EXPERIMENTAL INVESTIGATION ON CAISSON BREAKWATER SLIDING

Piero Ruol[1], Paolo Martin[1], Thomas Lykke Andersen[2] and Luca Martinelli[1]

This note presents wave flume experiments, carried out at Aalborg University, measuring the horizontal sliding distance of a vertical breakwater in 1:40 scale. Horizontal and uplift wave induced pressures were accurately measured simultaneously with the caisson movements. Caissons of different weight and same geometries are tested under regular and irregular waves. It is found that, under breaking conditions, the expected inaccuracy of the prediction of the force, inherent on the variability of the breaking process, induce unacceptable errors in the prediction of the sliding. This observation endorses other previous experimental results. Conversely, when the actual measured input force is used as input, the analytical Shimosako formula fit quite well the experimental sliding distance.

Keywords: breaker; impulsive force; sliding distance; vertical breakwaters

INTRODUCTION

This note focuses on the sliding phenomenon of vertical breakwaters subject to large breaking wave forces.

Several Authors agree that among the many possible failure mechanisms for vertical (or caisson) breakwater (Fig. 1), sliding is the most critical one (e.g. Oumeraci, 1994; Takahashi, Short course at ICCE 1996 and elsewhere; Goda and Takagi, 2000). Such sliding is typically induced by extreme breaking waves. Clearly, a very small sliding may not lead to actual failure and in fact, the caisson is considered to have failed its purpose in presence of an appreciable sliding, e.g. larger than 10-30 cm (Goda and Takagi, 1998-CEJ 2000). It is reasonable to believe that sliding larger than, say, 30-60 cm, will cause pocket effects that jeopardize the future life of the caissons, thus requiring urgent maintenance works. These arguments highlight the importance of a correct prediction the sliding distance. It is also clear that, in order to correctly predict the possible failure, the total displacement, as total of many sliding events, must be assessed.

Kim et al. (2004) checked the method proposed by Shimosako (see for example Shimosako et al., 1994) for the prediction of sliding occurrence and distance based on an experimental investigation. They found that, under breaking waves, the obtained predictions were accurate only in a restricted range of conditions. The result is not surprising, since the uncertainty of the breaking process is quite large and prediction of the dynamic response of vertical breakwaters under breaking conditions is full of uncertainty. It is anyway not clear whether the simple analytical formula by Shimosako is suited to the purpose or a more elaborate model should be considered.

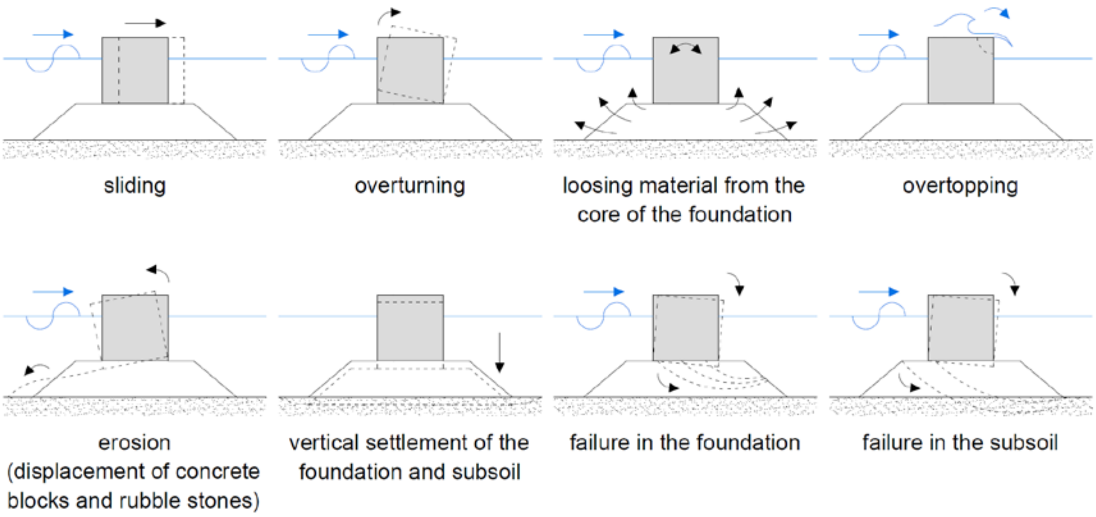

Figure 1 – Main failure modes for vertical caisson breakwaters.

[1] ICEA, University of Padova, v. Ognissanti, 39, Padova, 35129, Italy
[2] Department of Civil Engineering, Aalborg University, Sofiendalsvej 11, 9200 Aalborg SV, Denmark

The aim of this work is to measure the sliding distance of a vertical breakwater induced by a known (measured) load, in order to:

- better understand the involved processes;
- check the trustworthiness of existing formulae;
- extend the database against which the available predicting formulae can be compared.

These issues are important for reliability analyses and cost optimization.

Wave flume experiments investigated the effects of breakers on a vertical breakwater, measuring simultaneously:

- the horizontal and uplift pressure applied to the vertical wall;
- the sliding distance and rotation.

