



Aalborg Universitet

AALBORG UNIVERSITY  
DENMARK

## Wave Impacts on Caisson Breakwaters Situated in Multidirectionally Breaking Seas

Grønbech, J.; Burcharth, Hans F.; Frigaard, Peter; Kofoed, Jens Peter

*Published in:*  
Coastal Engineering

*Publication date:*  
1999

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

[Link to publication from Aalborg University](#)

*Citation for published version (APA):*  
Grønbech, J., Burcharth, H. F., Frigaard, P., & Kofoed, J. P. (1999). Wave Impacts on Caisson Breakwaters Situated in Multidirectionally Breaking Seas. In B. L. Edge (Ed.), *Coastal Engineering: Proceedings of the 26th Conference on Coastal Engineering* American Society of Civil Engineers.

### General rights

Copyright and moral rights for the publications made accessible in the public portal are retained by the authors and/or other copyright owners and it is a condition of accessing publications that users recognise and abide by the legal requirements associated with these rights.

- ? Users may download and print one copy of any publication from the public portal for the purpose of private study or research.
- ? You may not further distribute the material or use it for any profit-making activity or commercial gain
- ? You may freely distribute the URL identifying the publication in the public portal ?

### Take down policy

If you believe that this document breaches copyright please contact us at [vbn@aub.aau.dk](mailto:vbn@aub.aau.dk) providing details, and we will remove access to the work immediately and investigate your claim.

# Wave Impacts on Caisson Breakwaters situated in Multidirectionally Breaking Seas

Frigaard, P., Burcharth, H. F. & Kofoed, J. P.

## 1. Abstract

*The paper concerns horizontal wave forces on caisson breakwaters in multidirectional breaking seas. It is based on model tests performed in a 3D wave tank at Hydraulics and Coastal Engineering Laboratory, Aalborg University. The measured horizontal wave forces are compared to the Goda force. Good agreement with the Goda formula is found for waves not breaking directly on the structure, while increasing degree of breaking on the structure results in forces of up to 50 % higher than the Goda force. Furthermore, the reduction of the horizontal wave force on long structures due to the non full correlation of the wave pressure along the structure is investigated. A formula for the force reduction factor based on cross correlation coefficients is given as a function of the mean wave direction and the wave spreading.*

## 2. Introduction

In the design of caisson breakwaters it is common to use the rather simplified but well documented (Goda, 1974), equation for calculation of the wave forces. Alternatively, equations from (Takahashi et al., 1994), (Allsop & Vicinanza, 1996), deals with effects from impulsive forces. However, these models do not at the same time take into account both wave impacts and wave directionality.

The effects of wave breaking and impact forces on vertical structures have been investigated by several researchers in the past. However, the research on impact loading has mainly been based on 2D breaking waves, (Takahashi et al., 1994), (Oumeraci et al., 1995), (Allsop & Vicinanza, 1996), (McKenna & Allsop, 1998).

---

Hydraulics and Coastal Engineering Laboratory, Dept. of Civil Engineering,  
Aalborg University, Sohngaardsholmsvej 57, DK-9000, Aalborg, Denmark,  
Fax. +45 98142555, E-mail: peter.frigaard@civil.auc.dk

Attention has also been addressed to the effects of wave obliquity and multidirectionality on the wave loads on vertical caisson breakwaters situated in non breaking seas. Battjes (1982) gave a theoretical description of effects of short-crestedness on wave loads on long structures. Within the joint European (MAST-LIP-TAW) research project, a 3D model investigation was carried out at Delft Hydraulics to assess these effects. The results have been published by several researchers, among them Franco et al. (1995).

Few researchers (Grønbech et al., 1997), (Calabrese & Allsop, 1998), described the effect of obliquity on wave load. These studies included effects from impact forces.

So far, little attention has been paid to the effects of wave obliquity and multidirectionality on the reduction in the wave loads on long caisson breakwaters placed in deep water breaking seas. To assess these effects, a physical model study has been carried out (Hydraulics and Coastal Engineering Laboratory, Aalborg University, 1997 and 1998).

