters"**, by Lucia Margheritini and Peter Frigaard.

The paper presents the first direct comparison between two wave energy converters: the SSG, multiple reservoirs overtopping device and the OWC, oscillating water column. The comparison is based on real data from application of the concepts into breakwaters in 2 different locations.

The data are a result of advanced research on wave energy applications. Relevant issues related the overall performances of the two devices are presented. Results include comparison of power production, installation issues and costs and final price of electricity.

I would like to have this manuscript published by Journal of Renewable Energy.
Sincerely,
Lucia Margheritini

Technological and Commercial Comparison of OWC and SSG Wave Energy Converters Built into Breakwaters

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Abstract

The paper presents a comparison between two different wave energy technologies built into breakwaters: the Oscillating Water Column (OWC) and the Sea wave Slot cone Generator (SSG). The applications resulted to be equivalent and economically feasible with a cost of <0.33 €/kWh considering 10 years payback time. The comparison is based on two installations, one for each solution: OWC breakwater in Mutriku, Basque Country, Spain and SSG breakwater in Swakopmund, Namibia. The concepts differ from working principles: the first is based on wave to pneumatic energy conversion where air trapped in a chamber is compressed by the incoming waves to an air turbine, while the second is based on the overtopping principle, where potential energy of incoming waves is stored in a number of reservoirs at higher level than sea water level and converted into electricity by mean of low head hydro-turbines.

1. Introduction

Most agree that our future energy supply may not rely on fossil fuels but instead on diversify renewable energy resources. In this scenario the wave energy (WE) sector is gaining consideration especially pushed by the enormous power resource estimate to be up to 10 TW (Engineering Committee on Oceanic Resources — Working Group on Wave Energy Conversion (2003). A total of 96 companies working on WE worldwide is listed by EMEC today (http://www.emec.org.uk/wave_energy_developers.asp); this number corresponds roughly to the same amount of patents and concepts. Relative precise classifications of devices can be made based either on the working principle: buoys, oscillating water columns (OWC) and overtopping devices; or on the orientation to the main wave direction to the converter: point absorbers, attenuators, terminators and on the location of installation: shoreline, near shore and offshore. Such a wide diversification has not yet encountered any unique convergence. Because of the very different environmental conditions at sea, it is expected that there will not be a unique winning technology; nevertheless, from the market prospective it is important to compare the devices in order to be able to match each technology with the most suitable location. This is no easy task as there is a lack of long term data on power production but also because the devices are so different on materials, power take-off systems and dimensions. The greater WE resource is offshore, where waves can propagate without encountering any dissipation phenomena; in the near future concrete contribution to the energy demand will come from offshore installations rather than shoreline. Nevertheless it seems quite reasonable, considering the present status of technology, to turn to shoreline devices in order to address the first issues raised by the market such as comparison of technologies and reliability.

The present paper aims at comparing two different technologies into breakwaters, namely the OWC and the SSG by direct evaluation of cost and performance. At first the two systems and working principles will be presented. Secondly the comparison will use to the real-case applications of the concepts into breakwaters.

2. Overview of technology

In the following paragraphs OWC and SSG technologies will be presented in order to have an overview of the status of development before discussing their application on breakwaters. The two concepts present more similarities than any other couple of random concepts among wave energy technologies:

- Massive reinforced concrete structure.
- Specially design turbines.
- Shore-line and breakwaters applications.

It is worth to remember that we are in front of two different stages of development: while several prototype scale OWCs have already been constructed and operated with varying degrees of success over the last 30 years, SSG prototypes haven´t yet been realized. Nevertheless the SSG concept undertook more than 6 years of extensive laboratory testing and different installations have been analyzed meticulously (Margheritini et al. 2007). Also, it appears the two technologies focused on different challenges in the optimization of the power take off: the core of the research for OWCs is about Wells turbine while for the SSG the main issue has been the optimization of geometrical parameters (Margheritini et al. 2009a).

2.1 OWC technology

2.1.1 OWC working principle

The OWC device comprises a partly submerged concrete structure, open below the water surface, inside which air is trapped above the water free surface (Fig. 1). The oscillating motion of the internal free surface produced by the incident waves makes the air to flow through a self-rectifying axial-flow Wells turbine that drives an electric generator (Fig. 2).

Favorable features of the Wells turbines are:

- Torque is not sensitive to the direction of the air flow.
- High blade to air-flow ratio.
- Fairly good peak efficiency (0.7 for a full-sized turbine).
- Relatively cheap to construct.

The weak points of the Wells turbine are:

- Low or even negative torque at (relatively) small flow rates.
- Relatively large diameter for its power (2.6 m for the counter-rotating 500 kW turbine of Islay II Scotland, 2000). Drop (possibly sharp drop) in power output due to aerodynamic losses at flow rates exceeding the stall-free critical value. Recent research work indicates that this can be improved by a suitable geometry of the rotor blades (non-conventional, properly designed blade profiles). By using a variable-pitch turbine, blade stalling can be avoided or greatly reduced over a large range of flow rates, therefore allowing a substantially better time-averaged aerodynamic performance (of course this is paid for in terms of a more complex, more expensive and probably less reliable machine).
- Noise

Several versions of the Wells turbine have been object of considerable theoretical and/or experimental R&D, especially in Europe (UK, Portugal and Ireland), Japan, India and China.

The energy conversion chain consists of the following elements:

- wave to pneumatic chamber, 42%-59% efficiency.
- Wells turbine, 65% efficiency (average).
- Electrical generator and electrical equipment, 91% efficiency.

This gives an overall expected wave-to wire efficiency of 25% - 35%.
Examples of OWC prototypes are: in Norway (in Toftestallen, near Bergen, 1985), Japan (Sakata, 1990), India (Vizhinjam, near Trivandrum, Kerala state, 1990), Vizhinjam Fisheries Harbour, near Trivandrum (India, 1991), Portugal (Pico, Azores, 1999), UK (the LIMPET plant in Islay island, Scotland, 2000). OWCs can also be installed offshore as floating devices (Mighty Whale built in Japan). The integration of the plant structure into a breakwater has been realized already in the harbour of Sakata, Japan in 1990 and in Mutriku, Basque Country, Spain 2009. In none of these cases the overall efficiency exceeded 10%. This is understood to be largely due to poor original wave climate data and the consequential mismatch between the collector and turbine (I. Webb et al. 2005).

2.1.2 Geometrical optimization of OWC WE converter

The structure geometry is designed in order to obtain resonance in the pneumatic chamber. The selected geometric form of the collector chamber is considered to depend on the wave climate in each location. The geometrical parameters to be optimized in the construction of the OWC structure are (Fig. 3):

1) A: length of the pneumatic chamber.
2) B: opening to the pneumatic chamber.
3) C: immersion of the front wall.

The natural period of oscillation of practical OWC devices lies below the practical range of periods containing the most significant wave energy. Wave amplification away from this resonant period will thus be reduced.
The submergence depth, C, is related to the mass (and added mass) of the moving water column and determines the natural frequency of oscillation. For practical submergence depths of around 5m, the natural period of oscillation is approximately 5 seconds. Adding a horizontal section (A) to the chamber will increase the effective mass thus increasing the natural period. Sloping chamber results in a greater capture efficiency at shallow shore-line where energy is more concentrated into horizontal motion. A downside of having the sloping face is that water surface pitching rather than heave (the only part of the energy model that produces useful power) with consequent loss of captured power. Broaching is also an undesirable phenomenon and occurs when the water level falls below the level of entry lip and a direct air passage is opened between the working chamber and the atmosphere. Broaching causes loss of power take off and a sudden pressure change in the collector. The wave height at which this broaching occurs is a function of the lip penetration at still water, the state of the tide and the dynamic characteristics of the water column. Given that the selection of a lower lip (B), whilst increasing the natural period of oscillation of the OWC, will also be cutting out some of the incoming wave energy and reflecting it back, the highest lip level should be sought. The depth of wave troughs below still water level in the steep waves expected near-shore is around 1⁄3 the wave height.
The geometry and volume of the plenum should be selected in combination with the turbine to optimize power capture. A smaller air plenum will be subject to higher pressure and greater flow both of which must be considered in relation to the turbine characteristics. For reference on the hydrodynamic-aerodynamic optimization of OWCs see J. Weber, 2001.
The wave-to-pneumatic energy conversion may be studied theoretically/numerically, or by testing a physical model in a wave basin or wave flume. Numerical modeling is to be applied in the first stages of the plant design. One code, based on the boundary element method is named AQUADYN and was

developed at École Centrale de Nantes (ECN), in France, for naval and ocean engineering applications. It was modified, jointly at ECN and IST, in order to make it applicable to OWCs (A. Brito-Melo et. al 2001). The main limitations lie in it being unable to account for losses in water due to real (viscous) fluid effects (large-eddy turbulence) and not being capable to model accurately large amplitude water oscillations (nonlinear waves). For these reasons, model tests should be carried out in wave basin when the final geometry of the plant is already well established. They should provide information for the specification and design of the air turbine.

2.1.3 Design wave loads on OWC structure

The study of impacting loads on caisson structures has been the subject of recent research through the PROVERBS program (http://www-public.tu-bs.de:8080/~i5102401/proverbs.html). Caisson structures and their sub-elements have previously been designed for high impact pressures treated as quasi-static loading. However, the nature of the impacting load events is that they typically act over less than 100msec. An approach that considers the natural period of the element being designed in relation to the impact duration allows a much smaller quasi-static force to be designed for. Wave loading can be determined from diffraction theory for unusual geometries or from breakwater theory for reasonably uniform-fronted caisson or shoreline OWCs. It is preferable to site an OWC in an environment where impacting waves are less prevalent (K. Thiruvenkatasamy et. al 2005). The adoption of a sloping face to the OWC also limits the potential for a plunging breaker impacting the OWC. If plunging breakers are not present this allows the front face of the chamber to be designed for normal reflecting wave pressures such as those given by Goda, 1985.
In practical near-shore water depths, the design wave height is limited by the water depth. A maximum wave height of 0.78 x water depth is conventionally used (Sarpkaya , T. and Isaacson, M. 1981). The concept of a return period for 0.00% design thus has little meaning as the depth will govern for waves of the minimum return period appropriate for a design life of typically 25 years.

2.1.4 Influence of tide on OWC performance

In general, the tuning of oscillating water columns is more complicated when the device is situated in an area with a large tidal range. This is due to the changing natural frequency of the chamber and the requirement to alter the damping of the system.

2.1.5 Parameters influencing the cost of OWC device

The most important parameters having an effect on the investment cost of the OWC are:

- Local wave and tide climate (determines the geometry and size of the pneumatic chamber)
- Design wave height (determines the size of the structure)
- Water depth (determines the construction method and overall size of the caisson)

The capital cost of an OWC could be considered proportional to the maximum design water depth squared since the device has to become proportionally wider or heavier to resist wave loading whilst also getting taller. If the curve of available power versus water depth is combined with the average OWC capital cost, a further trend curve can be prepared as shown in Fig. This clearly shows that it is advantageous to site the device in around 10m water depth (Fig. 4).
Taking a typical water depth of 10m, the sensitivity of device cost to tidal amplitude could be assessed to be proportional to: Cost≈ $(\text{Depth} + \text{Tidal Range}^2)^2$.
If 4m tidal range is assumed compared to a location without tidal influence, a cost increase of 44% might occur. Hence it is important to concentrate on sites with the smallest tides (I. Webb et al. 2005).
Potential improvements could be achieved by considering a sloping face to avoid plunging breakers: this should be taken forward in future designs as it reduces the capital cost of the structure.

2.2 SSG technology

2.2.1 SSG working principle

The Sea-wave Slot-cone Generator (SSG) is a patented wave energy converter of the overtopping type (Fig. 5): incoming waves overtop a multiple level structure and water is temporarily stored in reservoirs at a higher level than sea water level (s.w.l.) offering the chance to exploit the potential energy in the stored water by mean of Multi Stage Turbine (MST) (Fig. 6).
The MST turbine will be 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. Successful testing has been carried out with a 1:4 scale prototype in the water laboratory at NTNU (Norwegian University of Science and Technology), Trondheim but the concept is still under optimization and alternatively low head hydro-turbines can be utilized of the same kind used in the Wave Dragon Knapp W. (2005).
The energy conversion chain consists of the following elements:

- wave to crests (Rcj, j=1, 2…n, n=number of reservoirs), 40%
- crests to reservoirs 75%,
- low head water turbines, 98% efficiency.
- Electrical generator and electrical equipment, 95% efficiency.

The losses on the "wave to crest" conversion are related to having 3 levels averaging al the wave heights in a wave climate: for example, waves higher than the second crest but not height enough to reach the third crest will lose part of their potential energy as they will be stored at a lower level than the wave height. In the second step of the energy conversion the losses are related to the spill out water from the limited capacity of the reservoirs and the further loss of potential energy from the crest to the water level in the reservoir.
The overall expected wave-to wire efficiency of 25%-35%.
The pilot project was meant to be built in the Island of Kvitsøy, Stavanger, Norway (Margheritini et al. 2009b), but at the very last phase, environmental concern was raised due to the required installation works that would see permanent alteration of the cliff at location. Consequently, public acceptance passed from being very positive to uncertain. Together with the motivation of challenging wave climate at location (19 kW/m, H_{max} 15 m), the non will of being part of a project with a not satisfying public acceptance led to the closing of the EU project in early 2008.
The integration of the SSG on breakwater has been taken into account for the renovation of the harbor in Plentzia, Basque Country, Spain and for Hanstholm, North Jylland, Denmark. Also, implementation of the SSG in breakwaters in Swakopmund, Namibia and Sines, Portugal has been deeply analyzed in order to identify issues related to performance, construction and installation (Oever 2008; Margheritini and Kofoed 2008a).

2.2.2 Geometrical optimization of SSG WE converter

The structure geometry is designed in order to maximize the overtopping flow into the reservoirs and depends on the wave climate at location. The geometrical parameters to be optimized in the construction of the SSG structure are (Fig. 7):

1) Rc1, 2 and 3: crest levels.
2) Inclination angle and length of the fronts.
3) HD1 and HD2: openings to the reservoirs.

Fronts and run up ramp angles are between 30° to 40° as it has been demonstrated that this increases the overtopping to the reservoirs (Le Mèhautè et al. 1968, and Kofoed 2002). The crest levels are defined through an iteration process that leads to maximization of hydraulic efficiency defined for the wave climate at location as:

$$\eta_{hyd} = \frac{\sum_{j=1}^{n} \rho g q_j R_{cj}}{\frac{\rho g^2}{64\pi} H_s^2 T_E} = \frac{P_{crest}}{P_{wave}} \qquad \text{.....(1)}$$

Where ρ=1020 kg/m^3, g = gravity acceleration, H_S is the significant wave height and T_E is the energy period = m-1/m0, where mn is the n-th moment of the wave spectrum. $R_{c,j}$ is the crest level of the respective reservoir and $q_{ov,j}$ is the total overtopping flow rate for the j-reservoir. The expression available now to calculate qj in Eq. 1 is the integration of the derivative overtopping discharge with respect to the vertical distance z (Kofoed 2002):

$$\frac{dq/dz}{\sqrt{gH_S}} = Ae^{B\frac{z}{H_S}} e^{C\frac{R_{c,1}}{H_S}} \qquad (2)$$

The coefficients A, B and C are fitted from laboratory tests, q is the average overtopping discharge per width [m^3/s/m], z is the vertical distance from the s.w.l., and the others parameters have been defined above. Laboratory tests are indeed important in the optimization process as at the present time they are the only method to define the length of the fronts and the HD parameters. Nevertheless, gain in efficiency after laboratory optimization is estimated to be of 30% (Margheritini and Kofoed 2008b), suggesting that model tests for optimization of the geometry are not compulsory if the commissioner is ready to compromise with such a level of uncertainties related to device performance. Alternatively to laboratory tests (or complementary to), the simulation program WOPSim 3.01 for overtopping of WECs could be utilized (Meinert 2008). The main inputs for the simulation program are geometry, wave and tide conditions and turbine strategy, characteristics and control. The turbine characteristics are chosen in order to handle overtopping flows resulting from the most probable wave conditions. 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.

2.2.3 Design wave loads on SSG structure

The influence of vertical or sloping coastal structures on the breaking wave phenomena is a very complicated problem. Wave loads and pressures on the SSG structure have been analyzed by mean of laboratory tests in different set ups but always on the same geometry and for the case of the SSG Pilot in the island of Kvitsøy. In that occasion the best reproduction of the surrounding bathymetry has been realized and the model device has been positioned on top of it equipped with pressure transducers once and with load cells the other time (Vicinanza et al. 2008).

The combined analysis of video-camera and pressures records made it possible to identify surging waves, characterized by a rapid rise of the wave along the three sloping front caisson plates (no breaking waves). A quasi-static loading time history is recognizable over all the front side plates and the pressure is almost hydrostatic ($p \approx \rho_w gHm$). The pressure values for 1/250 corresponds to non-exceedance levels of about 99.7%. The analysis results indicate that Weibull is a more suitable CDF to describe the probability distribution of pressures.

The SSG innovative structure cannot fit any standard design method, however in order to check the general tendency of these test results, the experimental pressures were compared with the design criteria suggested by the CEM (2000) for predicting pressure distribution on sloping top structures. Pressure measurements compared with the prediction method made for caisson breakwaters with sloping top showed 20–50% higher wave pressures than the Takahashi et al. (1994) design equation. One of the reasons is that the SSG model was fixed rigidly instead the design method was tested using sliding experiments. In fact the Takahashi et al. model caissons were fabricated from synthetic acrylic plates and had a bottom comprised of a concrete slab for simulating the friction factor. The fact that the tests show 50% higher wave pressures than the 'best' available design equation, suggests that design wave pressures is a topic needing careful attention, and not all experience from designing traditional maritime structures are usable. Laboratory tests are needed previous construction.

2.2.4 Influence of tide on SSG performance

The results of the present section are based on WOPSim 3.01 simulations (Meinert 2008) for specific locations, namely Sines, Portugal and Swakopmund at 6 and 11 m water depth, Namibia (E. ten Oever 2008; Margheritini and Kofoed 2008a). The efficiency decreases with increasing the tidal variation for a selected geometry.

If the presence of tide is neglected in the design of the SSG geometry, 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%)(Fig. 8). Nevertheless it is possible to take into account the tidal range into the design of the SSG device and therefore occur in minor losses especially for bigger tidal ranges. The optimization of structure for tide will result in different crest levels and no additional costs compared to the case with no tide (Fig. 9). For larger tidal ranges (>8 m.), it is advantageous to consider adding a reservoir in the design of the device.

2.2.5 Parameters influencing the cost of SSG device

The most important parameters having an effect on the investment cost of the SSG are:

- Local wave and tide climate (determines the number and size of reservoirs, in average passing from three to four reservoirs will see an increase of construction cost of 4%,)
- Design wave height (determines ballast and size of the structure)
- Water depth (determines the construction method and overall size of the caisson)

For large tidal ranges it is advisable to check the feasibility of an extra reservoir as the performance benefit of it. From Fig. 10, where we have 4 different tidal ranges (no tide, T1=1.6m, T3= 4.8 m and T6= 9.6 m) emerges that in average the gain is 5 points % passing from 2 to 3 reservoirs, 3 points % passing from 3 to 4 reservoirs and 2 points % passing from 4 to 5 reservoirs. But for the same wave condition, the gain in percentage is bigger for bigger tidal ranges, meaning that it is beneficial to add a reservoir in case of tide. For example, passing from 2 to 3 reservoirs in the case of Sines gives 17.6% gain for no tide and 44.8% gain for 9.6 m tidal range.