The peculiarities of the experiments are:

- the use of a high speed camera, triggered with many pressure gauges sampled at 1000 Hz,
- the structure is a “typical” European caisson breakwater (Franco, 1994), designed for non-breaking waves but here tested even for breaking waves larger than design (as prescribed by a risk analysis approach).

MODEL TESTS

Experiments have been performed in the Shallow Water Wave Flume at the Hydraulics and Coastal Engineering Laboratory of Aalborg University (DK). The flume is 26 m long, 1.5 m wide (Fig. 2).

The tested model is a caisson breakwater (representing a real prototype in scale 1:42.5 of the cross section of Playa Blanca caisson in Lanzarote, Spain, previously tested in the lab) divided in three parts (Fig. 3): two of them are fixed, while one module of scaled weight (Fig. 4) is allowed to slide.

The foundation consist of rubble mound with diameter D_{n50}=1 mm (Fig. 5) protected by rock armour). A layer of sand was glued to the bottom of the caisson and the friction coefficient under water between caisson and foundation was accurately measured to be 0.45. This value is actually equal to the usual value of the friction coefficient (i.e. 0.6), divided by the usual safety factor (S.F.=1.3).

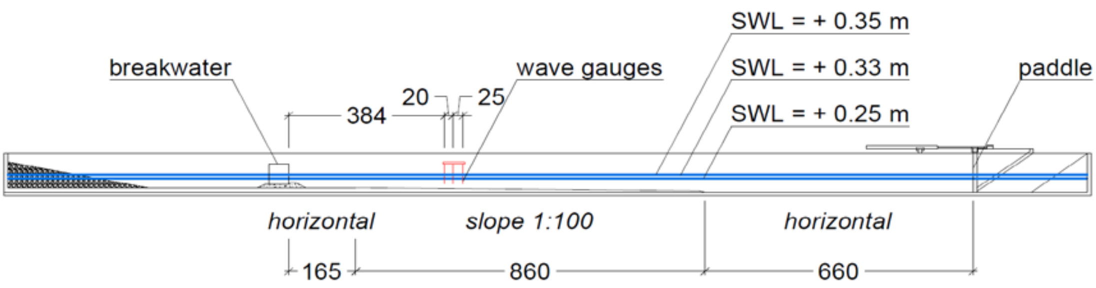

Figure 2 – Wave flume.

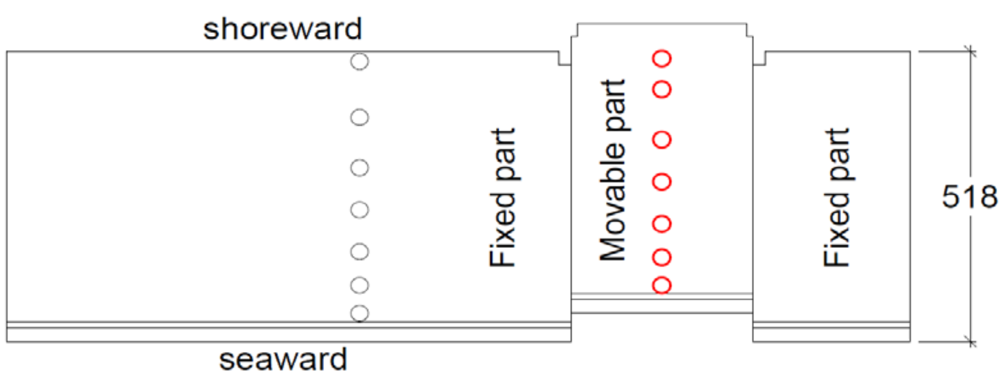

Figure 3 – Front view of the caisson breakwater.

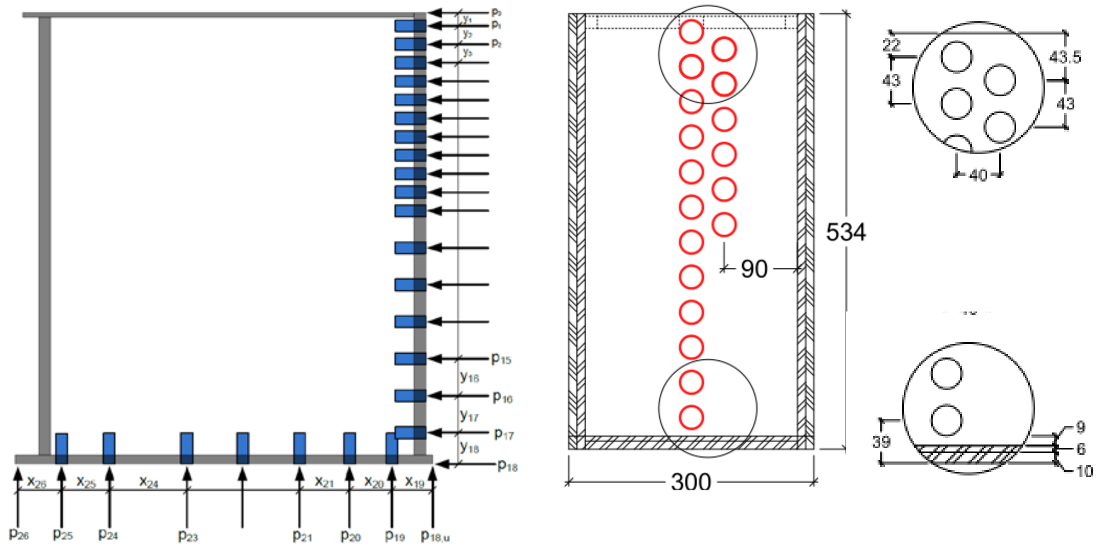

Figure 4 – Dimensions of the sliding module (in mm) and position of the 26 pressure transducers.