In addition ongoing research will in the near future focus further on this topic. Late autumn 1998 a joint European TMS research project entitled Coherence of impact pressures at vertical wall in multidirectional seas lead by Prof. Alberto Lamberti, University of Bologna, Italy, will be performed at HR Wallingford.

### 3. Experimental setup

The physical model tests were carried out in the 3D wave basin. A caisson breakwater model constructed in plywood was used in the tests. The model was placed on a one layer smooth foundation constructed in concrete. The idea of this foundation was to provoke wave breaking in front of the caisson or even to introduce wave breaking on the caisson. The cross section and a plan view of one of the models are seen in Figure 1.

The size of the model did not refer to any particular prototype structure, however, a Froude scaling of 1:20 - 1:25 seems appropriate for this type of structures.

Two different crest freeboards corresponding to dimensionless crest freeboards in the range 1.17 – 1.90 were applied in the tests.

Two different model layouts were used, obliquity of the model relative to the wave paddles where 15° and 30°, respectively. The reason for these two model layouts was the fact that model influence given by the layouts should be extracted.

To assess the effects of wave obliquity and multidirectionality of the waves, the wave induced pressures were measured on a 2.4 m wide test section, giving the opportunity to study these effects on a single vertical element of the test section as well as on the total width of the test section.

To eliminate the disturbance from wave diffraction at the two ends of the model, the model were extended beyond the test section.

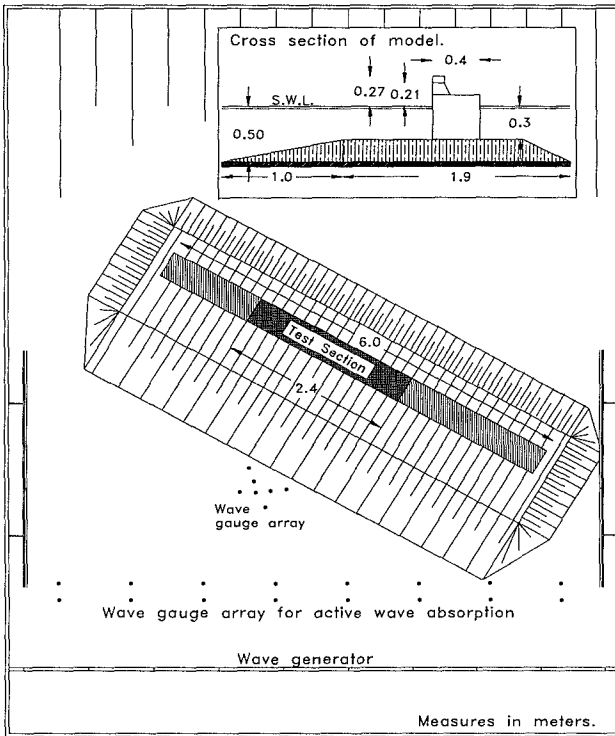


Figure 1 - Model setup.

Preliminary to the tests a numerical study was performed in order to evaluate the diffraction effect from the two ends of the caisson. A consequence of this study was a non-symmetrical placing of the test section in the whole structure, see Figure 1. Even though much effort during planning and model changes were done trying to minimize effects from diffraction it must be concluded that variations in the order of  $\pm 5\%$  of the incident wave height along the structure within the test section could be expected in a test. Consideration of this variation is rather important for the analysis of lateral force distribution.

In order to control the incident waves the basin were equipped with a full 3D active absorption system, see (Hald & Frigaard, 1997).

As the main objective of the study was to assess the effect of wave obliquity and multidirectionality in breaking seas, the changes on test conditions were mainly the incident mean direction of the waves and the directional spreading of the waves. The wave parameters are summarized in table 1.

Wave spectrum	JONSWAP, $\gamma=3.3$
Peak period $T_p$	1.2 sec
Significant wave height $H_s$	0.14 - 0.18 m
Crest freeboard $R_c$	0.21 and 0.27
Water depth near caisson $h$	0.3 m
Mean wave direction $\theta$	$0^\circ$ to $48^\circ$
Spreading of waves	Cosine squared, $\sigma = 0^\circ - 25^\circ$

Table 1 - Wave parameters.

