Estimation of Design Wave Loads on the SSG WEC Pilot Plant based on 3-D Model Tests

Kofoed, Jens Peter; Vicinanza, Diego; Osaland, Espen

Published in:
Proceedings of the Sixteenth (2006) International Offshore and Polar Engineering Conference

Publication date:
2006

Document Version
Publisher's PDF, also known as Version of record

Estimation of design wave loads on the SSG WEC pilot plant based on 3-D model tests

Jens Peter Kofoed
Aalborg University
Aalborg, Denmark
Diego Vicinanza
Second University of Naples
Aversa (Caserta), Italy
Espen Osaland
WAVEenergy AS
Aalgaard, Norway

ABSTRACT

This paper discuss wave loadings acting on the Wave Energy Converter (WEC) Seawave Slot-Cone Generator (SSG). The SSG is a new type of structure for wave energy conversion based on storing the incoming waves in several reservoirs placed one above the other. The wave forces on the main structure can be estimated using experiences from coastal protection structures, but the differences between the two types of structures are so large that more reliable knowledge on the wave forces is desired. Model tests were carried out to measure wave loads using realistic random 2D and 3D wave conditions. Pressure cells were placed in order to achieve information on impact/pulsating loadings. Data analysis identifies different structure response depending on wall geometry and location. Results discussed here derive from preliminary analysis conducted using only a part of the whole data set. The research is intended to be of direct use to engineers analyzing design and stability of the pilot plant under construction at Kvitsøy island, partly funded by the EU 6th framework program (WAVESSG).

KEY WORDS: SSG, breakwaters, 3D model tests, sloping walls, vertical walls, forces, pressures.

INTRODUCTION

As recently expressed by top European heads of state, the climate change is the largest challenge that we are facing today. In the short term, the Kyoto protocol and the RES Directive promote the use of Renewable Energy Sources (RES), and in the long term the decreasing reserves of fossil fuels will urge the use of RES.

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). Sea waves have one of the highest energy densities among the RES.

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

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. The water captured in the reservoirs will then run through the multi-stage turbine for electricity production (Fig. 1). The use of multiple reservoirs will result in a higher overall efficiency, compared to a single reservoir structure (Kofoed, 2002; Kofoed, 2005; Kofoed and Osaland, 2005).

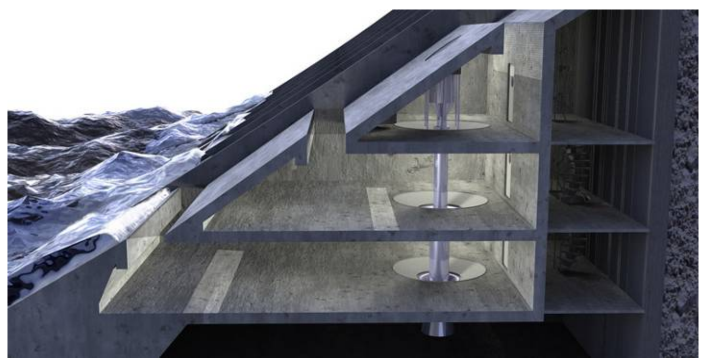

Figure 1: Scheme of Seawave Slot-Cone Generator (SSG).

WAVEenergy AS is currently carrying out a pilot project of the SSG wave energy converter at the island of Kvitsøy (Norway), partly founded by the European Commission FP-6-2004-Energy-3 (WAVESSG - Full-scale demonstration of robust and high-efficiency wave energy converter). The full-scale technical prototype of the SSG includes three reservoirs for capturing the ocean energy and is constructed as a robust shoreline device (Fig. 2).

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.
The pilot project regards a 10 m wide civil structure module of the SSG and will be installed within 2006.

Figure 2: Scheme of Seawave Slot-Cone Generator (SSG).

The purpose of the work described in this paper is to derive information on wave pressures/forces acting on sloping and vertical walls constituting the structure. The aim is to optimize the structural design and geometrical layout of the SSG under extreme wave conditions (Larsen and Kofoed, 2005; Vicinanza et al., 2006).

MODEL SETUP

Model tests have been performed in a wave tank at Aalborg University, in 1:60 length scale compared to the prototype. 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 (Fig. 3).

The wave generation software used for controlling the wave paddles is AWASYS5, developed by the laboratory research staff (http://hydrosoft.civil.aau.dk/AwaSys/).