Figure 5 – The fixed caissons and the rubble mound used as foundation.

The model is equipped with 2 displacement transducers attached to two rear hooks, vertically aligned, placed at 105 and 277 mm from the bottom of the caisson (the mutual distance being 172 mm), so that sliding and rotation could be measured simultaneously. Further, 18 pressure transducers on the front and 5 in the bottom of the caisson, both on the fixed modules and on the "free to slide" one (Figs. 4 and 5).

An array of three wave gauges were used to measure the incident waves.

Test programme

In order to evaluate the sliding distance under different breaking wave conditions, 3 regular and one irregular sea states have been generated, on 3 different depths (Table 1).

The third regular wave is designed so that the target wave height *H* is the breaker wave height for that water depth, i.e *H*=0.78 *h* (*h* being the water depth). The first two regular waves are not expected to break.

The irregular wave is designed so that some waves break in all conditions, although clearly the occurrence of breakers is different for the three depths.

Furthermore, in order to observe different system response to the same applied load, and thus test the efficiency of the predicting models, 3 different structures have been considered, with same geometry but mass equal to 103, 113 and 123 kg (at model scale).

A total of 39 tests were therefore carried out.

Table 1. Test programme (wave attacks)

H (m)	*T* (s)	Water depth *h*		
		0.25 m	0.33 m	0.35 m
0.10	1.12	X	X	X
0.20	1.53	X	X	X
0.78 h	2.00	X	X	X
Hs (m)	Tp (s)			
0.18	1.53	X	X	X

RESULTS

This Section, after a brief analysis of the sensitivity to the results to the frequency of acquisition and of the model, presents the results in terms of applied forces and sliding distance.

Sensitivity to the frequency of acquisition

The vertical and horizontal force is derived by integration of the pressures measured respectively by the gauges placed under the caisson and on the front face, as seen in Fig. 4, using standard methods. It is argued by Lamberti et al. (2011) that in these conditions the frequency of acquisition and the pressure transducers' distance need to be related, and that too small or too large frequencies of acquisitions can induce gross errors in the reconstructed force. In order to check that the frequency of acquisition is not too small, a typical analysis is presented in Table 2, where the maximum measured force is computed using different frequencies of acquisitions. From the table it is clear that when the logging frequency exceeds 300 Hz, the maximum force reaches a stable value.

Table 2. Sensitivity to the frequency of acquisition

Frequency [Hz]	Maximum force [N/m]	Difference with 1kHz case (%)
1000	924.9	-
500	923.0	0.20
333	921.0	0.42
250	902.6	2.41
166	877.8	5.17
125	874.5	5.50
111	864.9	6.48
100	858.6	7.16

In order to check that the frequency of acquisition is not too large, the vertical coherence of the peak pressure is compared to the pressure transducers' spacing. Fig. 6 shows the pressure distribution for the largest measured force, and it is evident that at least 3 gauges capture the peak pressure, and that therefore the frequency of acquisition is not so large that local pressure jets are misinterpreted as pressure applied to large areas.

Frequencies in Table 2 should be related to the system modes of oscillation. For this purpose, a standard preliminary investigation was carried out hitting a caisson by a hammer to simulate an impulsive load. Although the eigenmodes were not entirely identified due to errors in the measurement system, the main system eigenfrequencies could be identified to be in the range 120-250 Hz.

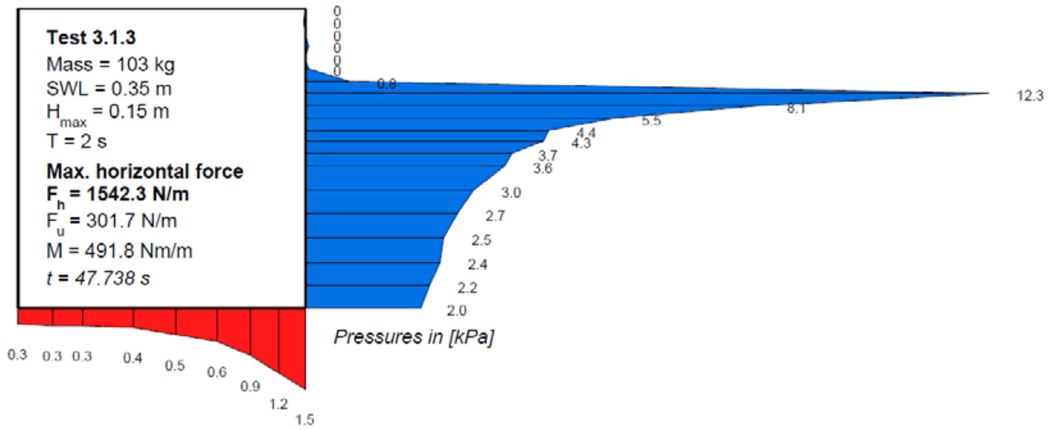

Figure 6 – Pressure distribution corresponding to the maximum measured force.