To obtain an adequately statistically validity of the test results, rather long time series were performed with no test series having less than 1800 waves, and on certain occasions up to 2500 waves.

#### 4. Wave analysis

Wave height is the most important parameter in the description of the wave forces. Due to high amount of reflection from the caisson and thereby the relatively confused sea, high priority were given to calculation of the incident wave height.

Wave elevations were measured by a wave gauges array consisting of 7 wave gauges located on deep water, see Figure 1. Using the Bayesian Directional Method, see Hashimoto & Kobune (1988), the wave field were separated into incident and reflected wave fields. Mean wave directions, spreading of the waves, significant wave heights and reflection coefficients at deep water were calculated from this incident wave field.

As the waves approached the more shallow water near the caisson they shoaled, refracted and started to break. Therefore, deep water wave parameters could not describe the waves sufficient in shallow water near the structure. Due to the wave reflection wave breaking and the rapid changes in the sea state shallow water wave parameters like wave direction, spreading of energy and wave height could not be calculated with high accuracy. Therefore, the front of the caissons was equipped with up to 4 wave gauges.

Using measured reflection coefficients of approximately 95 % an estimate of the shallow water significant wave height were calculated from  $H_s = 4/1,95 \sqrt{m_0}$ , with  $m_0$  = total amount of energy.

Mean wave directions and energy spreading of the waves referred to in the following were all calculated from the deep water incident wave field.

### 5. Statistical distribution of forces

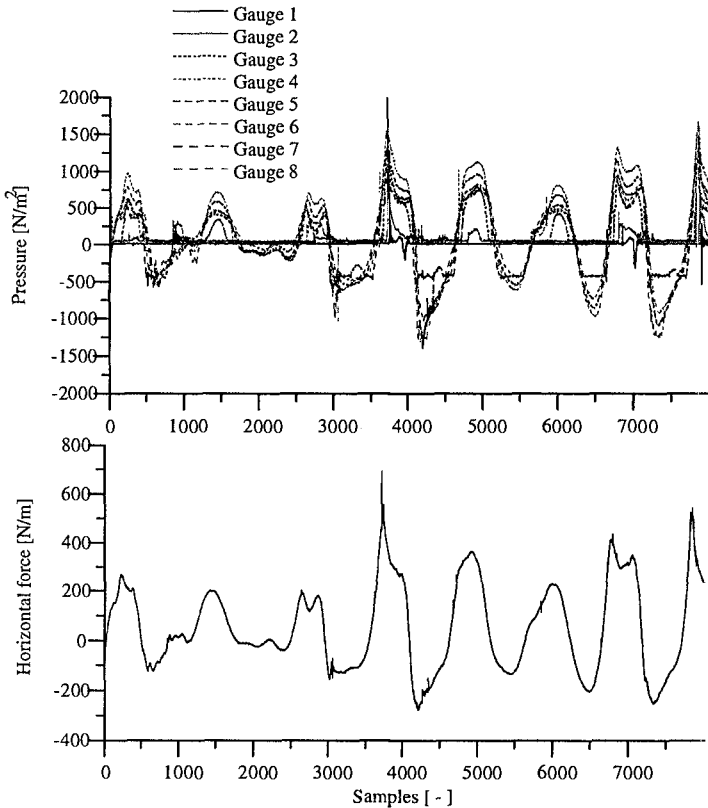


Figure 2 – Example of time series with a duration of 10 seconds showing sampled pressures in one section and the corresponding integrated horizontal force.

The forces in the sections were determined by integrating the measured pressures over the height of the model. In Figure 2, a time series of measured pressures in eight different levels and corresponding integrated horizontal force are shown. The pressures were sampled at 800 Hz. In order to compare the results of the tests with the prediction formula of Goda and to compare the results of tests with different wave heights, the measured forces were expressed in terms of the statistical force parameter  $F_{1/250}$  which is average of the force peaks occurring with a probability less than 0.4 %. Wave heights input into the Goda formula were measured at the structure exactly where the forces were measured.

