Designing the ShoreSWEC as a breakwater and wave energy converter

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Abstract
This paper presents the results of on-going research at Stellenbosch University on a novel wave energy converter (WEC) device called the ShoreSWEC. The ShoreSWEC forms part of a caisson breakwater and is essentially an adaptation of South Africa's first WEC device developed at Stellenbosch University in the seventies called the SWEC. A site evaluation and selection procedure was conducted and a location best suited for the deployment of the ShoreSWEC was identified in Table Bay. In order to effectively design the ShoreSWEC to optimally convert wave energy from the dominant wave conditions at the deployment location and to ensure that its structure will survive design storm conditions statistics of local short-term and long-term sea states are required. Due to the lack of recorded or modelled wave data at the site a numerical wave model was developed to simulate wave propagation from offshore into Table Bay. The transferred wave data was analysed to determine the design storm (extreme) conditions and the operational wave conditions. A breakwater design procedure based on coastal engineering design codes was performed to determine device dimensions required to ensure structure stability during extreme storm events. Further numerical modelling and experimental testing will be conducted to define the best device layout for optimal energy conversion from the operational wave conditions.

Keywords: Breakwater wave energy converter

1. From SWEC to ShoreSWEC

The Stellenbosch Wave Energy Converter device or SWEC was developed and tested at Stellenbosch University by Deon Retief and his team from the late seventies to early eighties. Retief et al conducted extensive experimental tests on two- and three-dimensional scaled models in the hydraulic laboratory of Stellenbosch University, but unfortunately no sea trails were ever perform and the project was shelved after the oil price stabilised (Retief 1982). The capital cost, potential environmental impacts and permitting requirements were mayor barriers in the way of full-scale deployment of the standalone SWEC device. In recent times an adaption of the SWEC, patented by Stellenbosch University, has been proposed to overcome these barriers. By incorporating the SWEC into the structure of a caisson breakwater in an existing port development the capital cost of the project is shared between the breakwater and wave energy device. Deployment of the device inside an existing port development also simplifies the permitting requirements and environmental impact assessments. This SWEC adaptation is aptly named the ShoreSWEC and comprise of a series of hollow, steel reinforced concrete chambers with openings below the water surface to allow wave driven flow to enter and exit the chambers. Oscillation of the water column inside the chamber forces entrapped air through a one-way valve to or from a high or low pressure conduit which runs along the length of the device. As a wave propagates along the device the chambers are sequentially activated and the generated airflow accumulated is used to drive a turbine/generator set to produce electricity (refer to Figure 1 for the patent drawings of the ShoreSWEC).

The main objective of this study is to determine the wave energy conversion potential of the ShoreSWEC and to define its structural geometry and dimensions that will ensure maximum efficiency and survivability in South African oceanic waters. A thorough understanding of the prevailing wave conditions at the deployment location is of utmost importance and therefore a numerical wave model was developed to simulate wave propagation from offshore to a selected site in Table Bay. Development of the numerical wave model and its output is presented in the following sections.

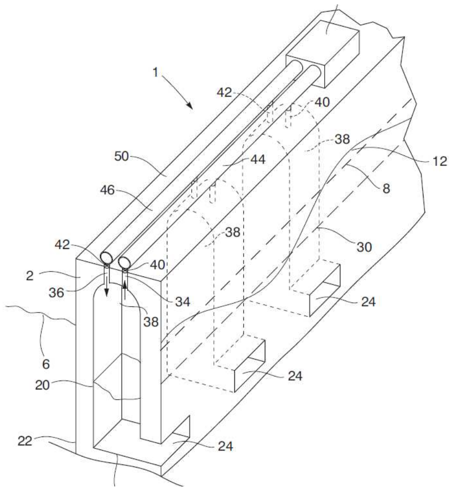

Figure 1: ShoreSWEC chambers, valves pressure conduits and turbine/generator housing

2. Wave conditions at Granger Bay

In order to determine statistics of long term wave conditions at the selected site in Table Bay a numerical wave model, called SWAN, was used to simulate wave propagation from offshore to the site. SWAN, a third-generation wave model developed at Delft University of Technology, solves action density equations over a computational domain to compute the change of wave height and direction due to the variation in depth as waves travel from deep sea into shallower water (refer to the SWAN user manual for more information). The offshore wave data used as input into the SWAN model is output from global wave models of the U.S National Weather Service's National Centers for Environmental Prediction (NCEP http://www.ncep.noaa.gov/ 24/10/2011). The historic output of global wave models (which are normally validated and adjusted if necessary), is known as hindcast wave data.