Applied loads

The horizontal and vertical load have been found by pressure integration.

Fig. 7 shows the typical "church roof" shape of the load in time, in case of breaking waves applied on a vertical wall. The impulsive contribution may be described by a triangular shape with base (total duration) of the order of 0.1-0.3 s.

The final displacement of the mobile caisson is very small. Else, it would have been possible that the regressed caisson had trapped some water, causing a "pocket effect" and thus inducing a different phenomenon. The pocket effect can be understood analyzing the different experimental results obtained by of Hattori et al. (1994) on a vertical wall and by Kisacick et al. (2014) for a confined wall, where pressures do not abide. Hattori et al. (1994) found large pressure impulses, related to the air content, that were not considered to be applied simultaneously on the wall. Kisacick et al., (2014), for a upper confined wall, found large pressure peaks with a wide spatial coherence.

Table 3. Experimental rise time for horizontal forces

Author	Scale	Rise time t_r relative to larger forces	Comments
Blackmore & Hewson, 1984	Prototype	0.15 s	t_r related to impulse (constant impulse)
Kirkgöz, 1991	Small scale	-	t_r related to impulse (qualitative)
Marinski and Oumeraci, 1992	Small scale	0.01-0.13 T	Based on h_s/H, reference to Russian papers
Martinelli and Lamberti, 2002	1:100, basin	0.1 T_p	t_r related to impulse, statistics given
Bullock et al., 2004	1:4	0.007 s	-
Wolters et al., 2004	1:4	0.0004–0.05 s	-
Bullock et al., 2007	1:4	0.08–0.2 & 0.1–0.45 s	Low aeration & high aeration case
Cuomo et al. 2010	Large scale	0.1 s	t_r related to impulse

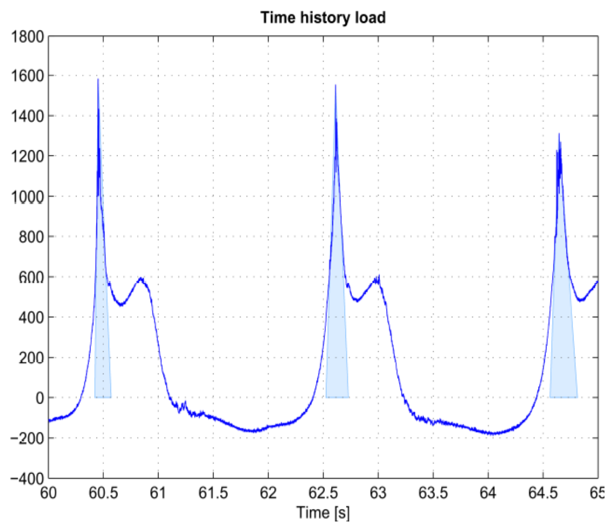

Figure 7 – Typical recorded horizontal load.

Martinelli and Lamberti (2002) present the statistics of the force time history based on a comprehensive set of wave conditions. Table 3 summarizes a few other studies on this issue. Blackmore (1982) present an overview of older studies, dated back to 1937.

The measured loads are compared to Goda formulae, with and without the contribution proposed by Takahashi for the horizontal loads (for reference see for example Goda, 2010).

Figs. 8 and 9 shows the results of the comparison between the measured loads and moments and the predictions based on Goda design formulae. The measured design loads are obtained selecting the average of 1/250 upper quantile for each test, although under regular wave conditions the statistic value of this operation is somewhat doubtful. Tests labeled “irregular” are the irregular waves in Table 1, and may contain few breakers, that are bound to be important in the averaging procedure, but breakers are not severe. Regular non-breaking waves (first two rows in Table 1) are labeled “regular” and are essentially regular non-breaking, whereas tests labeled “breaking” (third row in Table 1) are regular breaking waves.

Since the 12 wave attacks in Table 1 are applied to three structures of different weight but same geometry, in the quasi-static deterministic scenario the dataset of maximum loads would is formed by 3 sets with the same measured values. This is obviously not found in breaking conditions, due to the intrinsic nature of the breaking process, but is approximately true for the 6 non-breaking tests, and in fact the three points relative to the same tests are quite overlapped in the graph.

Fig. 8 also shows that the prediction is suitable to interpret our tests, except in case of (regular) breaking waves. The expected overestimation of the load, equal to 0.90 and 0.77 for horizontal and uplift loads respectively (see Coastal Engineering Manual 2011, Tab VI-5-53), was somewhat observed for the nonbreaking cases.

Goda formula does not apply to breaking wave conditions, and in fact it severely underestimates the applied load. Takahasi provides a well-known extension to the Goda formula to include such conditions. It should be noted that the extension does not apply to uplift forces, whereas the data in Fig. 8 show that also the uplift forces are not well interpreted by the original Goda formula.

