Wave-induced Uplift Loading of Caisson Breakwaters

Andreas Kortenhaus[1], Hocine Oumeraci[2], Søren Kohlhase[3] & Peter Klammer[1]

ABSTRACT

Wave induced uplift pressures and forces have been investigated by small- and large-scale model tests. Detailed analyses of waves at the structure, forces and structure motions showed that conventional design formulae neglect details of pressure development which contribute to the total uplift force. Direct comparison between quasi-static, dynamic forces and the commonly used formulae show that dynamic effects exhibit a significantly different behaviour in comparison to those observed during quasi-static wave attack. The most relevant of the effects which may be useful for setting up a numerical model for the prediction of uplift pressures are described.

1. INTRODUCTION

Wave induced uplift forces for design purposes are commonly calculated by empirical or semi-empirical formulae based on the assumption that the maximum uplift pressure occurs at the seaward bottom edge of the caisson and decreases linearly to zero at the shoreward edge. Generally, it is also implicitely assumed that the maximum horizontal peak force occurs simultaneously with its maximum uplift counterpart and that the caisson structure is fixed and does not exhibit any motion during wave loading. Moreover, the influence of the characteristics of the rubble foundation (permeability, thickness etc.) is totally ignored.

The wave induced flow in the rubble foundation has been studied in a large-scale and a small-scale-model by using a fixed and a movable caisson break-

[1] Dipl.-Ing., Franzius-Institut, University of Hannover, Appelstr. 9A, 30167 Hannover, Germany

[2] Prof. Dr.-Ing., University Professor, Techn. University Braunschweig, Beethovenstr. 51a, 38106 Braunschweig, Germany

[3] Prof. Dr.-Ing., Institut für Wasserbau, University of Rostock, Philipp-Müller-Str., 23966 Wismar, Germany

water. The tests principally aim at a better understanding of the generation of uplift pressure under standing, breaking and broken wave conditions and at the establishment of a basis for the development of a conceptual model for the calculation of uplift pressures.

The present paper primarily intends to discussing some of the experimental results. Based on these results an attempt is made to explain the physical processes underneath a vertical caisson structure subject to breaking and nonbreaking wave attack.

2. EXPERIMENTAL SET-UP

Small-scale and large-scale model tests have been conducted on a caisson breakwater supported by a rubble foundation (Fig. 1). These investigations were

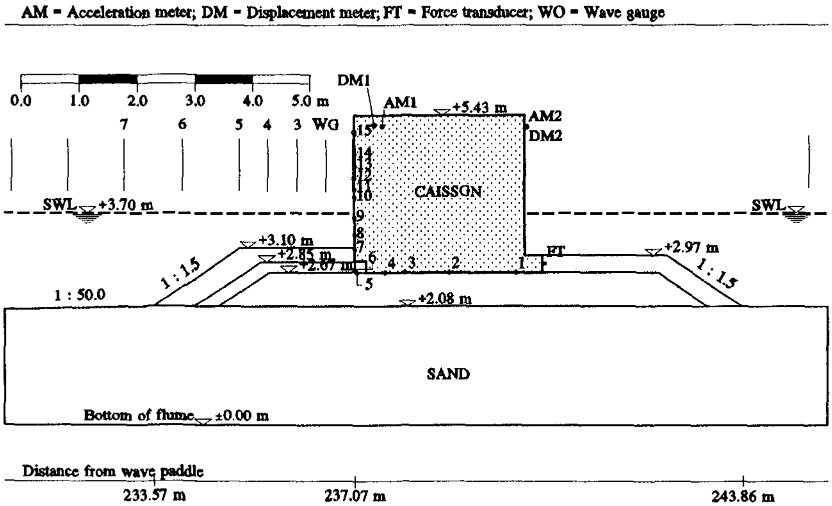

Fig. 1: Large-scale model of caisson breakwater

principally directed towards the examination of the effect of the motions of the caisson on the uplift pressure development. For this purpose, five pressure gauges were installed underneath the structure and two displacement meters were placed at the rear and front top of the caisson, respectively. Further seven pressure gauges were installed along the toe berm in front of the caisson in order to determine the impact pressure gradients along the rubble toe, and hence to better understand the mechanisms associated with the uplift generation. Tests with a fixed and a movable caisson were performed. Further simultaneous measurements of waves, accelerations, total forces and pore pressures in the rubble foundation were conducted.

