

# CHAPTER 113

## Impact Loading and Dynamic Response of Caisson Breakwaters

### - Results of Large-Scale Model Tests -

H.Oumeraci<sup>1)</sup>, H.W. Partenscky<sup>2)</sup>, S. Kohlhasse<sup>3)</sup>, P. Klammer<sup>4)</sup>

#### **Abstract**

The results of large-scale model tests on impact loading and dynamic response of a caisson breakwater are presented.

Hydraulic model tests are performed in which the impact loading is induced by breaking waves on the structure. Horizontal impact forces, uplift forces and the related overturning moments are determined. The transmissibility of the impact loads and the accelerations of the structure are investigated (dynamic response).

The hydraulic model tests are supplemented by pendulum tests on the same caisson breakwater model used in the hydraulic model tests in order to determine the characteristics of the structural model itself. The added mass of water oscillating with the structure, the stiffness of the foundation and the damping ratio are evaluated from these tests.

#### **Introduction**

Based on the lessons drawn from past failures and on a literature review, a research programme on monolithic structures subject to breaking waves has been established (OUMERACI et al, 1991). The main objective of this programme is the development of design guidelines as well as the evaluation of counter-measures for increasing the overall stability of the structure and its foundation. In fact, the stability of vertical and composite breakwaters has been yet approached by the existing codes of practice and design methods of different countries as a static problem (GODA, 1985). Therefore, the underlying philosophy of these investigations is to show that a caisson breakwater/foundation system is a purely dynamic problem which cannot be simply treated statically. For this purpose, large-scale hydraulic model tests and pendulum tests on a caisson breakwaters were performed, supplemented by a dynamic analysis of the structure. The paper is principally intended to present and discuss some of the most relevant results of the hydraulic and pendulum tests. The results of the dynamic analysis of caisson breakwater by using a simple numerical model, together with the experimental results, will be presented in a further paper (OUMERACI & KORTENHAUS, 1993).

---

<sup>1)</sup> Dr.-Ing., Senior Researcher, SFB 205, University of Hannover, Appelstr. 9A, 3000 Hannover

<sup>2)</sup> Prof. Dr.-Ing., Dr.-Phys., formerly Managing Director Franzius-Institut

<sup>3)</sup> Prof. Dr.-Ing.habil., Franzius-Institut

<sup>4)</sup> Dipl.-Ing., Research Engineer, SFB 205

**Experimental Set-up and Test Conditions**

Hydraulic model tests and pendulum tests have been performed in the Large Wave Flume (GWK) in Hannover on a caisson breakwater, with a rubble mound foundation lying on a sand bed.

**a) Hydraulic Model Tests**

The hydraulic model tests were conducted by using regular and irregular waves with wave heights and periods up to 1.20m and 7s, respectively. The measurements were simultaneously performed on two independent caisson structures. Total forces were measured on the first caisson. The second caisson was installed on a rubble mound foundation lying on a 1.4 m-thick sand layer (Fig. 1).

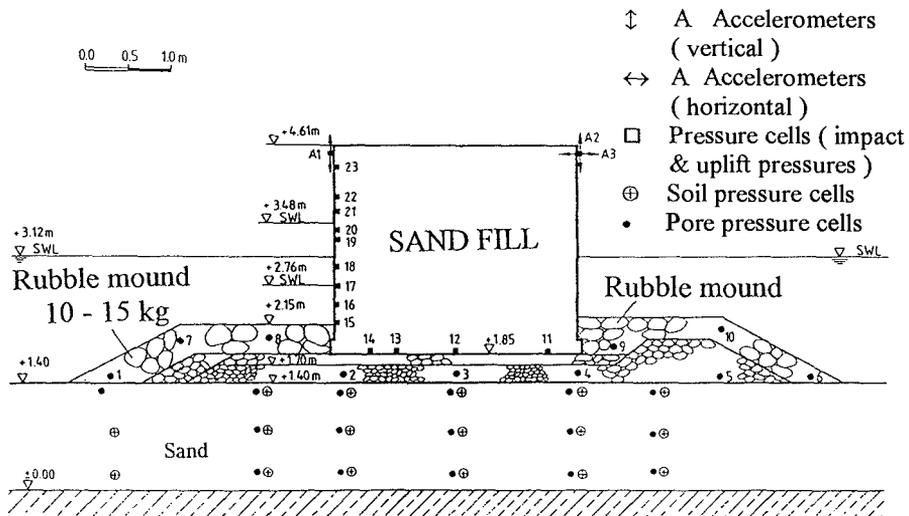


FIG. 1: CAISSON BREAKWATER TESTED IN THE LARGE WAVE FLUME (GWK)

