CHAPTER 186

DYNAMIC FORCES DUE TO WAVES BREAKING AT VERTICAL COASTAL STRUCTURES

by

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1. Introduction

In the past 20 years, considerable effort has been devoted to replacing the widely used approaches of HIROI, MINIKIN, NAGAI, PLAKIDA and others /1,2,3,4/, for the design of vertical breakwaters under the impact of breaking waves, with improved and more exact calculation methods. However, almost all new theoretical and empirical approaches lacked the support of prototype measurements or test results from model measurements at a larger scale. The difference between the proposed design criteria and classical approaches is sometimes so great that engineers do not have a reliable method for the design of a vertical or composite breakwater. Figure 1 shows the resulting wave forces per unit width due to different theories as a function of the design wave height H.

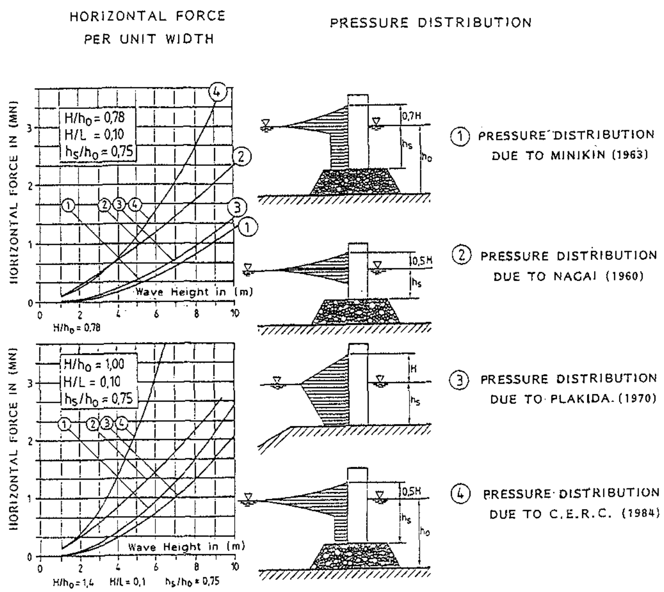

Fig. 1

Horizontal wave forces per unit width due to breaking waves according to different theories /5/

The results show clearly how much the different theories deviate from one another. Therefore, there seems to be an urgent need to re-examine the existing approaches and to support them by means of prototype or large-scale model measurements. The photos of Figures 2 and 3 show some classical examples of waves breaking against coastal structures.

Fig. 2
Impact of waves on the south west breakwater of Colombo/Sri Lanka

Fig. 3
Wave breaking against Catania breakwater on Sicily/Italy /4/

Although there have been no spectacular failures of vertical breakwaters in recent years, the existing and widely used design criteria fail to include some newly defined and recognized factors which may promote or cause damage to structures of this kind.

There factors include:

- the simplified assumption of the dynamic pressure distribution on the structure caused by breaking waves,

- the imprecise determination of the design wave height,
- the scale effects of past model tests with respect to the air entrainment of breaking waves, and
- incorrect dimensioning criteria for waves approaches the structure at an angle (Mach reflection).

There are basically three possible major failure modes for vertical and composite breakwaters (Fig. 4). These are sliding and overturning of the upright section as well as failure of the foundation.

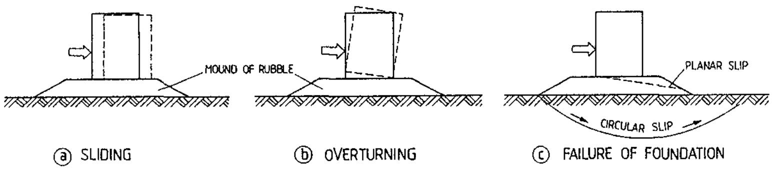

Fig. 4
Possible modes of failure of vertical and composite breakwaters

In all three cases, the impact of the breaking wave at the vertical face of the structure is one of the main causes for the damage. It is therefore extremely important to determine the instantaneous pressure distribution at the vertical wall due to the breaking wave.

2. Modes of wave impact

The resulting pressure distribution at the structure due to a breaking wave depends to a high degree on the mode in which the incoming wave collides with the wall. In this respect, two different cases must be distinguished.

In the first case, the wave begins to break in front of the wall and the tongue of the plunging breaker hits the vertical face enclosing a certain amount of air (Fig. 5).