In the design of the model layout for non braking and breaking conditions approximately 0 % of the waves and 8 % of the waves were assumed to break on the ramp or at the caisson, respectively. Figure 2 show that some impulsive loading from the breaking waves were measured but only some deviations relative to pulsating forces were seen.

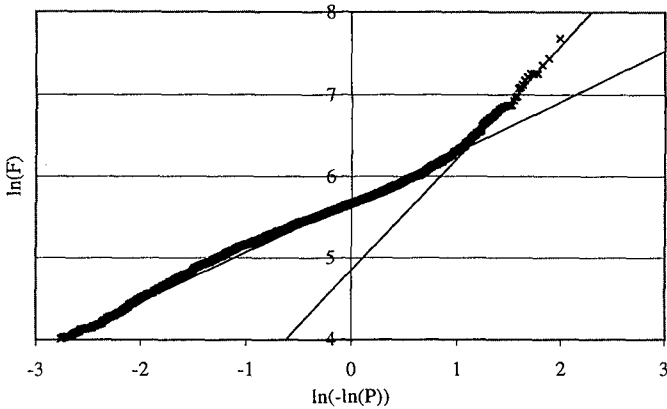


Figure 3 – Example of statistical distribution of horizontal force peaks.

McKenna & Allsop (1998), stated that the statistical distribution of the pulsating force peaks may be described well by the Weibull distribution, and that a change in the gradient of the Weibull plot indicates the onset of impulsive wave impacts.

Figure 3 show that this deviation happened for force peaks with a probability of exceedence  $P$  less than approximately 3 - 4 % which corresponds rather well to the observed number of waves breaking near the caisson or at the caisson.

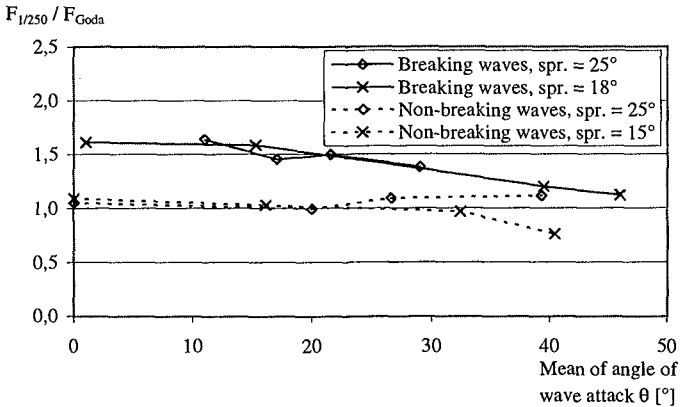


Figure 4 - Force comparisons.

Figure 4 show that measured forces corresponds very well to forces predicted by the Goda equation. As long as the waves were non breaking this was the case for all tests no matter wave direction and wave spreading. Other researchers, (Franco et al. 1995) and (Grønbech et al., 1997). found poorly agreement between measurements and the Goda force. They reported deviations up to 20 %. This is because they calculated the Goda force from the deep water parameters where the present study uses shallow water wave parameters.

The measured force for the breaking waves deviate approximately 50 % from the forces predicted by the formula of Goda. Apparently, the Goda formula underestimates the forces from the breaking waves. Though, here it must be remembered that the test conditions for the breaking waves present somewhat the most severe possible sea condition, which in practice is the upper limit for the number of breaking waves. Such conditions were never meant to be covered by the Goda formula. Goda (1984), described how to avoid such a condition in the design.