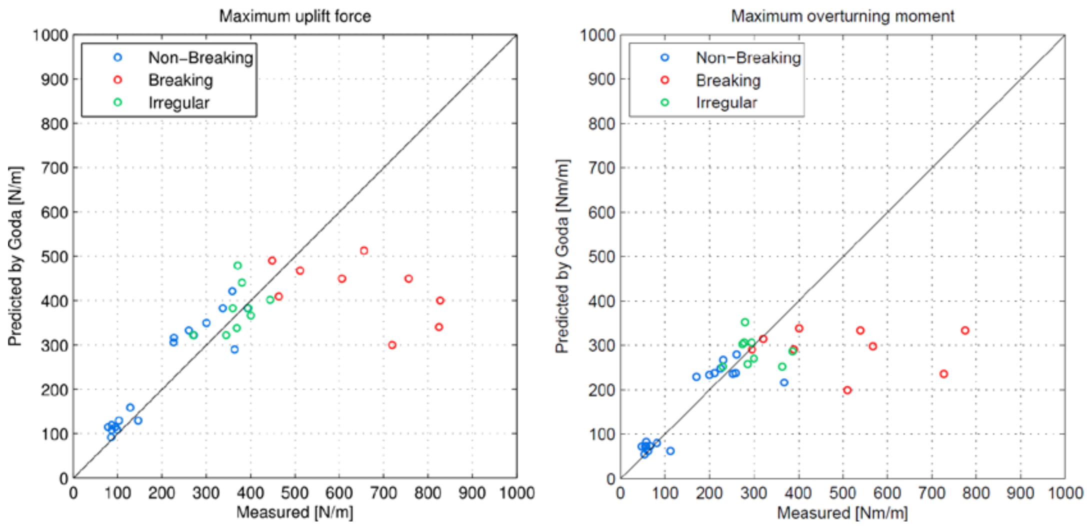

Figure 8 – Maximum horizontal and vertical load per each test, compared to Goda Formula.

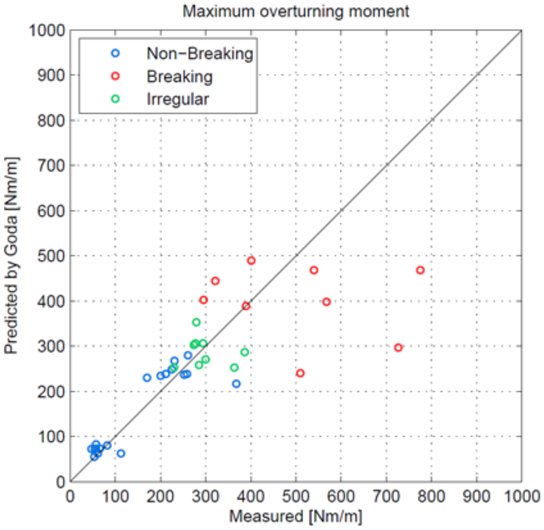

Figure 9 – Maximum moment per each test, compared to Goda Formula.

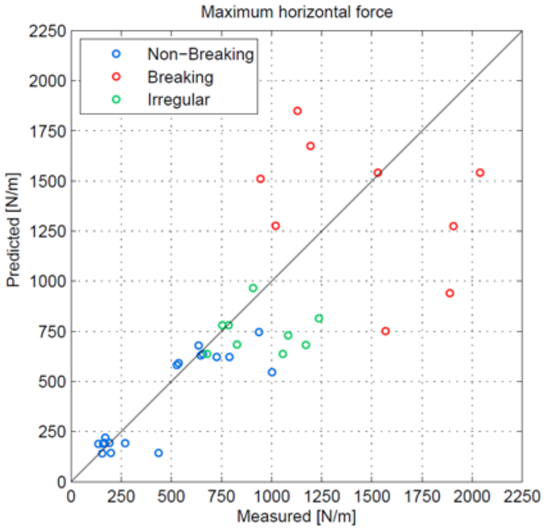

Figure 10 – Maximum horizontal load per each test, compared to Goda –Takahashi formula.

Fig. 10 shows the comparison based on the Goda-Takahashi formula. It is clear that the extended formula interprets well also the maximum loads due to breaking wave condition, but the experimental scatter is too large to have a good prediction.

Loads and sliding

Following Shimosako et al. (1994), a sliding force has been defined. Starting from the relation that describes the sliding limit, it is straightforward to derive a sliding force F_S as:

$$F_S = F_h + \mu F_u > \mu\, W' \qquad (1)$$

where F_h is the horizontal force, μ is the friction factor, F_u is the simultaneous vertical force due to the wave induced uplift pressures, W' is the weight reduced for buoyancy. The sliding occurs when the sliding force exceeds the threshold $\mu\, W'$.

