# Spatial Distribution of Wave Pressures on Seawave Slot-Cone Generator

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## SPATIAL DISTRIBUTION OF WAVE PRESSURES ON SEAWAVE SLOT-CONE GENERATOR

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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.

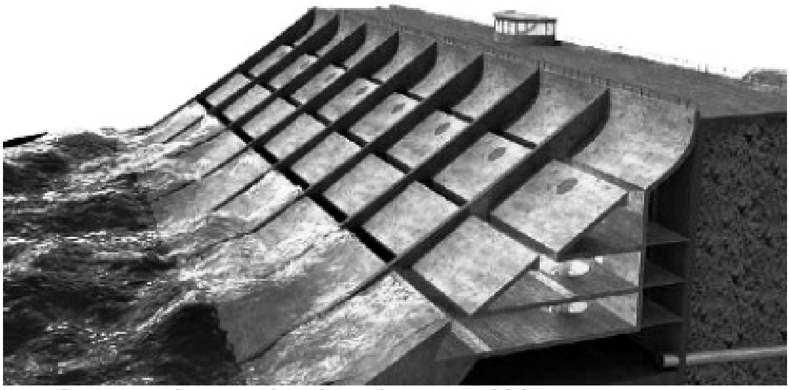

**Figure 1. Scheme of Seawave Slot-Cone Generator (SSG).**

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).

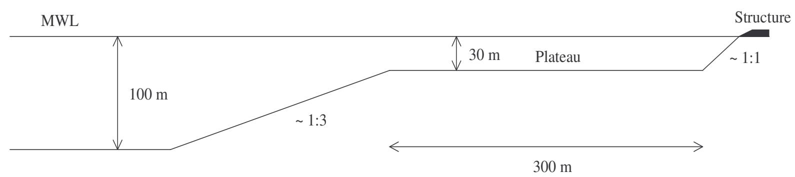


**Figure 2. Rough sketch of the foreshore.**

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).

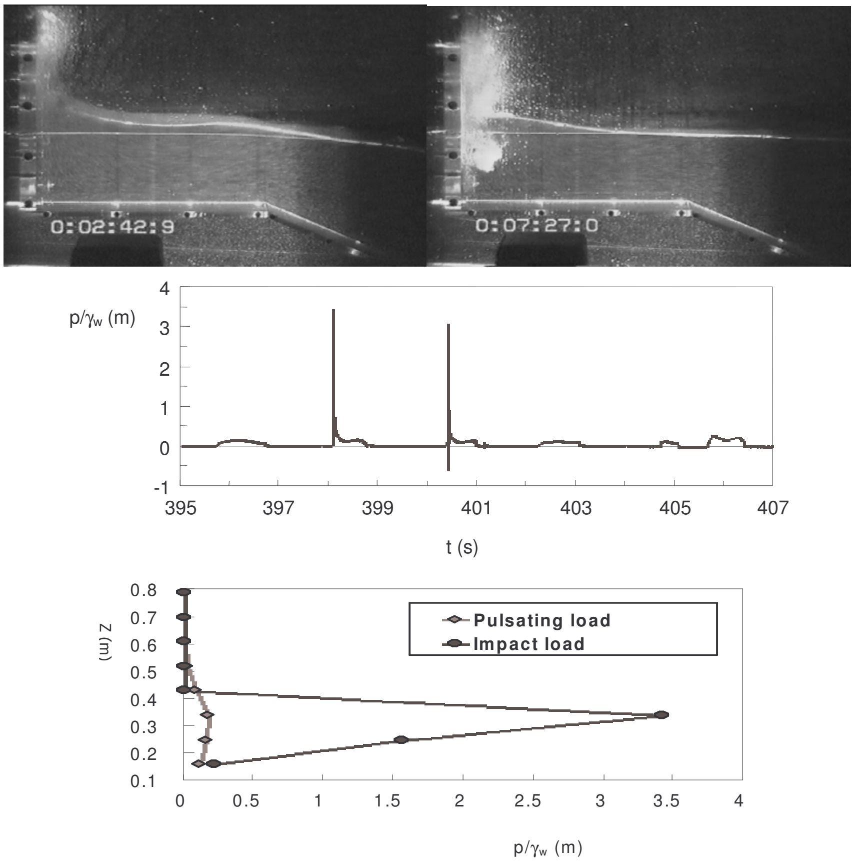


**Figure 3. Quasi-static and impact time history and pressure spatial gradients (after Vicinanza, 1997a, b; Vicinanza, 1999).**