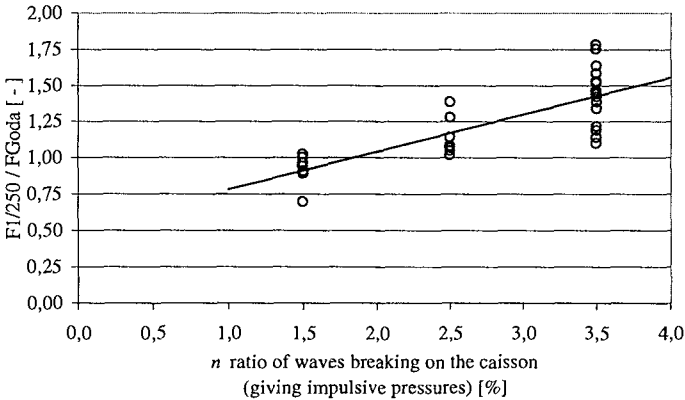


Figure 5 – Measured forces compared to the Goda force for different degrees of wave breaking on the caisson.

Allsop et al. (1996), also found that the Goda equation underestimated impact loadings from breaking waves and they demonstrated very good correlation with the equation originally proposed by Allsop & Vicinanza (1996):

$$F_{1/250} / (\rho_w g h^2) = 15 (H_s/d)^{3.134} \text{ for } 0.35 < H_s/h < 0.6 \tag{1}$$

In the present study relatively good agreement to the Allsop & Vicinanza equation were found but the equation seemed to overestimate the measured forces. For the test cases the Allsop & Vicinanza equation estimate forces approximately double the estimates from the Goda equation.

Figure 5 shows that force ratio as a function of the number of waves breaking on the caisson. From plots like the one shown in Figure 3 it is possible to find the number of waves breaking on the caisson.

The performed tests were divided in three groups, non breaking waves, moderate breaking waves and breaking waves, respectively. In Figure 5 all force ratios are plotted relative to the average number of waves breaking on the structure in the whole group of wave conditions. A clear trend described by the following equation is found.

$$F_{1/250} / F_{Goda} = 0.6 + 25n \tag{2}$$



## 6. Lateral distribution of forces

The horizontal wave load over a length of caisson will be reduced relative to forces measured in one section due to the non full correlation of the wave pressures along the structure.

By measuring the vertical distributions in nine sections spaced by 0.3 m and one section 0.1 m wide, the lateral distribution of the horizontal forces was studied. This is important due to the unknown correlation between the lateral distribution of the horizontal pressure and the lateral distribution of the horizontal force. The wave pressures were measured by a set of pressure transducers placed in the test section as shown below.

Figure 6 indicates 10 sections with 8 - 9 pressure transducers in each section. Never the less due to a restriction of 50 available pressure transducers only measurements in four sections were done simultaneously.

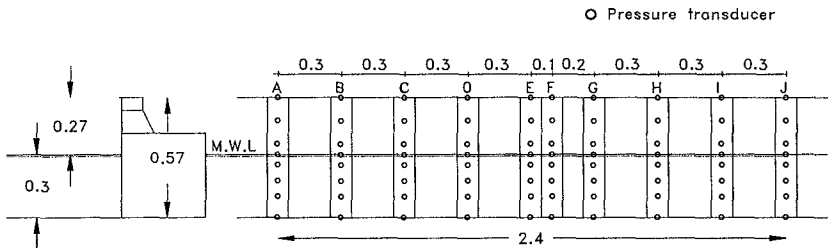


Figure 6 - Pressure transducer placement. Measures in meter.

From simultaneous measurements of force time series in different sections the reduction factor  $r_F$  can be calculated (actually  $r_F$  describes the non reduction in the force). Given a structure with length  $l_s$  the reduction factor is traditionally calculated as:

$$r_F(l_s, P) = \frac{\int_0^{l_s} F(x, t) dx}{l_s \cdot F_P(t)} \quad (3)$$

Notice that the reduction factor might be depending on the probability level  $P$  for the calculation.

Calabrese & Allsop (1997) and Burcharth (1998), calculated force reduction coefficients simply by doing statistical analysis of lateral integrated force time series, and then take  $r_F$  as the ratio between  $F_{1/250}$  from the lateral integrated force time series and  $F_{1/250}$  from the force times series measured in one section.