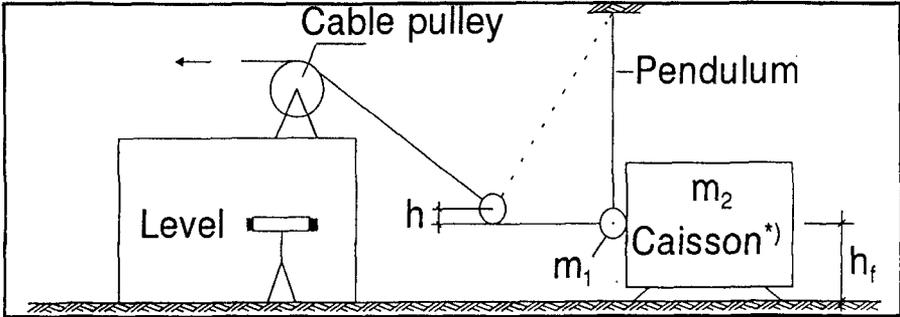
Simultaneous measurements of the following items were carried out:

- (a) incident and reflected waves, (b) impact pressure on the caisson front, (c) uplift pressure, (d) wave-induced pore-water pressure in the foundation, (e) total wave-induced stress in the sand layer, (f) dynamic response of the caisson (accelerations) and (g) total forces.

Pressure and load transducers with high natural frequencies were used. Sampling rates up to 11kHz were adopted for impact pressures and forces.

**b) Pendulum Tests**

The hydraulic model tests were supplemented by pendulum tests (Fig. 2) using the same caisson/foundation model (Fig. 1) in different water depths.



\*) Same Model as shown in Fig. 1

FIG. 2: EXPERIMENTAL SET-UP FOR PENDULUM TESTS

The impulsive loads induced by the pendulum and the response of the structure and its foundation were simultaneously recorded. The main objective of these tests was to determine the hydrodynamic mass as well as the damping and the subgrade reaction coefficients to be considered in a dynamic analysis of the caisson/foundation system. Two test series were conducted on the caisson part lying on the rubble mound foundation: Tests under dry conditions and tests with different water depths.

**Discussion of Results of Hydraulic Model Tests**

**a) Impact Pressure Histories and Distribution**

The characteristics of the impact pressure histories simultaneously recorded at nine elevations of the caisson front are strongly related to the types and kinematics of the waves breaking on the structure (OUMERACI et al, 1993). Spatial pressure distributions on the caisson front for time steps as small as  $\Delta t = 0.1\text{ms}$  can be plotted from the measured data, and the related total forces are calculated from pressure integration. An example of such pressure distributions resulting from a breaking wave with  $H = 0.85\text{m}$  and  $T = 4\text{s}$  is given in Fig. 3 for  $\Delta t = 6.3\text{ms}$ .

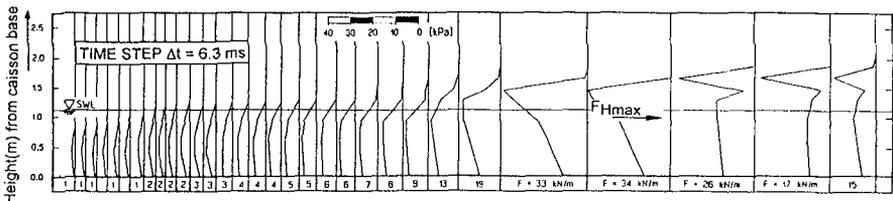


FIG. 3: PRESSURE DISTRIBUTION ON CAISSON FRONT

These pressure distributions, together with the simultaneously measured pressure histories at different wall elevations not only give a complete picture of the impact process, but also allow us to identify the most relevant characteristics of the shape of the breakers impinging on the wall.

### b) Uplift Pressure Histories and Distributions

Uplift pressures are subject to much less variation in time than the pressures on the caisson front (OUMERACI et al, 1991). For waves breaking on the caisson, the recorded uplift pressure histories exhibit a dynamic shape, even if much less sharper than that of the impact pressure on the caisson front. In this case, the uplift pressure is not linearly distributed and not equal to zero at the rear edge of the caisson, as usually assumed. This is shown for instance by Fig. 4 which represents the uplift pressure distribution at the instant of wave breaking ( $H = 0.85\text{m}$ ,  $T \approx 4.0\text{s}$ ).