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 plexiglass with dimension of 0.471 x 0.179 m. The three front plates were positioned with a slope of $\alpha = 35°$. The model was installed on a 3D concrete model of the cliff located in the middle of the basin at 5 m from the paddles. The miniature of the cliff is 1.17 x 1.67 m (Fig. 4).

The bathymetry near the model is defined as follows:

- 5 m plateau length (300 m in prototype);
- 0.65 - 0.68 m water depths at plateau toe;
- 0.50 – 0.53 m water depth at cliff toe;
- 1:1 cliff slope.

The purpose of the plateau is to enable the reproduction of extreme sea states in front of the model which is not possible on a flat bottom in the wave tank. The plateau is not modeling the topography at the location but focus is achieving the target wave characteristics in front of the structure.

Seven resistive wave probes were located on a pentangle array placed on the plateau. Fourteen Kulite Semiconductor pressure cells were used to measure the pressure in a total of 25 positions on the structure.

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 - 6).

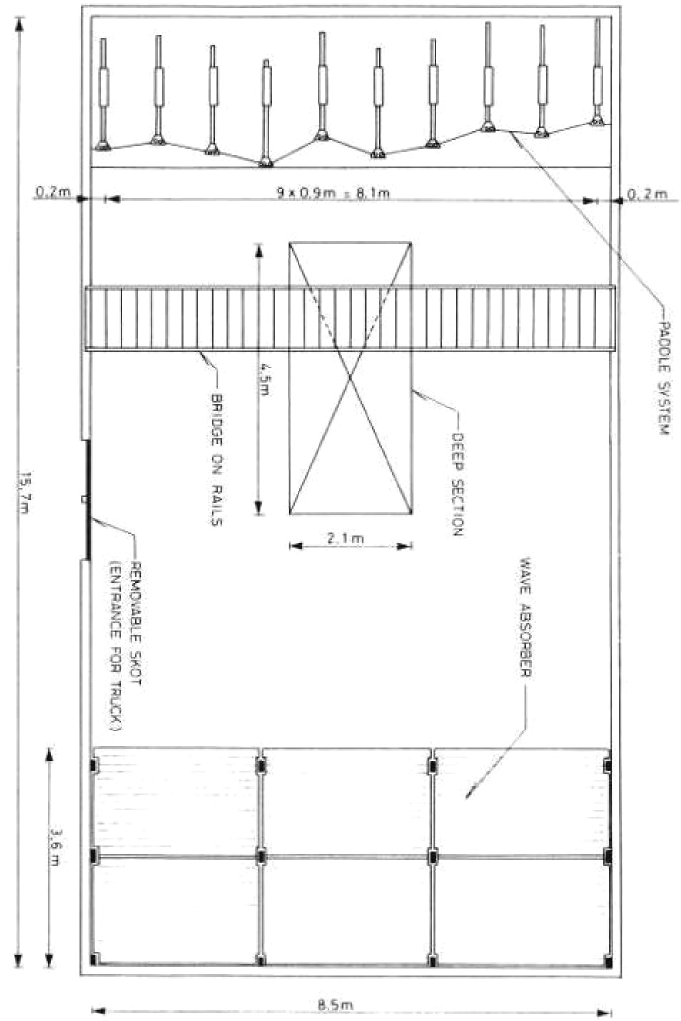

Figure 3: Scheme of the deep 3-D wave basin.

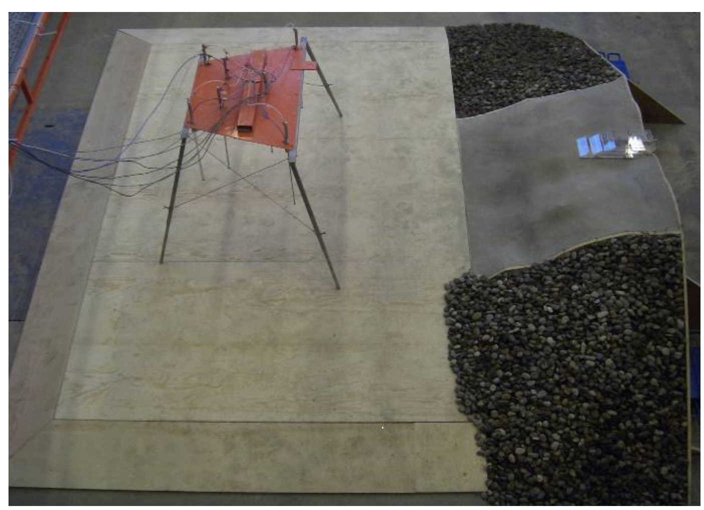

Figure 4: Bathymetry and model set up.