Though, such a method for calculating the force reduction will be influenced by diffraction patterns along the structure leading to too low reduction factors, which is unsafe. Furthermore, it is impossible to perform tests with high spatial resolution, so results will be based on a coarse resolution. Generally this effect will result in too high reduction factors. It is not possible assess these effects.

In the present study the spatial correlation coefficient used as the basis for calculation of the force reduction factor is calculated as:

$$\rho(x_1 - x_2) = \frac{\frac{1}{T} \sum_{t=1}^T F(x = x_1, t) F(x = x_2, t) \Delta t}{\sigma_{F(x=x_1)} \sigma_{F(x=x_2)}} \quad (4)$$

Here  $T$  is the part of the time series that the spatial correlation coefficient is based on ( $T$  can consist of disconnected parts of the time series).  $x_1$  and  $x_2$  are coordinates of the sections where the time series are measured along the structure.  $\sigma_{F(x=x_1)}$  is the variance of the  $F$  part of the time series.

The advantages of using the cross correlation coefficients are that diffraction effects are removed from the measurements, and it is possible to get a very high spatial resolution by repeating the tests with different distances between the sections. Results will not be affected by smaller changes in the seastates.

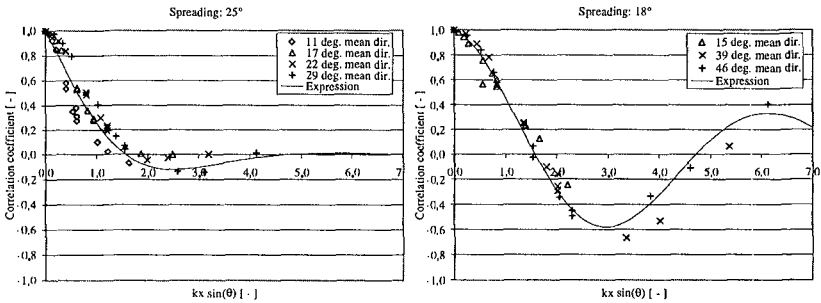


Figure 7 – Spatial correlation coefficients and fitted spatial correlation function  $\rho_1 = \rho_1(kx \sin(\theta))$  based on all data points in the time series.

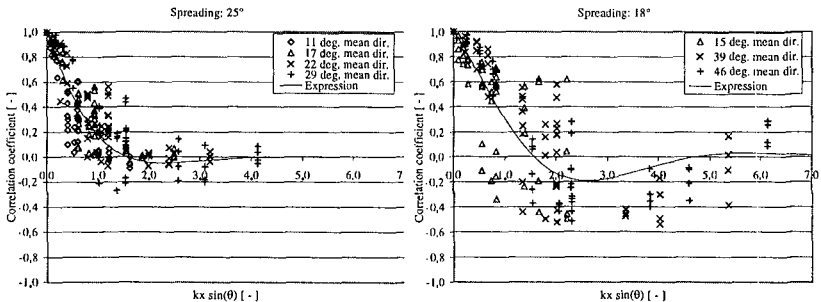


Figure 8 – Spatial correlation coefficients and fitted spatial correlation function  $\rho_2 = \rho_2(kx \sin(\theta))$  based on all force peaks in the time series.

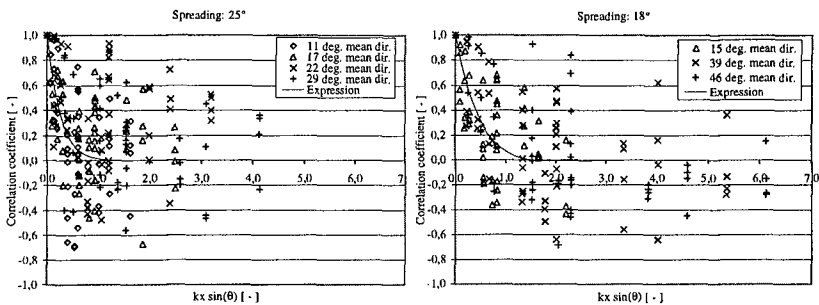


Figure 9 – Spatial correlation coefficients and fitted spatial correlation function  $\rho_3 = \rho_3(kx \sin(\theta))$  based on force peaks with a probability of occurrence less than 0.4 %.