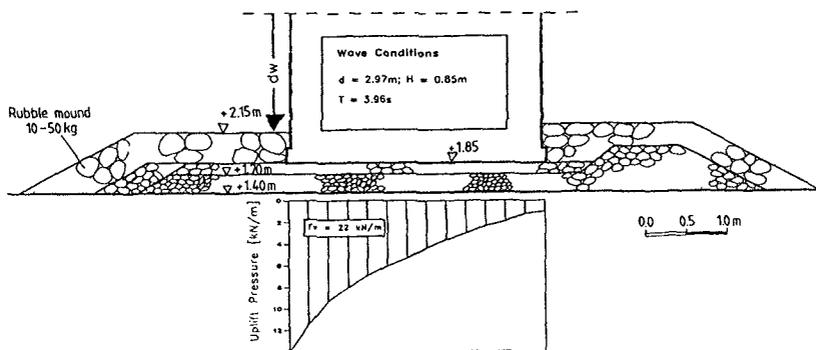


FIG. 4: UPLIFT PRESSURE DISTRIBUTION FOR BREAKING WAVES

### c) Total Forces and Overturning Moments

The total horizontal and uplift forces were obtained by spatially integrating the impact pressures measured on the caisson front (horizontal force  $F_h$ ) and the uplift pressure (vertical force  $F_v$ ). An example of the total horizontal force  $F_h$  and its overturning moment  $M_h$  around the rear edge of the caisson is shown in Fig. 5. It can also be seen that during the impact process, the location of the point of application of  $F_h$  is almost constant and slightly under still water level.

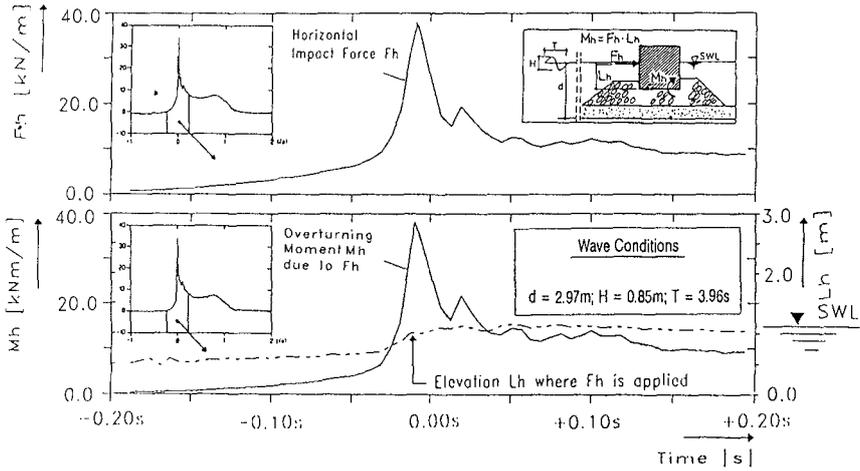


FIG. 5: HORIZONTAL FORCE AND INDUCED OVERTURNING MOMENT

The uplift force and related overturning moment which are caused by the same wave which induced the force and moment in Fig. 5 are also shown in Fig. 6. It can be seen that the point of application of the uplift force during impact is located rather at  $\frac{1}{4}$  than  $\frac{1}{3}$  of the caisson width from the seaward edge, due to the non-linear uplift pressure distribution as shown in Fig. 4. This will result in a larger contribution of the uplift force to the total overturning moment. In fact, depending on the caisson size, the water depth, the thickness of the rubble mound foundation, the breaker type and the magnitude of the horizontal force, this contribution may be higher than that of the horizontal impact forces (Fig. 7).