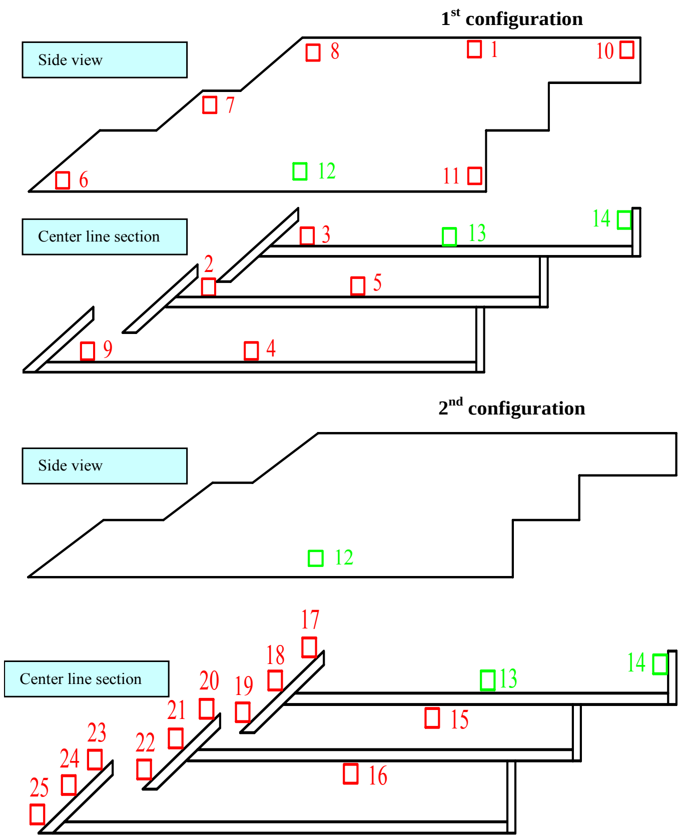

Figure 5: Tests configurations and pressure cells locations (green identify transducer locations used in both configuration).

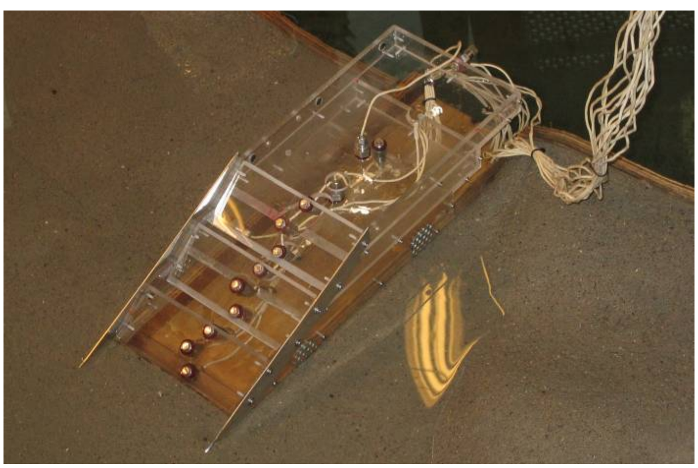

Figure 6: SSG model and pressure cells locations.

EXPERIMENTAL PROCEDURE

Table 1 shows the 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 Tab. 1), with various levels of directional spreading (n). The directional spreading function adopted is expressed by a cosine power form:

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

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 were very good (within 2% for the considered tests). For the 3D wave analysis the BDM procedure were used.

Table 1. Summary of wave conditions.

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

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 (Larsen and Kofoed, 2005) corresponding to test 3 in Table 1.

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 was thus performed twice.

A preliminary visual test analysis (Fig. 7) 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 (see pressure transducers from 17 to 25 – Fig. 5);
- impact of water jet, resulting from massive wave overtopping directly hitting the vertical rear wall in upper reservoir, characterized by evident wave slamming (see pressure transducer 14 – Fig. 5).