Spatial correlation coefficients between the measured integrated pressures in sections with varying lateral distance have been calculated in three different ways, see Figure 7 – 9.

First spatial correlation function  $\rho_1$  were calculated from the whole of the forces, i.e. using all data points. Second spatial correlation function  $\rho_2$  were calculated from the parts of the force time series were the peaks of the forces were situated. Finally, the third spatial correlation function  $\rho_3$  were calculated only from the parts of the force time series were the highest peaks (peaks with a probability of occurrence lower than 0.4 %) were situated.  $k$  is the wave number based on the peak period of the waves at shallow water.

It is obvious that the calculated spatial correlation coefficients show increasingly scatter when they are calculated from fewer data points. Also the figures indicates that the spatial correlation for the highest forces is lower than the spatial correlation for the whole time series. This means that the reduction factor for the highest peaks is lower than the reduction factors for the whole time series.

The reduction coefficients for the design forces were calculated using the spatial correlation function  $\rho_3$ :

$$r_F = \frac{1}{l^2} \int_{-\frac{l}{2}}^{\frac{l}{2}} \int_{-\frac{l}{2}+\xi}^{\frac{l}{2}+\xi} \rho_3 dx d\xi \tag{5}$$

It is assumed that the force peak will hit anywhere along the caisson with even probability. This means that the caisson in the calculation is placed randomly relative to the spatial correlation function.

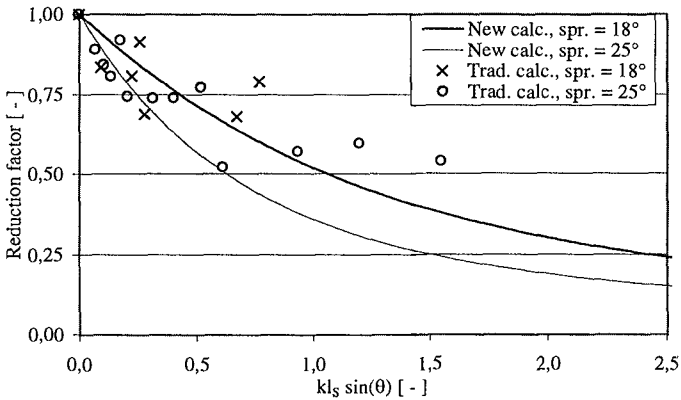


Figure 10 – Reduction factors for the horizontal wave loads.

Figure 10 show calculated reduction factors based on spatial correlation functions (eqn. (4)) compared to reduction factors based on a traditional method of calculation (eqn. (3)). For low values of the dimensionless structure length  $kl_s \sin(\theta)$  it is seen that a traditional calculation will underestimate the reduction factor where as for high values of the dimensionless structure length the traditional calculation will overestimate the reduction factor.

The calculated reduction factors can be described by the following equation:

$$r_F = \frac{1}{1 + a(kl_s \sin(\theta))^{1.2}} \begin{cases} \text{for } \sigma = 18^\circ, a = 1.0 \\ \text{for } \sigma = 25^\circ, a = 1.8 \end{cases} \text{ for } \theta > 5^\circ \quad (6)$$

7. Conclusion

Regarding the effect of wave breaking on vertical caisson breakwaters, an increase compared to non-breaking waves and the predictions by the Goda formula of the wave forces was seen. It was found that the ratio of the waves breaking on the caisson controls the amplification of the force relative to the Goda force.

Because no full correlation exists between measured horizontal forces in sections with varying lateral distance, a force reduction for wide caisson sections are found. It has been shown that force reduction coefficients must be based on measured cross correlation coefficients. Expressions for reduction factors as function of wave angle and wave spreading are given.

8. Acknowledgements

This work has been carried with support from the framework programme *Dynamics of Structures* of the Danish Technical Research Council and the EU Commission MAST III project *Probabilistic Design Tools for Vertical Breakwaters*.

