WAVE FORCES ON VERTICAL COASTAL STRUCTURES

Hans-Werner PARTENSCKY
Franzius-Institute, University of Hannover, Germany

LECTURE SPECIALE

A number of serious cases of damage were experienced by large vertical breakwaters before and after World War II. Since then, caisson breakwaters have been widely used only in Japan, mainly due to the difficulties of quarrying sufficient quantities of sound large rock units. Elsewhere, this breawater type has been rarely used or practically abandoned in favour of rubble mound breakwaters. However, after the catastrophic failures of a series of large rubble mound breakwaters in the last 15 years /1/, potential possibilities for the revival of vertical breakwaters are appearing to emerge /5/.

This revival could be promoted by the following relatively recent developments :

a) Considerable improvement of the technology of caisson structures and their fondation, particularly in the offshore oil industry. However, potential possibilities for further developments exist with respect to reducing wave impact forces, construction time, foundation work and costs.

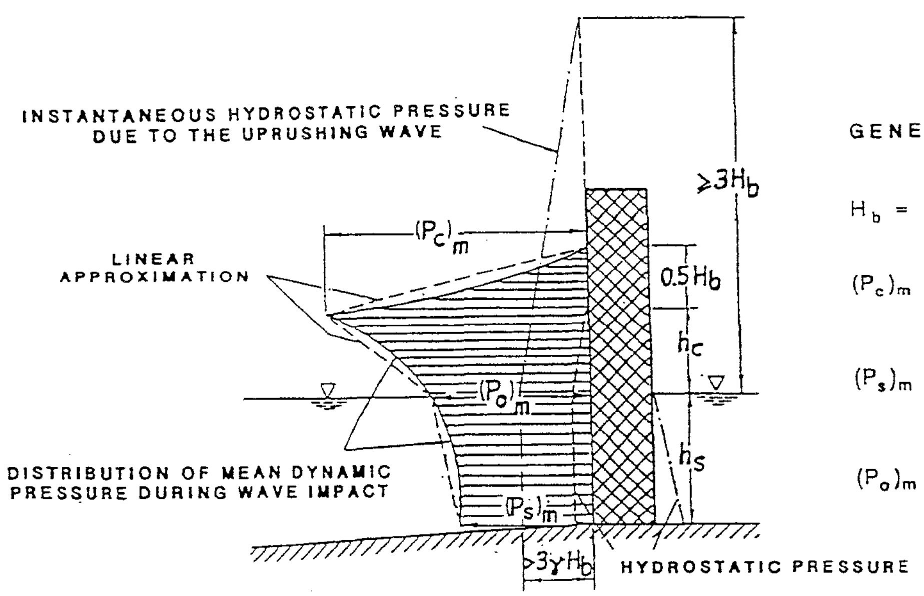
b) An increase of the need for protective structures in deep water, due to the increase of ship sizes in the last decade. The economic advantage resulting from the use of composite breakwaters and considerable savings in construction time is thus obvious.

c) An improvement of model techniques and the availability of large scale testing facilities in which all the dynamic, hydraulic and hydro-geotechnical aspects can be simultaneously investigated.

d) Experience in the numerical modelling of wave-structure-soil interactions, particularly in the offshore oil industry.

Considering these developments and the state of the art on breaking wave impact loads on vertical structures, the Franzius-Institute, University of Hannover, has started an extensive research program on caisson breakwaters which is supported by the German Research Council (DFG) within the Hannover Special Research Unit (SFNB 205). The main objective of this program is to improve the understanding of the physical processes involved and to elaborate more reliable design methods for vertical structures subject to breaking wave forces.

In former studies, instantaneous wave pressures were measured at a rigid vertical wall installed in the Hannover Large Wave Channel by means of some 25 highly sensitive pressure cells. The results obtained were supported by means of an analytical approach /2/ as well as by the results of a numerical model /3/. The resulting pressure distribution recommended for the design of coastal structures subject to the impact of breaking waves is shown in Fig. 1.



GENERAL EQUATIONS:

$$H_b = 0{,}78\,h_s \;;\; t_m \geq 0{,}04\,s$$

$$(P_c)_m = \frac{\rho \cdot H_b}{2 \cdot t_m} \cdot (g\,h_s)^{1/2}$$

$$(P_s)_m = \frac{\pi \cdot \rho \cdot H_b^2 \cdot (g\,h_s)^{1/2}}{t_m \cdot L \cdot \cosh(2\pi h_s/L)}$$

$$(P_o)_m = \frac{\pi \cdot \rho \cdot H_b^2}{t_m \cdot L} \cdot (g\,h_s)^{1/2}$$

Fig. 1 : Recommended pressure distribution for rigid

However, these studies did not include the effects of air content in the collapsing wave, the inclusion of air pockets as well as the effect of the elasticity of the structure and its foundation. These effects, in general, will dampen the momentum exchange between the water mass in motion and the structure. This would result in smaller peak pressures compared with Figure 1, and consequently also in smaller wave forces per unit width.