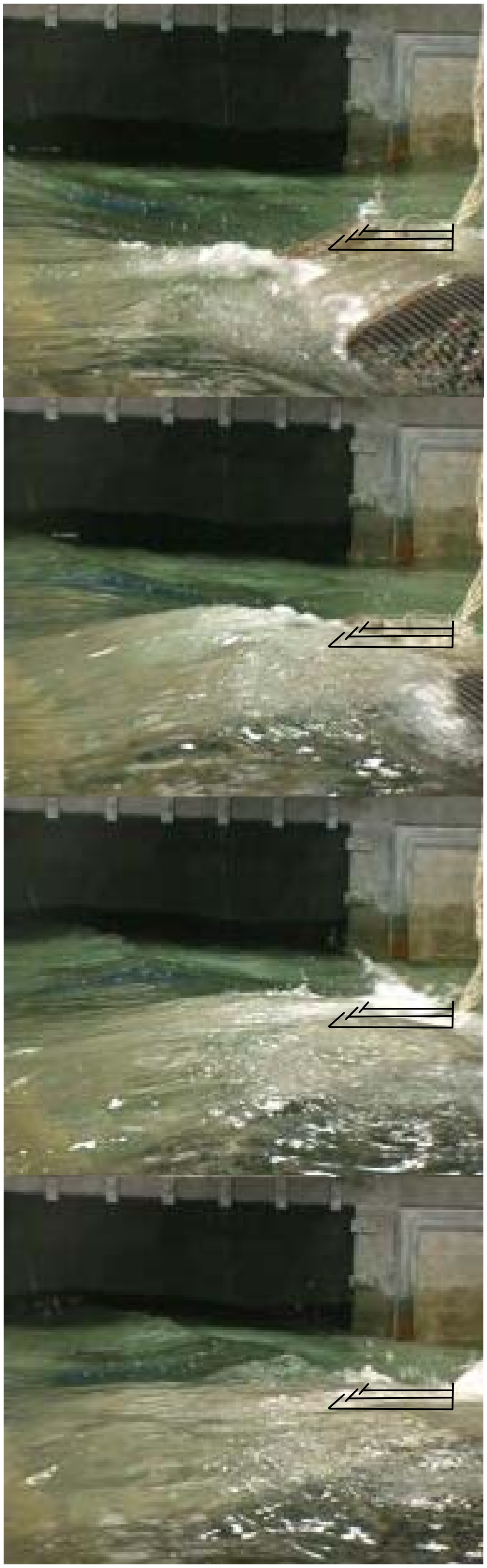

Figure 7: Sequence of video frames from test 4 (time between frames: 0.2 s).

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 200Hz. A second run was carried out at sampling rate of 1200Hz.

Results discussed here derive from preliminary analysis conducted using tests 1, 2, 3, 4 (Tab. 1), expected to represent the most severe wave loading.

RESULTS

The loading experienced by a breakwater depends strongly upon the form of the largest waves reaching the structure. The forms and magnitudes of wave pressures/forces 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, 1996a,b; Vicinanza, 1997a,b).

Within the impact loading regime, it is now well established that the form of the breaker at the wall is of crucial influence on the form of the pressure and force impulse at the structure. Thus the designer needs to know whether impact loads may occur, and if so, the form of the wave responsible.

Identification of loading regime

The first part of the experimental data analysis was finalized to identify the loading regime on different structure locations.

In Figure 8 pressure time history recorded by transducers mounted on the front sloping walls under the extreme wave attack is shown (Test number 4 in Tab. 1). 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.

To confirm this behavior, for the same test, a comparison was made between pressure signal acquired at 200Hz and at 1200Hz (Fig. 9).

The multiple tests at different rate permitted the level of repeatability of the events within the tests to be verified: the mean of the four largest events (pressure 1/250) identified from the pressure measurements for the first run were observed in the second runs as almost the same value as shown in Figure 10 - 11 for test 4 (36.66 mbar against 36.94 mbar).

A completely different behavior was recognized from time history analysis of the pressure transducer at the rear wall in the upper reservoir. Comparison with front plate transducer signal show evident rapidly-varying in time and high pressure peaks typically described as “impact” (Fig. 12).

Moreover, multiple tests at different rate show from one side the repeatability of the events, but on the other, the extreme impulsive variability of the peaks not recognizable at the slow rate acquisition (Fig. 13).

The exceedance level probability plot show the high discrepancy of values for the different acquisition rates. The difference within the tests is illustrated in Figure 14 - 15 for test 4 (57.60 mbar against 97.37 mbar).