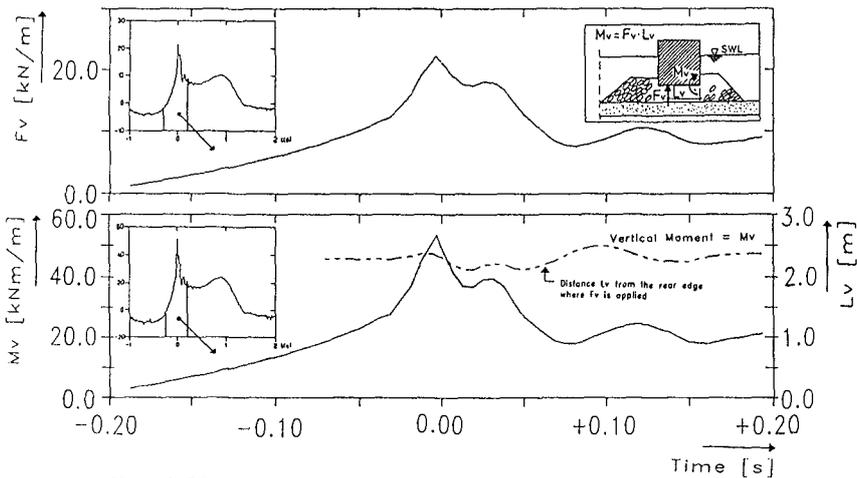


FIG. 6: UPLIFT FORCE AND INDUCED OVERTURNING MOMENT

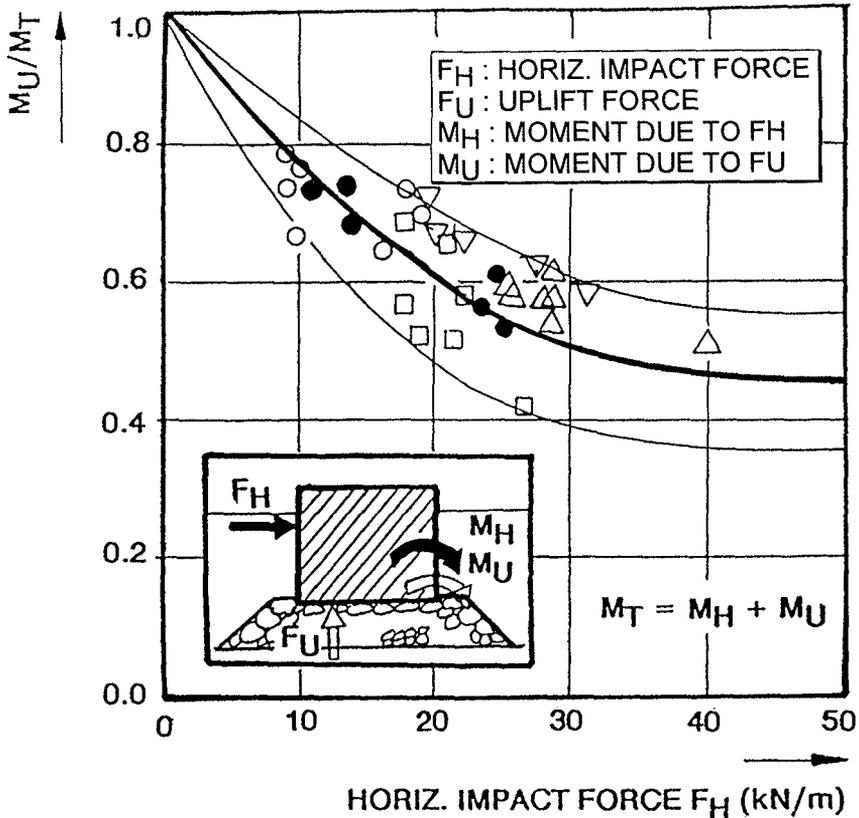


FIG. 7: CONTRIBUTION OF UPLIFT FORCE TO THE TOTAL OVERTURNING MOMENT

#### d) Dynamic Response of the Structure

##### Transmissibility of Impact Loads

The dynamic response of the first part of the model caisson behind which the total horizontal reaction force was measured is described by the transmissibility of the impact load. This transmissibility is defined as the ratio of the reaction force measured behind the caisson to the total impact force on the caisson front obtained by pressure integration over the front area. An example of the results obtained is given in Fig. 8.

This and further similar results show that for the conditions tested, the transmissibility of the impact forces is in the range of 0.10 to 0.80 whereas the transmissibility of the force impulses is about twice the transmissibility of the related peak forces. This, however, strongly depends on the shape of the impact force history which is known to be primarily determined by the shape of the breaker impinging on the wall (OUMERACI & KORTENHAUS, 1993).