However, if the natural period of oscillation of the structure in question lies in the same order of magnitude as that of the impact load ($0.02\ s \leq t_T \leq 0.06\ s$), an amplification of the pressure force due to resonance must be expected (Fig. 2).

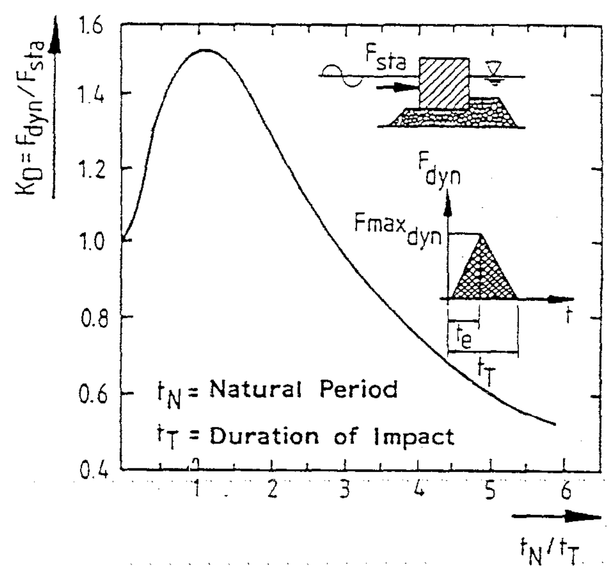
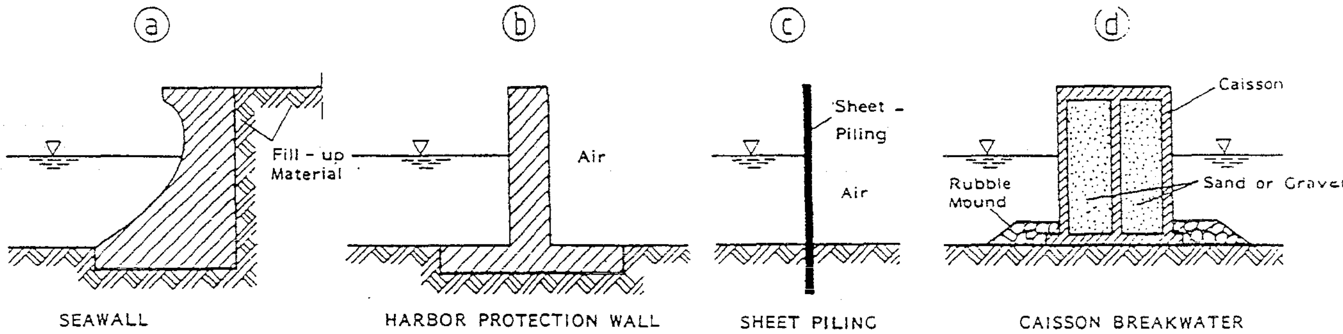
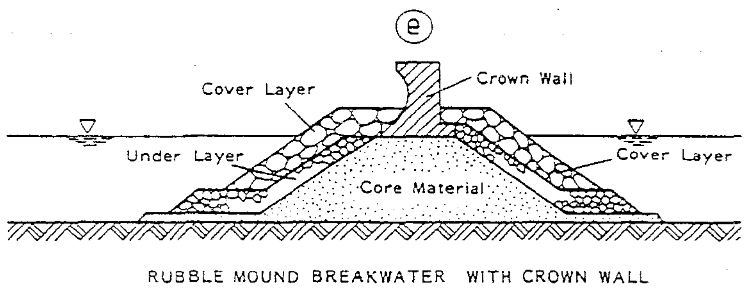


Fig. 2 : Amplification factor for dynamic load

Figure 3 shows some vertical structures with their natural period of oscillation, the latter depending essentially on the kind of construction and its foundation.





Kind of Structure	Natural Period
(a) Seawall	T = ?
(b) Harbor Protection Wall	T = ca. 0,1s
(c) Sheet Piling	T = 0,1 bis 0,01s
(d) Caisson Breakwater	T = 0,2 bis 0,3s
(e) Rubble Mound Breakwater with Crown Wall	T = ?

Fig. 3 : Typical vertical coastal structures and their natural periods of oscillation

The philosophy of this paper is to question the foundamental requirements and conditions for caisson breakwaters to become more competitive than the rubble mound type. This is illustrated by showing and discussing some results of the ongoing investigations at the University of Hannover.