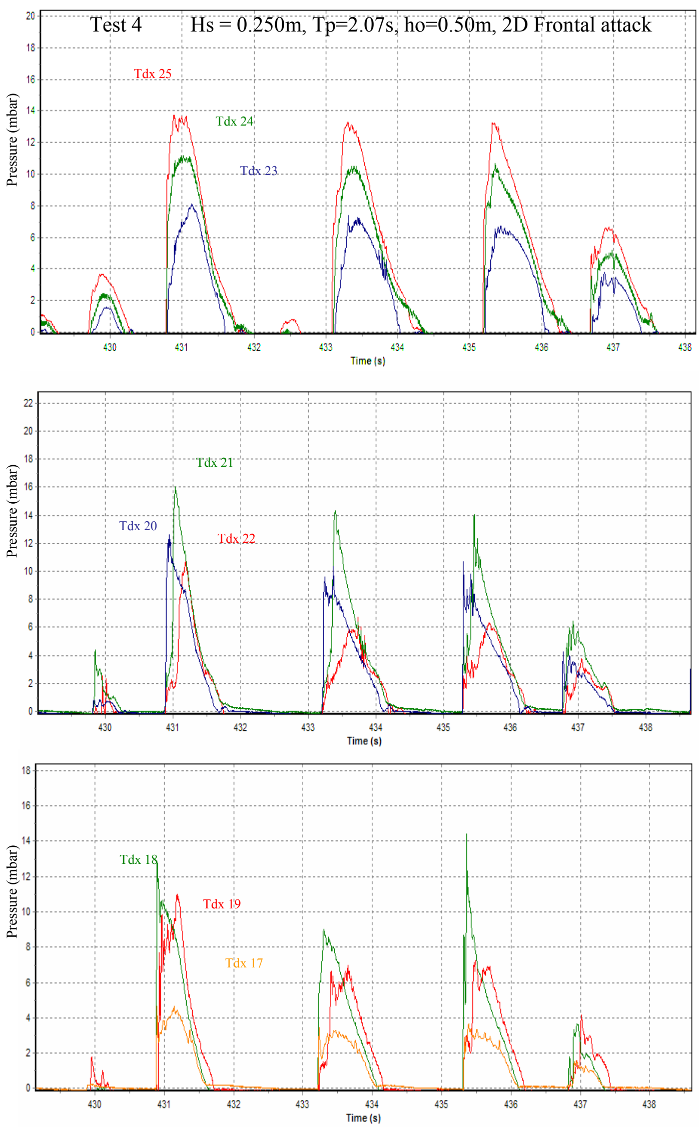

Figure 8: Pressure time history at the transducers on the front plates.

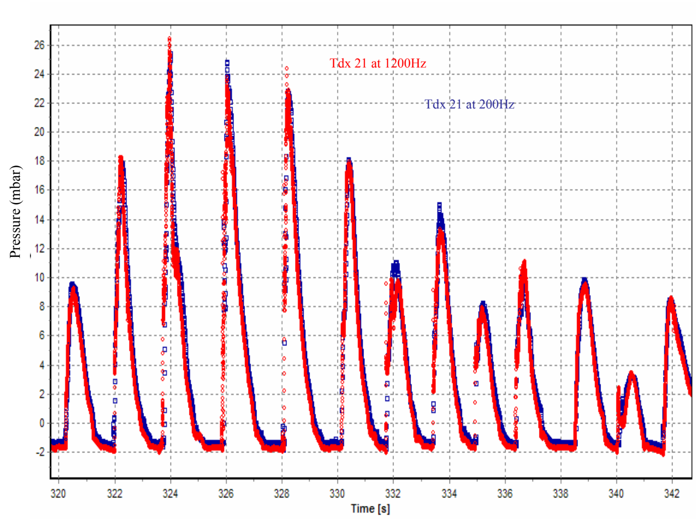

Figure 9: Pressure time history acquired at 200Hz and at 1200Hz.