Since the small-scale model tests are not suitable for the quantitative evaluation of the uplift pressure, flow in the porous rubble foundation is subject to scale effects due to too large viscous forces, large-scale model tests were also performed for different water depths and wave conditions.

Three different wave types were generated throughout the tests: solitary waves, regular waves and random waves (TMA spectra). The wave heights, wave periods and water depths used in the large-scale model tests are summarized in Table 1.

Table 1: Test programme for large-scale model tests

SOLITARY WAVES					
Wave heights	steps	**Wave periods**	steps	**Water depth**	steps
0.40 - 0.60 m	0.10 m	-	-	3.10 - 3.50 m	0.10 m
0.65 - 1.10 m	0.05 m	-	-	3.50 - 3.90 m	0.20 m
REGULAR WAVES					
Wave heights	steps	**Wave periods**	steps	**Water depth**	steps
0.30 - 1.10 m	0.20 m	3.50 - 6.50 s	1.00 s	3.70 - 4.30 m	0.20 m
RANDOM WAVES (TMA spectra)					
Wave heights	steps	**Wave periods**	steps	**Water depth**	steps
0.30 - 1.10 m	0.20 m	3.50 - 6.50 s	1.00 s	3.70 - 4.30 m	0.20 m

The small-scale model was about four times smaller than the large-scale model.

3. CLASSIFICATION OF BREAKER TYPES

Three major breaker types were identified from the experiments leading to essentially different response of the structure (Fig. 2). All these breaker types could be observed - independently from the type of waves generated. Generally there are more than three different breaker types (*Schmidt et al., 1992*). However, only the three main types shown in Fig. 2 for two time steps t_1 and t_2 are considered below in order to make more clear the physical processes involved.

Four parameters were found suitable to describe the main features of the force histories of both impact and uplift pressures (Fig. 3). These parameters are:

- $\mathbf{t_r}$: rise time of impact force
- $\mathbf{\Delta t}$: time difference between maximum of impact force and maximum of uplift force

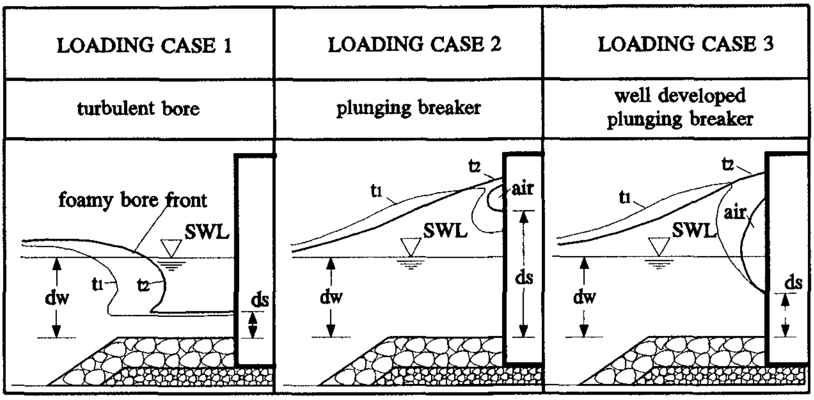

Fig. 2: Main breaker types considered

- $\eta_{Fm,q}$: ratio of quasi-static impact force $F_{h,qus}$ and maximum impact force $F_{h,max}$
- $\eta_{Fh,v}$: ratio of maximum uplift force $F_{v,max}$ and maximum impact force $F_{h,max}$

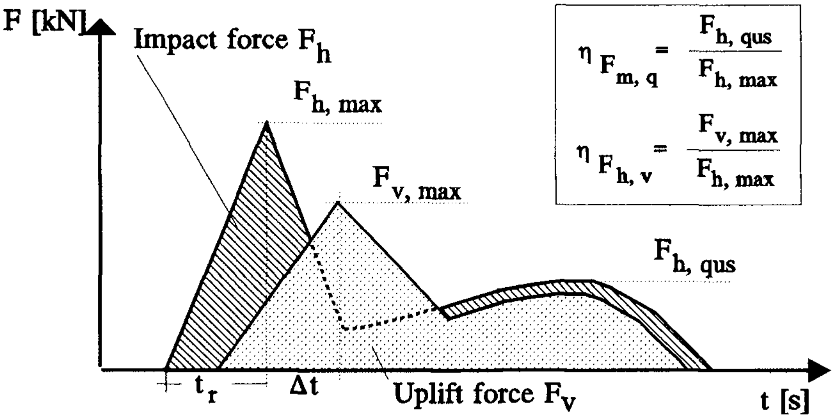

Fig. 3: Parameters for identification of loading cases

4. TYPICAL TEST RESULTS

Force histories

Typical force histories for impact and uplift forces for the three different loading cases can be seen in Figs. 4 to 6 where the aforementioned descriptive parameters are given.