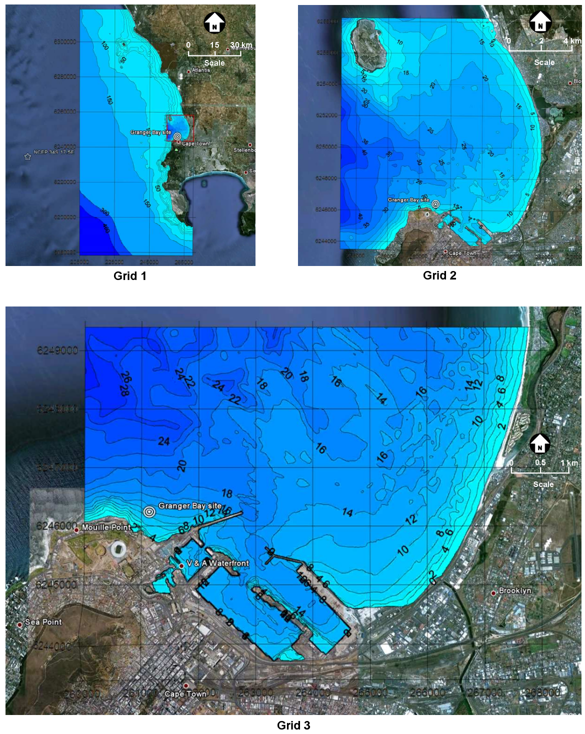

Figure 2: Computational domains of numerical wave model of Table Bay

Output from the SWAN wave model was used to effectively transfer 11 years of hindcast NCEP wave data from deep water to the site in Table Bay (refer to Figure 2 for the computational domains used by the SWAN wave model). The geometric layout of the ShoreSWEC will be designed to optimally convert wave energy from the prevailing/operational wave conditions as derived from the transferred hindcast wave data. The derivation of the operational wave conditions and a discussion of these conditions follow.

2.1. Operational wave conditions

The NCEP hindcast wave data is a collection of three hourly wave records of significant wave height (H_s), peak wave period (T_p) and peak wave direction (D_p) for the recording period of February 1997 to August 2008. The data was analysed by binning it according to T_p (1s intervals for T_p = 3 to 19s) and D_p (22.5° intervals for D_p = 0 to 237.5°). For each combination of T_p and D_p an average H_s was determined and simulated with SWAN over the computational domains as shown in Figure 2. The depth induced variation in H_s and D_p as computed by SWAN was then used to transfer the entire NCEP wave data set, effectively generating 11 years of wave data at site of interest. From the transferred wave data the operational wave conditions were derived.

2.1.1. Peak wave period

As waves travel from deep to shallower water its period is unaffected by the change in water depth and therefore it was assumed that the NCEP values of peak wave period will remain constant over the computational domain. Upon analysis of the NCEP wave period data it was found that the dominant values ranges from 9 to 13 s with the most frequently occurring value of 11 s occurring for 26% of the 11 year recording period. This corresponds to wave period data measured at the Slangkop wave recording station (refer to Figure 3).

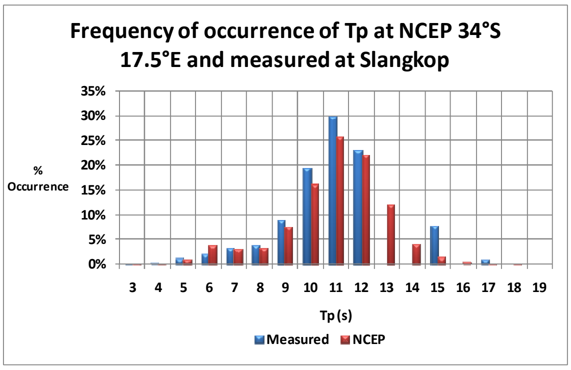

Figure 3: Frequency of occurrence of T_p at NCEP 34°S 17.5°E and Slangkop recording station

The prevailing wave period greatly influences the optimal design of the ShoreSWEC. The total device length must be greater than the wavelength associated with wave periods of 9 to 13s which in 10 m water depth is approximately 80 to 120 m. This will ensure a variation in pressure and consequent airflow between chambers. To utilise resonance the geometry of the device and its resulting natural period must match the operational wave period. Wave power is a function of wave period and therefore longer period waves have more power available for generation. A further important design consideration of the ShoreSWEC is its orientation relative to the dominant wave direction.