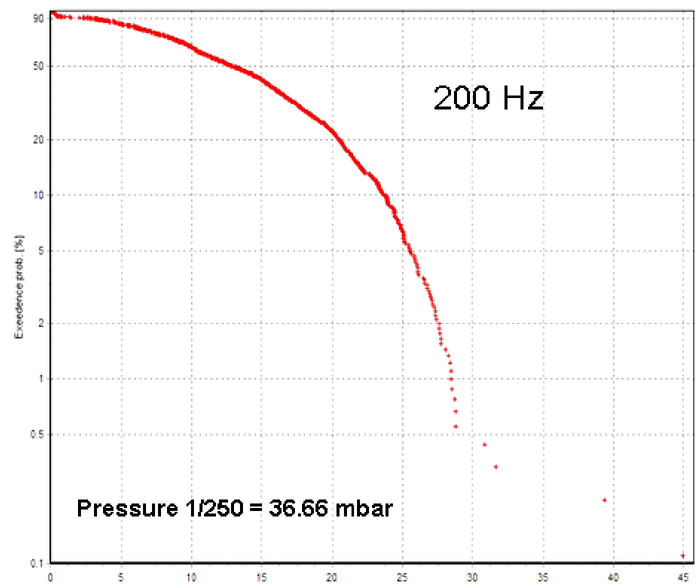

Figure 10: Exceedance probabilities of peak pressures at transducer 21

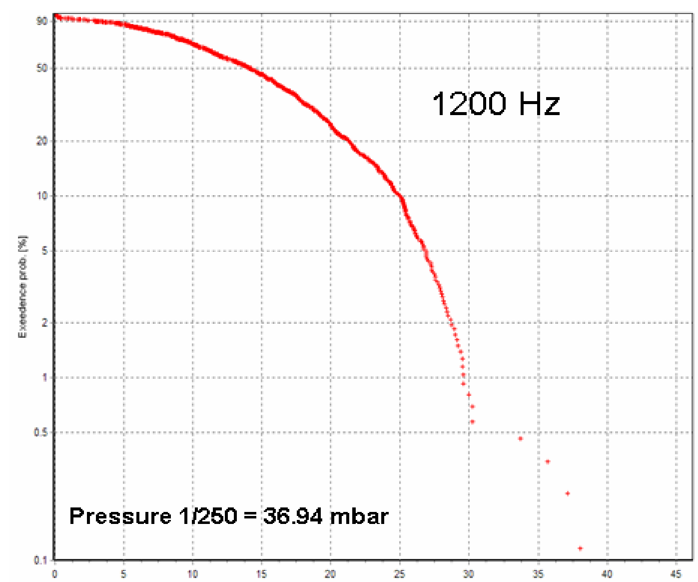

Figure 11: Exceedance probabilities of peak pressures at transducer 21

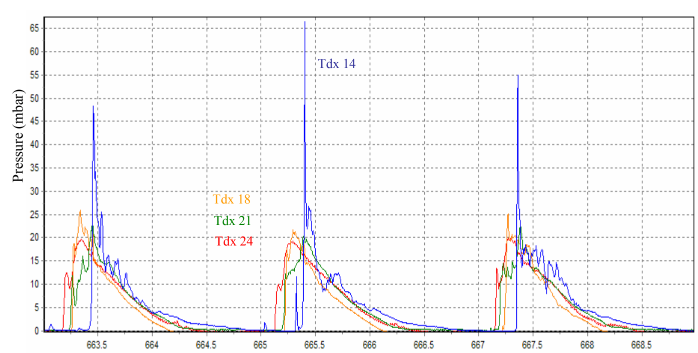

Figure 12: Comparison between transducers on the front plates and on the rear wall.

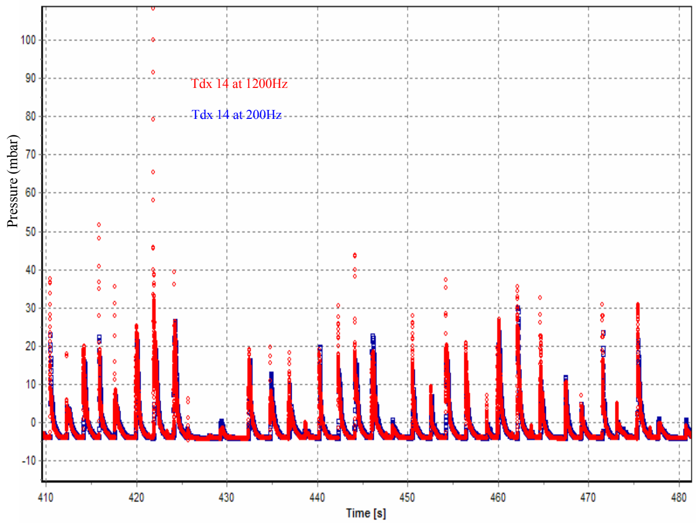

Figure 13: Pressure time history acquired at 200Hz and at 1200Hz.