The coast of the structure largely depends on the size i.e. the amount of concrete used. Some improvements could be taken into account for future installations. Due to the geometry and the nature of the SSG, overtopping could be less than calculated with normal overtopping criteria based on actual crest level. This indicates that the structure can be potentially lower than a conventional structure which will reduce the costs. In the same way, with the sloping face of the device the wave pressures can be used in advantage of stability of the structure reducing size and costs.

3. Integration of OWC and SSG on breakwaters

The integration of wave energy converters on breakwaters presents some advantages:

- sharing of construction costs.
- Access and therefore operation and maintenance are easier compare to an offshore situation.
- Production of clean energy.
- Recirculation of the water inside the harbor i.e. improvement of water quality (only in case of SSG) as the outlet of the turbines would be in the rear part of the breakwater.
- Potential lower visual impact as a consequence of a lower crest level (only in the SSG case).

With regard to reflection performance of the integrated structure, preliminary comparison results from OWC and SSG laboratory tests show that in both cases we are in presence of highly reflective structures (Zanuttigh et al. 2009) with reflection coefficient never lower than 40% and that can rise up to 90%. It is then a design issue to construct a proper toe protection layer to avoid scour holes if not a berm to reduce the reflection.

Following two case studies will be presented, one for each technology. In the case of the OWC the breakwater installation in the small harbor of Mutriku, Basque Country, Spain will be analyzed presenting the construction, installation and costs of the application. For the SSG, a study on the possible implementation of the device in the Swakopmund marina in Namibia is reported. Also in this case construction, installation and costs will be presented as result of an extensive feasibility study carried out by the developing company WaveEnergy AS. The additional costs are defined as the costs related to the construction and installation of the WECs on breakwater that would not occur in case of a traditional harbor protection. Examples of extra costs are: electrical equipment, turbines, extra concrete etc…

3.1 OWC technology in Mutriku breakwater

The data presented in the following sections have been made available by Ente Basco de la energia (EVE) during the construction phase of the breakwater in Mutriku. The Basque Country, has a target for wave energy that foreseen the installation of 5 MW by 2010. Mutriku has 4000 inhabitants and is situated on the coast, east from Bilbao, Basque Country, Spain. The protection of the port was provided by three small breakwaters, two of which separated by an entrance to the inner port. The implantation of the new breakwater 500 m long will protect the port from the waves whose most frequent direction is N and NE (Fig. 11). The body of the breakwater is constituted by a structure with sloping sides (slope 3/2), made up of rock layers of various sizes: the outer protecting layer of the rubble mound part of the breakwater is made, on the side facing the sea, of blocks of natural stone of 15 and 25 t. In head of the breakwater, of conical shape, the protecting layer is made of blocks of natural stone of 45 t. Water depth at location is 6m with rapid change to 15 m at the head of the breakwater. Wave climate at location has been estimated to be 6 kW/m.

The new breakwater will also contribute to achieve the 5MW wave energy installation target as the Basque Government, Transport Department and Public Works (BTDG) decided to implement the OWC technology in the protection for a total length of 100 m in front of the rouble mound blocks (Fig. 12 and Table 1).

The active section is made up of 16 OWC caisson connected at the turbines outlets levels 4 by 4. Due to the limited water depth the installation was challenging not allowing floating the caisson into place because of the limited draft. Therefore each caisson has been realized as a 3D puzzle of 16 pre-fabricate slides resulted by sectioning the caisson horizontally in 16 points (Fig 13). The slides were anchored two by two, gaps filled in with gravels and concrete. The weight of 1 slide is 45 t. The front and the interior are water proved in order to host the electrical equipment. Total height is 16.5 m above s.w.l.. The total installed capacity is 2.96 kW/m. The cost of OWC-breakwater is 60000 €/m.

The turbine part was taken care by WAVEGEN and Voit SIEMENS. Basque government paid additional costs for civil works while Ente Vasco de la Energia (EVE) paid cost for laboratory tests (commissioned to INHA) and facilities. The construction, design and installation were taken care by ASMATU.

3.2 SSG technology in Swakopmund breakwater

The data presented in the following sections are result from the case study commissioned by WAVEenergy AS to Delta Marine Consultants about SSG-breakwater application. The study included comparisons of the SSG solution with traditional harbor protections such as caissons and rubble mound breakwaters from the construction, installation and economic point of view. The results are estimated to be correct with a degree of uncertainty of ±15%. Swakopmund is a small marina in Namibia which has to be constructed as part of resort development. The port is located on a gentle foreshore and the main breakwater is located at a limited water depth of approximately 6m. Wave climate at location is 15 kW/m. The new breakwater is 240 m long, perpendicular to the dominant wave direction South-South West.

The capturing crest levels are: Rc1=1m, Rc2=2.3 m and Rc3=3.16 m. Installed capacity is 12.5 kW/m. The crest level is at 6m and was determined by the operational overtopping conditions (conditions with operability of less than 0.1% exceedence). For operational conditions overtopping rates are kept below 10l/m/s. To allow some wave overtopping during design conditions, the breakwater does not have mooring facilities at the leeside. (Fig. 14 and Table 2). The front ramp is cut vertically at -3.8 m below sea water level for construction reasons.

On top, roadway for access is included in the design with 1m high protection walls. The first reservoir will be connected over the entire breakwater creating one large basin averaging the wave overtopping over the length of the breakwater; this appears to be advantageous also for the turbines operation. The second and third reservoirs have a section of the 10 m wide.

The total concrete quantity for a concrete caisson is higher for the SSG-breakwater than for a conventional caisson (because of floor slabs). Other differences are the higher center of gravity and eccentric location of the center of gravity that have consequences on the draft of the floating caisson into position. Indeed the draft revealed to be critical for such shallow water so that the construction will perform two separated parts (Fig. 15): the lower part reaches to the first slot and consists of elements of 5 m wide; the upper part is 10 m wide. The upper part fixed to the lower by the overhang of the upper part over the lower, to prevent uplift of the upper part it is secured by tension anchors in the walls. Pressure relieve openings under the ceilings of the reservoirs are part of the design. These openings will facilitate the inflow of water and reduce the extreme wave pressure.

The parts will be transported into position by the custom made placement crane (Fig 16). The cost of the SSG-breakwater including turbines and generators has been estimated to be of 76900 €/m while the price for a rubble mound breakwater in the same location has been estimated to be 28100 €/m.

For a 6 m water depth breakwater it is unlikely that a caisson type kind of harbor protection would be used. If considering the construction of the breakwater at 11m water depth instead, the capturing crest levels are: Rc1=1m, Rc2=2.5 m and Rc3=4 m. Installed capacity is 12.8kW/m. The crest level is at 8 m. with protected roadway. The structure has to be a prefabricated caisson to be loaded into place as the dimensions are too large to construct this structure in a similar way as the -6m caisson. The cost of the SSG-breakwater including turbines and generators has estimate to be 150700 €/m while the price for a traditional caisson at Swakopmund location is 124500 €/m.

4. Cost comparison

The comparison will be presented in terms of additional costs and payback time derived by the implementation of OWC and SSG technologies in three different breakwater locations (Table 3).

In the case of the OWC-breakwater in Mutriku, the additional cost compared to a rubble mound solution have been estimated to be 20000 €/m. Considering a production of 6000 kWh/m/y and a payback time of 10 years that seems reasonable taking into account the life time of harbor protection constructions, the cost results of 0.33 €/kWh.
In the case the SSG-breakwater in Swakopmund, the additional cost compared to a rubble mound solution have been estimated to be 48800 €/m. Considering a power production of 19000 kWh/m/y (Margheritini and Kofoed 2008b) and a payback time of 10 years, the cost is 0.27 €/kWh.
Of course for shallow water depths the traditional solution for harbor protection would converge on a rubble mound breakwater which is cheaper. Nevertheless if we compare a WEC installation on breakwater at a greater water depth, where traditionally a caisson type of breakwater would be adopted for harbor protection, the additional costs are reduced. This can be seen in the case of SSG-breakwater in Swakopmund at 11m water depth. The additional cost is estimated to be 26200 €/m. Considering a power production of 18000 kWh/m/y) (Margheritini and Kofoed 2008b) and again a payback time of 10 years, the cost results of 0.16 €/kWh.
It is to be noted that the power production in Swakopmund at 11 m water depth is lower than the one at 6 m water depth. This is because for construction reasons the front ramp had to be cut at -3.8 m. below the surface. This is penalizing less the power production in shallow water (Margheritini and Kofoed 2008b).

5. Conclusions

OWC and SSG working principles have been described as well as the parameters influencing the design. A comparison of OWC-breakwaters and SSG-breakwaters based on reliable data based on construction, installation and performance studies and predictions have been made. The main findings are summarized as follow:

- WECs built into breakwaters are economically feasible.
- Deep water applications (caisson type) have better payback time.
- Seen from an economical point of view OWC are comparable to SSG with respect to the present knowledge. The demonstrable differences are within the level of uncertainties.
- WECs must be included into the breakwater design since the early stage of the project in order to be able to fully benefit of the acquired knowledge on performance, construction and installation.
- The breakwater mounted WECs may offer some additional performance to the protection structure: clean energy production; recirculation of water in the harbor and lower visual impact in the case of the SSG solution.

It has also been concluded that the wave loading issue has relevant impact in the capital cost of both the structures. Opposite to traditional sea defense structures wave energy structures are designed in a way so they face and challenge the sea as much as possible. The design of near-shore caisson structures requires a detailed understanding of the wave loading to which they are subjected. While the OWC geometry fits into standard design methods, SSG geometry is atypical and wave loading on the structure can not yet be predicted with sufficient accuracy. For OWC and SSG shore-line and breakwaters applications the tide issue is relevant both to the power production and to the cost of the devices. In the SSG case the tidal range can be taken into account in the optimization of the geometry resulting in a reduction of negative effect that either way the tide would have in the power production.

Acknowledgments

This work has only been possible with the support of Monika Bakke from WaveEnergy AS. The impressive degree of development preceding the installation of the device and the amount of money spent to acquire valuable data will surely have a payback in terms of success of future projects. The authors would like to thank the Ente Vasco per la Energia (EVE), in particular Yago Torre-Enciso and José Ramón López for the amazing hospitality during our visit and the collaboration that led to the results presented in this paper.

References

Engineering Committee on Oceanic Resources — Working Group on Wave Energy Conversion (2003), John Brooke, ed., Wave Energy Conversion, Elsevier, pp. 7, ISBN 0080442129;

(http://www.emec.org.uk/wave_energy_developers.asp)

L. Margheritini, D. Vicinanza, and J.P. Kofoed. Hydraulic characteristics of seawave slot-cone generator pilot plant in Kvitsøy (Norway). Proceedings of the 7th European Wave and Tidal Energy Conference : Porto, Portugal, 11-14 September 2007. Porto : European Ocean Energy Association, 2007. 8 s.

I. Webb, C. Seaman and G. Jackson. Marine Energy Challenge. Oscillating Water Column Wave Energy Converter Evaluation Report. The Carbon trust 2005 - Submission Reference 115214-00

Weber J.W. & Thomas G.P. (2001). An investigation into the importance of the air chamber design of an Oscillating Water Column wave energy device. Proceedings of International Society of Offshore Engineers, ISOPE 2001, Stavanger, Norway.

A. Brito-Melo, T. Hofmann and A.J.N.A. Sarmento. International Journal of Offshore and Polar Engineering, Vol. 11, No. 2, June 2001 (ISSN 1053-5381)

PROVERBS http://www-public.tu-bs.de:8080/~i5102401/proverbs.html

Goda, Y. (1985), Random Seas and Design of Maritime Structures, University of Tokyo Press, Tokyo, Japan.

Sarpkaya , T. and Isaacson, M.; Mechanics of Wave Forces on Offshore Structures, van Nostrand Reinhold, 1981

K. Thiruvenkatasamy, S. Neelamani, and Michio Sato. Nonbreaking Wave Forces on Multiresonant Oscillating Water Column Wave Power Caisson Breakwater. Journal of Waterway, Port, Coastal and Ocean Engineering, Vol. 131, No. 2, March/April 2005, pp. 77-84.

Knapp W. (2005). Water Turbines for Overtopping Wave Energy Converters CA-OE workshop on Component Technology and Power Take-off. Uppsala, Sweden.

L. Margheritini; L. Victor; J.P. Kofoed; P. Troch. Geometrical Optimization for Improved Power Capture of Multi-level Overtopping Based Wave Energy Converters. / Proceedings of International Society of Offshore Engineers: ISOPE-2009. 2009a.

L. Margheritini; D. Vicinanza; P. Frigaard. SSG Wave Energy Converter : Design, Reliability and Hydraulic Performance of an Innovative Overtopping Device. / Renewable Energy. 2009a ; vol. 34, nr. 5, 01.11.2008. s. 1371–1380

E. ten Oever. The SSG- breakwater concept. Delta Marine Consultans. Report number 952350-rap-u-005
L. Margheritini; J.P. Kofoed. Aalborg . Parametrical numerical study on Breakwater SSG application. Aalborg University. Department of Civil Engineering, 2008a. (DCE Contract Reports; 47).

Mansard and Funke, 1980. E. Mansard and E. Funke, The measurement of incident and reflected spectra using a least squares method. In: Proceedings, 17th International Conference on Coastal Engineering, Sydney Vol. 1, pp. 154–172.

Kofoed J. P. (2002). Wave Overtopping of Marine Structures – Utilization of Wave Energy. Ph. D. Thesis, defended January 17, 2003 at Aalborg University. Hydraulics & Coastal Engineering Laboratory, Department of Civil Engineering, Aalborg University.

Meinert P. (2008), update of Meinert P. (2006). User Manual for SSG Power Simulation 2 DCE Technical Report No. 44, ISSN 1603-9874, Department of Civil Engineering, Aalborg University.

L. Margheritini; J.P. Kofoed. Aalborg. Case study on Breakwater SSG application. Aalborg University. Department of Civil Engineering, 2008b. (DCE Contract Reports; 44).

D. Vicinanza; L. Margheritini ; P. Contestabile; J.P. Kofoed ; P. Frigaard,. Seawave Slot-cone Generator : an Innovative Caisson Breakwaters for Energy Production. / 2009. 12 s. Konferencen: The International Conference on Coastal Engineering, nr. 31, Hamburg, Tyskland, 31. august 2008 - 5. september 2008.

CEM (Coastal Engineering Manual), 2002. U.S. Army Corps of Engineers. 2002. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).

Takahashi, S., S., Hosoyamada, and S. Yamamoto. 1994. Hydrodynamic characteristics of sloping top caissions. Proceedings of International Conference on Hydro-Technical Engineering for Port and Harbour Construction, Port and Harbour Research Institute, Japan, vol 1, pp. 733–746.

Zanuttigh, Barbara ; Margheritini, Lucia ; Martinelli, Luca. 2009. Analysis of Wave Reflection from Wave Energy Converters Installed as Breakwaters in Harbours. / Proceedings of the 8th European Wave and Tidal Energy Conference: Uppsala, Sweden l. European Ocean Energy Association, 2009

Figure

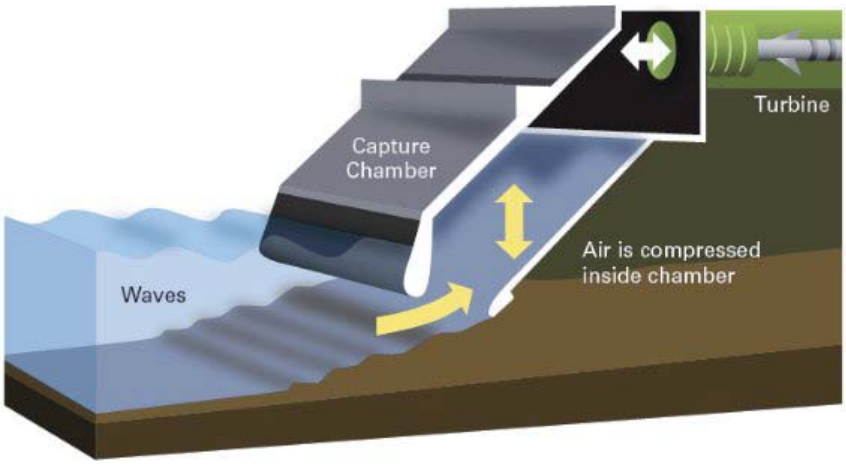


Figure 1. OWC working principle.

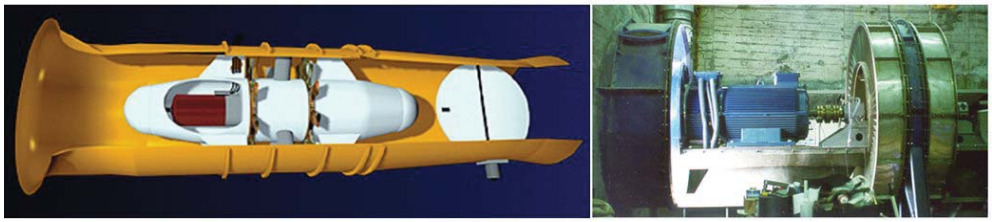


Figure 2. Wells turbine with symmetric blades (on the right: Pico plant, Azores, PT).

Figure

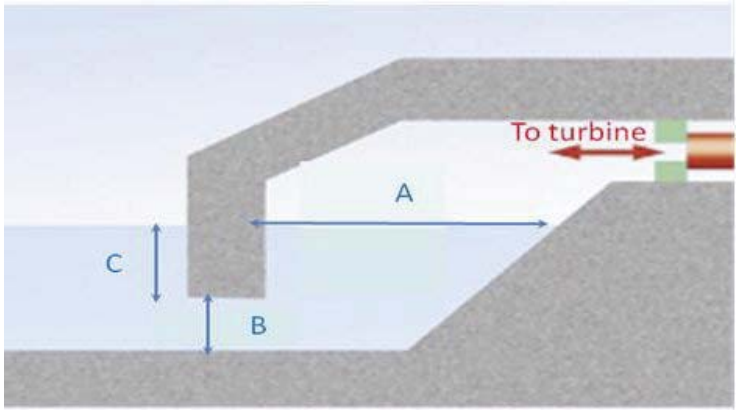


Figure 3. Definition sketch OWC´s main geometrical parameters.

Figure

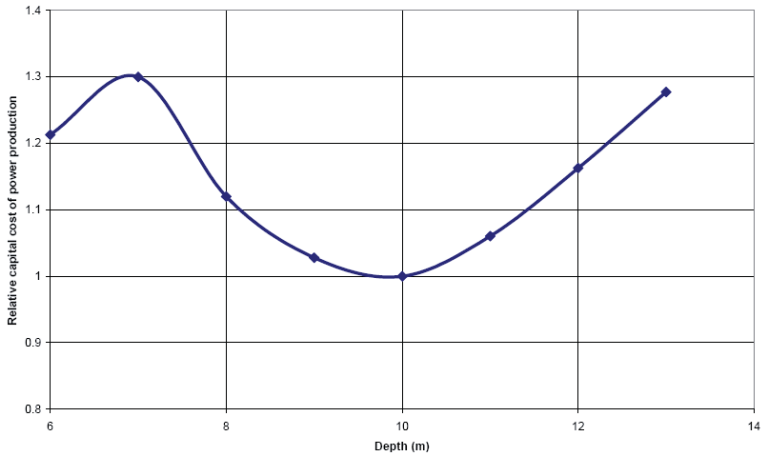


Figure 4. Cost of power production for OWC depending on water depth (I. Webb et al. 2005).