2.1.2. Peak wave direction

The transferred peak wave directional data as simulated with SWAN was analysed and it was found that there is very little variability in the peak direction of waves as it reaches the site. For 95% of the recording period waves approach the site from the northwest and the device must therefore be longitudinally orientated towards the northwest to ensure wave travels along its length. The significant wave height is the last operational wave parameter analysed at the site.

2.1.3. Significant wave height

It is important to determine the generation capacity of the ShoreSWEC at the site from the dominant wave height conditions and to optimise the device layout for maximum conversion efficiency from these dominant conditions. In general the values of significant wave height are low due to the sheltering effect of Mouille Point on the site from the dominant southwesterly waves. The most frequently occurring wave heights range from 0 to 1 m with the most frequently occurring wave height of 0.4 m occurring 36% of the recording period (refer to Figure 4 for percentage occurrence of H_s at the site).

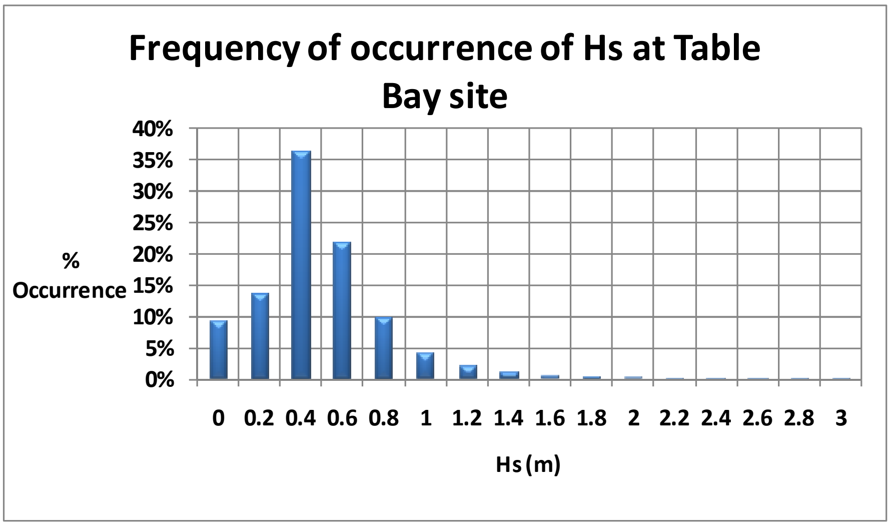

Figure 4: Frequency of occurrence of Hs at the deployment location

Currently a 3D computational fluid dynamic (CFD) model is being developed to evaluate the conversions efficiency of the device from the operational wave conditions. A scaled model is also going to be built and tested under these conditions in the wave flume to further optimise the design layout. The width and height of the ShoreSWEC structure must be sufficient to ensure stability against sliding, overturning and to avoid excessive overtopping during extreme storm events. SWAN was again used to simulate design storm events over the computational domain in an effort to determine design wave conditions at the deployment location.

2.2. Design wave conditions

The ShoreSWEC's target operational design life is 50 year which is typical for caisson breakwaters. The design wave height must have a longer return period to ensure an acceptable level of risk and structural stability during extreme storm events. According to Rossouw (1989) the most probable value of wave height with a return period of 100 years for the South African coastline is:

$$H_{m0_{100\,years}} = 12.0\,m$$

H_{m0} above is equivalent to significant wave height. The probability that this 100 year wave will be exceeded in any 100 year period is 63%. Due to this relatively high probability the British Standard for breakwater design (BS 6349-7:1991) recommends that the stability of the structure must be checked for a design wave with a 5% probability of exceedance. For the 50 year design life of the ShoreSWEC the 5% probability of exceedance corresponds to a design wave with a recurrence interval of 1000 years also known as the 1000 year wave. Rossouw (1989) found the 1000 year wave for South Africa to be:

$$H_{m0_{1000\,years}} = 13.92\,m$$

The 1 in 11 year design wave heights were determined offshore from the NCEP data for each directional bin. It was found that the greatest offshore wave height in the 11 year period is 10.31 m from westsouthwest. The maximum wave height for each directional bin was expressed as a percentage of the westsouthwest storm and multiplied by the 1000 year wave height. SWAN runs were done for the 1000 year waves from each directional bin and it was found that the design storm from the northwest gave the highest wave height at the site of 5.5 m. The design wave period was determined as prescribed by Det Norske Veritas (1977) as a function of the design wave for each direction by:

$$3.6\sqrt{H_{m0}} < T_p < 5.5\sqrt{H_{m0}}$$

The design peak wave period for the northwest design storm was calculated from the upper limit of the equation above as 15 s. The next step is to determine the width and height of the ShoreSWEC structure as prescribed in the caisson breakwater design procedure outlined in (BS 6349-7:1991) and the Coastal Engineering Manual (2006) for the design wave height and period conditions.

3. Breakwater design of ShoreSWEC

The width requirements of the ShoreSWEC will be assessed by first assuming conservatively it acts like an impermeable vertical face caisson breakwater. By adding an incline to the top part of the seaward structure greater stability against sliding and overturning is achieved, thus reducing the required width however more wave overtopping will occur. Patterson et. al. (2009) suggests modifications to the wave pressure distribution formulae to incorporate the effect of waves entering the capture chamber of the device. All three these approaches are discuss in further detail in the following sections.

3.1. Conventional vertical face caisson breakwater

Wave generated horizontal and uplift pressures and forces on a vertical wall structure can either be transmitted by non-breaking or breaking waves. Breaking waves cause an impulsive load which can be very large and it is therefore best to deploy the device in water depth greater than that of the breaker/surf zone. For this reason the ShoreSWEC will be deployed in approximately 10 m water depth which is beyond the extreme breaker zone.

Goda (1974) developed a design approach for estimating the pressure distributions and corresponding forces and overturning moments on vertical walls due to irregular waves (Figure 5 shows the horizontal and uplift pressure distribution). Goda's design approach as outlined in BS (6349-7:1991), CEM (2006) and Goda (2010) was used to determine the device dimensions required to give satisfactory factors of safety against sliding and overturning. Some basic assumptions were made:

Table 1: Assumptions for vertical wall caisson design

Assumption	Value	Comment
Recurrence interval	50 years	Typical for caisson breakwater
Design wave height	5.54m	1 in 1000 year storm from the northwest
Design wave period	15 s	As defined by Det Norske Veritas
Water depth	10 m	Near structure head
Storm surge	2.5 m	Includes 2 m tidal range
Wave direction	80°	Assume device is orientated from NW to SE

Further assumptions made relating to the rubble mound foundation, berm width (B_m) and freeboard (h_c) are shown below in Figure 5.