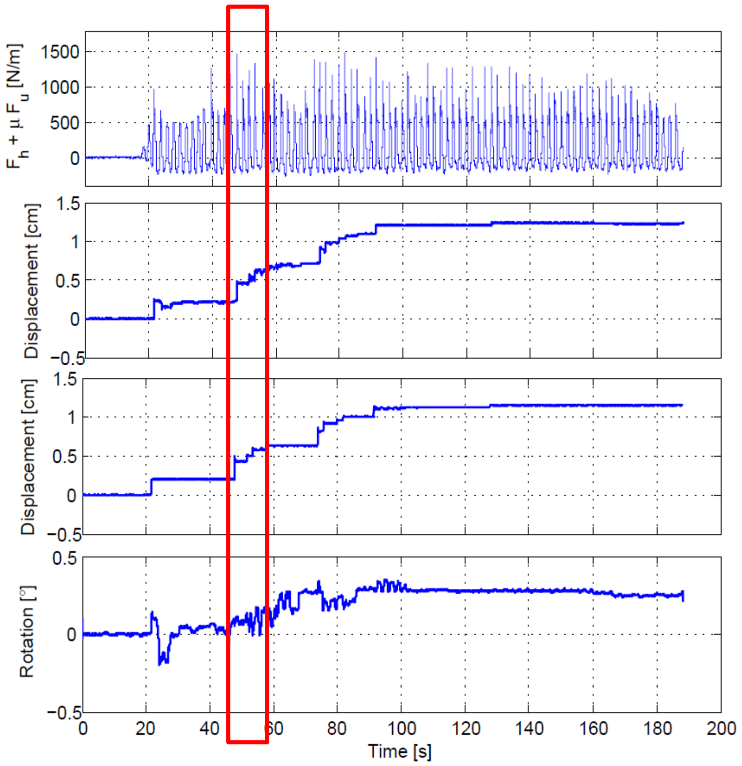

Figure 11 – Typical results and analysed range (red box).

Fig. 11 (upper panel) shows the measured sliding force compared to the "quasi-static" sliding limit indicated by a horizontal red line (L.H.S. of Eq. 1). The second and third panels show the movements of the caisson as measured by the two displacement transducers. The lower panel confirms that the rotation is almost zero, the maximum rotation being lower than 0.5°, and that in fact the observed process is merely sliding. The pressure distribution in Fig 11 is given in more detail in Fig. 6.

Fig. 11 also shows that the first sliding event might be larger than expected, and that after some time the caisson does not move any more. The first is possibly due to a non-perfect initial positioning of the caisson in the breakwater, the second to a possible blockage effects (e.g. sand between the gap between the mobile and the fixed caissons), that increases the overall friction. Cleary it would be difficult to understand the measured residual sliding in absence of a continuous measurement of the caisson movements. It was decided that the analysis should focus on a small time range, highlighted by the red box in Fig. 11.

Fig. 12 shows a snapshot of the wave impacting the vertical wall, for the maximum recorded horizontal force. The rightmost panel shows that the displacement at the instant of the peak force, marked with the red circle, has not yet occurred: in fact the 0.25 mm sliding takes place immediately after. After such impact, a large flip-through (= upward water jet) was observed.

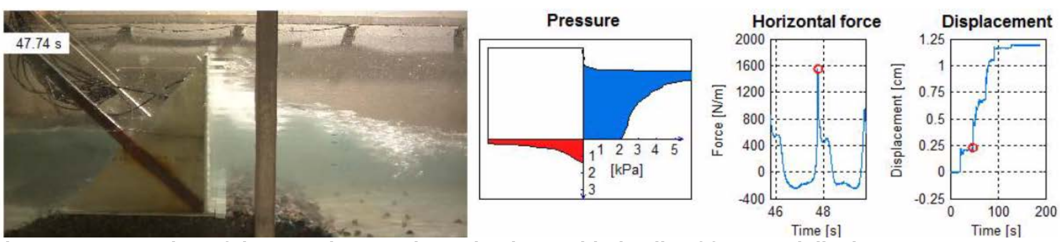

Figure 12 – Snapshot of the wave impact shown in Fig. 6, with details of force and displacement.

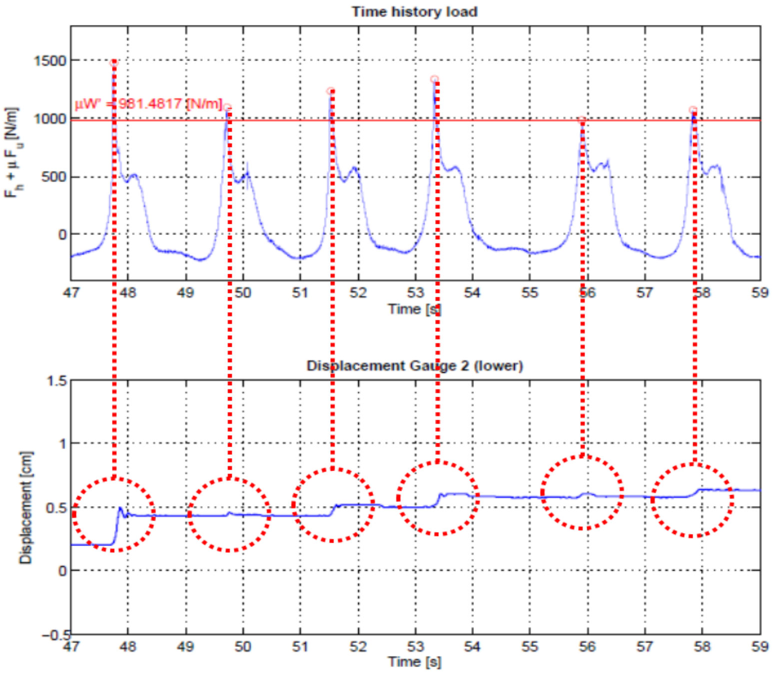

Figure 13 – Simultaneous record of sliding forces and displacements.