## 9. References

Aalborg University, Denmark, Hydraulics and Coastal Engineering Laboratory (1998). *An overview of performed laboratory tests related to the investigation of wave overtopping and horizontal wave forces acting on vertical breakwaters in non-breaking and breaking multidirectional seas*. Proc. EU MAST III project PROVERBS, 2<sup>nd</sup> Overall Project Workshop, Naples, 1998.

Aalborg University, Hydraulics and Coastal Engineering Laboratory (1997): *Results of Caisson Breakwater Tests in Multidirectional Breaking Seas (Internal Laboratory Test Report)*., Aalborg University, Dept. of Civil Engineering, Denmark, December 1997.

Allsop, N. W. H. & Vicinanza, D. (1996): *Wave impact loadings on vertical breakwaters: development of new prediction formulae*. 11<sup>th</sup> Int. Harbour Congress. Antwerp.

Allsop, N. W. H., McKenna, J. E., Vicinanza, D. & Whittaker, T. T. J (1996): *New design methods for wave impact loadings on vertical breakwaters and seawalls*. Proc. of teh 25<sup>th</sup> int. conf., Florida, ASCE.

Battjes, J. A. (1982): *Effects of short-crestedness on wave loads on long structures*, Applied Ocean Research, Vol. 4, No. 3.

Burcharth, H. F. (1998): *Identification and Evaluation of Design Tools, Report of Sub-Group A*. PIANC PTCH WG 28 on Breakwaters with Vertical and Inclined Concrete Walls, Aalborg University, 1998.

Calabrese, M. & Allsop, N. W. H. (1997): *Impact loadings on vertical walls in directional seas*. Proc. Task 1 Technical Workshop, Edingburg, EU MAST III project PROVERBS.

Franco, C., Franco, L., Passoni, G., Restano, C., van der Meer, J.W. (1995). *The effect of wave obliquity and short-crestedness on the horizontal and uplift forces on caisson breakwaters*. In: Final proceedings, MAST II, (MAS2-CT92-0047), MCS-Project: Monolithic (Vertical) Coastal Structures, Paper 4.8, 22 pp.

Goda, Y. (1974): *A new method of wave pressure calculation for the design of composite breakwaters*. Report Port and Harbour Research Institute, Vol. 12, No. 3 (in Japanese) or Proc. 14<sup>th</sup> Conf. Coastal Eng., Copenhagen, 1974, pp. 1702 – 1720.

Grønbech J., Hald T., Frigaard P. and Burcharth H. F. (1997) *Wave Loading and Overtopping on Caisson Breakwaters in multidirectional breaking seas*. IAHR seminar on Multidirectional Waves and their interaction with Structures, San Francisco, 1997.

Hald, T. & Frigaard, P. (1997): *Alternative method for active absorption in multidirectional waves*. IAHR Seminar Multidirectional waves and their Interaction with Structures, San Francisco, ISBN 0-660-17093-0.

Hashimoto, N., and Kobune, K. (1988): *Directional spectrum estimation from a Bayesian approach*. In Proc. 21<sup>st</sup> International Conference on Coastal Engineering, Malaga, Spain, 1988, pp. 62 – 76, ASCE.

McKenna, J. & Allsop, W. (1998): *Statistical distribution of horizontal wave forces on vertical breakwaters*, 26<sup>th</sup> International Conference on Coastal Engineering, Copenhagen, 1998.

Oumeraci, H., Hewson, P., Juhl, J. and van der Meer, J.W. (1995). MCS-Project: *Multi-Disciplinary Research on Monolithic Coastal Structures*, Final proceedings, MAST II, (MAS2-CT92-0047), MCS-project: Monolithic (Vertical) Coastal Structures. Papers Task 1-2.

Takahashi, S., Tanimoto, K. and Shimosako, K. (1994): *A proposal of impulsive pressure coefficient for design of composite breakwaters*. Proc. of Int. Conf. on Hydro-technical Eng. for Port and Harbour Construction. Port and Harbour Research Institute, Yokosuka, Japan.