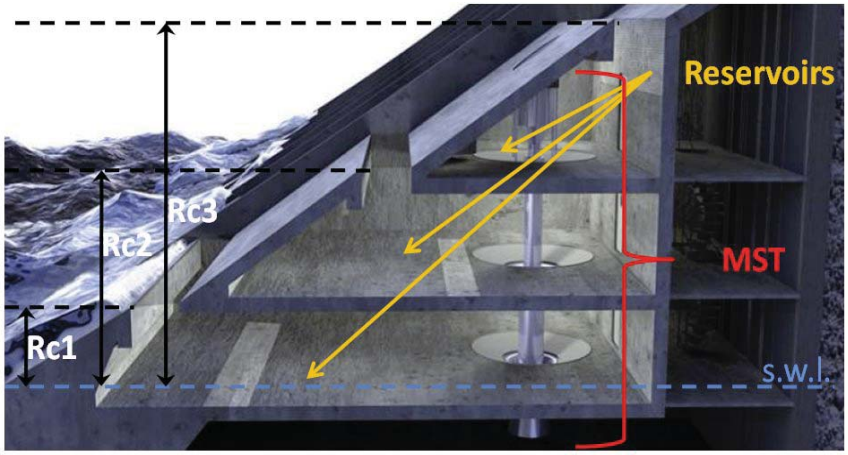


Figure 5. SSG working principle.

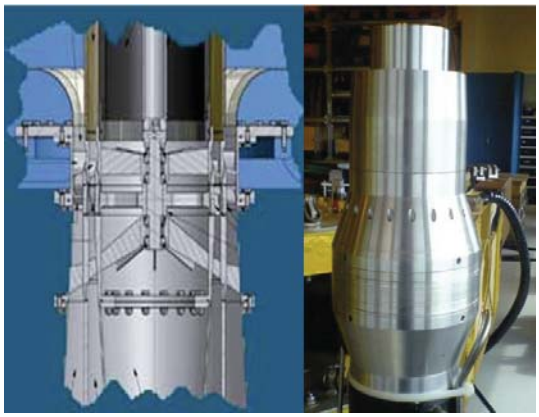


Figure 6. MST turbine, (on the right in scale 1:4).

Figure

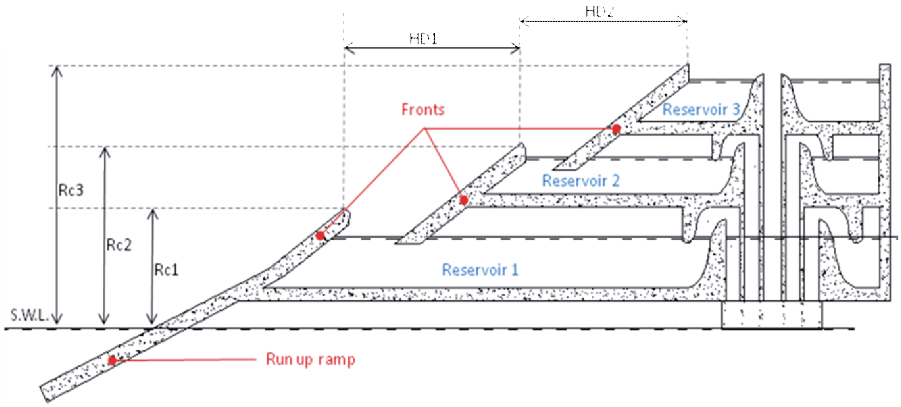


Figure 7. Definition sketch SSG´s main geometrical parameters.

Figure

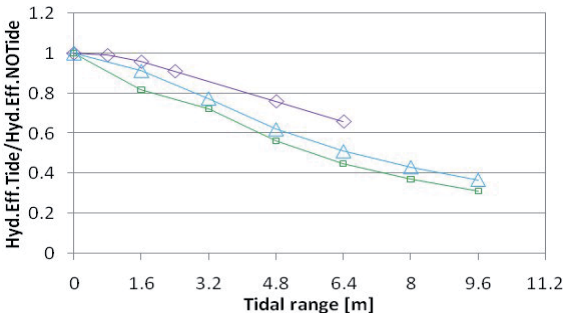


Figure 8. Influence of tide on the performance of the SSG device for different wave conditions.

Figure

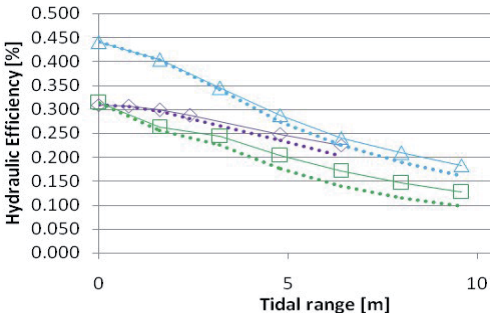


Figure 9. Improvement in performance of the SSG device when taking into account the tidal range in the crest levels design (comparison dotted lines to continuous lines).

Figure

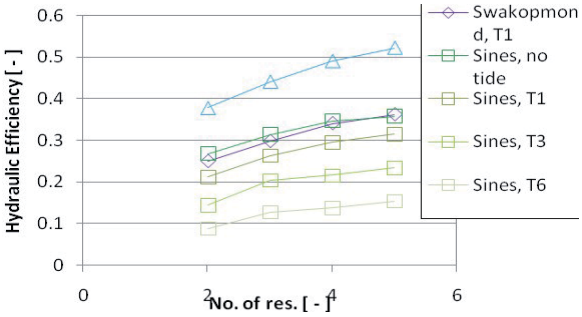


Figure 10. Performance of the SSG device for different number of reservoirs and wave and tide conditions.

Figure 11. Mutriku OWC breakwater.

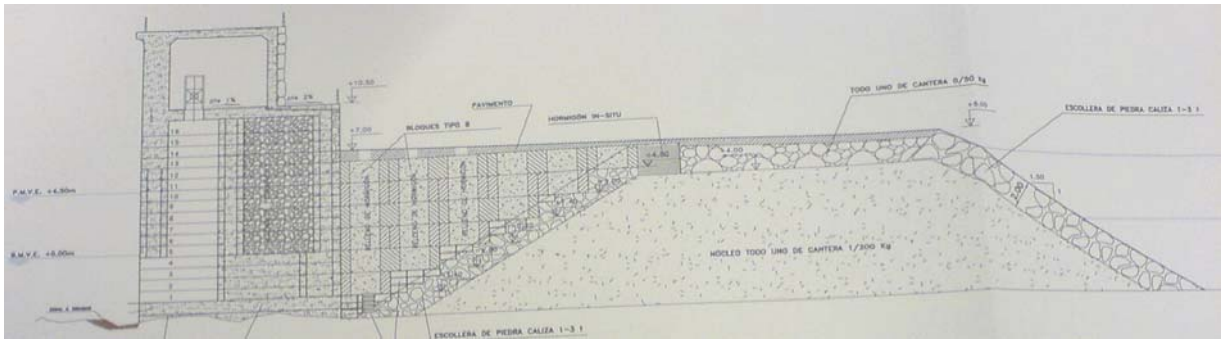


Figure 12. Section of Mutriku breakwater installed in front of the old rubble mound breakwater.

Figure 13. Installation of the OWC caissons composed by 16 pre-fabricate slides, Mutriku, 6 m water depth.

Figure 14. Swakopmund SSG breakwater.

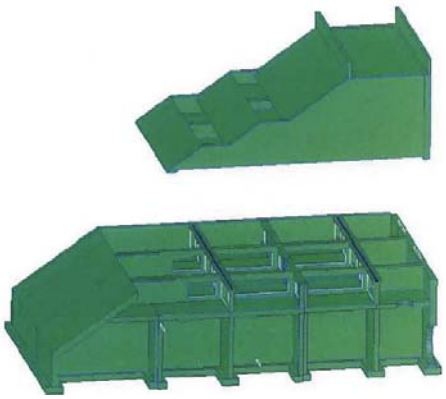


Figure 15. Upper and lower part of SSG breakwater caisson in Swakopmund, 6 m water depth (E.ten Oever 2008).

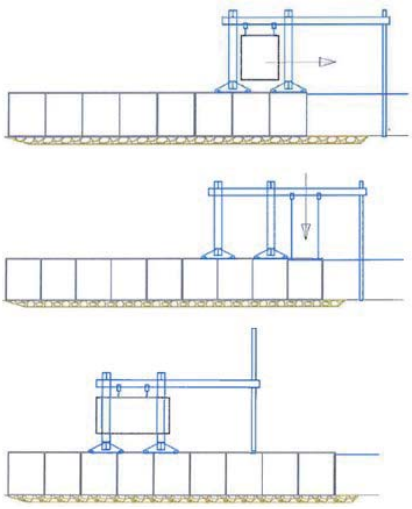


Figure 16. Installation of upper and lower parts of the SSG caisson in Swakopmund, 6 m water depth (E. ten Oever 2008).

Table 1. Main characteristics of the OWC installation in Mutriku.

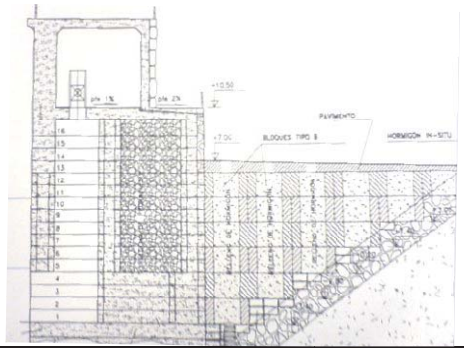
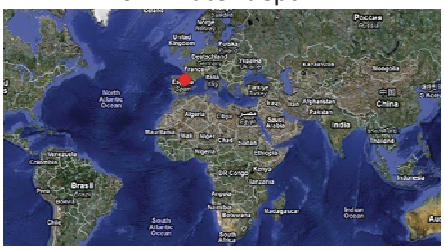
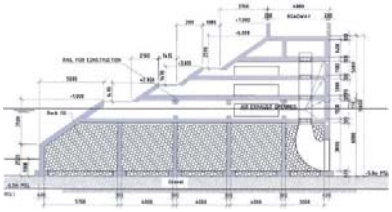
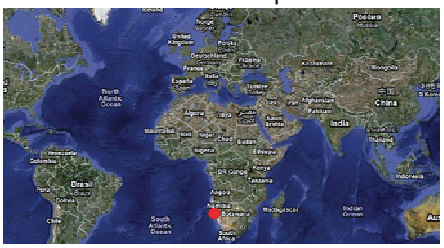
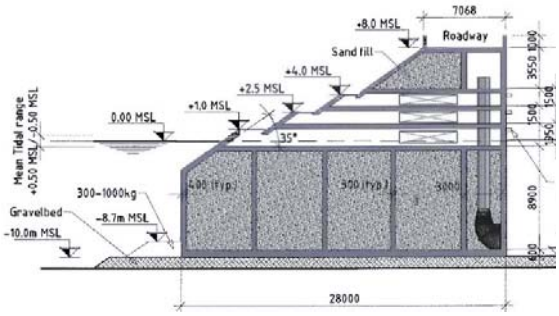
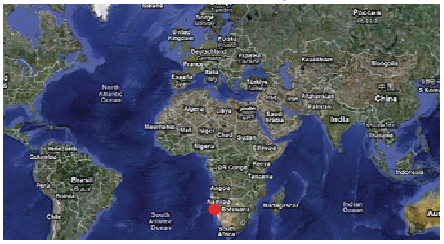
DEVICE: OWC in Mutriku	
Extraction technology	Oscillating water column
Installed capacity	2.96 kW/m
Wave climate at location	6 kW/m
Water depth at the installation site	6 m and 15 m at the head of the breakwater.
Design wave height	7 m, considering the head of the breakwater.
Tidal range	-

Table 1. Main characteristics of the SSG installation in Swakopmun.

DEVICE: SSG in Swakopmund	
Extraction technology	Overtopping WEC
Installed capacity	12.5 kW/m
Wave climate at location	15 kW/m
Water depth at the installation site	6 m
Design wave height	5.7 m
Tidal range	1 m.

Table

Table 1. Cost of electricity calculated for 10 years payback time.

Technology	Location	Cost of electricity
	Mutriku, Spain, 6 m water depth. 	0.33 €/kWh
	Swakopmund, Namibia, 6 m water depth. 	0.27 €/kWh
	Swakopmund, Namibia, 11 m water depth. 	0.16 €/kWh

Analysis of wave reflection from wave energy converters installed as breakwaters in harbours

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Zanuttigh, Barbara ; Margheritini, Lucia ; L. Gambles and Martinelli, Luca

Thesis author´s contribution:
The Thesis author is the second author of this paper. She is responsible for the tests and data on SSG reflection. She is also responsible for the collaboration with WaveGen in the person of L. Gambles, who provided data on OWC analyzed by first author Barbara Zanuttigh with the help of co-author Luca Martinelli.

Analysis of wave reflection from wave energy converters installed as breakwaters in harbour

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Abstract

Amplification and renovation of harbours, none the last for the need of straitening existing structures because of the increased storminess due to climate change, is a practice that is repeating itself all around the world.

To this purpose, integration of breakwaters and WECs based on two different technologies, one based on the overtopping principle and the other of OWC type, revealed to be suitable with different advantages compared to offshore installations, among the others: sharing of costs, cheaper accessibility and maintenance, lower loads on the structure, i.e. better survivability.

Nevertheless these devices must comply with the requirements of harbour protection structures and thus cope with problems due to reflection of incoming waves, i.e. dangerous sea states close to harbors entrances and intensified sediment scour, which can lead to structure destabilization.

The present paper aims to analyse wave reflection from OWC and SSG converters, based on experimental results obtained in 2D and 3D facilities.

The applicability of formulae available in the literature and derived from costal structures experience are checked.

Consideration on induced scour and structure stability are also carried out, and solution for design improvements are finally drawn.

Keywords: wave reflection, sea slot cone generator, oscillating water column, scour, experiments, formulae.

Nomenclature

B	=	cross-shore width of the OWC reservoir
h	=	water depth at the structure toe
H_{m0}	=	significant wave height at the structure toe
HD	=	width of the SSG reservoir mouth
K_r	=	reflection coefficient
L_o	=	wave length based on spectral wave period T_{m-1}
L_p	=	wave length based on peak wave period T_p
R	=	reduction factor
R_c	=	freeboard of the crest of the OWC device or of the SSG reservoirs
T_m	=	average wave period
T_p	=	peak wave period
α_d	=	down slope
α_{inc}	=	slope in the run-up/down
γ_f	=	roughness factor in the overtopping discharge
ξ_o	=	surf similarity parameter based on L_0

1 Introduction

For all the countries around the world, harbours represent a significant economic hub, due to their capability of attracting foreign direct investments, of sustaining tourism activities and of creating industrial and transportation employment. The increase of sea level and storminess induced by occurring and expected climate change pushes for the reinforcement of existing sea banks and breakwaters.

In the meantime, within the crisis weakening the economies of the developed world, an urgent need arises for stimulating economic growth by investing in the clean energy economy and in a sustainable environment.

In this frame, it is particularly relevant the investigation of a proper design of WECs that can be used in the amplification and/or renovation of harbours. In this paper, two types of WECs, one based on the overtopping principle and one of the Oscillating Water

Column (OWC) type will be examined, with focus on wave reflection and induced scour at the WEC toe that can progressively decrease their stability as in case of traditional breakwaters.

Breakwater failures due to scour is reported by various authors, a.o. by [19], [14] and [6].

WECs based on the overtopping principle are Wave Dragon [9], Wave Plane [4] and SSG [10], [17], [18]. Looking in particular at SSG, studies were carried out so far on the design of these devices with respect to the loads and to the storing energy capacity that can be obtained by using multi-level reservoirs. No specific analysis was performed regarding wave reflection from these kind of structures, even if many experimental data exist.

For the purpose of wave reflection analysis, OWC converters might be roughly assimilated in principle to perforated wall breakwaters [8]. Many works exist regarding wave reflection of normally incident waves from single perforated wall structures [32], [23] and from structures with two or multiple perforated front walls [5], [28]. Effects of wave obliquity were also investigated from single perforated wall breakwaters [12], [26] and from breakwaters with two or multiple perforated front walls [15], [13]. As to irregular waves, experimental tests were conducted by [25], to examine the reflection coefficient of a perforated caisson sitting on a rubble mound, and by [3], to analyse the effect of irregular head-on waves on perforated caissons and single screens with different porosity.

Aims of this paper are to investigate for the first time the magnitude of wave reflection from nearshore WECs and to examine how and how far formulae for predicting the reflection coefficient available in the coastal engineering literature can represent wave reflection from near-shore WECs.

The paper presents the two experimental datasets adopted for the analysis, one related to 2D tests carried out at Aalborg University on a multi-level SSG device and the other one related to 3D tests carried out at Wavegen laboratory on a OWC device. Tested geometries and wave conditions are summarised.

The formulae available for smooth slopes [20], [30] are compared with the results obtained from the tests on SSG and proper indications and corrections are provided for design purposes.

2 The SSG device and tests

Tests were carried out in the shallow water wave flume at the Hydraulics and Coastal Engineering laboratory of the Department of Civil Engineering of Aalborg University.

The flume is 25 m long, 1.5 m wide and 1 m deep. The flume is equipped with a piston type wave generator with a stroke length of approximately 0.7 m. The software used for controlling the paddle system to generate regular and irregular waves is AwaSys developed by the same laboratory [1].

The multi-level SSG, 0.514 m wide, consists of 3 horizontal metal plates inclined of 35° with respect to the horizontal. In front of the SSG, a wooden run-up ramp 0.89 m, long inclined of 35°, leads the waves to the model. This slope of 35° was proven to be the optimal one for maximizing wave overtopping [11].

The structure was confined in the flume by two wooden walls, approximately 2 m long, to guide the waves avoiding spurious reflection at the structure side. An artificial dissipating beach was realized outside these wall.

A frontal picture of the set-up is shown in Fig. 1.

The plates in the SSG can be removed to vary the number of reservoirs from 1 to 3 and can slide one respect to the others in order to change the , i.e. the distances HD_1 and HD_2 (see the sketch in Fig. 2).

The majority of the tests were carried out with 2 reservoirs and the results will be presented only for this configuration.

Thirteen different geometries were tested with 0.30 m< HD_1<0.053 m, keeping fixed the crest levels R_{c1} and R_{c2} respectively at 0.033 m and 0.072 m.