Fig. 13 shows a zoomed view of the sliding wave force (defined as in Eq. 1) and of the horizontal displacement in the time interval pointed out in Fig. 11. In the upper panel, the threshold limit for sliding is indicated by a red line. It may be observed that the sliding distance increases almost every time the threshold is exceeded, with the exceptions of cases 2 and 5 (at t = 50 s and 56 s), that are correctly interpreted assuming an elastic macroscale movements of the caisson on the foundation.

The quasi-static definition of sliding force accurately predicted the sliding triggering condition, although this is due to absence of dynamic amplification effects (dynamic force amplification was almost 1.0).

Table 4 presents the total sliding measured during the part of the test that is considered reliable (see red box in Fig. 11), in terms of horizontal displacement (*D*) and rotation (*R*) and it may be observed that the rotation is negligible.

Figure 14 presents a comparison between the measured and computed values of sliding. Two methods are used for the computations. In the one labeled "a-priori", the force is predicted using Goda-Takahashi and the impulse duration is predicted using the suggestion of Shimosako formula (described for instance in Shimosako et al. 1994), although it is clearly not suited to breaking wave conditions. The obtained sliding per wave is multiplied by the number of waves in the time series. As also found by Kim et al (2004), this approach overpredicts the measurements. The results obtained with a second method are labeled "measure-based". In this second case, the sliding distance is evaluated wave per wave, using the same formula but using the actual value of the force and rise time. Fig. 15 details the results of the second computation. The upper panel shows the sliding force and the lower panel the evaluated sliding.

Table 3. Measured sliding (D) and rotation (R).

Mass [kg]	d [cm]	H [cm]	T [s]	D [cm]	R [deg]
103	33	25	2	0.46	-0.20
103	35	27	2	1.15	0.26
113	33	25	2	0.03	-0.08
113	35	27	2	0.11	0.22

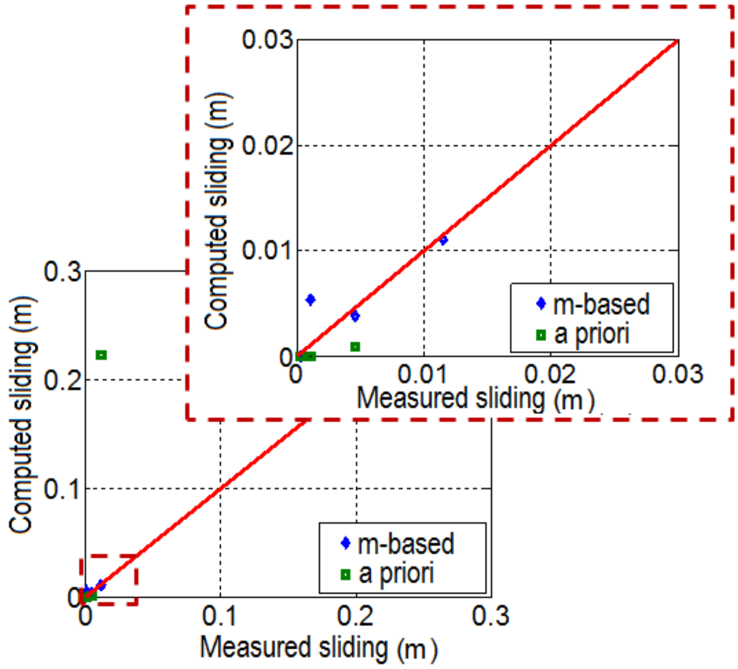

Figure 14 – Comparisons between measures and predictions.

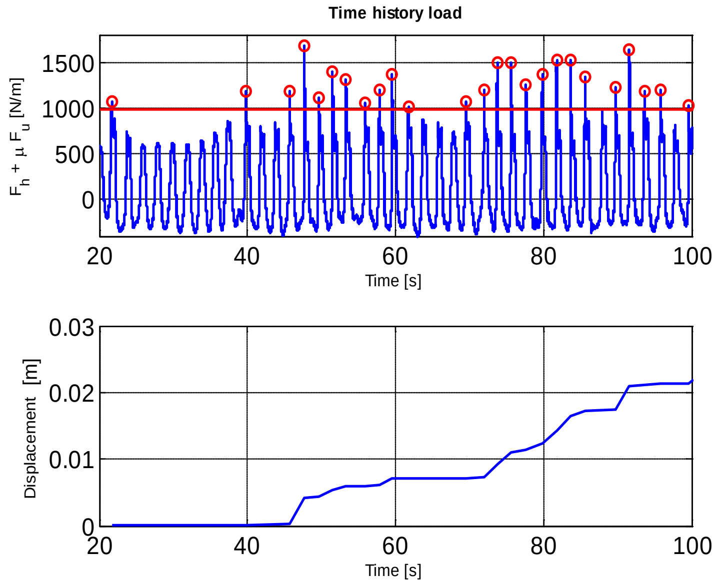

Figure 15 – Wave flume.