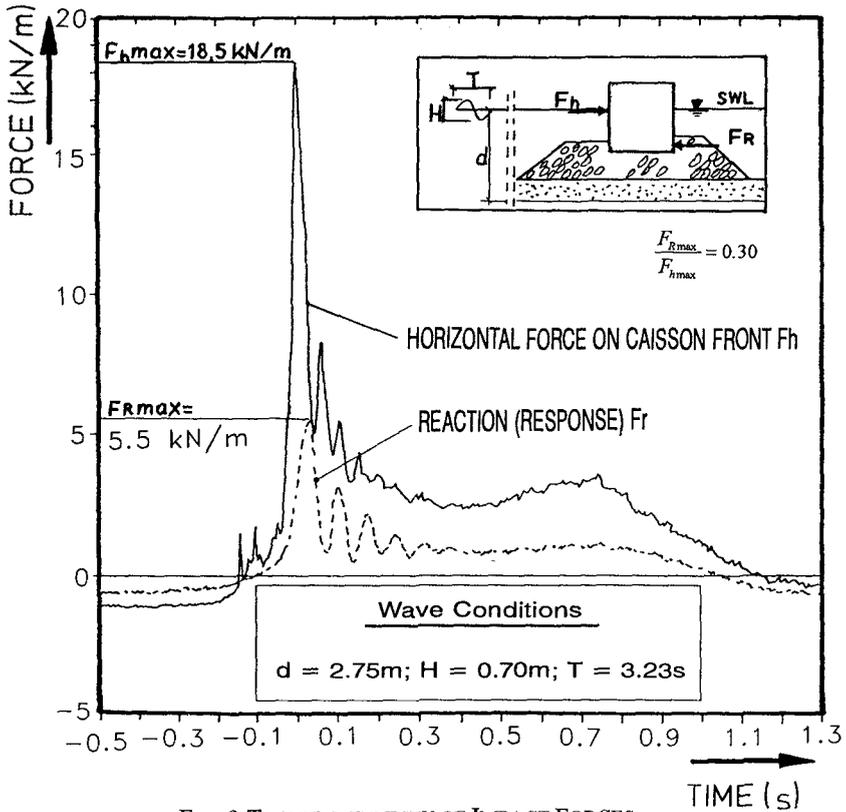


FIG. 8 TRANSMISSIBILITY OF IMPACT FORCES

Accelerations of the Structure

The response of the second caisson part is described by the accelerations of the caisson (vertical and horizontal direction at the rear top and horizontal direction at the front top of the caisson). A typical horizontal acceleration trace is given in Fig.9, showing that accelerations in the order of some decimeters/s<sup>2</sup> may occur. It is also seen that the structure oscillates with periods  $T_s \approx 0.06 - 0.08\text{s}$ .

Pore Water Pressure in Rubble Mound Foundation and Sand Layer

Pore-water pressure in the foundation and total stress in the sand layer induced by waves and wave impacts on the caisson represent further means to characterize the response of the foundation. The pore pressures in the rubble mound foundation are found to have peak values which are an order of magnitude lower than those of the impact pressure on the caisson front (OUMERACI, 1991).

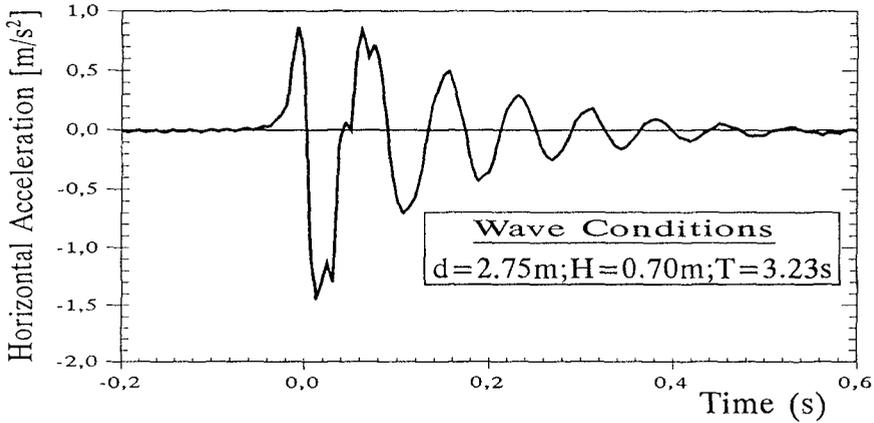


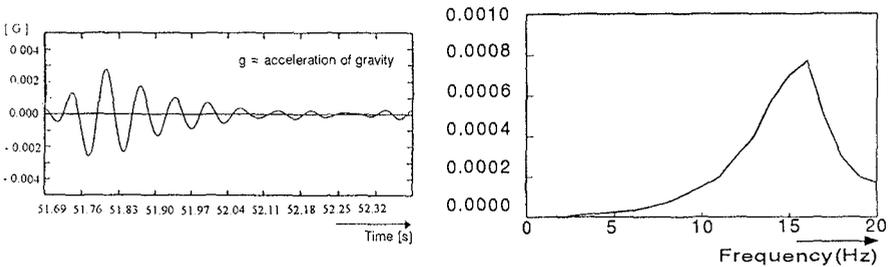
FIG. 9: HORIZONTAL ACCELERATIONS AT THE REAR TOP OF THE CAISSON

The pore pressure histories in the sand foundation exhibit a shape which is almost similar to that of the pressure induced by a partial clapotis. The corresponding total stress is characterized by the sharp peaks at both the seaward and shoreward bottom edges and by low frequency oscillations after the peak which correspond to the rocking motions of the caisson recorded by the accelerometers.

**Results of Pendulum Tests**

**a) Tests under Dry Conditions**

An illustrative example for the results related to the rocking motions of the caisson subject to pendulum impact under dry conditions (without any water in the flume) is given in Fig. 10a showing the time series, and Fig. 10b showing the corresponding spectral representation. This means that the natural frequency of the structure under dry conditions is  $f_n^* \approx 15\text{Hz}$ .



**a) Time Series**

**b) Spectral Representations**

FIG. 10: ACCELERATION OF STRUCTURE OSCILLATIONS (DRY CONDITIONS)

The damping of the structure oscillations under dry conditions is shown by Fig. 11 where the oscillation amplitude  $A$  is plotted against dimensionless time corresponding to the number of oscillation cycles.