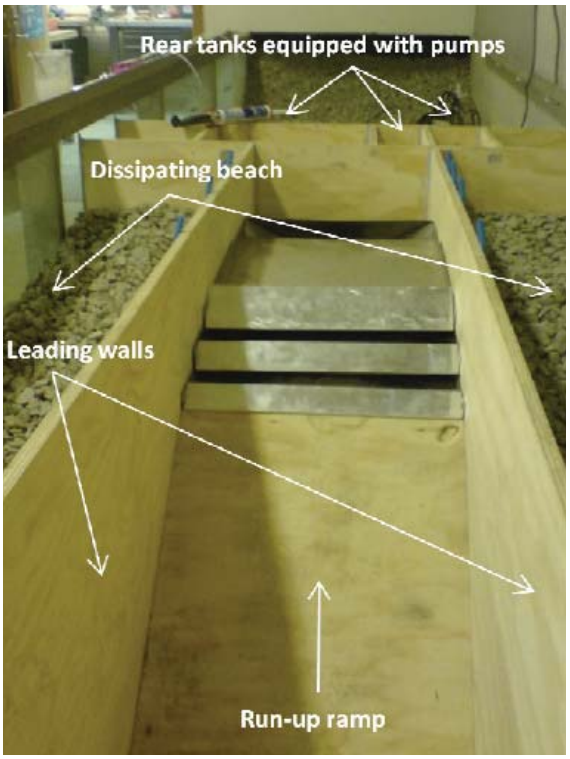

Figure 1: Frontal view of the SSG device with 3 reservoirs.

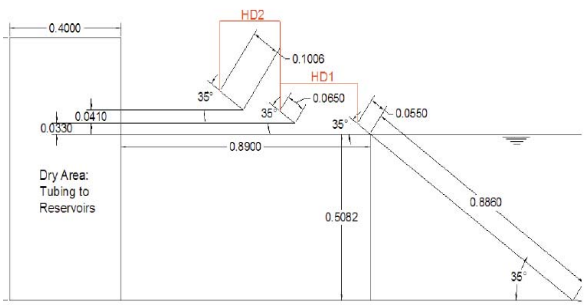

Figure 2: Sketch of the SSG, measures in m, side view.

Tested wave attacks (see Tab. 1) were 2D irregular waves with Jonswap spectrum (3.3 peak enhancement

factor). Wave conditions (W1, W2, W3, W4) were selected among the most common in the North Sea (probability of occurrence greater than 5%).

Wave heights H_{m0} were in the range 0.03-0.13 m, water depth was kept constant h=0.51 m (at the structure toe) and additional wave peak periods T_p were reproduced (W1a; W2a,b; W3b,c; W4b,c) to investigate the effect of wave steepness.

Generated waves were measured with 3 resistance type wave gauges in front of the structure, the closest one placed at 1.96 m from the model. The data acquisition was performed at 50 Hz. For the wave analysis the software WaveLab 2.94 [2] developed at Aalborg University was used by adopting Mansard and Funke method [16].

Wave	H_{m0} [m]	T_p [s]
W1	0.03	1.02
W1a	0.03	2.07
W2	0.07	1.28
W2a	0.07	2.92
W2b	0.07	0.92
W3	0.10	1.53
W3b	0.10	1.13
W3c	0.10	2.53
W4	0.13	1.79
W4b	0.13	1.31
W4c	0.13	2.92

Table 1: Target wave attacks in the Aalborg lab.

3 Wave reflection at SSG device

Most of the existing literature on wave reflection from coastal structures relates the reflection coefficient K_r to the surf similarity parameter ξ only

$$\xi_o = \tan\alpha \Big/ \sqrt{H_{m0}/L_0} \qquad (1)$$

being α the structure off-shore slope, H_{m0} the significant wave height at the structure toe and L_0 the wave length at the toe based on the spectral wave period $T_{m-1,0}$. Wave length is computed according to Guo formulation [7].

Among the available formulae, we recall for smooth slopes the work by [20]

$$K_r = (a_1 \cdot \xi_o^2)/(b_1 + \xi_o^2) \quad \text{with } a_1=1,\ b_1=5 \qquad (2)$$

and the recent analysis performed by [30] on an extensive homogeneous database

$$K_r = \tanh(a \cdot \xi_o^b) \qquad (3)$$

where a and b are directly dependent on the roughness factor γ_f in the overtopping discharge formula and for impermeable slopes -as in this case- assume the values a=0.16, b=1.43.

When there is a non-straight slope to deal with, which is the slope to be included in the definition of ξ, Eq. (1), to achieve an adequate representation of the reflection process?

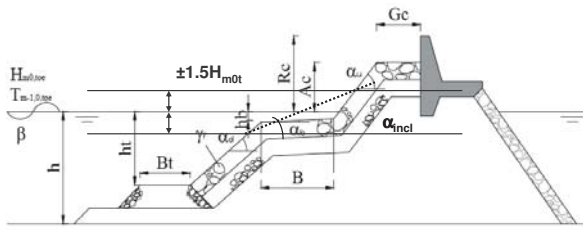

Figure 3. Structure parameters in the database, based on CLASH schematization, from [31].

The problem of identifying the correct representation of the slope in the Iribarren parameter was first analysed by [30], [31] for composite slopes and structures with berm.

Main results can be summarized as follows:

- what reflects is the slope below sea water level (SWL);
- for combined slopes an average slope has to be included in ξ;
- wave reflection is influenced by wave breaking and run-up. The lower the run-up the greater the reflection, and the greater the energy dissipation by breaking on a berm, the lower the reflection. The presence of a toe and/or a berm should thus be accounted for whenever it may affect these processes, more specifically when the berm is placed in the run-up /down area +/-1.5·H_{m0}.

In the attempt to consider the presence of the berm even when it is at SWL or above it, the Authors thus suggested to use the following average structure slope:

$$\xi_o = \frac{\left[\tan\alpha_d \cdot (h - 1.5H_{m0}) + \tan\alpha_{inc} \cdot 1.5H_{m0}\right]/h}{\sqrt{H_{m0t}/L_0}} \qquad (4)$$

$$\xi_o = \tan\alpha_{incl} \Big/ \sqrt{H_{m0}/L_0}$$

where the second expression is used only when the water depth h is such that $h \leq 1.5 \cdot H_{m0}$.

The weighted average slope in Eq. (4)

- is performed over the water depth at the structure toe h;
- makes use of the average slope in the whole run-up/down α_{incl}.

Figure 4 shows the wave reflection coefficients derived from measurements at the SSG together with the database for smooth slopes by [30]. In this figure, the value of ξ is calculated based on Eq. (4).

The improvement that is obtained by adopting the average slope in Eq. (4) instead of the downstream slope α_d can be seen by comparing the predictions by Eq. (2) with the two slopes, respectively in Figures 5 and 6.

More specifically, the overall performance of the formulae, Eq.s (2) and (3), is synthetically described below in terms of the percentage rms errors

- 11.6% rms-error given by Eq. (2) where ξ is evaluated based on $\alpha = \alpha_d$
- 8.6% rms-error given by Eq. (2) where ξ is evaluated based on Eq. (4)

- 16.9% rms-error given by Eq. (3) where ξ is evaluated based on $\alpha = \alpha_d$
- 12.6% rms-error given by Eq. (3) where ξ is evaluated based on Eq. (4)

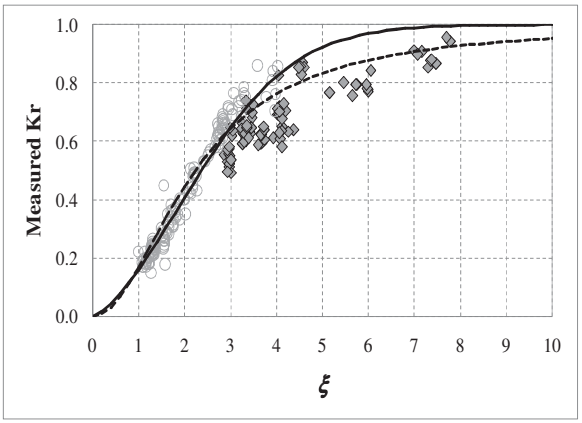

Figure 4. Measured values of the reflection coefficient at the SSG structure (diamonds) and measured values for smooth straight slopes (circles) from the reflection database by [30]. Dashed line is Eq. (2), solid line is Eq. (3).

Both Eq.s (2) and (3), but especially Eq. (3) tend to overestimate the measured values whatever is the expression adopted for the surf similarity parameter. The overall performance of both formulae is significantly improved by using Eq. (4). So the importance for reflection process of what is happening in the run-up/down area is confirmed.

Eq. (2) provides a much greater accuracy in the predictions.

Anyway, it is worthy to note that in these tests no measurements of the roughness factor γ_f is available, so that in Eq. (3) the standard values of a and b for the impermeable concrete slopes were selected.

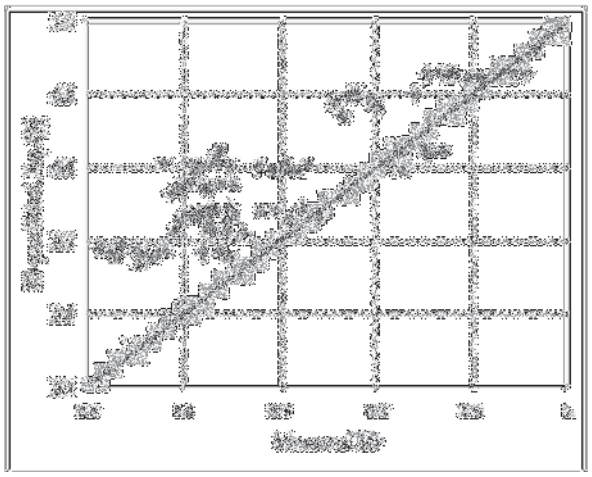

Figure 5. Comparison among measured values of the reflection coefficient and predictions obtained by [20], Eq. (2), being in ξ the slope $\alpha = \alpha_d$.

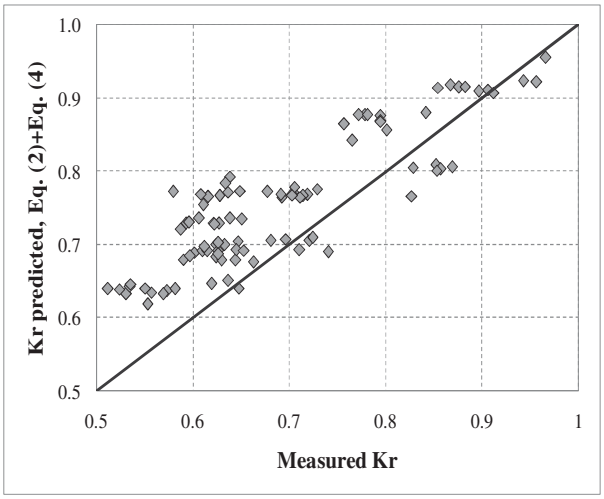

Figure 6. Comparison among measured values of the reflection coefficient and predictions obtained by [20], Eq. (2) and Eq. (4).

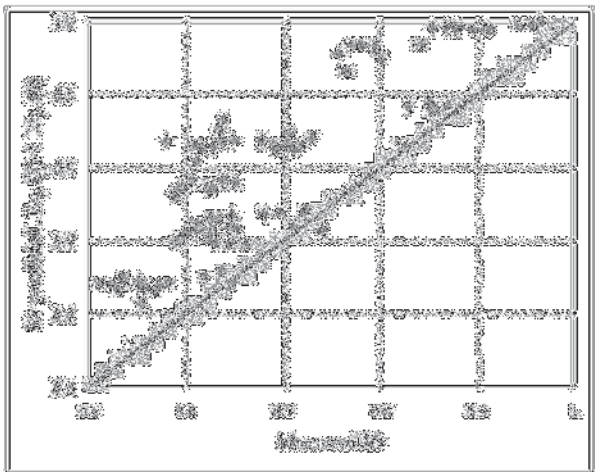

Figure 7. Comparison among measured values of the reflection coefficient and predictions obtained by [30], Eq. (3) and Eq. (4).

In the case of the SSG device, wave run-up and run-down appear to be particularly relevant to the reflection processes because differently from the case of a berm, the 'step' in the structure slope does not provide any dissipation.

On a berm, waves usually break and dissipate so that the wider the berm the lower the reflection offered by the upper slope – and the greater the phase delay between the waves reflected from the down and upper part of the structure.

In the SSG, the 'step' in the slope is the mouth of the reservoir, so that part of the waves disappear into the reservoir and these waves obviously cannot produce any reflection from the upper part of the SSG slope. Moreover, the width of the reservoir mouth is not comparable, even at prototype scale, with the traditional berm width so that waves reflecting from the slope of the second reservoir are essentially in-phase with the waves reflecting from the first reservoir.

Further analysis is thus required to correctly represent the contribution to wave reflection from the

SSG. Based on the observations just drawn above, it cannot be disregarded what happens in the whole run-up/down area. To this aim two contemporary measures can be adopted:

- the average structure slope including the run-up/down, i.e. Eq. (4); this slope is still adopted because includes the effect of the run-up/down but weights more what happens below SWL;
- a reduction factor for the reflection coefficient, to account for the water volume 'lost' inside the first reservoir, which is always placed in the run-up area

$$R = \frac{R_{c1} - R_{c2} + HD_1 + h}{R_{c1} + HD_1 + h} \tag{5}$$

The performance of the formulae by [20] and [30] results very similar with this correction R in terms of rms-error, but the latter gives a much better accuracy in terms of Wilmot index I_w [29]:

$$I_W = 1 - \frac{\sum_{k=1}^{N} \left(Xc_k - Xm_k \right)^2}{\sum_{k=1}^{N} \left[\left| Xc_k - \overline{Xm} \right| + \left| Xm_k - \overline{Xm} \right| \right]^2} \tag{6}$$

where Xc and Xm are the computed/estimated and measured values respectively, and the overbar denotes the average. If I_w equals 1 there is a perfect agreement among computations/estimations and measurements whereas if I_w equals 0 there is no match.

More in details

- a rms-error of 6.6% and a I_W of 87.8% are given by Eq.s (2) and (5), with ξ estimated from Eq. (4),
- a rms-error of 6.3% and a I_W of 91.4% are given by Eq.s (3) and (5), with ξ estimated from Eq. (4).

The predictions are compared in Figures 8 and 9 for Eq.s (2) and (3) respectively. Eq. (2) with the inclusion of the reduction factor, Eq. (6), tends to provide non-cautious estimations of K_r when K_r is greater than 0.65.

So far we have focused on the prediction capacity of existing formulae only. A second but not less important aspect of this analysis consists in the consideration that wave reflection induced by the SSG is always high, not less than 50% and on average equal to 68%.

Traditional rubble mound breakwaters or breakwaters with armour units such as tetrapods, cubes, etc., usually give a 30-40% wave reflection. Wave reflection from caisson breakwaters indeed is around 45% and up to 90% so that these values are close to the ones obtained from SSG devices. In both cases of rubble mound and caisson breakwaters however proper rocky toe protections or perforated screens are designed in order to reduce wave reflection and the induced scour at the structure toe.Indeed for SSG devices it should be properly planned the placement of a toe protection. This protection has to cope with two opposing aspects. On one side, it has not to be too high, in order not to induce wave breaking and thus dissipate incident wave energy that can be transferred to run-up and then potential energy into the SSG. On the other side, it has to assure the stability of the structure by avoiding mechanisms of failure induced by the scour hole that may occur at its toe. To achieve both purposes in presence of a soft bottom may be rather difficult.

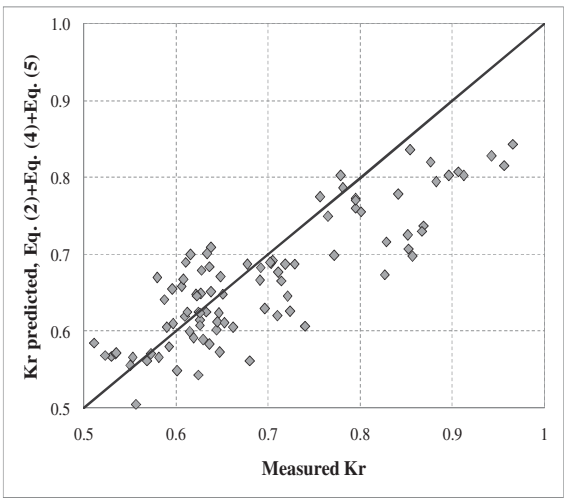

Figure 8. Comparison among measured values of the reflection coefficient and predictions obtained by [20], Eq. (2) and Eq. (4) corrected by the reduction factor R expressed by Eq. (5).

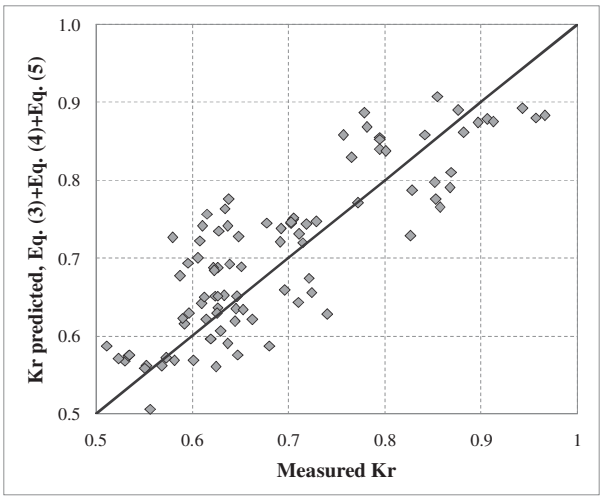

Figure 9. Comparison among measured values of the reflection coefficient and predictions obtained by [30], Eq. (3) and Eq. (4) corrected by the reduction factor R expressed by Eq. (5).

The design is indeed strongly influenced by the degree of wave reflection. The stability formulae proposed by [27], for a degree of reflection similar to that of a rubble mound breakwater, are much less severe than those proposed by [22] for the case of a vertical breakwater, where a toe protection block may be needed. In extreme cases, a proper excavation and reinforcement of the original bed may be necessary. A block may be required, placed over at least two layers of rubbles, with weight and perforation computed according to Tanimoto formula [24].

An ideal kind of protection may consist of geosynthetic bags placed on the bottom and filled in by sand extracted from submarine borrow areas. Such protection can be composed by few rows of bags in the

cross-structure direction -ideally covering a distance of around $L_0/4$ from the toe [21]- and 2 layers of bags along the water depth, taking care of the ratio H_{m0}/h.

4 The OWC device and tests

The tests were carried out in scale 1:40 in the wave tank at Wavegen laboratory. The tank is 20m long x 6m wide x 1.5m deep. It is equipped with a piston type wave generator with a stroke length of approximately 0.7 m. The wavemaker is composed by 8 independent paddles, to generate directional waves and spread/ short-crested seas. However for the purposes of these tests only uni-directional waves were performed.

Bottom slope consists of ramps with different inclinations, being the average slope 1:70.

Picture and cross section of the tested OWC devices are shown in Figures 10 and 11 respectively. Two cases were tested: a device with three OWC chambers, covering a long-shore width L_b=36 m, and a similar device for a total L_b=110 m (measures at prototype scale). The structure is 15.24 m high and the chamber inlet extends up to 4.96 m from the bottom.

The OWC was fixed directly to the tank floor which has a slope of 5°. The structure, mounted on the tank centre line was not confined by any leading walls However an artificial dissipating beach was fitted directly behind the device

Sixteen wave attacks, consisting of 2D irregular seas with Bretschneider spectrum, were performed for both structures (see Tab. 2) for a total of 32 tests. Wave heights H_{m0} were in the range 1.48-6.04 m, mean wave periods T_m were in the range 6.5-15.5 s and water depth was kept constant h=7.4 m (at the structure toe).