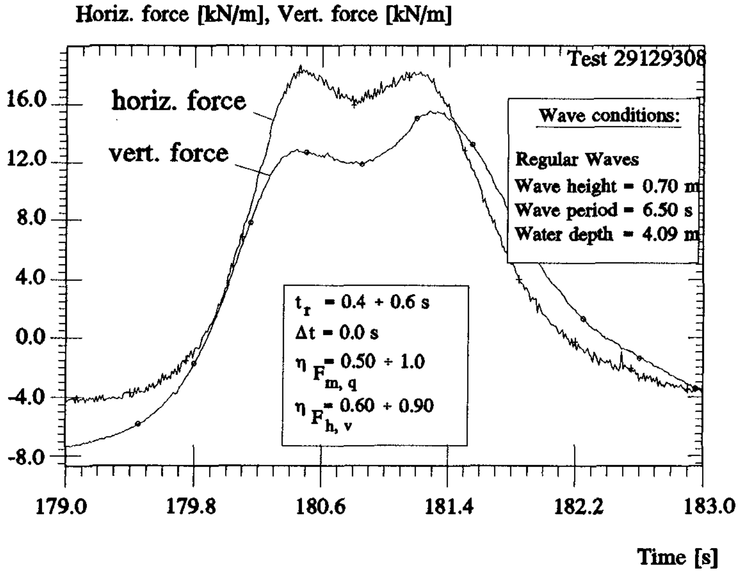

Fig. 4: Typical force histories for loading case 1 (turbulent bore)

The force histories show a high force peak ($F_{h,max}$) caused by the wave hitting the structure and a lower force maximum $F_{h,qus}$ which occurs when the water of the wave running up the wall falls down again. Between these maxima there is a minimum force (trough) which coincides with the maximum wave run-up.

In loading case 1 (turbulent bore) the wave breaks before reaching the structure, inducing a turbulent air-water mixture. This can be observed from the high frequency oscillations in the time history of the impact forces. There is no time lag between the maxima of impact and uplift force.

The uplift force in loading case 2 shows a double peak for the vertical force which however do not always occurs when a plunging breaker hits the structure. This double peak force is due to the uplift of the structure, which will be discussed in the next chapter.

Loading case 3 shows a very sharp and high peak in the impact force history followed by high frequency oscillations which are due to oscillations of the

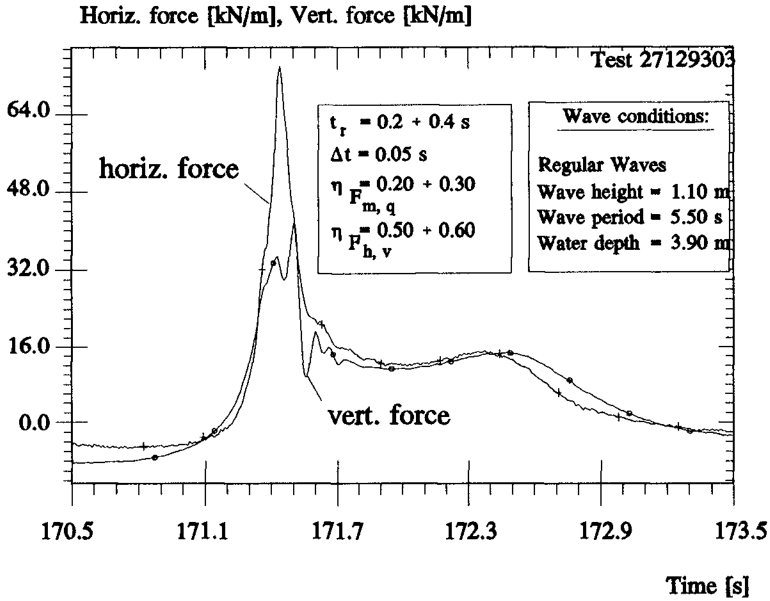

Fig. 5: Typical force histories for loading case 2 (plunging breaker)