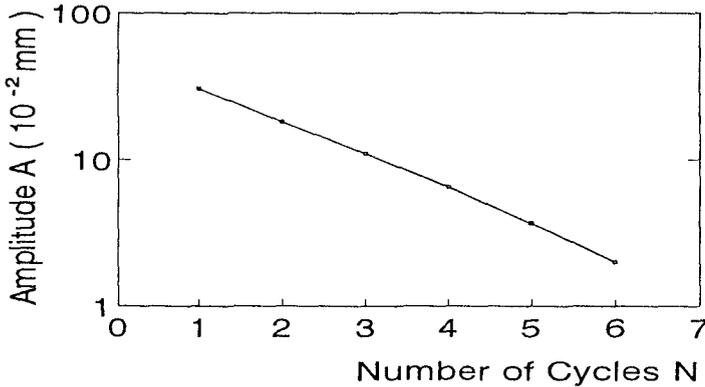


FIG. 11: DAMPING OF STRUCTURE OSCILLATIONS (DRY CONDITIONS)

From Fig. 11 and further similar results, the logarithmic decrement

$$\delta = \frac{1}{n} \left[ \frac{A(t)}{A(t+n \cdot T)} \right] = \frac{2 \pi D}{\sqrt{1-D^2}} \quad (1)$$

has been determined to  $\delta = 0.446$  which leads to a damping ratio  $D = 0.071$ . Since:

$$f_n = f_n^* \left[ \sqrt{1-(D)^2} \right] \quad (2)$$

where  $f_n^*$  is the natural frequency of the undamped oscillations, it can be assumed that  $f_n = f_n^*$  and that the logarithmic decrement  $\delta = 2\pi D$ .

### b) Tests with Different Water Depths

The pendulum tests were conducted for different water depths under the same loading conditions as for the tests under dry conditions.

As already mentioned, the main objective of these tests was to determine the dynamic parameters needed for dynamic analysis, namely :

- the added mass of water which is forced to oscillate with the structure subject to impact loads,
- the damping ratio of the oscillating structure,
- the stiffness of the foundation.

Added Mass of Water

The hydrodynamic mass is generally calculated by using the following formulae derived on the assumption of an incompressible and irrotational potential flow (OUMERACI & KORTENHAUS, 1993):

$$m_{ad,hor.} = 0.543 \cdot \rho_w \cdot d_w^2 \tag{3}$$

$$m_{ad,rot.} = 0.218 \cdot \rho_w \cdot d_w^2 \tag{4}$$

where:

$m_{ad,hor.}$ ,  $m_{ad,rot}$  = added mass of water for horizontal and rotational motion of the structure, respectively  $[kg/m]$

$\rho_w$  = density of water  $[kg/m^3]$

$d_w$  = water depth at the wall  $[m]$

Since the frequency of oscillations  $f_n$  of the structure is directly related to the total mass of the oscillating system  $M_t = M_c + m_{ad}$  ( $M_c$  = caisson mass /m), the added mass of water for each water depth  $d_w$  can be determined from the corresponding measured frequency  $f_n$  by the following relationship:

$$\frac{M_c + (m_{ad})_2}{M_c + (m_{ad})_1} = \left( \frac{f_{n_1}}{f_{n_2}} \right)^2 \tag{5}$$

where indices 1 and 2 are related to water depth  $d_{w1}$  and  $d_{w2}$ , respectively.

The variations of the frequency of oscillations for which the relative water depth  $d_w/d_h$  obtained from the pendulum tests is shown in Fig. 12. By using Eq. (5), the relationship between the added mass and the water depth can be found.

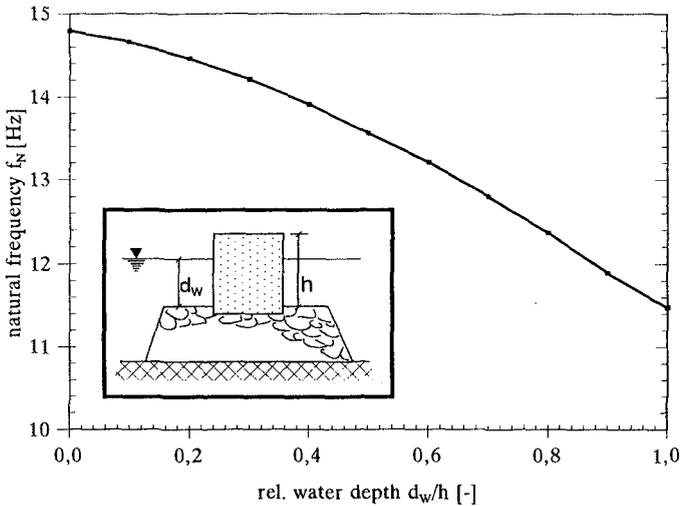


FIG. 12: EFFECT OF WATER DEPTH ON OSCILLATION FREQUENCY

Generally, the added mass calculated by using Eqs. (3) & (4) are slightly underestimated as compared to the experimental results (OUMERACI & KORTENHAUS, 1993).