Generated waves were measured with 3 resistance type wave gauges along the middle axis of the tank, the closest one placed at 3.14 m from the model (model scale measure). The data acquisition was performed at 20 Hz. Wave analysis was carried out both in time domain and in frequency domain by adopting Mansard and Funke method [16].

5 Wave reflection at OWC device

Wave reflection from the OWC device is analysed based on the most relevant parameters highlighted by previous works for vertical breakwaters, perforated caisson breakwaters and perforated screens.

For vertical breakwaters, these parameters can be the crest freeboard to incident wave height ratio R_c/H_{m0} [3] and the incident wave height to water depth ratio H_{m0}/h [25].

For perforated caissons and screens, these parameters are respectively the chamber cross-shore width to wave length ratio B/L_p and the water depth to wave length ratio h/L_p [3].

Wave length is evaluated from the formulation provided by Guo [7].

It is worthy to note that in these tests R_c/H_{m0} is always greater than 1.0 so that no significant effect related to overtopping can be expected (for sure when $R_c/H_{m0}\geq 2$). Moreover, both the water depth h and the chamber cross-shore width B are constant, so that no real effect of these parameters can be observed.

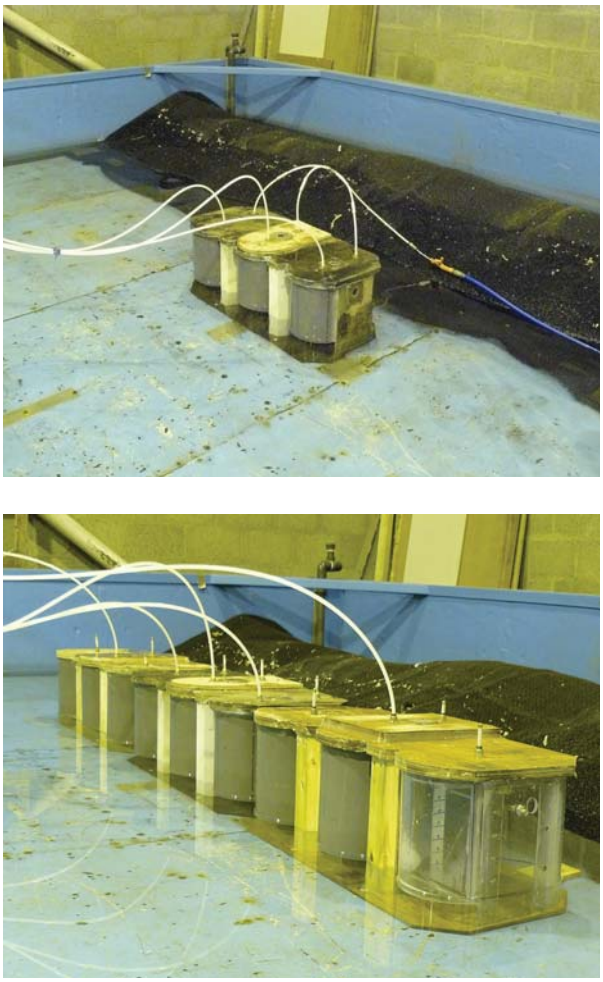

Figure 10: Pictures of the OWC devices in the wave tank.

Wave	H_{m0} [m]	T_m [s]
1	1.48	6.5
2	1.57	7.5
3	1.79	8.5
4	1.52	9.5
5	1.84	10.5
6	2.18	11.5
7	2.43	12.5
8	2.74	13.5
9	2.96	14.5
10	4.81	15.5
11	3.38	9.5
12	4.08	10.5
13	4.80	11.5
14	5.34	12.5
15	5.96	13.5
16	6.04	14.5

Table 2: Target wave attacks in the Wavegen laboratory.

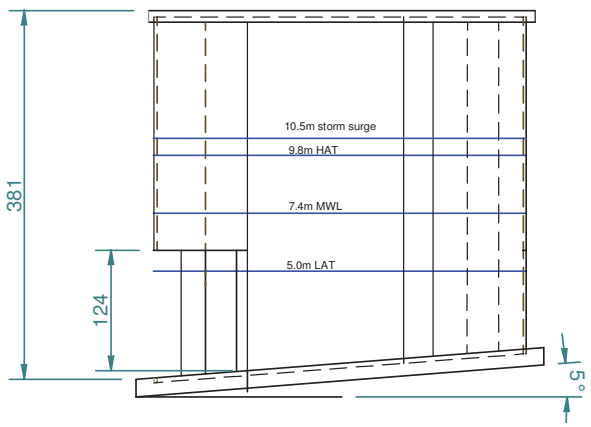

Figure 11: Cross-shore scheme of the OWC device. Measures in mm at laboratory scale.

If we look at Figures 12 and 13, wave reflection from the breakwaters of different long-shore width L_b show a completely different behavior.

As a general consideration, in the case of L_b=36 m, values of K_r are much lower (range of K_r between 32-39%) than in case of L_b=110 m (range of K_r between 40-54%).

In case of the breakwater with L_b=36 m, K_r substantially decreases with B/L_p and does not show any dependence on H_{m0}/h being the slight tendency of K_r to increase with H_{m0}/h in the order of measurement errors .

In case of the breakwater with L_b =110m, K_r shows a mirror-like tendency, since it increases with B/L_p and clearly decreases with H_{m0}/h, i.e. with wave breaking.

It can be also appreciated that under similar conditions the values of K_r for L_b =110m are characterized by a much greater scatter than in case of L_b =36 m.

The difference in the behavior for different values of L_b can be explained by two facts. The OWCs were not confined by leading walls, so that reflection from the beach can contribute to the reflection measured from the wave gauges especially in the case of L_b =36 m. Moreover, the wave gauges for measurements are placed quite far from the devices when L_b =36 m (the closer gauge is at a distance around 3 times the device width), so that the reflected wave will for sure not be a plane one and this phenomenon will be more marked the narrower the device width with respect to the wave tank width.

As a consequence, the results obtained for L_b =36 m can be misleading whereas the more reliable results are for L_b =110 m. It is indeed necessary to remark also in this case that the absence of leading walls may lead to a partial contribution from the dissipating beach that should reduce the global value of K_r (this consideration is based on the available results in presence of the beach only).

The dependence of K_r on B/L_p for the OWC device in Fig. 12 cannot be compared to the analysis carried out for perforated caissons with a single porous screen found by [3] under irregular head-on waves. In fact, the OWC porosity is much greater then the maximum screen porosity considered by the Authors (25%).

If one decides to relate K_r essentially to wave length, i.e. to B/L_p, polynomial functions of the fourth order (solid lines in Fig. 12) can be obtained merely by data approximation. It is given in the following the expression for L_b=110 m only due to the restrictions applied to the dataset and discussed above

$$K_r = 5204 \cdot \left(\frac{B}{L_p}\right)^4 - 2434 \cdot \left(\frac{B}{L_p}\right)^3 + 393 \cdot \left(\frac{B}{L_p}\right)^2 - 24 \cdot \left(\frac{B}{L_p}\right) + 0.9$$

This simplified fitting (cut at the first figure after the dot) provides a representation of K_r with rms-errors of 1.8% and 2.6% respectively, but obviously do not lead to any general prediction capacity.

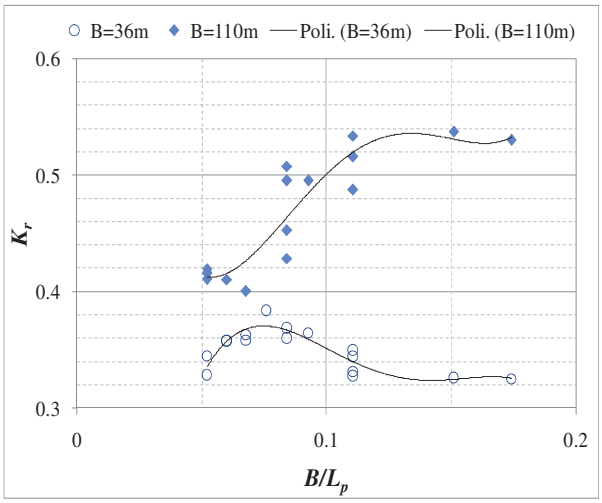

Figure 12: Dependence of the measured reflection coefficient on the chamber width to wave length ratio.

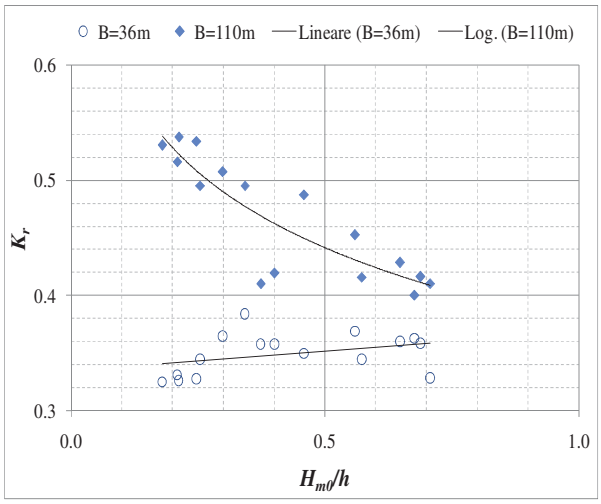

Figure 13: Dependence of the measured reflection coefficient on the incident wave height to water depth ratio.

The values of the reflection coefficient provided by the OWC device are much lower (never exceeding 55%) than the ones obtained from the tests on the SSG device previously presented (always exceeding 50%). It is worthy to note that the results from the tests are not

directly comparable, since the facilities are different and in particular one is a wave tank and the other one is a wave flume, i.e. 3D effects can take place.

Indeed also for OWC devices it should be properly planned the placement of a toe protection. The toe protection has to cope with the design of the OWC and thus with the position of the chamber inlet that is very close to the sea bottom.

In order not to oppose to waves, the protection should be placed within the bottom in front of the structure. It can be prepared as follows: the sandy bottom below the structure should be replaced with suitable material to reduce settlement. The excavation at the structure toe, running parallel to structure length L_b should extend in the off-shore direction for a distance of around $L_0/4$ from the toe [21]. Note that the crest of the toe protection has to be aligned with the sea bottom level.

Geotextile can be lied down, to separate the sand from two layers of gravels/rocks, properly sized for stability of the toe protection layers [22]. Further, hydraulic stability may be investigated being relevant to avoid the possible movement of the rocks.

6 Conclusions

This paper analysed wave reflection from a SSG and an OWC device, based on 95 and 32 tests in a wave flume and in a wave tank respectively.

In case of the SSG device, available existing formulae from coastal experience [20], [30] provide sufficiently accurate predictions of K_r, provided that a proper representation is given to the structure slope and that a proper reduction factor is introduced.

The best agreement between measurements and predictions can be obtained by using Eq. (3) with the inclusion of an average structure slope, Eq. (4), that accounts of the slope in the run-up/down area and with the inclusion of an original reduction factor developed specifically for the SSG, Eq. (5).

In case of the OWC device, the available experimental data allowed to highlight the dependence of K_r on wave length L_p: with increasing L_p, K_r decreases, being constant the chamber cross-shore width B. More tests are needed to check the effects of the chamber width B to wave length ratio (resonance problems) and of the water depth to chamber height ratio (change of structure porosity).

The reflection coefficient K_r for both OWC and SSG devices is never lower than 40% and can rise up to 90%. It is consequently a significant design issue to construct a proper toe protection layer avoiding scour holes at the structure toe and consequent possible structure failure by sliding. It is generally recommended an in depth analysis of the sea bottom, the excavation and coverage with geotextile in case of very fine sand and clay, and the construction of a stable protection with rocks or geosynthetic bags.

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References

[1] Aalborg University, 2007a. AwaSys 5 homepage. http://www.hydrosoft.civil.auc.dk/AwaSys.

[2] Aalborg University, 2007b. WaveLab 2 homepage. http://www.hydrosoft.civil.auc.dk/wavelab.

[3] Allsop, N.W.H. and S.S.L. Hettiarachchi. Wave reflections in harbours; the design, construction and performance of wave absorbing structures. Report OD 89, Hydraulics Research, Wallingford, 1988.

[4] Frigaard, P., Trewers, A., Kofoed, J. P., Margheritini, L. *Conceptual Design of Wave Plane*. Aalborg University : Department of Civil Engineering, DCE Contract Reports 38, 2008.

[5] Fugazza, M. and Natale, L. Hydraulic design of perforated breakwaters. *J. Waterw. Port Coastal Ocean Eng*., 118: 1-14, 1992.

[6] Gunbak, A.R., Gokce, T., Guler, I. Erosion and protection of Samandag Breakwater. *J. Coastal Research, Proc. of Skagen Symposium*, vol. 2, 753–771, 1990.

[7] Guo, J. 2002. Simple and explicit solution of wave dispersion equation, *Coastal Engineering*, 45, 71-74.

[8] Jarlan, G.E., 1961. A perforated vertical wall breakwater. *Dock and Harbour Authority XII* (486), 394– 398.

[9] Kofoed, J. P.; Frigaard, P.; Friis-Madsen, E. ; Sørensen, H. C.. Prototype Testing of the Wave Energy Converter Wave Dragon". *Renewable Energy*. vol. 31 (31), 181-189, 2006.

[10] Kofoed, J. P. Vertical Distribution of Wave Overtopping for Design of Multi Level Overtopping Based Wave Energy Converters. *Proc. XXX Int. Conf on Coastal Engineering*, San Diego, 2006.

[11] Le Méhauté, B., R. C. Y. Koh, and L. Hwang. A systhesis of wave run-up. *Journal of waterways and coastal engineering division*, ASCE Vol. 94 (No. WW1, proc. Paper 5807), 77-92, 1968.

[12] Li, Y.C., Liu, H.J., Teng, B., Sun, D.P. Reflection of oblique incident waves by breakwaters with partial-perforated wall. *China Ocean Eng*. 16 (3), 329–342, 2002.

[13] Li, Y., Dong, G., Liu, H, Sun, D. The reflection of oblique incident waves by breakwaters with double-layered perforated wall, *Coastal Eng*., 50, 47–60, 2003.

[14] Lillycrop, W.J., Hughes, S.A. *Scour Hole Problems Experienced by the Corps of Engineers; Data*

Presentation and Summary. Miscellaneous Papers, CERC-93-2. U.S.A.C.E, Coastal Engineering Research Center, Vicksburg, MS. 1993.

[15] Losada, I. J., M.A. Losada and A. Baquerizo, An analytical method to evaluate the efficiency of porous screens as wave dampers, Applied Ocean Research, 15, 207–215, 1993.

[16] Mansard E. and E. Funke, The measurement of incident and reflected spectra using a least squares method. Proc. 17th Int. Conf.on Coastal Eng., Sydney, 1, 154–172.

[17] Margheritini, L., Lander, V., Kofoed, J.P. and P. Troch, Geometrical optimization for improved power capture of multi-level overtopping based wave energy converters. *Proc. ISOPE 2008*, in press.

[18] Margheritini, L. ; Vicinanza, D. ; Frigaard, P. SSG Wave Energy Converter: Design, Reliability and Hydraulic Performance of an Innovative Overtopping Device. *Renewable Energy*; vol. 34 (5), 1371–1380, 2009.

[19] Oumeraci, H. Scour in front of vertical breakwaters, review of problems. Proc. of Int. Workshop on Wave Barriers in Deep Waters. Port and Harbour Research Institute, Yokosuka, 281–307, 1994.

[20] Seelig, W.N. and J.P. Ahrens, 1981. *Estimation of wave reflection and energy dissipation coefficients for beaches, revetments and breakwaters*. CERC Technical paper 81-1, Fort Belvoir, U.S.A.C.E., Vicksburg, MS.

[21] Sumer, M., Fredsøe, J., Lamberti, A., Zanuttigh, B. Dixen, M., Gislason, K., Di Penta, A. Local scour and erosion around low-crested structures, *Coastal Eng*., 52 (10-11), 995-1025, 2005.

[22] Takahashi, S. *Design of vertical breakwaters*, Reference Document N34, Port and Harbour Research Institute, Japan, 1996.

[23] Takahashi, S., Y. Kotake, R. Fujiwra and M. Isobe, Performance evaluation of perforated-wall caissons by VOF numerical simulations, *Proc. of the 28th Coastal Eng. Conf.*, Cardiff, 1364–1376.

[24] Tanimoto, K., Haranaka, S., Takahashi, S., Komatsu, K., Todoroki, M. and Osato, M. An experimental investigation of wave reflection, overtopping and wave forces for several types of breakwaters and sea walls. *Tech. Note of Port and Harbour Res. Inst*., Ministry of Transport, Japan, No. 246, 38 pp. (in Japanese, with English abstract), 1976.

[25] Tanimoto K. and S. Takahashi. Design and construction of caisson breakwaters--the Japanese experience, *Coastal Engineering*, 22 (1-2), 57-77, 1994.

[26] Teng, X.T. Zhang and D.Z. Ning, Interaction of oblique waves with infinite number of perforated caissons, Ocean Engineering, 31, 615–632, 2004.

[27] Van der Meer, J.W., K. d'Agremond and E Gerding. Toe Structure Stability of Rubble Mound Breakwaters. *Proc. of the Advances in Coastal Structures and Breakwaters Conference*, Institution of Civil Engineers, Thomas Telford Publishing, 308-321, London, UK, 1995.

[28] Williams, A. N., A.E.M. Mansour and H.S. Lee, Simplified analytical solutions for wave interaction with absorbing-type caisson breakwaters, Ocean Engineering, 27, 1231–1248, 2000.

[29] Wilmott, C. J. 1981. On the validation of models. *Phys. Geogr.*, 2: 184-194.

[30] Zanuttigh B. and J.W. Van der Meer, 2008. Wave reflection from coastal structures in design conditions. *Coastal Engineering*, Elsevier, 55 (10), 771-779.

[31] Zanuttigh, Van der Meer, J. W., Lykke Andersen, J. W., Lara, J. L and I. J. Losada. Analysis of wave reflection from structures with berms through an extensive database and 2DV numerical modelling. *Proc. XXXI Int. Conf. on Coastal Eng.*, 2008, in press.

[32] Zhu S.T. and A.T. Chwang, Investigations on the reflection behaviour of a slotted seawall, Coastal Engineering, 43, 93–104, 2001.

A method for EIA scoping of Wave Energy Converters - based on classification of the used technology

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Lucia Margheritini, Anne Merrild, Peter Frigaard

Thesis author´s contribution:

The Thesis author is the first author of this paper. She is responsible for the review of the existing literature and information on EIA of WECs. The Author is also responsible for the selection of relevant parameters for the used to present a new classification of WECs based on their expected impact on different environmental receptors. Second Author Anne Merrild is responsible for the methodology and terminology presented in the paper as well as the presentation of the screening and scooping processes. Co-author Peter Frigaard is responsible for the communication with Danish Energy Agency and idea behind the paper.

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Abstract: During the first decade of the 21st Century the World faces spread concern for global warming caused by rise of green house gasses produced mainly by combustion of fossil fuels. Under this latest spin all renewable energies run parallel to achieve sustainable development and among them wave energy has an unequivocal potential. Technology is ready to enter the market and contribute to the renewable energy sector. Yet, frameworks and regulations for wave energy development are not fully ready, experiencing a setback caused by lack of understanding of the interaction of the technologies and marine environment, lack of coordination from the competent Authorities regulating devices deployment and conflicts of maritime areas utilization. The EIA within the consents process is meeting point for technology, politics and public and central in the realization of full scale devices. This paper presents the development of a classification of wave energy converters that is based on the different impact the technologies are expected to have on the environment in order to simplify the scoping process for developers and authorities.