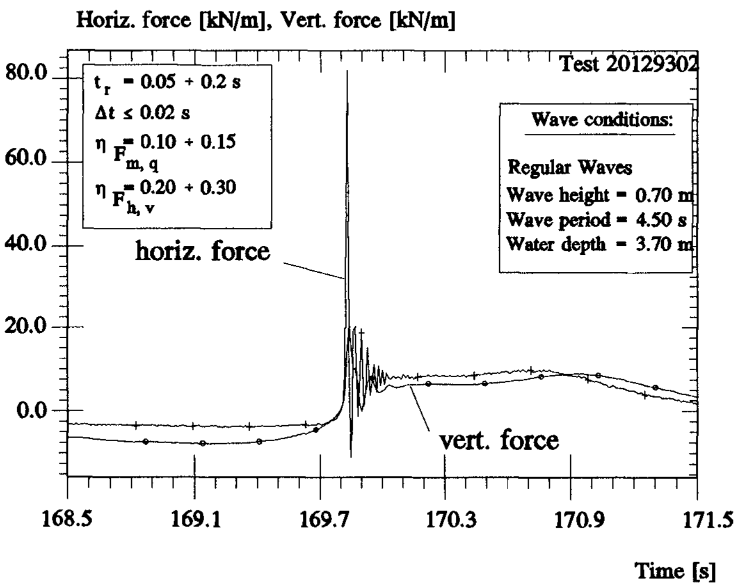

Fig. 6: Typical force histories for loading case 3 (plunging breaker with air entrapment

entrapped air pocket at the vertical face of the structure. Very often a double peaked force in the impact force history occurs. The first peak is induced by the beaker tongue hitting the structure whereas the second occurs when the rest of the wave front impinges on the structure.

Pressure distributions

Typical horizontal impact pressure distributions on the front and the bottom of the caisson caused during loading case 2 are given in Fig. 7. In this figure the motion of the structure is amplified by a factor of 200 to make clear:

- whether the structure moves at all,
- the comparison of all loading cases,
- at which time the structure starts to move.

The following typical features of impact und uplift pressures are observed for loading case 1:

- there are almost no temporal changes in the figures (quasi-static behaviour)
- maximum impact pressure is approximately at still water level
- measured impact pressure distribution is very close to the theoretically predicted (SAINFLOU) distribution for a standing wave with little scatter due to reflection coefficients which are actually less than 100%.
- uplift pressures are almost linear but not decreasing to zero pressure at the shoreward edge of the structure
- velocity flow in the rubble foundation has been estimated by an improved Forchheimer equation (*Van Gent, 1993*) to about 5 to 10 cm/s. Therefore, pressures due to velocity head can be neglected

For loading case 2 (Fig. 7), the typical features which are worth to mention are:

- point of application of impact force is in the range of still water level, but the magnitude of the force is much higher
- there are mainly horizontal motions of the structure, but only some slight rotational motions
- compression wave underneath the structure
- lever arm of uplift force is at 2/3 of the width of the structure from its heel

Loading case 3 is an extreme example for plunging breakers with air entrapment inducing a very high short time impact peak followed by high amplitude oscillations. Impact and uplift pressures exhibit the following typical distribution:

- maximum impact is reached within a very short time
- pressure oscillations due to air entrapment occur about 0.2 s after maximum peak
- no motions of structure is induced since peak pressures are too short
- relatively low uplift pressures (showing phenomenon of compression wave)