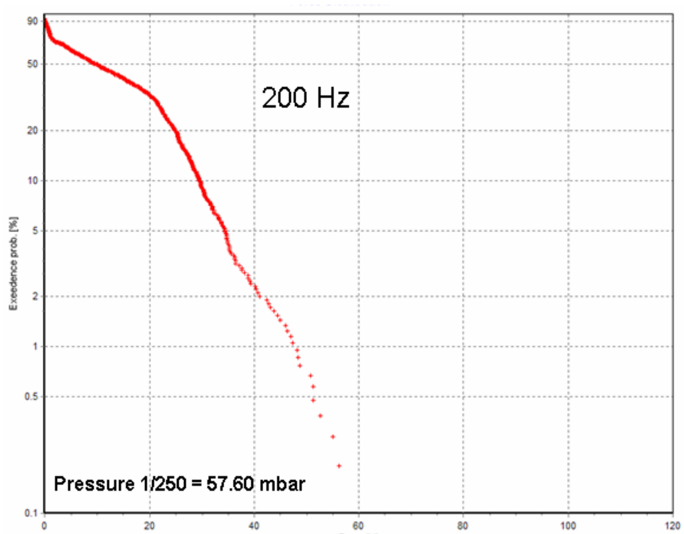

Figure 14: Exceedance probabilities of peak pressures at transducer 14

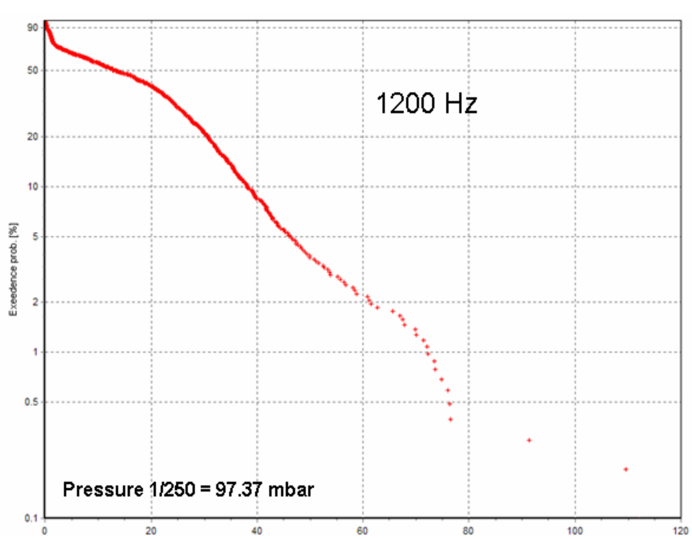

Figure 15: Exceedance probabilities of peak pressures at transducer 14

Spatial distribution of wave pressure

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 summarized in Table 2 appear to indicate that pressures on front plates are quasi static ($p_{1/250} \sim \rho_w\ g\ H_{max}$) or pulsating loads generated by non-breaking waves.

The wave loading on the rear vertical wall are varying over 2 - 3 ρ_w g H_{max}. 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.

Table 2. Summary of model tests pressure 1/250.

Plate	Sampling rate (Hz)	Tdx	Model tests			
			1	2	3	4
			$p_{1/250}$ (kN/m^2)	$p_{1/250}$ (kN/m^2)	$p_{1/250}$ (kN/m^2)	$p_{1/250}$ (kN/m^2)
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

The shape of the spatial pressure distribution on the front plates and at the rear wall is shown in Figure 16. The pressure distribution assume a typical trapezoidal shape (Goda, 1974; Goda, 1985) for plates 2 and 3 while a slightly different shape is visible for plate 1.

Vertical pressure distribution for the rear side wall shows a high peak value in the central part of the wall. Obviously one transducer is not enough to describe the spatial pressure distribution but it is quite reliable to define a substantial increase in the pressure gradient from the previous schematization.

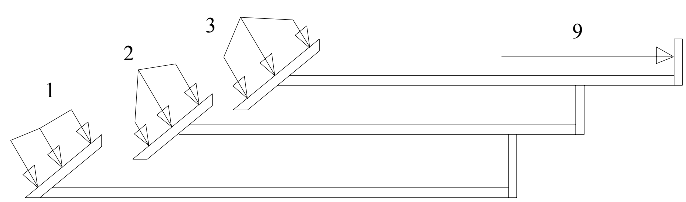

Figure 16: Measured pressure distribution at exceedance of 1/250 (for transducer number refer to Fig. 5).

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 is therefore very probable that the higher impact pressure measured in these model tests can be scaled to lower values.

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

In Table 3 the Froude scaled pressures value for the analyzed tests are reported.

Table 3. Summary of pressure 1/250 scaled to prototype.