A review and analysis of the major causes of failures experienced by vertical breakwaters provide the following most relevant lessons :

a) The existing design methods are still based on static calculations. They should be replaced by dynamic approaches, since the design of a caisson breakwater is a purely dynamic problem for which the wave-breaking load-time function, the dynamic properties of the materials involved and the elastic response of the structure are needed.

b) The existing methods for the evaluation of the loads and the response of the structure are not sufficient. They should necessarily be accompanied by the further development of caisson breakwater technology towards reducing impact loads and facilitating construction and foundation work.

The research program carried out by the Franzius-Institute in the Large Wave Channel therefore concentrates on the following aspects.

- measurements of impact pressures and total forces,
- measurements of uplift and pore pressures in the rubble mound and sand foundation,
- stress measurements in the sand bed,
- analysis of the breaking wave kinematics in front of the caisson breakwater,
- determination of the dynamic characteristics of the structure (pendulum tests with variable water depths).

Figure 4 shows the cross-section of the caisson model used in the Large Wave Channel with its instrumentation.

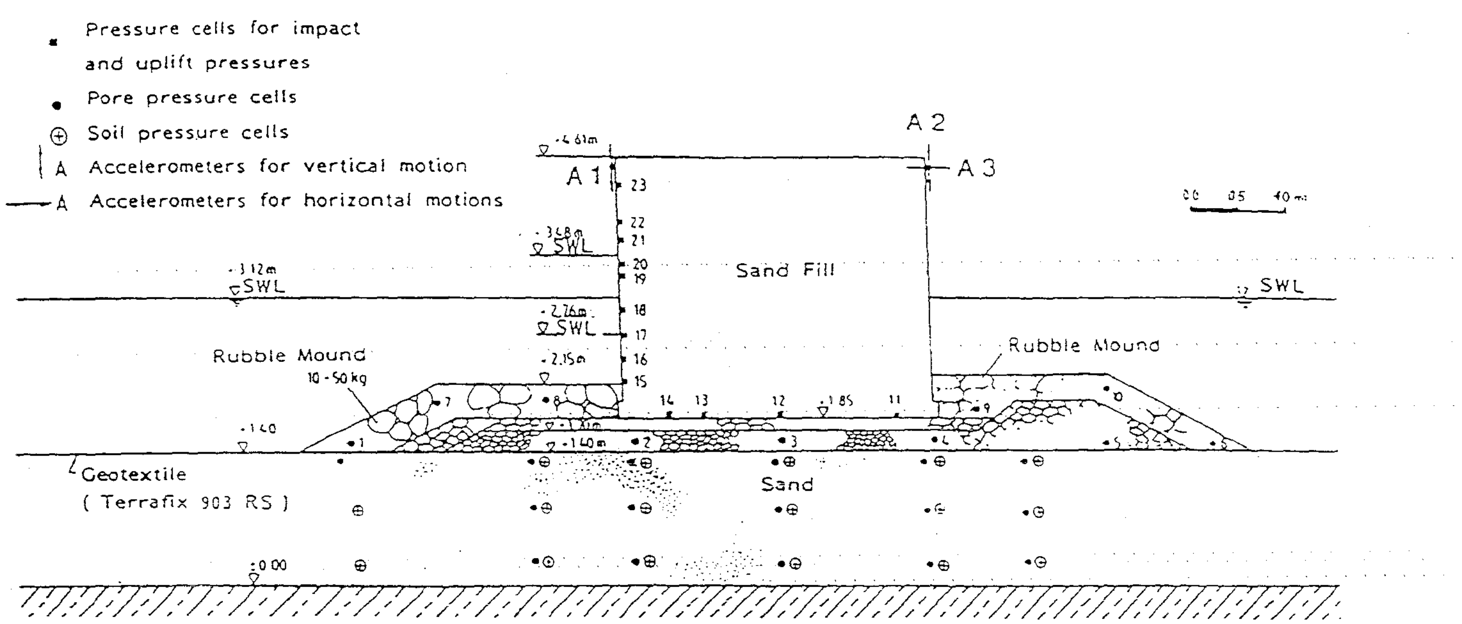


Fig. 4 : Cross-section of the investigated caisson model /5/

Investigations are carried out on caisson models with a plane with a cylindrical front. Figure 5 shows some results on simultaneous pressure distributions and resulting wave and uplift forces as well as on accelerations and motions of the caisson.