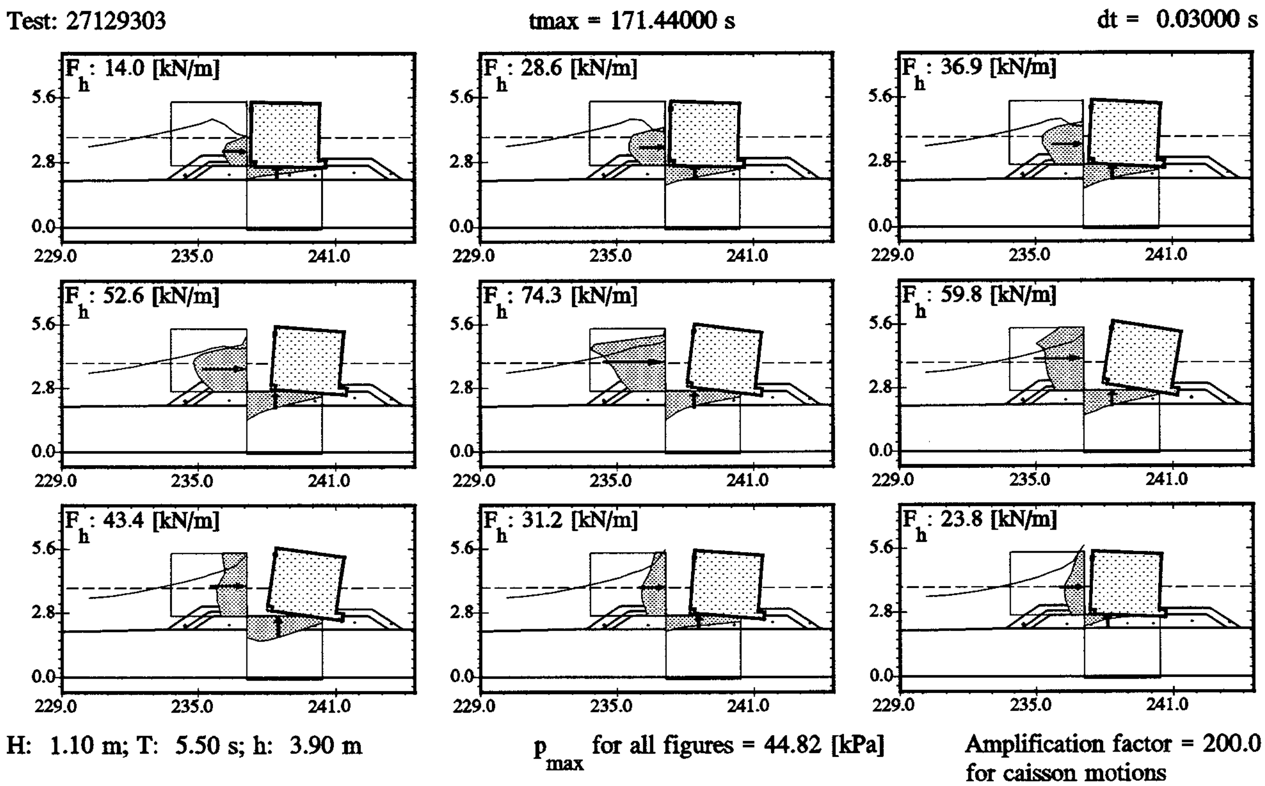

Fig. 7: Typical pressure distribution for loading case 2

Comparison to Goda formulae

For the prediction of horizontal wave and uplift forces the Goda formulae are widely used (*Goda, 1985*). Calculations using these formulae have been performed for the loading cases described in section 3 to show the range of quasi-static and dynamic loads. The results of these calculations are compared to the measurements for each of the loading cases in Figs 8 to 10.

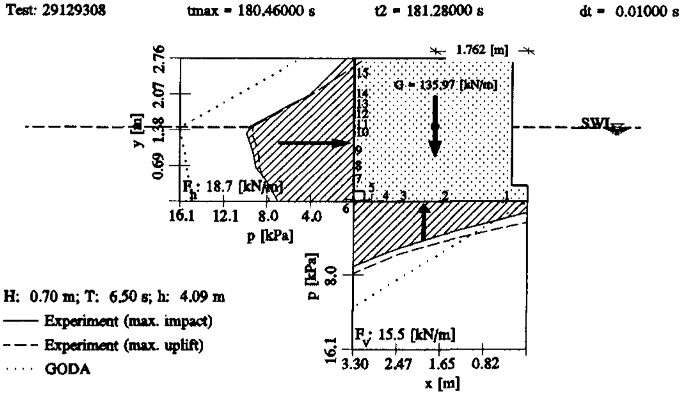

Fig. 8: Comparison of loading case 1 with GODA's formulae

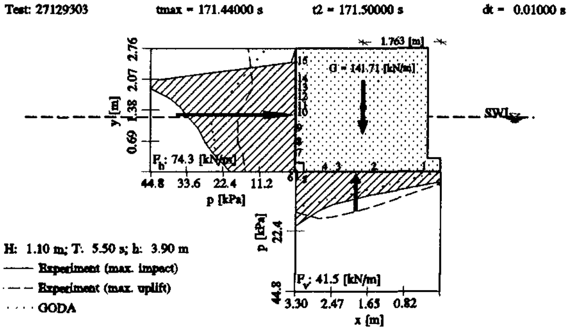

Fig. 9: Comparison of loading case 2 with GODA's formulae

For quasi-static loads (loading case 1) the results obtained by Goda formulae are conservative whereas the pressure peaks in the case of highly dynamic loads exceed the calculated pressure by a factor up to 5. Goda's formulae predict approximately correct uplift pressure values at the seaward edge of the breakwater for loading cases 1 and 3, but not for dynamic loading case 2. In this case the maximum uplift force occurs when the compression wave is underneath the structure and has reached 2/3 of the width of the structure from its heel.