Stiffness of Foundation

The stiffness coefficient related to the horizontal motions ( $K_x$ ) and to the rotational motions ( $K_\phi$ ) have also been determined from the results of the pendulum tests for different water depths. These results are compared to the approximate method of SAVINOV in Fig. 13 ( see MARINSKI & OUMERACI, 1992).

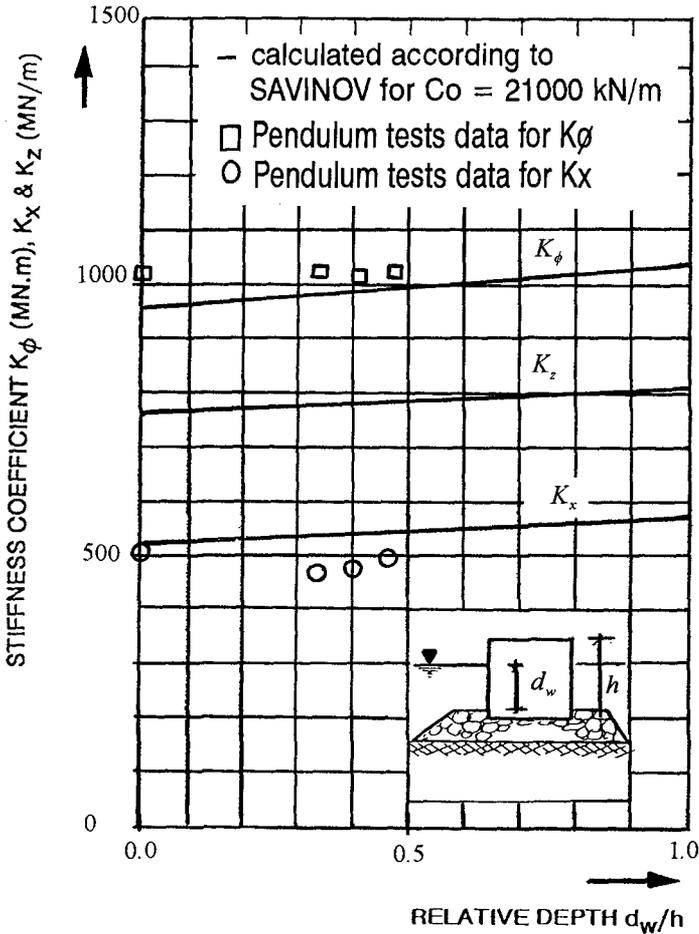


FIG. 13 - STIFFNESS COEFFICIENTS VS. WATER DEPTH

It is seen from Fig. 13 that the experimental results do not exhibit any variation with water depth, but show a relatively good agreement with the approximate method of SAVINOV.

**Damping Ratio**

The damping ratio  $D_x$  and  $D_\phi$  (for horizontal and rotational motions, respectively) have been determined for different water depths  $d_w$  at the caisson front by using the following relationships:

$$D_{x,\phi} = \sqrt{\left(\frac{\delta_{x,\phi}}{4\pi^2 + \delta_{x,\phi}^2}\right)} \tag{6}$$

where:

$\delta_x, \delta_\phi$  = logarithmic decrement for the horizontal and rotational motion, respectively:

$$\delta_x = \ln \left[ \frac{x(t)}{x(t + T_x)} \right] \tag{7}$$

$$\delta_\phi = \ln \left[ \frac{\phi(t)}{\phi(t + T_\phi)} \right] \tag{8}$$

$x(t), \phi(t)$  = amplitude of horizontal and rotational oscillations, respectively

$T_x, T_\phi$  = period of horizontal and rotational oscillations, respectively

$t$  = time

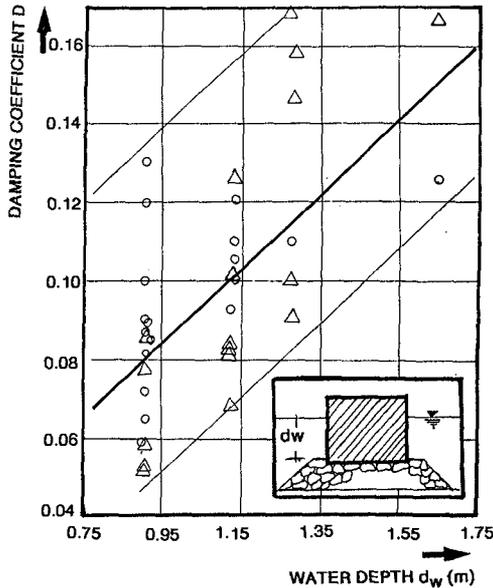


FIG. 14: DAMPING COEFFICIENT VS. WATER DEPTH

The damping ratio  $D_x$  and  $D_\phi$  obtained from the pendulum tests are plotted in Fig. 14 against water depth  $d_w$ .