Cover Page

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Title Page

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1 Introduction

Wave Energy Converters (WECs) has been undergoing a significant development since the oil crisis in the 1970s, and have been subject to extensive studies. The technologies have been optimized, tested and Pilot wave energy projects have been realized. The obtained knowledge and experiences has lead to a development status where WEC technologies are today ready to pass from the testing phase to full scale deployment and contribute to the renewable energy sector. A

total of 96 companies and energy concepts worldwide are listed by European Marine Equipment Council (EMEC) today; more than 56% of the WECs are located in Europe. WECs vary in technological concept and design. There are as many as 49 different concepts identified only within Europe. The growth and interest in expanding the wave energy sector is based on its potential estimated to be up to 10 TW. Depending on what is considered to be exploitable, this covers from 15% to 66% of the total world energy consumption referred to 2006 (Engineering Committee on Oceanic Resources, 2003; Cruz et al. 2008). Forty-nine different wave energy concepts are under development today only within Europe (Fig 1). In order to gain permit from the related planning authorities to place a full scale WEC at a specific site, an Environmental Impact Assessment (EIA) is an administrative procedure that a project will usually have to pass (Zubiate et al., 2005). As WECs deployed in full scale is an early practice, only few EIA's of WEC's has been carried out. It is argued by the developers, that only minor environmental impacts can be expected by deployment of WECs, and that most impacts are associated with the establishment and decommissioning phase. (Sørensen and Russel, 2008). Never the less a European Directive requires that the European countries at least conduct an initial EIA screening to investigate weather or not a WEC is mandatory to conduct a full EIA.

Today Environmental Impact Assessments has been carried out for the following wave energy devices:

- AquaBuOY based on the deployment September 2007, Oregon, US (Weinstein et al. 2007) (Fig 2).
- Wave Dragon 1:4½ prototype deployed by 2003 in Nissum Bredning, Denmark (Hansen et al. 2003) (Fig. 2).
- Wave Dragon based on the expected deployment off the west coast of Wales, UK by 2010 (Russell and Sorensen 2007).

The deployment of the AWS west of Portugal in 2005 (Beirão at al. 2007) was established without any accessible EIA. So was the case with the deployment of Pelamis in Portugal, 2008 (Fig. 2)and a number of prototype shoreline devices of the OWC kind that have already been constructed and operated with varying degrees of success over the last 30 years around the World.

Further information on EIA exists for:

- The EMEC test centre in Orkney, UK.
- The Wave Hub project north of Cornwall, UK established in 2008 (Harrington and Andina-Pendás 2007).

The EMEC test centre has been created for the purpose of testing full scale grid connected prototype devices on a limited amount of time (Fig. 3). Being a tests site, the devices are not required to be subjects of full EIAs, but an Environmental Statement demonstrating the developers are aware of the issues and potential environmental impacts of their devices. Wave Hub is an innovative demonstration site for generation of wave energy located in the South West of England, north coast of Cornwall (Harrington and Andina-Pendás 2007). It consists of an offshore electrical "socket" to connect arrays of wave energy converters to the national grid via under-sea cables (Fig. 3).

In European EIA systems, the involvement of the public, as well as the competent authority and other responsible government agencies, is an integral part of the process. Normally it is the competent authorities together with the developer of the project and his consultants in corporation that carries out the first two steps of an EIA, namely the *screening* and the *scoping* and sets the plan for the following process (Kørnøv et al., 2007). The organisation and quality of the communication between the developer and the authorities is depended on the national legislation in the actual country as well as on the administrative body. As implementation of full scale WECs is still at an early stage, the planning authorities in the European context have in

general not a specific frame or body in place to handle the applications. This increases the risk that conflicts arises from the communication and thereby the risk that there will be a lack of coordination among developer, consenting bodies, authorities and statutory consultees. (Kørnøv et al., 2007; Cashmore, 2004) At the present time Denmark is the only European country who has an administrative body in place to coordinate the planning and implementation of offshore wind and WECs: the Danish Energy Agency. Before 2009 the Danish Energy Agency allowed the deployment of 4 wave energy converters as demonstration plants and prototypes: (Wave Dragon, Wave Star, Wave Plane and Poseidon Organ) with very smooth procedures. For the development projects EIA screenings was carried out with the conclusion that it was not necessary to conduct EIAs for the projects. The conclusion was sent in hearing to the statutory consultees and if the responses to the conclusion were positive, then the project could continue its implementation with no more environmental investigations. For one of the four demonstration projects the affected municipality asked for investigation of a specific doc species, and when this was conducted and showed that the doc's would not be impacted, no further investigations was demanded and no EIA was carried out. In the UK an administrative body similar to the Danish is under construction. At European level so far activities in the maritime environment have been managed by separate policies but now EU is calling for more integrated approach with a maritime spatial planning. Offshore energy production seats within the list of main activities to be coordinated, including wave energy. The program foresees also coordination between Member States that will lead to less bulky procedures and lower administrative costs.

This paper aims at classifying different WEC technologies depending on the potential environmental impacts in order to identify what environmental receptors are important to assess in an EIA. The mean is to make it easier for developers and authorities to carry out the scooping of this type of projects. The classification is made by analyzing the potential environmental impacts

of different technologies. Then the suggested categories and the related expected impacts are compared with existing EIAs to test the system. Finally it is discussed how the administrative body in the countries in which the WECs are implemented, impact on the EIA process.

2. Method and materials

To identify which receptors are important to EIA of WECs, this paper takes its point of departure in the legislative context. The first part of this paper; section 3, investigates the demands to EIA with focus WECs within the EU. The related EU directive which is the basic EIA frame for all European Countries, is presented and the demands to the content of EIAs is described with special focus on the first two steps of the process where the scope and content of EIA is decided upon. Section 3 also looks into the state of the art on identification of environmental parameters in relation to WECs. The most detailed material regarding environmental impacts of WECs today is based on the EMEC guidelines for EIA of WECs. The EMEC results are used to set up an impact matrix potential for the screening and scoping of WECs. This part of the analysis is based on EU legislation and EIA educational materials as well as on existing reports and papers of EIAs of WECs. Other relevant reference material partially presented in this paper comes from the work under development within the EU-funded projects EquiMar (7FP, 2008-2011, under grant agreement n° FP721338) in which the Department of Civil Engineering of Aalborg University is partner for the Work Packages (WP) 3 and 4.

The next part of the paper, section 4 is presenting an analysis of technological differences of WECs and related potential environmental impacts. The existing technologies within EU are scanned in relation to 4 plus 1criterions:

1) Distance from shore (onshore, intermediate, offshore)
2) Stability elements (simple moorings, complex moorings, gravity foundations, piles)
3) Obstruction to water column (little, some, very)

4) Power takeoff.

5) Obstruction to the sea surface in case of wave farms (only mentioned).

In relation to these criterions it is identified how the technological differences affect the likelihood of potential environmental impacts. Based on the expected similar impact of the converters on the environment a classification of WECs is presented. This is done by comparing the technology with the impact matrix based on the EMEC Guidance in section 3. Finally, a comparative analysis is conducted between the expected environmental impacts derived from the classification against the results from the EIA of WECs that has been conducted at the present time (Wave Dragon Wales and AquaBuOY Oregon).

Based on the comparative analysis, it is concluded in section 4 that a table underlining the principal areas of concern for different group of technologies as presented, makes it possible to simplify the scoping procedure and to provide an easier understanding of the technologies to the authorities.

3. EU Directive on EIA

EIA is an environmental management instrument implemented worldwide. EIA was introduced in The European Community in 1985 via the directive: "Council Directive 85/337 /EEC- on the assessment of the effects of certain public and private projects on the environment" (85/332/EEC) and later with an amending directive in 1997 (97/11/EEC). The European Directive describes the aim of EIA as: "…providing the competent authorities with relevant information to enable them to take a decision on a specific project in full knowledge of the projects likely significant impact on the environment..." This way EIA's function includes two fundamental aspects, one is technical and regards the question of how to make the best description and assessment of different impacts. The other aspect regards the question of how to make the EIA inform and influence decision making. (Kørnøv et al., 2007) EIA often functions as a "framework for negotiation and

compromise" and it plays an important role in the consenting process of a lot of projects (Cashmore, 2004).

The Directive covers the provision for 25 countries and counting. Among others Denmark and United Kingdom has implemented EIA into their national legislation systems. (Kørnøv et al, 2007)

As earlier mentioned in this paper, several wave energy projects in Europe are facing the stage of realisation where they are to deploy in large or full scale. WECs in full scale are expected to be subjects of EIA. A full EIA process based on the Directive includes the steps shown in fig. x the arrows illustrate how the EIA process is iterative. The EU Directive prescribes eight steps to conduct a full EIA process including: Screening, Scoping, Baseline studies, Identification of alternatives, Prediction and evaluation, Mitigation, Documentation and hearing and finally Monitoring all illustrated in Fig. 4.

As the focus of this paper is on the technical aspect of EIA regarding the delimitation and the coverage of EIAs in relation to ocean energy devices, the first two steps, Screening and Scoping of the EIA process is of the main interest and is further described in the following sections including performance tools.

3.1 Screening and Scoping

The screening is the first stage in the process of EIA. There are different ways of conducting screenings. In the EU directive an inclusive list of projects where EIAs are mandatory is given in Annex I. And for projects where EIA can be mandatory depending on the size and significance of the environmental impacts are listed in Annex II (97/11/EEC). Projects listed in Annex II should undergo further screening to assess weather or not the project will impact significant on the environment and if yes, a full EIA will be required. Annex II includes among others wave energy constructions as the following is listed: *"Energy industry" and "Industrial installations for the production of electricity, steam and hot water".* (97/11/EEC)

Usually the screening is conducted by the related authorities when a developer or company is applying for a site permit for a specific project. (Kørnøv et al., 2007) If the screening process leads to the conclusion that an EIA should be carried out, the next step in the process is to decide which receptors are important to include in the EIA and at what level of details they should be assessed. This is the scoping phase. Shortly put, the mean by scoping is to identify:

1. the appropriate time and space boundaries of the EIA study
2. the information necessary for decision-making and
3. the important issues to be considered in an EIA
4. the significant effects and factors to be studied in detail

The EU Directive defines a broad concept of the environment and points at the following groups of environmental receptors to be investigated in relation to the scoping of the assessments: human beings, fauna and flora, soil, water, air, climate and the landscape, material goods and the cultural heritage and the interaction between these factors. (97/11/EEC) There are different methods for scoping. The EU Commission made guidelines for scoping (EU- Commission 2001) including general checklists or impact matrixes. Also existing environmental statements of relevance for the specific action, Consultation of (environmental) authorities, NGOs, the public and consultants and experts are ways of approaching the scoping of a project. The scoping process itself can vary in scope, complexity and time taken. A method used to identify part 3. and 4. as described above is the Impact Matrix method, where the rows represent the environmental receptors and the columns represent the stressing activities (stressors). This method will be applied to wave energy projects. The cells of the matrix then express the potential for disturbance of the entity on the rows from the activity in the column. A matrix created following the EMEC Guidance has been realized. This summarizes all the activities and entities that may interact in case of WECs.

4. Development of an Impact matrix from the EMEC EIA Guidance

EMEC presented a very detailed list of information that developers must provide with respective key impact issues associated with different aspects of the device (EMEC EIA Guidance for WECs). The proposed criteria to assess the environmental impact of wave energy converters is based on the exposure, defined as “the contact or co-occurrence of a stressor with a receptor” and the effects describing the ability of a stressor to cause adverse effects. The criteria for the assessment of the EI can hence be presented as a combination of exposure and effects as major, moderate, minor, negligible, no impact and positive impact (table 1).

The effect on the environment of the device must be accessed for installation, operation and maintenance and decommission phases. Also, the assessment of accidental events must be taken into account (table 2).

Based on the huge variety of existing WECs it appears obvious that the environmental effects of WECs are strongly dependant on the technology in addition to the location of the project. A summary of receptors and activities potentially involved in the deployment of WECs is reported from the EquiMar project (7FP, 2008-2011).

ALTERATION IN WATER COLUMN PATTERNS Effect on currents and waves: The impacts on currents and waves are strongly dependant on technology and location of the projects with maximum effects closest to the installation and near the shoreline (Boehlert et al. 2008).Sediment Dynamics: Disturbance on sediment dynamic can occur during operation as a consequence of modification on water circulation i.e. in current velocities or wave heights but also directly during installation or decommission. The effects which occurred during installation are usually temporary and their significance is proportional to the amount and type of bottom substrate disturbed.

INTERFERENCE WITH BENTHIC HABITATS. It occurs during installation and decommission as direct result of disturbance of anchoring of the construction vessels, digging and refilling the trenches of the power cables and installation of permanent anchors, pilings or other mooring systems. When

installation is completed, disturbed areas are supposed to re-colonize by the same organisms assuming that the substrate and habitats are restored to a similar state but uncertainties from indirect impact of alteration in water circulation may be more extensive and long-lasting.

ARTIFICIAL REEF EFFECTS. The extensive and rapid colonization of ocean energy structures by macro-benthic communities has been also established, particularly on the device foundations installed in coastal sandy areas. It is important to determine if this change is beneficial or not for the existing local conditions. The offshore energy units should be regarded as artificial reefs and as such its design can play a critical role in species establishment. The influence of foundation surface orientation of on epibenthic colonisation was also examined and observations of the use by fish and crustaceans were carried out during three years (Langhamer, et al. 2009.).

WATER QUALITY INTERFERENCE. When talking about chemical effects of wave energy devices it is important to distinguish between spills as a source of chemical, low probability but high impact, versus continuous release of chemicals for example in fouling paints. The rapid and heavy growth of marine fouling of wave energy devices is considered of particular concern. There are currently only three options to deal with marine fouling: use of antifouling coatings, in situ cleaning using high pressure jet spray by divers or remotely operated vehicles and removal of the device from the water surface for cleaning on site or onshore and reapplication of antifouling coatings. Nowadays the use of tri-butyl tin compounds on coatings has been proposed to phase out and research has been carried out to develop less toxic antifouling coatings (Dunagan et al. 2007). Chemicals can move over a large area, depending on the site circulation pattern. Although this type of effects are, like others, strongly site-specific, information is needed on the toxic compounds to be used, potential amounts that could be released, responses of the biological receptors and the fate of contaminants.

NOISE DISTURBANCE. Construction, operation and decommission of large mechanical structures will inevitably produce sound that may disturb or even cause physical damage to wildlife in the vicinity. It is worth to mention that for some devices the noise can be of disturbance for the local communities.

The construction phase is of particular concern if pile driving is required. The effects of pile driving operations on fish have received little attention (Hawkins 2006) but ongoing work is being conducted by CEFAS and Cranfield University funded by COWRIE. Early work that demonstrated that the rise and decay time is very important and that a combination of rapid rise and decay (~1 ms) and a sound pressure of ~229 dB re to 1 µPa are required to be lethal (Wardle et al. 2001)and it is unlikely that piling operations will cause mortalities directly.

Noise disturbance on marine mammals: physical/physiological effects may include hearing threshold shifts and auditory damage. Behavioural responses, including fright, avoidance and changes in behaviour and vocalisation patterns have been observed in baleen whales, odontocetes and pinnipeds; in some cases at range of tens or hundreds of kilometres from loud industrial noises. There are important gaps in our knowledge. For example, the characteristics of the sound signature of these new and developing technologies are poorly known and how they propagate at different ranges and depths are poorly understood. Work is needed to estimate safe levels of exposure for different marine mammal species

Noise disturbance on fish: even if physiological damage is unlikely to be caused by marine renewable energy devices construction, behaviour may be disturbed. Many species of fish use sound both for communication and for detecting prey and predators. There could, however, be physiological damage either temporary or permanent that could seriously affect subsequent survival(Blaxter and Hoss 1979; Hoss and Blaxter 1982).

ELECTROMAGNETIC FILELDS. As the offshore renewable energies have been developing and maturing it became clear that the most practical way to transport the energy produced is to wire it to land through underwater cables. However cables are also expected to link devices between themselves and possibly a common hub, depending on the park design. Therefore a significant proportion of seabed in offshore parks is expected to have the presence of cables. The Electromagnetic field (EMF) is a broader term that includes the Electric Field (E Field), measured in µV/m, which is usually contained within the cable insulation and the magnetic (B- Field) measured in µTeslas which is detectable on the outside of the cable. In turn, the B-field can create an Induced Electrical Field (iE Field) when conductive animals move through it. Some marine species have the ability to detect and some use EMF fields for orientation and detection of other animals (predator-prey interactions), (Murray 1974).

The offshore wind industry in the UK has been funding important environmental work through the Collaborative Offshore Wind Research Into The Environment (COWRIE) including a comprehensive study on EMF fields. The COWIE 1.5 report concluded that an interaction between electro-sensitive species and the EMF fields caused by offshore wind cables is likely to occur. However it is very hard, with current data, to estimate if there can be a species or an ecological impact from EMF.

INTERFERENCE WITH MARINE ANIMAL MOVEMENTS AND MIGRATIONS. Device dependent, especially dependent on the size of the installation. Disturbance and collision are considered the most concerned issues but also the permanent loss of habitat due to displacement (avoidance), barrier effects (e.g. fragmentation effects on units of the ecological habitat network such as breeding or feeding areas) and increased consumption of energy reserves during migration due to avoidance reactions, are to be taken into account. Construction of large industrial scale generation systems could potentially disrupt the movement patterns of marine wildlife.

SOCIO-ECONOMIC ISSUES: PUBLIC OPINION, ACCEPTANCE AND PARTICIPATION. Opinion studies conducted in Europe and United States indicate that the public is generally supportive of developing alternative energy sources specifically onshore and offshore wind energy (Coyle 2007; Ladenburg 2006 and Dong Energy 2006). A review on public acceptance of offshore wind energy in Denmark and United Kingdom indicates some fairly strong trends in public opinion which can be resumed in the following topics (Michel et al. 2007):

1) The public is in favour of offshore wind energy also in the region where they reside;

2) Visual impacts appear to be the primary issue of public concern;

3) Offshore wind park development appears to gain public approval as the community is exposed to operational projects

4) Early local input to the planning process is critical to gain public acceptance.

Although there is uncertainty on the public support for wave power, it should be reasonable to assume that similar conclusions would be obtained for wave energy installations.

From a socio-economic point of view wave energy farms may induce negative attitudes and create conflicts with other activities due to space-use conflicts such as fisheries, subsistence fishing, marine recreational activities, proximity of designated conservation areas and other alternative energy facilities.

5. Relevant parameters for EIA of wave energy converters

Different wave energy technologies have few things in common one with the other that can be listed as follow: electrical transmission infrastructure, electrical system, subsea conversion station / system and energy storage; unless the device is onshore, these installations may be responsible for electromagnetic fields and of the impact on benthic habitats. Other common features are: shore connection, shore facilities and in most of cases use of antifouling, the last could be responsible for degradation of water quality. Impact assessment of possible degradation of

benthonic habitats caused by laying submarine cables, disturbance of sensitive spices exposed to electromagnetic fields and risk of degradation of water quality as consequence of usage of antifouling must be completed for all the devices with exclusion only of onshore devices that do not use antifouling, such as onshore Oscillating Water Column devices (OWC) or Sea wave Slot cone Generator (SSG) (Margheritini et al. 2009).