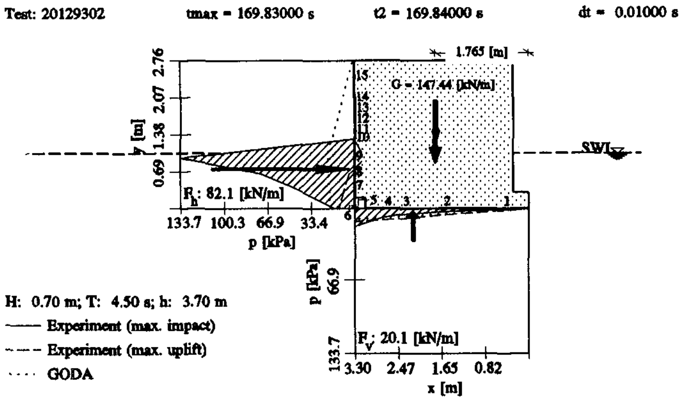

Fig. 10: Comparison of loading case 3 with GODA's formulae

5. OBSERVED UPLIFT PHENOMENA

Compression wave

For loading cases 2 and 3 a compression wave is induced beneath the structure by the wave impact on the vertical face of the breakwater. To obtain more information about the propagation, velocity of this wave underneath the structure and its effect on the porous rubble foundation the uplift pressure recorded beneath the structure have been plotted for a single wave in Fig. 11. The maxima of the obtained curves were determined and time differences have been plotted versus the

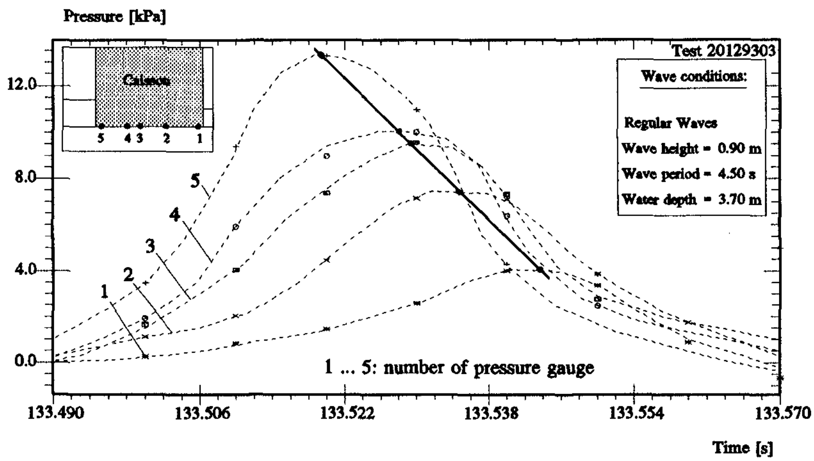

Fig. 11: Analysis of compression wave underneath the structure

spacial difference dx_i between the respective pressure gauges. This procedure was repeated several times for a test with regular waves (Fig. 12). Similar results have been found for further regular and random waves, showing that the velocity of the compression wave is not affected by the wave parameters.

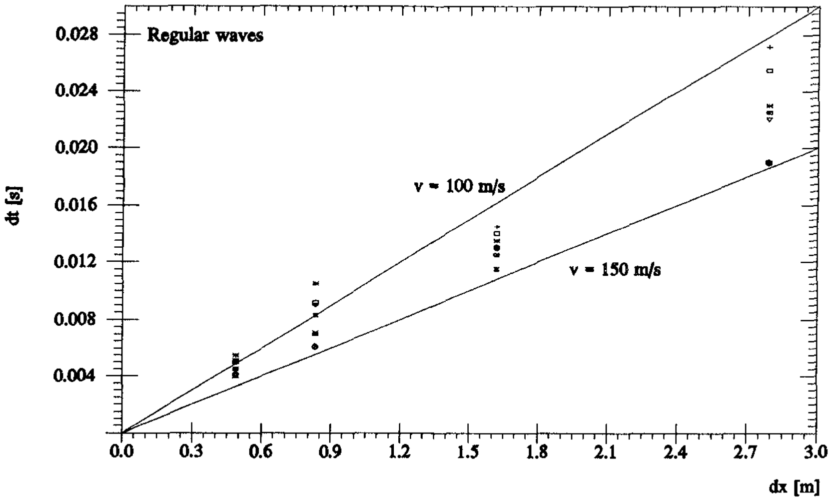

Fig. 12: Velocity of compression wave beneath the structure

Suction underneath the structure

Due to the very high impacts inducing a strong horizontal and rotational motion of the structure a gap forms at the front base (Fig. 13). Pressure decreases