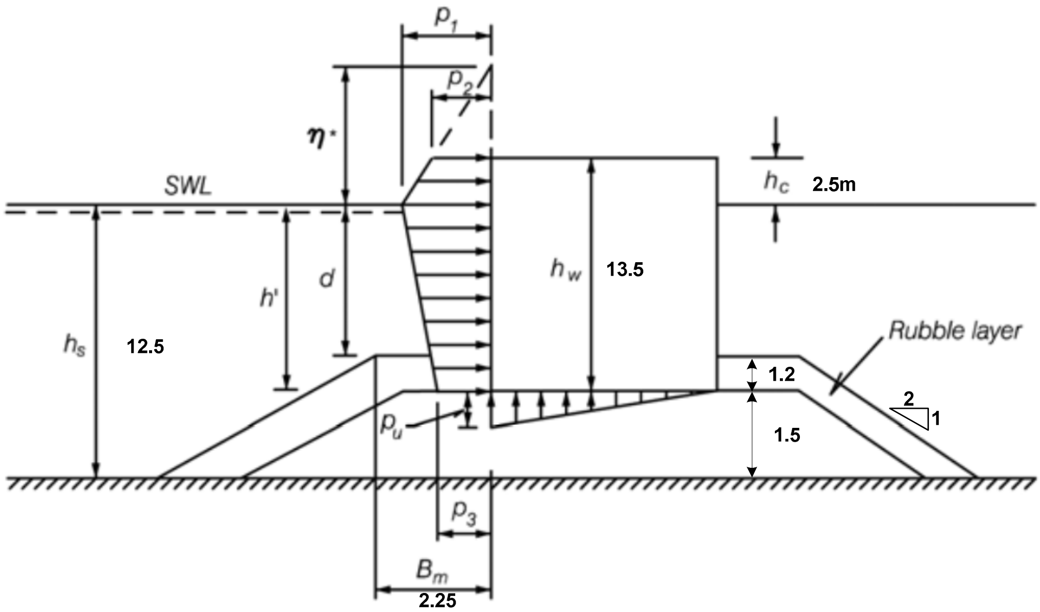

Figure 5: Goda formula for vertical face caisson breakwater in irregular waves (CEM 2006)

The horizontal and uplift forces were calculated and the safety factors against sliding, overturning and heal bearing pressures were evaluated. A structural width of 11.5 m gives a safety factor of at least 1.5 against sliding, overturning and heal bearing failure as recommended by BS (6349-7:1991). In an effort to reduce the required width and consequent material cost a sloped face caisson was also considered.

3.2. Sloped face caisson

A sloping seaward face caisson transmits wave forces horizontally as well as vertically downwards effectively reducing the uplift force. A smaller required width is therefore possible compared to a conventional vertical wall caisson however a greater height is needed to ensure acceptable levels of overtopping. Takahashi (1988) recommends that the crown height of a sloping structure must be at least the design wave height above the water level. A sloped face ShoreSWEC will be 0.5 m higher than a vertical wall one. Takahashi and Hosoyamada (1994) modified Goda's formulas which show that a 8 m wide structure will provide sufficient stability.

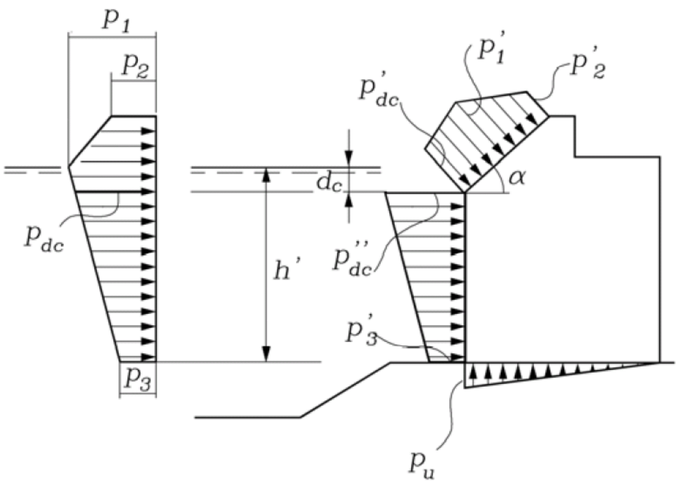

Figure 6: Wave loads on sloping top structures (Takahashi and Hosoyamada 1994)

Patterson et. al. (2009) argues that Goda's formulae does not take into account the impact of the collector chamber on the wave loading and proposed some modifications to the formulae.

3.3. Wave power breakwater

The uplift pressure generated underneath the structure in its permeable foundation is a result of the impermeable barrier the vertical wall of the caisson presents to incident waves. For a wave power caisson a portion of the incident wave energy will enter the capture chamber reducing the potential for pressure build up underneath the base of the structure. Patterson et. al. assumes that the uplift force will only be generated underneath the ballast chamber beyond the capture chamber as outlined in Figure 7.