It can be seen that the experimental D-values show a large scatter, but there is a clear tendency for the damping coefficient to increase with water depth.

### **Summary of Results, Concluding Remarks and Perspectives**

The results achieved so far in the hydraulic model tests, supplemented by the pendulum tests on the same model may be summarized as follows.

The suggestion commonly found in the literature that the effect of impact pressures on the stability of caisson breakwaters is not significant, could not be confirmed by the present study. In fact, the occurrence of a sharp force peak followed by a quasi-static load generally leads to a higher response of the structure than that of the quasi-static force alone (MARINSKI & OUMERACI, 1992).

The shape of the wave breaking on the structure primarily governs the characteristics of the impact load. Double-peaked impact forces which are induced by well-developed plunging breakers are found to be the most critical loads (OUMERACI et al, 1992).

Dynamic uplift pressures caused by wave impacts are not linearly distributed and appear to be important for the dynamic stability analysis of the structure.

Free damped nonlinear oscillations of the structure foundation system are essentially induced by the impulsive load due to breaking waves. This nonlinearity is probably due to the plastic deformations of the foundation as well as the hydrodynamic mass and geodynamic mass which both increase with the amplitudes of the oscillations of the structure.

The oscillations of the structure are transmitted to the foundation. The total stress recorded in the foundations shows a sharp peak, followed by smaller oscillations corresponding to the free rocking oscillations of the structure.

The total stresses recorded in the sand layer beneath the caisson structure and its rubble mound foundation are of two types: one is caused by the shock wave propagation into the soil foundation and the other one by the free oscillations of the structure following the impact.

The logarithmic decrement of the free oscillations of the structure-foundation system subject to breaking wave impact loads ranges from  $\delta = 0.35$  to 1.1 (average value of  $\delta$  is 0.62).

Accounting for the added mass of water oscillating with the structure may result in an increase up to 25 % of the natural frequency of the rocking motions, depending on the loads, the water depth and the size of the structure.

The stiffness coefficients ( $K_x$  and  $K_\phi$ ) determined by the pendulum tests agree relatively well with those determined by the approximate method of SAVINOV.

The damping coefficients determined by the pendulum tests exhibit a wider variation ( $D= 0.08 - 0.16$ ) than the subgrade reaction coefficients.

The results of these investigations can be used, together with a numerical model for the simulation of the dynamic response of the structure, in order to perform a sensitivity analysis of the most relevant parameters (shape and different characteristics of the loading, added mass, damping and stiffness coefficients etc.) of the system (OUMERACI & KORTENHAUS, 1993).

In the ongoing further analysis of the data from both hydraulic and pendulum tests, more effort is being concentrated on :

- the dynamic response of the foundation as related to the caisson oscillations;
- scaling problems related to the loading and response of the structure;
- providing a complete set of reliable data for the validation of numerical models.

### **Acknowledgements**

This study is part of an extensive research programme within the Coastal Engineering Research Unit "SFB 205" which is supported by the German Research Council (DFG), Bonn. Additional support is provided by the European Community within the Research Program MAST G6-0032.

### **References**

- Goda, Y.: Random seas and design of maritime structures, University of Tokyo Press, 1985
- Marinski, T.; Oumeraci, H.: Dynamic response of vertical structures to breaking wave forces-Review of the CIS design experience, ASCE, ICCE 92, Venice, Italy, 1992 (these Proceedings)
- Oumeraci, H.; Partenscky, H.-W.; Tautenhain, E.; Nickels, H.: Large-scale model investigations: a contribution to the revival of vertical breakwaters - ICE, Proc. Conf. in Coastal Structures and Breakwaters, 1991, pp. 207-220.
- Oumeraci, H.; Klammer, P.; Partenscky, H.-W.: Classification of breaking wave impact loads on vertical structures. ASCE, J. Waterway, Port, Coastal and Ocean Eng. Div. (To appear 1993).
- Oumeraci, H. :Dynamic loading and response of caisson breakwaters - Result of Large-scale model tests. MAST G6-S/ Research in Coastal Structures/ Project 2, Res. Rep. Sept. 1991, pp. 33 + Appendices
- Oumeraci, H.; Kortenhau, A.: Analysis of the dynamic response of caisson breakwaters . Paper submitted to "Special Issue on Vertical breakwaters" in Coastal Engineering, Elsevier Amsterdam, 1993
- Oumeraci, H.; Partenscky, H.-W.; Tautenhain, E.: Breaking wave loads on vertical gravity structures. Proc. 2nd Int. (Conf. Offshore & Polar Eng., ISOPE 92, Vol III, San Francisco, 1992 pp. 532 - 539.