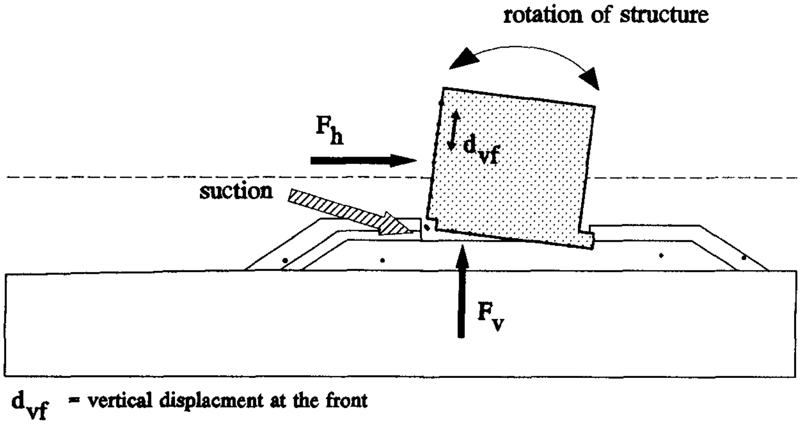

Fig. 13: Suction beneath a rotating caisson structure

immediately and water flows with relatively high velocities into the gap thus increasing the risk of erosion of the rubble materials (suction). This phenomenon was only observed for high impacts and cannot be considered as typical for normal impact loading. However, it may be a very critical case, due to the aforementioned erosion potential of the high velocity gap flow.

Fig. 14 shows simultaneously recorded uplift forces and displacements at the vertical face front of the structure. It can be seen that the increase of the displacement suddenly leads to a decrease in the uplift force (dashed line in Fig. 14). The uplift force is obtained from the integration of all pressure records at the bottom of the structure. Similar decrease can also be observed in the pore pressure records.

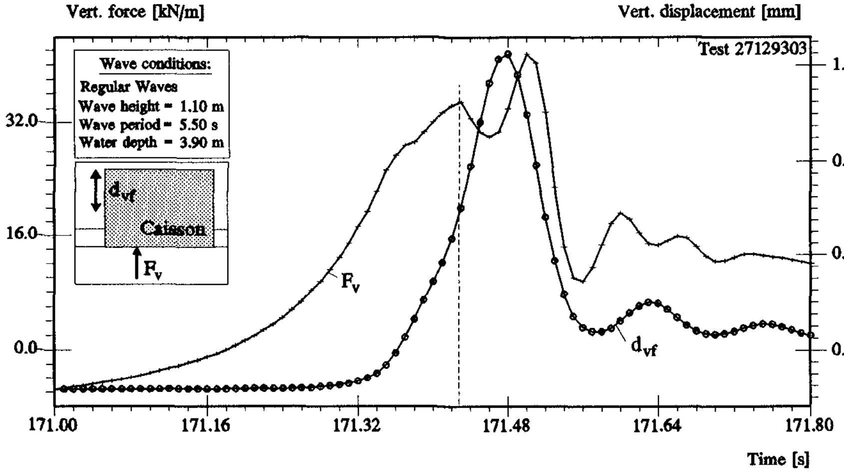

Fig. 14: Time histories of uplift force and vertical displacement at the caisson front

The reduction of uplift force caused by the suction is shown in Fig. 15 where an increase of vertical displacements at the front for small values creates an almost linear increase of uplift forces. Three almost parallel lines were plotted to show this dependency. Furthermore, it can be seen that there is a clear relationship between the water depth and the uplift force where higher water depths at the toe of the rubble mound in front of the structure will result in smaller uplift forces. However, for larger displacement values a sudden bend can be found leading to an increase of uplift forces at a lower rate with increasing displacements. A more detailed analysis in the near future will show whether there are similar relations with respect to different water depths. To obtain quantitative statements more variations in wave parameters have to be considered.