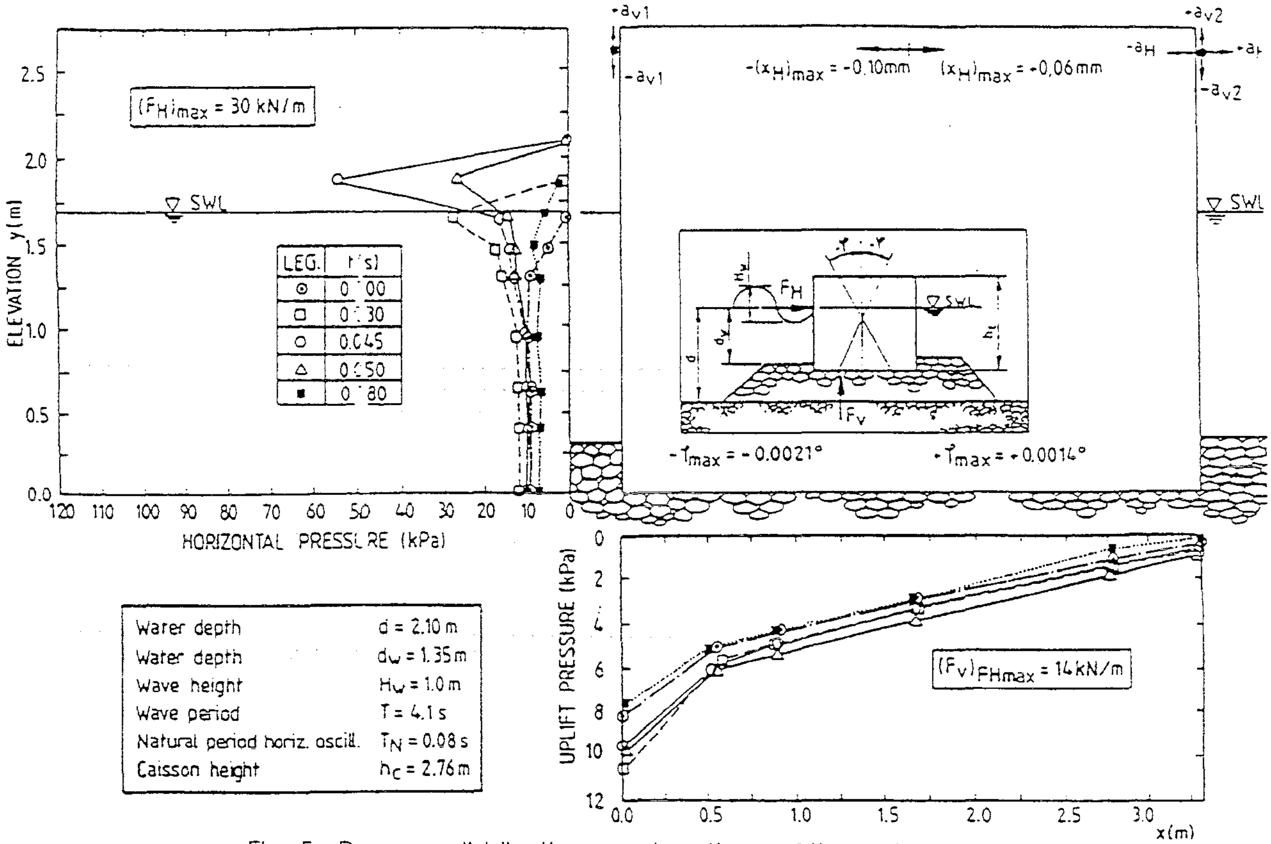


Fig. 5 : Pressure distributions and motions of the caisson due to a breaking wave /5/

Figure 6 shows the variation in pressure distribution at the structure during the impact process (time step t = 5.9 ms) /4,5/.

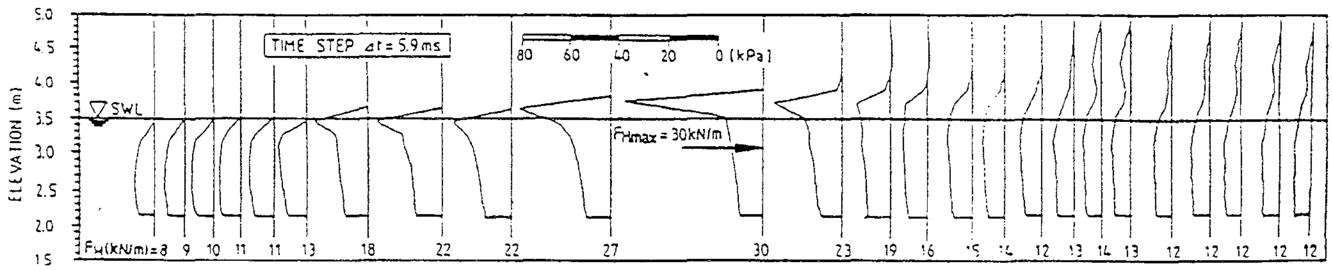


Fig. 6 : Development of impact pressures during the wave impact /5/

The research program in the Large Wave Channel in Hannover is still under way and will be extended till 1994. The results will contribute to a better understanding of the impact process due to breaking waves and might finally lead to some new design criteria for vertical coastal structures.

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