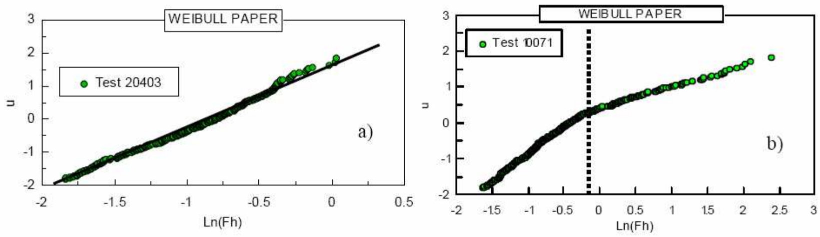


**Figure 4. Weibull plot for conditions: a) quasi –static b) impact (after Vicinanza, 1997b).**

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).

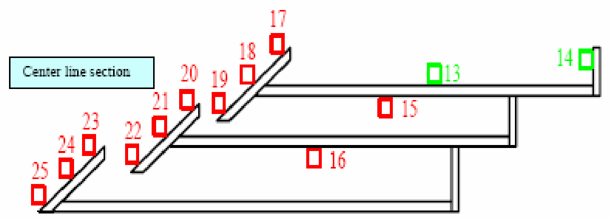


**Figure 5. Tests configurations and pressure cells locations at center line section.**

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.

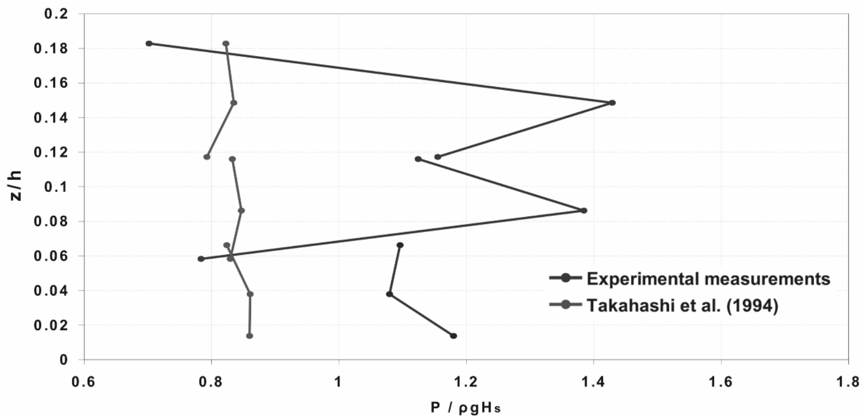


**Figure 6. Takahashi et al. (1994) formula compared test 3.**

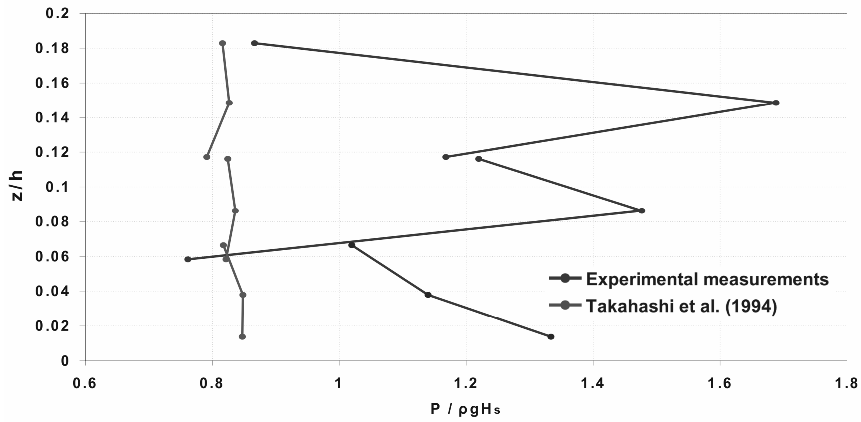


**Figure 7. Takahashi et al. (1994) formula compared to test 4.**

## 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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Breakwaters
Coastal structures
Dynamic response
Inlets
Long waves
Overtopping
Wave generation