Plate	Tdx	Prototype			
		1	2	3	4
		$p_{1/250}$ (kN/m^2)	$p_{1/250}$ (kN/m^2)	$p_{1/250}$ (kN/m^2)	$p_{1/250}$ (kN/m^2)
9	14	309	330	470	584
3	17	51	82	89	129
	18	88	156	181	251
	19	87	138	146	174
2	20	89	125	142	182
	21	112	162	175	220
	22	67	93	99	113
1	23	63	93	139	152
	24	84	124	137	170
	25	97	146	149	199

Comparison with 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) as illustrated in the Figure 17.

This method is not directly applicable to the tested structure but it is the engineering tool that comes closest (CEM 2.01). The reason to compare measurements against this prediction method is to check the order of magnitude of tests results.

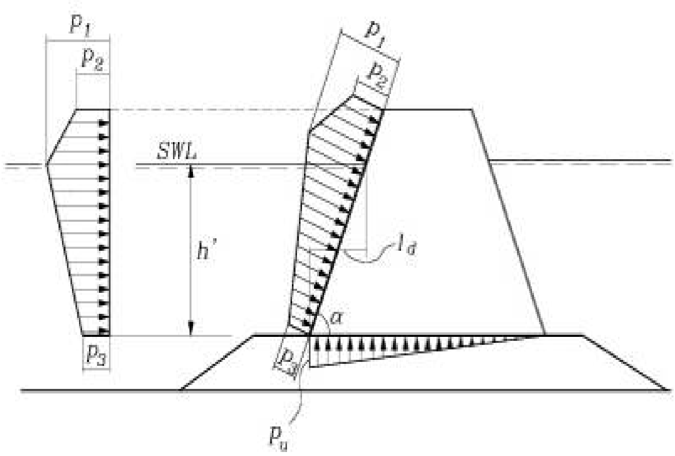

Figure 17: Pressure distribution on inclined wall after Tanimoto and Kimura (1985)

Comparison for the extreme wave condition tested (test 4) are reported in Table 4.

The overall agreement is, despite the violation of the T&K presumptions, quite good. Predicted values are, averaging each plate values, about 10% greater than the measured ones. Pressure gradients are greater in model then predicted by T&K.

Table 4. Summary of pressure 1/250 scaled to prototype.

Plate	Tdx	Prototype		
		Test 4	Tanimoto and Kimura (1985)	
		$p_{1/250}$ (kN/m^2)	see Fig. 17	$p_{1/250}$ (kN/m^2)
3	17	129	p_2	175
	18	251	p_1	202
	19	174	p_3	203
2	20	182	p_2	186
	21	220	p_1	202
	22	113	p_3	200
1	23	152	p_2	194
	24	170	p_1	202
	25	199	p_3	200

CONCLUSIONS

A new type of concrete caisson breakwater is employed as wave energy converted based on the known principle of overtopping and storing the wave energy in reservoirs above sea level. SSG has been model tested for the first time at the Aalborg University with the main aim to give advice to the structure designers on wave loading acting on different parts of the structure.

Mainly two different behaviors 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 250 kN/m^2 that are comparable with the one predicted by Tanimoto and Kimura (1985) for inclined impermeable walls.

The impact loading on the vertical rear wall in the upper reservoir indicate to modify that part in a gentler sloping wall. Anyway the final design of the pilot caisson should include a monitored central vertical area to record pressure impact in order to evaluate scale effects.

REFERENCES

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.

Allsop, N.W.H., Vicinanza, D., M. Calabrese & L. Centurioni, (1996a), "Breaking Wave Impact Loads On Vertical Faces", 6th Int. Conf. Isope, Los Angeles.

Goda, Y. (1974), "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. (2002),"Wave Overtopping of Marine Structures – Utilization of Wave Energy", Ph. D. Thesis, Aalborg University, ISSN 0909-4296, Series Paper No. 24.

Kofoed, J. P. (2005), "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, April, 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, San Diego.

Kofoed, J. P. and Osaland E. (2005), "Crest level optimization of the multi level reservoir overtopping based wave energy converter Seawave Slot-cone Generator, 6th European Wave & Tidal Conference, Glasgow.

Kortenhaus, A., and Oumeraci, H. (1998), "Classification of Wave Loading on Monolithic Coastal Structures," Proceedings of the 26th International Coastal Engineering Conference, Vol 1, pp 867-880.

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.

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., Kofoed, J. 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, Dept. of Civil Eng., Aalborg University, March, 2006.