The traditional classification of wave energy converters based on working principles (overtopping devices, point absorbers and oscillating water columns), location (shoreline, near shore, offshore) or orientation to the main wave direction (terminators, attenuators, buoys) fails to address the relevant parameters related to environmental issues and can be misleading if used to assess the environmental impact of the different technologies.

For example, two oscillating water columns may use different stability elements: the Mighty Whale (Japan, 1998, Fig. 5) it is a floating structure moored to the sea bed while the Pico plant (Pico, Azores, 1999, Fig. 5) it is an onshore device with gravity foundation. Pelamis and Archimides Wave Swing (Fig. 5) are both design to be installed in deep waters but while the first one is emerging from the surface, the second is several meters submerged. In the same way, the Oyster (Fig. 5) and Wave Dragon are both terminators but the first one is submerged wave activated body while the second is floating overtopping device.

In the variety of locations, shapes, sizes and working principles that the wave energy sector presents at this stage it is difficult to identify common guidelines for the different technologies. Nevertheless it is possible to clearly recognize five parameters relevant for the EIA of WECs. These parameters are described in the following paragraphs. The assessment tables presented in the next paragraphs are to be considered a simplification and conservative with respect to the final impact assessment. The tables should be used by developers, authorities and stakeholders as

primary indication or fast consultation on relevant issues for the EIA of a specific technology once that basic information on the installation is known.

***D*: Distance from shore**

It is possible to classify the devices by location (Fig. 6):

- Onshore devices. All the devices installed on land, on harbours or any device installed within the swash and surf zone.
- Intermediate water devices. All the devices installed further than the surfzone or in any case within 5 km from land.
- Offshore devices. All the devices installed further than 5 km from land.

The *D* parameter has direct consequences on the following receptors:

- Local communities (visual impact and recreational use of the sea).
- Coastal processes such as current velocities and wave heights and sediment dynamics and coastal spices.
- Navigation and fishery.

It is possible to state that the major impacts for local communities occur for onshore devices as they are directly exposed to the different phases of the device lifetime. Major impacts related to coastal processes are also expected to occur for near shore devices (Boehlert et a. 2008) while the impact on navigation is nil.

In the case of intermediate waters devices, moderate impact can be expected for local communities and costal processes while major impact for navigation. This assuming any kind of interaction of the local community's with the device area. Nevertheless intermediate waters tend to be busy with recreational and economical activities and because of this major cannot be excluded.

For offshore devices negligible impacts are expected on local communities, while on navigation can be major. The evaluation of the impact on coastal process must be further investigated even though it seems reasonable to consider a minor-negligible impact as the majority of the available wave energy at a deep water offshore site can be lost naturally through frictional effects with the sea bed before it gets to the shoreline.

Table 3 summarizes the above statements based on the exposure method for all the phases during the lifetime of the installation (installation, operation, decommission).

***S*: Stability elements**

It has been stated that most impacts are associated with establishment and decommission phase of WECs. Considerable impact can be attributed to the installation of stability elements. Four elements can provide stability to WECs depending on the device:

- For floating devices, anchors/mooring that allow different degrees of movements are used. Mooring lines can be simple or complex and especially in case of wave energy parks they can form intrigued underwater patterns (Fig. 7).
- For bottom supported structures, piles or gravity foundations are used.

Simple mooring are here considered the ones that see not more than 3 lines to and from the single device.

The *S* parameter has direct influence in the following receptors:

- Benthonic habitats
- Geology
- Archeology
- Water column spices

Piling represents inevitably the most intrusive practice, considering also the noise during installation phase and the permanent impact on benthonic habitats, geology and possibly

archeology. He impact on water column spices is considered to be limited to installation phase and therefore minor.

Moorings are in general the less impacting practice presenting a negligible impact on geology but in case of complex mooring or wave farms and large scale installations the mooring lines may impact water column species and even navigation. All the other impacts are considered to be minor and eventually temporary and related to installation and decommission.

Gravity foundations are considered to have moderate impact on the benthonic habitats and possibly archeology, estimating potential recovery within 2 years from installation, while negligible impact is expected to occur on water column spices. Moderate impact on geology has here been attributed to this practice as less intrusive than piling but usually interesting a bigger area than moorings or anchors. Table 4 presents the assessment of the technologies by categorization of *S* parameter.

***z/d*: obstruction to water column**

Being *d* the water depth at location and *z* the draft of the wave energy device if floating or the extension from the sea bottom if bottom based (Folley et al. 2007), the $\left|\frac{z}{d}\right|$ parameter expresses the relative obstruction of the water column (vertical) by the device; *z* is positive for floating devices and negative for bottom based devices (Fig. 8). The absolute value of the obstruction parameter is included between 0 and 1, assuming it is never equal to 0 and equal to 1 for total obstruction of the water column. Being the water column the habitat of most sea species as well as the area for propagation of currents and wave, and being the presence of wave energy devices a possible degradation of the natural conditions of this environment, it seems reasonable to introduce a factor that alerts on the restrictions that these receptors may run into.

This parameter is also directly related to the operation phase of the devices more than any other phase.

Depending on the *z/d* parameter, the WECs can be classified as follow:

- Little obstructive, for $0 < \left|\frac{z}{d}\right| \leq 0.1$
- Obstructive, for $0.1 < \left|\frac{z}{d}\right| \leq 0.3$
- Very obstructive, for $\left|\frac{z}{d}\right| > 0.3$

The *z/d* parameter has direct consequences on the following receptors:

- Water column spices
- Costal processes
- Navigation and fishery
- Local communities (recreational activities)

It must be noticed that it is difficult to define the obstruction parameter for onshore devices so that we will assume that for those devices *z/d* < 0.1. This seems reasonable as onshore devices have relevant impact only on the above receptors listed also for the *D* parameter.

It is clear that currents and waves propagates with different mechanisms at different depths and that it is relevant if the partial obstruction interests the lower or higher part of the water column. Consequences on the higher or lower obstruction of the water column on the EI are anticipated in the assessment table.

The major impacts occur for very obstructive devices. Water column species, from marine mammals to fish are likely to be seriously affected by the presence of the devices, especially considering parks installations. Behaviours such as avoidance or permanent loss of habitat are of major concern. Considering that such installation are suitable mainly for offshore locations, the impact on coastal processes and local communities is considered moderate but major for fishery and navigation.

For obstructive devices, also considered suitable only for offshore installations, the effect on local communities is also moderate but minor on coastal processes taking into account a smaller water column obstruction. The expected effect on navigation and fishery is moderate or major fro bottom based and floating structures respectively.

For bottom based little obstructive devices the impacts on the listed sensitive receptors minor or negligible. For floating little obstructive devices, impact in navigation is major but on coastal processes, water column spices and local communities is minor or negligible. The last statements for little obstructive devices are valid for offshore devices. The results of the classification on the assessment are presented on Table 5.

***w/a*: obstruction to sea surface**

It is important to remember that the final goal of the sector is to realize wave energy farms and extract bigger quantities of energy. In this prospective the EIA must take into account impacts related to its extension, like conflict of utilization of the sea resource with other sectors, the hydrodynamic processes, sediment distribution and movement, routes of large sea spices such as mammal.

Being z/d a parameter that refers to two dimensional conditions, it seems important to mention the introduction of another parameter relevant for wave energy farms involving installation of a number of devices. Being *a* the total area occupied by the pack and *w* the sum of the areas (above view) of the structures, the parameter *w/a* expresses the horizontal obstruction of wave farm installations and implements the *z/d* parameter. This parameter is related only to offshore or intermediate devices, being *w*=0 for onshore devices. The parameter *w/a* is expected to have potential influence in the following receptors:

- Navigation and fishery
- Interference with marine animal movements

- Coastal processes
- Local communities

P: Power takeoff system

The power takeoff systems in wave energy involve moving parts, either directly activated by wave motion (hydraulic ram, elastomeric hose pump and air turbines) or by wave energy potential (hydroelectric turbines) and a system to convert the mechanical energy into electricity (generators). The sage of conversion are obviously technology dependent and so their expected impact on receptors.

The main disturbance to the environment derived by the power takeoff of the devices is caused by noise and accidental oil spillage.

In first attempt it is possible to address these issues to specific power takeoffs. Hydraulic ram, hydroelectric turbines and air turbines are noisy and especially air turbine can be of high disturbance for local communities. Hydraulic ram and elastomeric hose pump system may generate oil spillage as consequence of malfunctioning. Being the underwater noise and the water quality interference subjected to large uncertainties, no summarizing table of assessment of expected environmental impacts will be presented but it is important to address the above mentioned issues and relevant baseline studies are conduct supported by appropriate monitoring programs.

6. Validation of assessment method

As already mentioned, because of the early stage of wave energy devices not many technologies underwent through a full EIA. Following the EIA of the AquaBuOY and Wave Dragon are used to validate the assessment method presented in this paper. The validation is presented in Tables 6 and 7 and therefore is a summary, often too simplified, of the extensive and detailed work of the

two developers (Weinstein et al. 2007 and Russel 2007). It is anyway believed that the Tables 6 and 7 report the relevant conclusions of the work of the developers correctly.

It emerges that the assessment method succeeds on addressing the relevant parameters for the environmental impact of WECs. Also the classification of devices depending on the introduced parameters correctly addresses the impact of the specific devices. The main discrepancies regard respectively the minor and negligible impact on water column spices and coastal processes claimed by AquaBuOY developer while based on the assessment of this paper major and moderate impacts are expected. This may be due to the small installation that was assessed in the EIA report (4 devices) while the assessment in this article is made keeping the large farm installation into mind. The same goes for the expected impact on navigation of the Wave Dragon device.

7. Suggestion for the EIA process

No clear indications from the Authorities exists on what to provide for the EIA of wave energy devices as for them is difficult to spot common approaches to such a enormous variety of technologies; for this reason the risk is that WE companies are dragged into a time consuming and too expensive process for the EIA and Consents process that may kill the project and consequently the technology. The case of Wave Dragon is eloquent: the developer had to take care of communication to the public, Statutory Consultees and Consenting Bodies. The process has been prohibitively slow. Coordination of the interlocutor is required.

As argued in the introduction, at European level there is not frame or body in place to handle the applications for wave energy deployment. This increases the risk that conflicts arises from the communication and thereby the risk that there will be a lack of coordination among developer, consenting bodies, authorities and statutory consultees. In the other hands, the presence of such a body in Denmark, represented by the Energy Agency, demonstrated to be beneficial to the process. It is then suggested that not the developer but a competent governmental body is in

charge of the coordination among the consulters (Fig. 9). Companies in the market are relatively small, and so are the capitals available to run the business, relying on private investors, and funds from different R&D projects. Many if not all the companies behind the different technologies present the same history: after 5-10 years of research and development, pilot plants and prototypes are being constructed. At this point the developers had spent at least 15 mill Euro in average (Kofoed et al. 2008).

8. Conclusions

Selection of relevant parameters for EIA of wave energy converters has been made; those are:

1. *D* parameter, indicating the distance of the installation from shore
2. *S* parameter, indicating the kind of element used for stabilizing the device.
3. *z/d* parameter, indicating the relative water column obstruction (vertical) caused by the presence of the device
4. *w/a* parameter, indicating the relative horizontal obstruction of a wave energy farm and.
5. *P* parameter, indicating the kind if power takeoff utilized in the installation.

Classifications of wave energy devices have been made for each parameter. It is possible to assess the EI of WECs by the presented classification. The impact assessment on receptors that are present in more than one table must be carefully considered as result of interaction of different relevant parameters.

All the relevant receptors involved in WE are represented in the impact assessment tables and the expected impacts are quantified based on the classification. Beneficial impacts regard artificial reef effects, save in CO_2 emissions or improvement of local economy or local community life. For devices built on breakwater the improvement of the civil structure and its functions is also a beneficial impact. Those effects have not been listed as they are very project dependent or subjected to uncertainties, (see, for example, artificial reef effects´ paragraph).

Suggestion for the creation of a management body between the developers and the authorities responsible for the consent of wave energy deployment has been made arguing the relative benefits based on the Danish case.

It is demonstrated that the above listed tools can improve the EIA process for WECs.

References:

Blaxter, J.H.S., Hoss, D.E., 1979. The effect of rapid changes of hydrostatic pressure on the Atlantic herring Clupea harengus L. II. The response of the auditory bulla system in larvae and juveniles. Journal of Experimental Marine Biology and Ecology 41, 87-100.

Boehlert, G. W., MacMurray, G.R., Tortorici, C. E. (eds.), 2008. Ecological effects of wave energy development in the Pacific Northwest. U. S. Department of Commerce, NOAA Technical memorandum NMFS-F/SPO-92. 174 p.EU Commission (2001): Guidance on EIA Scoping. Luxemburg

Cashmore Matt, (2004): The Role of science in environmental impact assessment: process and procedure versus purpose in the development of theory. Environmental Impact Assessment review 24: 403-426. Elsevier, UK

Coyle, A. M., 2007. Scroby Sands Stakeholder Dialogue, Presentation by Powergen. 1-16.

Cruz J.; Gunnar M., Barstow S., Mollison D. (2008), Joao Cruz, ed., Green Energy and Technology, Ocean Wave Energy, Springer Science+Business Media, pp. 93, ISBN 978-3-540-74894-6)

Dong Energy, Vattenfall, Danish Energy Authority and the Danish Forest and Nature Agency, 2006. Danish Offshore Wind – key Environmental Issues. November. 144 pp.

Engineering Committee on Oceanic Resources — Working Group on Wave Energy Conversion (2003), John Brooke, ed., Wave Energy Conversion, Elsevier, pp. 7, ISBN 0080442129;

Folley M., T.W.T. Whittaker and J. van´t Hoff 2007: “The design of small seabed-mounted bottom-hinged wave energy converters”. Proceeding at the 7th European Wave and Tidal Energy ConferenceEWTEC07.

Hansen Lars K., Christensen Lars & Sørensen Hans Chr. 2003: Experiences from the Approval Process of the Wave Dragon Project Proceedings from the 5th European Wave Energy Conference, University College Cork, Ireland 2003

Harrington Nick and Andina-Pendás Inma 2007: Environmental Impact and Appraisal – Gaining Planning Consent for the South West of England Wave Hub. Proceeding at the 7th European Wave and Tidal Energy Conference (EWTEC07).

Hawkins, A., 2006. Assessing the impact of pile driving upon fish. In: Irwin, C., Garrett, P., McDermott, K. (Eds.), Proceedings of the 2005 International Conference on Ecology and Transportation, Center for Transportation and the Environment, North Carolina State University, Raleigh, NC, pp. p. 22. (Abstract).Holm, K.J., Burger, A.E., 2002. Foraging Behavior and Resource Partitioning by Diving Birds During Winter in Areas of Strong Tidal Currents. Waterbirds 25, 312-325.

Hoss, D.E., Blaxter, J.H.S., 1982. Development and function of the swimbladder-inner ear-lateral line system in the Atlantic menhaden, Brevoortia tyrannus (Latrobe). Journal of Fish Biology 20, 131-142.

Iain Russell, Wave Dragon 2007: Commercial Wave Energy Device, Environmental Statement Volume 2: Environmental Statement, by PMSS on behalf of Wave Dragon Wales Ltd.

Kofoed, Jens Peter ; Frigaard, Peter ; Hernandez, Lorenzo Banos. In 2008: “Development of Wave Energy Devices - Past, Present and Future : The Danish Case.” / ECOR Symposium. Engineering Committee on Oceanic Resources: Marine Renewable Energy: St. Johns, Canada, Oct. 23-24, 2008 : Specialists Panel. Institute for Ocean Technology, Canada, 2008. 12 p.

Kørnøv L., Thrane, M., Remmen, A., Lund, H. (2007): Tools for sustainable development, 2007, Aalborg Universitetsforlag p. 355-399.

Ladenburg, J., 2006. Attitudes Towards Wind Power Development in Denmark. Institute of Food and Resource Economics, Denmark

Langhamer, O., Wilhelmsson, Engstrom, J., 2009. Artificial reef effect and fouling impacts on offshore wave power foundations ad buoys – a pilot study. Estuarine Coastal and Shelf Science, 82: 426-423.

Margheritini, Lucia ; Vicinanza, Diego ; Frigaard, Peter. SSG Wave Energy Converter: Design, Reliability and Hydraulic Performance of an Innovative Overtopping Device. / I: Renewable Energy. 2009a ; vol. 34, nr. 5, 01.11.2008. s. 1371–1380

Michel, J. Dunagan, H., Boring, C., Healy, E., Evans, W., Dean, J. M., McGillis, A., Hain, J., 2007. Worldwide Synthesis and Analysis of Existing Information Regarding Environmental Effects of Alternative Energy Uses on the Outer Continental Shelf. U.S. Department of the interior, Minerals Management Service, Herndon, VA, MMS OCS Report 2007-038. 254 p.

Murray, R.W. 1974. The ampullae of Lorenzini, In Electroreceptors and other specialized organs in lower vertebrates, (ed. A. Fessard): 125-146, pringer-Verlag, New York.

Pedro Beirão, Mario J. G. C. Mendes, Duarte Valerio and José Sá da Costa: “Control of the Archimedes Wave Swing using Neural Networks”. Proceeding at the 7th European Wave and Tidal Energy Conference (EWTEC07).

Russell Iain and Sorensen Hans Chr. 2007: Wave Dragon : Results From UK EIA and Consenting Process. Proceeding at the 7th European Wave and Tidal Energy Conference (EWTEC07).

Sørensen H. C. and Russel I, (2008) Work Session 5 Environmental, Economics, Development Policy and Promotion of Opportunities, EU Commission, Luxemburg, Project no. 502701, CA-OE, Co-ordination Action on Ocean Energy

The Council of the European Union (1985): Council Directive 85/337/EEC of 27 June 1985 on the assessment of effects of certain public and private projects on the environment

The Council of the European Union (1997): Council Directive 97/11/EEC of 3 March 1997 amending Directive 85/337/EEC on the assessment of effects of certain public and private projects on the environment

Wardle, C.S., Carter, C.G., Urquhart, G.G., Johnstone, A.D.F., Ziolkowski, A.M., Hampson, G., Mackie, A.M., 2001. Effects of seismic air guns on marine fish. Continental Shelf Research 21, 1005-1027.

Weinstein Alla, Nielsen Kim, Zandiyeh Kambiaz, Bensted Jonathan 2007: The AquaBuOY Hose-Pump – theory and experimental results. Proceeding at the 7th European Wave and Tidal Energy Conference (EWTEC07).

Wood Christopher, (2003) Environmental Impact Assessment. A Comparative Review, Pearson/Prentice Hall p. 31-41.

Zubiate L., Villate J.L., Torre-Enciso Y., Soerensen H.C., Holmes B., Panagiotopoulos M., Neumann F., Rousseau N., Langston D. 2005: Methodology for site selection for wave energy projects. Proceeding at the 8th European Wave and Tidal Energy Conference (EWTEC08).

Table

Table 1. Summary of EMEC environmental impact assessment guidelines for WECs.