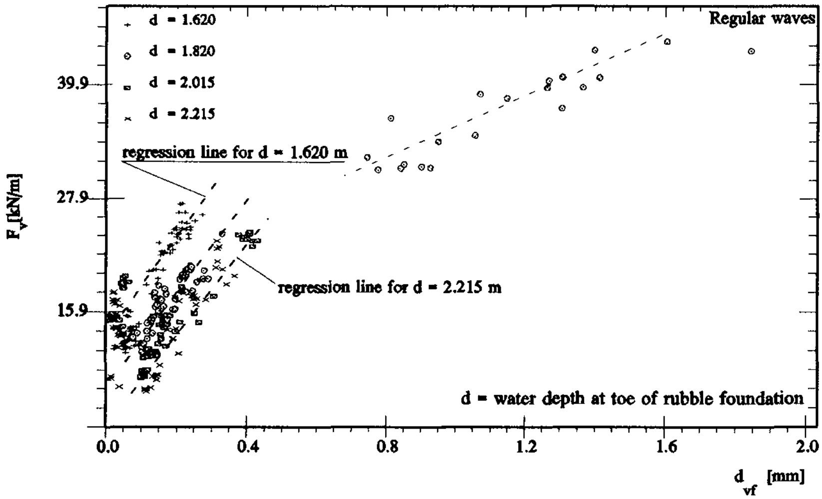

Fig. 15: Reduction of uplift force by suction underneath the structure

Pressure at shoreward edge

In all measurements of both breaking and nonbreaking waves the pressure at the shoreward edge of the breakwater was different from zero. This phenomenon was also observed under model and prototype conditions, but could not be explained (*Marchi et al., 1975*). Several reasons may be considered to explain this phenomenon:

- water level variations behind the breakwater were induced by a flow through the rubble foundation thus leading to hydrostatic pressures at the shoreward edge
- flow induced velocities are responsible for this pressure (velocity head $v^2/2g$)
- pressures may be induced by flow resistance in the rubble foundation, since the rubble structure at the rear is higher than the bottom level of the caisson

Since water level variations and flow through the rubble foundation induces pressures which are in the range of 5% of the measured pressures at the rear edge, the latter reason seems to provide the most reasonable explanation. This has to be confirmed by further model tests or numerical modelling.

6. SUMMARY OF RESULTS AND FUTURE RESEARCH TASKS

- The widely used assumption of linear pressure distribution underneath the structure is almost valid for quasi-static wave loading. For impulsive loading, however, a non linear distribution occurs. This is due to the effect of com-

pression waves beneath the structure where the maximum uplift pressure no longer occurs at the seaward edge of the structure;
- the assumption of zero pressure at the shoreward edge of the structure seems to be only valid for very low rubble foundation and should be more thoroughly examined for higher rubble substructures;
- for extreme wave conditions suction at the seaward bottom edge of the structure may occur leading to lower uplift pressures and forces at the base of the structure and in the rubble foundation, thus increasing the risk of erosion due to high velocities of the gap flow.

The influencing parameters to be investigated in the future are:
- permeability of the rubble foundation
- geometry of the structure and the rubble foundation (thickness and height)

A numerical model should be developed which accounts for a) the (turbulent) flow and pressures in the rubble mound; b) breaking and nonbreaking wave conditions and c) variations in geometry and permeability of the rubble foundation. This numerical model could be calibrated by the experimental results obtained by this study and might be used for an extensive parameter study. The results of this parameter study will help to develop simple prediction formulae for both uplift pressure distribution and forces as a function of various parameters (water depth, wave parameters, breaker type, properties of rubble material and geometry of structure and rubble foundation).

ACKNOWLEDGEMENTS

Support by the EEC within the research programme MAST II (MAS2-CT92-0047) is gratefully acknowledged. The support by the German Research Council (DFG) within the SFB 205 at the Unversity of Hannover is also gratefully acknowledged.

REFERENCES

GODA, Y. (1985): Random seas and design of maritime structures. Tokyo: University of Tokyo Press, 323 pp.

MARCHI, E.; E. RAITERI; G. SCARSI and S. STURA (1975): Storm wave pressures on the breakwater of Genoa harbour - measurement station. *Proceedings XVIth Congress of the International Association for Hydraulic Research (IAHR)*, vol. 1.

SCHMIDT, R.; H. OUMERACI and H.-W. PARTENSCKY (1992): Impact loads induced by plunging breakers on vertical structures. *Proceedings 23rd International Conference Coastal Engineering (ICCE)*, vol. 2, pp. 1545-1558.

VAN GENT, M.R.A. (1993): Stationary and oscillatory flow through coarse porous media. *Communications on Hydraulic and Geotechnical Engineering*, Report no. 93-9, Delft University, 62 pp.