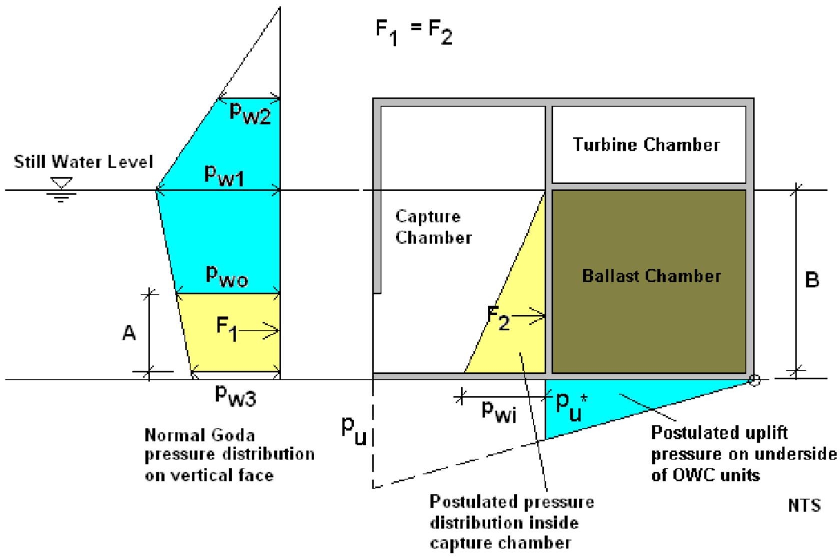

Figure 7: Modified Goda pressure distribution for OWC breakwater (Patterson et. al. 2009)

Applying Patterson's adaptation of Goda formulae to the ShoreSWEC design shows that a reduced width of 11 m and 7.5 m will provide sufficient stability for a vertical wall and sloped face structural geometry respectively.

4. Conclusions and recommendations

A numerical wave model was developed to determine the operational and extreme wave conditions at the deployment location. The structural geometry of the ShoreSWEC will be designed to optimally convert wave energy from the operational wave conditions through experimental tests and CFD modelling. The operational significant wave height at the site ranges from 0 to 1 m and typical values of peak wave period ranges from 9 to 13 s.

The width requirements of the ShoreSWEC were evaluated according to coastal engineering design codes for the design wave height and period conditions. Three design approaches were considered all based on the work of Goda (1973) to determine the pressure distribution on the structure of the device. Conservatively assuming the ShoreSWEC acts as a conventional vertical face caisson the required width as prescribe by Goda was calculated to be 11.5 m. Adding an incline to the seaward face of the structure adds stability and reduces the required width to 8 m. To incorporate the influence of waves entering the capture chamber Patterson et. al. modified Goda's formulae to reduce the uplift force. This reduces the required width for the vertical and inclined caisson to 11 and 7.5 m respectively. The final structural geometry of the ShoreSWEC will potentially be similar to that of Patterson et. al. as shown in Figure 8.

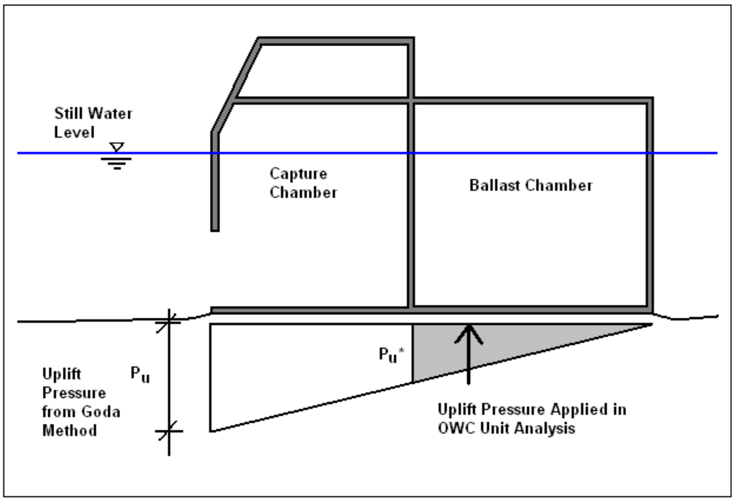

Figure 8: Potential layout of ShoreSWEC (Patterson et. al. 2009)

The next phase of the research project will focus on experimentally testing a scaled vertical wall and inclined face model of the ShoreSWEC in the wave flume under the operational wave conditions. A 3D CFD model will be developed in conjunction to validate the experimental results.

Acknowledgements

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