	Ecological effects	Socio-economic effects
Major	Degradation to the quality or availability of habitats and/or wildlife with recovery taking more than 2 years *(e.g. widespread seabed excavations, erosion)*	Change to commercial activity leading to a loss of income or opportunity beyond normal business variability/risk potential short term effect upon public health / well-being, real risk of injury *(e.g. loss of important fishery area, dive site, creation of seabed or floating debris)*
Moderate	Change in habitats or species beyond natural variability with recovery potentially within 2 years *(e.g. seabed excavations in a small area)*	Change to commercial activity leading to a loss of income or opportunity within normal business variability/risk Possible but unlikely effect upon public health/well-being. Remote risk of injury *(e.g. small exclusion area away from or small part of actively used areas)*
Minor	Change in habitats or species which can be seen and measured but is at same scale as natural variability *(e.g. low level noise from devices)*	Possible nuisance to other activities and some minor influence on income or opportunity. Nuisance but no harm to public *(e.g. short term congestion at harbours)*
Negligible	Change in habitats or species within scope of existing variability and difficult to measure or observe *(e.g. localized avoidance of structures by wildlife)*	Noticed by, but not a nuisance to other commercial activities. Noticed by but no effects upon the health and well-being of the public *(e.g. additional shipping at sea)*
No interaction	None	None
Positive	An enhancement of ecosystem or popular parameter *(e.g. enhance biodiversity, save in CO2 emissions)*	Benefits to local community *(e.g. contract to use local skills and expertise on a project)*

Table

Table 2. Example of stressors and activities in general impact matrix of WECs.

RECEPTOR / ACTIVITY	Geology and Geomorphology	Sediment distribution and movement	Hydrography and hydrographic processes	Landscape and seascape	Atmosphere	Coastal spices	Water column spices	Seabed spices	Sea bird	Commercial sea and harbor uses	Local economy	Recreation and communities	Archeology
Installation													
Mooring/foundation system													
Electric transmission infrastructure													
Vessel presence													
Operation and maintenance													
Mooring/foundation system													
Electric transmission infrastructure													
WE device presence													
Heating and cooling system													
Chemical coating													
Noise emissions													
Vibrations													
Light													
Decommission													
Vessel presence													
Mooring/foundation removal													
Electric transmission infrastructure removal													
Accidental events													
Oil spill													
Sinking													
Uncontrolled floating													
Collision													

Table 3. Assessment table based on Distance from shore.

D parameter	Local communities	Coastal processes and coastal spices	Navigation and fishery
Onshore devices	**Major**	**Major**	**Nil**
Intermediate water devices	**Moderate**	**Moderate**	**Major**
Offshore devices	**Negligible**	**Minor**	**Major**

Table 4. Assessment table based on the type of Stabilizing element.

S parameter	Benthonic habitats	Geology	Archeology	Water column spices
Simple moorings	**Minor**	**Negligible**	**Minor**	**Minor**
Complex moorings	**Minor**	**Negligible**	**Minor**	**Moderate**
Gravity foundations	**Moderate**	**Moderate**	**Moderate**	**Negligible**
Piles	**Major**	**Major**	**Major**	**Minor**

***z/d*: obstruction to water column**

Table 5. Assessment table based on the obstruction parameter.

z/d parameter	Water column spices	Navigation and fishery	Coastal processes	Local communities
Little obstructive z>0	**Minor**	**Major ***	**Minor ****	**Negligible****
Little obstructive z<0	**Minor**	**Minor**	**Minor**	**Negligible**
Obstructive z>0	**Moderate**	**Major**	**Minor**	**Moderate**
Obstructive z<0	**Moderate**	**Moderate**	**Minor**	**Moderate**
Very obstructive z>0	**Major**	**Major**	**Moderate**	**Moderate**
Very obstructive z<0	**Major**	**Major**	**Moderate**	**Moderate**

*no interaction for onshore devices **Major for onshore devices

Table 6. Comparative analysis between the assessment of the AquaBuoY installation in Oregon derived from the classification presented in this paper (expected impacts) and the assessment presented by the developer (stated impacts).

AquaBuOY			
Classification		Expected impacts	Stated impacts
D: Offshore device	☺	1. Negligible impact on local communities. 2. Minor impact on coastal processes. 3. Major impact on navigation.	1. Development of shore station represents a permanent visual impact of the project but because of dimensions and distance from shore of the buoys this impact will not be significant. 2. Negligible impact on coastal processes. 3. Fishing and navigation exclusion zone needs to be established.
S:Complex mooring	☺	1. Minor impact on benthonic habitats. 2. Negligible impact on Geology. 3. Minor impact on archaeology. 4. Moderate impact in water column spices.	1. Negligible impact on benthonic spices. 2. No substantial changes in the bathymetry or temporary for deployment phase. 3. - 4. It is unknown if the mooring system may represent a point of entanglement for marine life especially for farms.
z/d: Very obstructive, *z*>0	☺	1. Major impact on water column spices. 2. Major impact on navigation and fishery. 3. Moderate impact on costal processes. 4. Moderate impact on local communities.	1. Minor impact on water column spices. 2. Fishing and navigation exclusion zone needs to be established. 3. Negligible impact on costal processes. 4. No detrimental impact on recreational activities.
P: elastomeric hose pump system	☺	1. May generate oil spillage in case of malfunctioning.	1. The system does not use hazardous materials. As such project operation will not affect water quality.

Table 7. Comparative analysis between the assessment of the Wave Dragon installation in Wales derived from the classification presented in this paper (expected impacts) and the assessment presented by the developer (stated impacts).

Wave Dragon			
Classification		Expected impacts	Stated impacts
D: Intermediate water device	☺	1. Moderate impact on local communities. 2. Moderate impact on coastal processes. 3. Major impact on navigation.	1. Moderate. 2. Minor because of short term effects. 3. Negligible.
S:Complex mooring	☺	1. Minor impact on benthonic habitats. 2. Negligible impact on Geology. 3. Minor impact on archeology. 4. Moderate impact in water column spices.	1. Minor to moderate. 2. Not issue of concern. 3. Negligible effect. 4. Minor to moderate impact.
z/d: Little obstructive, *z*>0	☺	1. Minor impact on water column spices. 2. Major impact on navigation and fishery 3. Minor impact on costal processes. 4. Negligible impact on local communities.	1. Minor impact. 2. Minimal. 4. Minor and short term effects. 3. Negligible.
P: hydroelectric turbines	☺	1. Noise.	1. Minor significance.

Figure

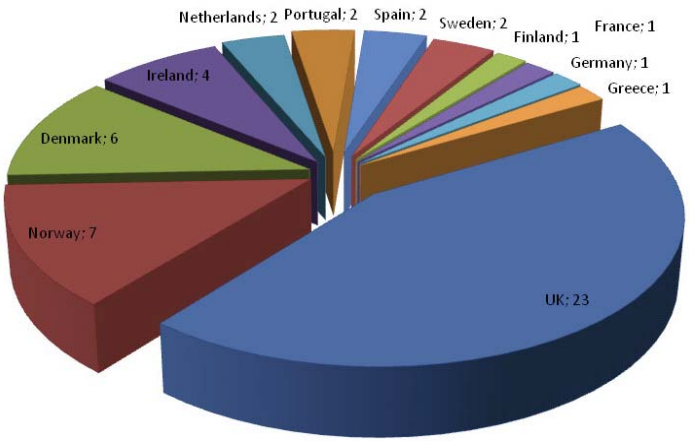

Figure 1. Number of wave energy technologies developed per Country in Europe. Total is 52, with 3 technologies being in collaboration between 2 Countries, for a total of 49 different concepts in EU.

Figure

Figure 2. From the left: Pelamis Portugal, Wave Dragon Nissum Bredning, DK, AquaBuOY Oregon, US.

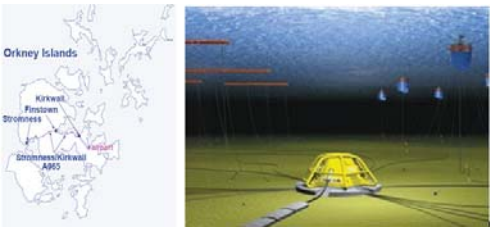

Figure 3. On the left, EMEC facility location. On the right, offshore underwater electrical “socket” of the Wave Hub Project.

Figure

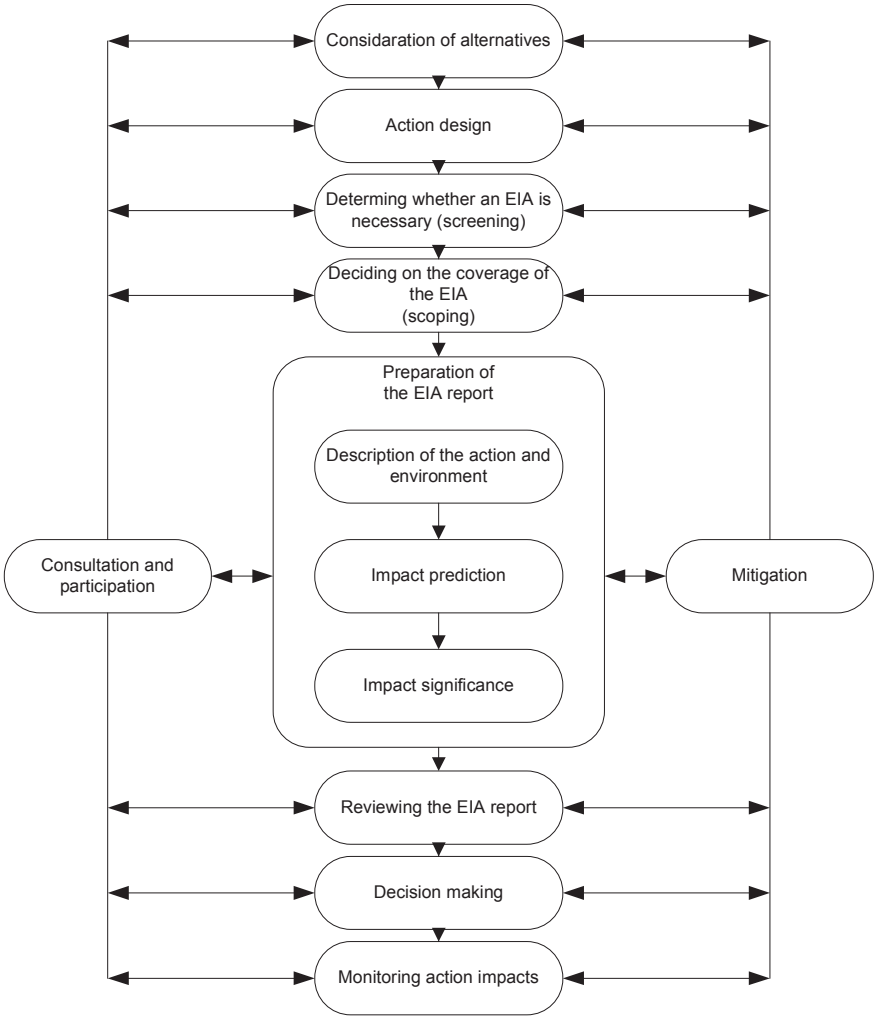

Figure 4. The environmental impact assessment process (after Wood 2003 p.7)

Figure

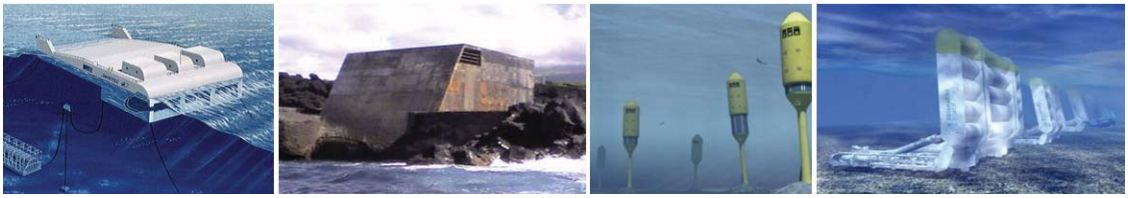

Figure 5. From the left: Mighty Wale, Pico plant, wave Swing and Oyster.

Figure

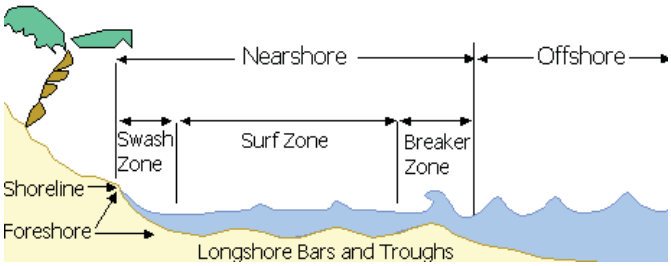

Figure 6. Illustration from the Division of Nearshore Research (http://lighthouse.tamucc.edu/Main/HomePage).

Figure

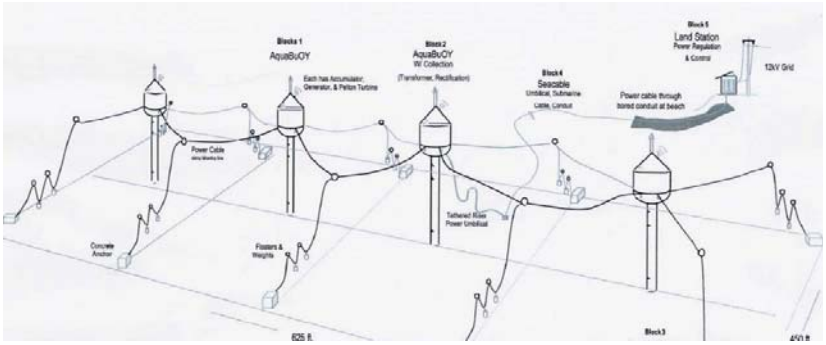

Figure 7. AquaEnergy Group Ltd. AquaBuOY multi-buoy configuration, dimensions and distances not in scale.

Figure

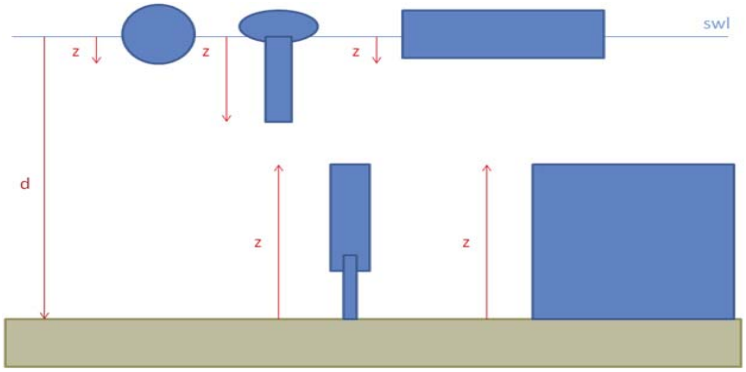

Figure 8. Definition sketch for z for different kind of devices.

Figure

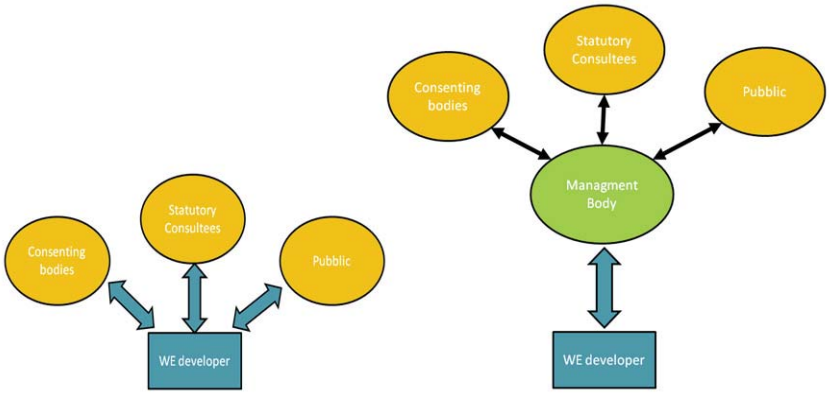

Figure 9. Schematics of the existing (left) and suggested (right) structures for managing the applications for deployment of WECs.

Other author´s publications related to the SSG device

Case study for Breakwater SSG application. / Margheritini, Lucia ; Kofoed, Jens Peter. Aalborg : Aalborg University. Department of Civil Engineering, 2008. 39 s. (DCE Contract Reports; 44).

Design, Construction, Reliability and Hydraulic Performance of an Innovative Wave Overtopping Device. / Frigaard, Peter ; Andersen, Thomas Lykke ; Margheritini, Lucia ; Vicinanza, D.. I: Transportation Engineering/Coastal & Harbour Engineering/Hydraulic Engineering : Proceedings of the 8th International Congress on Advances in Civil Engineering, ACE 2008, Famagusta, North Cyprus, 15-17 September 2008. Famagusta, North Cyprus : Eastern Mediterranean University Press, 2008. s. 551-558.

Estimation of wave conditions at Svåheia SSG pilot site. / Kofoed, Jens Peter ; Margheritini, Lucia ; Stratigaki, V. ; Troch, P.. Aalborg : Aalborg University. Department of Civil Engineering, 2008. 154 s. (DCE Contract Reports; 52).

Parametrical numerical study on Breakwater SSG application. / Margheritini, Lucia ; Kofoed, Jens Peter. Aalborg : Aalborg University. Department of Civil Engineering, 2008. 45 s. (DCE Contract Reports; 47).

Preliminary geometrical design for Breakwater SSG application at Gorliz, Spain. / Margheritini, Lucia ; Kofoed, Jens Peter. Aalborg : Aalborg University. Department of Civil Engineering, 2008. 11 s. (DCE Contract Reports; 51).

3D tests on overtopping for SSG wave energyconverter. / Margheritini, Lucia ; Kofoed, Jens Peter. Aalborg : Aalborg University. Department of Civil Engineering, 2007. 23 s. (DCE Technical Report; 18).

Instrumentation of the model in scaled 1:10 to prototype of the AquaBuOY wave energy converter : real sea testing at Nissum Breding, Denmark. / Margheritini, Lucia ; Frigaard, Peter. Aalborg : Aalborg University. Department of Civil Engineering, 2007. 7 s. (DCE Technical Report; 15).

Model tests on overall forces on the SSG pilot plant. / Margheritini, Lucia ; Morris, Alex. Aalborg : Aalborg University. Department of Civil Engineering, 2007. 26 s. (DCE Technical Report; 31).

In-situ Measurements of Surface Elevations in Tail Water Channel for SSG Pilot Plant at Kvitsøy. / Kofoed, Jens Peter ; Margheritini, Lucia. Aalborg : Aalborg University. Department of Civil Engineering, 2006. 21 s. (DCE Contract Report; 10).

Specification for Instrumentation : Data Acquisition and Control System for SSG Pilot Plant at Kvitsøy : Norway. / Margheritini, Lucia ; Kofoed, Jens Peter. Aalborg : Aalborg University. Department of Civil Engineering, 2006. 67 s. (DCE Technical Report; 19).

Specification for Instrumentation : Data Acquisition and Control System for SSG Pilot Plant at Kvitsøy : Norway. / Margheritini, Lucia ; Kofoed, Jens Peter. Aalborg : Aalborg University. Department of Civil Engineering, 2006. 19 s. (Hydraulic and Coastal Engineering; 46).

Status and performance of the SSG Wave Energy Converter. / Margheritini, Lucia ; Kofoed, Jens Peter ; Frigaard, Peter. I: 4th Coordinated Action on Ocean Energy (CA-OE) Workshop : Performance Monitoring of Ocean Energy Systems. 2006. 10 s.