The prediction of the measured-based sliding events are much more accurate, proving that the dynamic formula proposed by Shimosako is sound and reliable, if the exact input values are used. Clearly, the large but tolerable uncertainty on the prediction of the force, inherent in the breaking process, and the use of an appropriate formula for the rise time, generate a very large and intolerable bias on the average sliding. Therefore, for design purposes, some method is needed to correct for this bias. Only after these problems are solved, it is worthwhile to use dynamic models that account for all the details neglected by the Shimosako formula.

CONCLUSIONS

Physical model tests on the force applied to caisson breakwaters were carried out in the wave flume of the Civil Department of Aalborg University (DK).

The possible sliding induced by breakers was specifically investigated, and it was concluded that:

- the applied force is well interpreted by existing formulas; nevertheless uplift forces seem to be underestimated by existing formulations for breaking wave conditions
- the predicted incipient sliding condition is in good agreement with measurements
- final sliding distance was here not yet compared to calculation (further work)
- the smallest sliding induced by a single wave was of the order of the diameter of the rubble mound material

REFERENCES

Blackmore (1982): Evaluation of wave forces on seawalls, PhD. Thesis, Plymouth Polytechnics.

Blackmore P.A., Hewson P.J. 1984. Experiments on full scale wave impact pressures. Coastal Engineering, 8, 331-346.

Bullock G., C. Obhrai, D.H. Peregrine, H. Bredmose Violent breaking wave impacts Part 1: results from large-scale regular wave tests on vertical and sloping walls Coast. Eng., 54 (8) (2007), pp. 602–617

Bullock G., C. Obhrai, G. Wolters, G. Müller, H. Peregrine, H. Bredmose (2004): Characteristics and design implications of breaking wave impacts Proc. of the 29th Inst. Conference on Coastal Engineering, ICCE 2004Vol. 4 (2004), pp. 3966–3978

Cuomo G., W. Allsop, T. Bruce, J. Pearson (2010) Breaking wave loads at vertical seawalls and breakwaters, Coast. Eng., 57 (4) pp. 424–439.

Franco, L. (1994): Vertical breakwaters: the Italian Experience. *Coastal Engineering*, 22, 31-55.

Goda (2010): Random Seas and Design of Maritime Structures. Advanced Series on Ocean Engineering. World Scientific, 3rd Ed., 732 pp.

Hattori M., Arami A., Yui T., (1994). Wave impact pressure on vertical walls under breaking waves of various types., 22 (1–2), 79–114

Kim, T.M., Takayama, T. & Miyawaki, Y. (2004) Laboratory Experiments on the Sliding Distance and Tilting Angle of a Caisson Breakwater Subject to Wave Impact, Proc. 29th Int. Conf. on Coastal Eng., ASCE, pp 3762-3773.

Kirkgöz, M.S. (1991): Impact pressure of breaking waves on vertical and sloping walls. Ocean Engr., 18, 45-49.

Kisacik D., Troch P., Van Bogaert P., Caspeele R. (2014): Investigation of uplift impact forces on a vertical wall with an overhanging horizontal cantilever slab, Coastal Engineering, 90, 12-22

Lamberti A., L. Martinelli., M.G. Gaeta, M. Tirindelli, J. Alderson, 2011: Experimental spatial correlation of wave loads on front decks, Journal of Hydraulic Research Vol. 49, No. S1, pp. 81–90.

Marinski J.G. & H. Oumeraci (1992): Dynamic response of vertical structures to breaking wave forces - review of the CIS design experience. Proc. ICCE1992, ASCE, vol. 23, Part 2, pp. 1357-1370

Martinelli L., A. Lamberti (2011). Dynamic response of caisson breakwaters: suggestions for the equivalent static analysis of a single caisson in the array, Coastal Engineering Journal 53, (**1**), 1-20.

Oumeraci H. (1994): Review and analysis of vertical breakwater failures: lesson learned. *Coastal Engineering*, 22, 3-29.

Shimosako K., S. Takahashi, K. Tanimoto (1994): Estimating the sliding distance of composite breakwaters due to wave forces inclusive of impulsive forces Proc. of 24th Int. Conf. Coastal Eng, ASCE, Kobe (1994), pp. 1580–1594

Shimosako, K. & Takahashi, S. (2000). Application of Expected Sliding Distance Method for Composite Breakwaters Design, Proc. 27th Int. Conf. on Coastal Eng., ASCE, pp 1885-1898.

Wolters G., G. Müller, G. Bullock, C. Obhrai, H. Peregrine, H. Bredmose (2004): Field and large scale model tests of wave impact pressure propagation into cracks Proc. of the 29th Inst. Conference on Coastal Engineering, ICCE 2004vol. 4 (2004), pp. 4027–4039

Zhang S., Yue D.K.P., Tanizawa K. (1996): Simulation of plunging wave impact on a vertical wall., J. Fluid Mechanics, 327, 221-254.