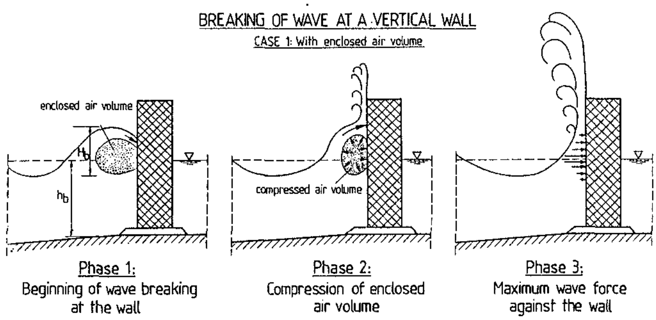

Fig. 5
Wave breaking against a vertical structure with an enclosed air volume

This air volume is quickly compressed during the impact process before the uprushing water carries it upward and away. Figure 6 shows the impact of breaking waves at the sea wall of Westerland/Sylt (F.R.G.) during the stormflood of January 4, 1976. In this case, a considerable amount of air is enclosed in the uprushing water mass which dampens the instantaneous wave pressure exerted at the wall.

Fig. 6

Impact of waves at the Westerland sea wall, Island of Sylt, F.R.G., during the stormflood of January 1976 /5/

In the second case, the wave collides with the structure with an almost vertical front face and only little air is enclosed between the wall and the wave front (Fig. 7).

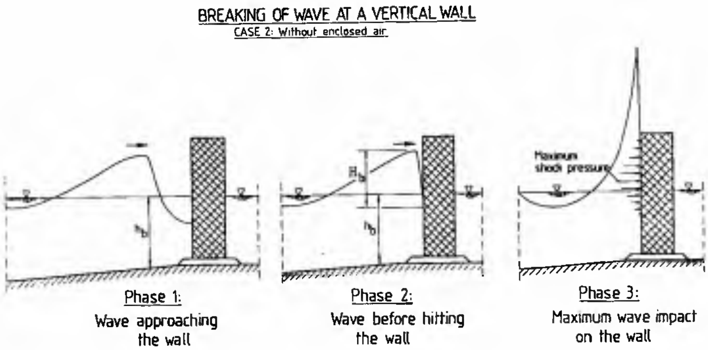

Fig. 7

Wave breaking against a vertical wall without enclosed air

In this case, the forward momentum of the impinging wave is fully transformed into a pressure force which acts on the vertical face of the structure. Measurements showed that the highest pressure values would be exerted at the wall in this case. Figure 8 shows an example of a breaking wave with little air entrainment.

Fig. 8

Wave breaking against Catania breakwater in Sicily/Italy

Preliminary results of measurements obtained in the wave flume of the Franzius-Institut and in the Large Wave Channel showed that the force resulting from the instantaneous pressure distribution at the wall can rise to more than 10 times the hydrostatic pressure force corresponding to the breaker height. This result is in good agreement with observations made by GODA et al. /7,8/.

The peak pressure at the structure is exerted at some distance above the mean water level depending upon the breaker type. It is of very short duration (0.01 s to 0.03 s), and can easily reach values of several hundred kN/m², depending upon the height of the breaking wave and the amount of air entrapped.

According to OUMERACI /9/, the impact pressure depends essentially on the air content and can be determined by the following equation:

$$p_{dyn} = K_L \cdot \rho \cdot g \cdot H_b \qquad (1)$$

in which: p_{dyn} = maximum dynamic pressure due to the wave impact (kN/m²)
H_b = breaking height of the wave (m)
ρ = density of the water (kN·s²/m⁴)
g = acceleration of gravity (m/s²)

K_L is a kind of air content coefficient which is given by:

$$K_L = 5.4 \left(\frac{1}{\kappa_L} - 1\right) \qquad (2)$$

with κ_L = relative air content (Fig. 9).

Using equation (1), the dynamic pressure exerted at a vertical wall by a breaking wave of 1.5 m of height with an almost vertical front face (little air content, κ_L = 0.1 to 0.2) would amount to p_{dyn} = 300 to 700 kN/m², depending upon the volume of air enclosed. The order of magnitude of these values is in good agreement with the results of measurements obtained from tests in the Large Wave Channel in Hannover (see Table 1).

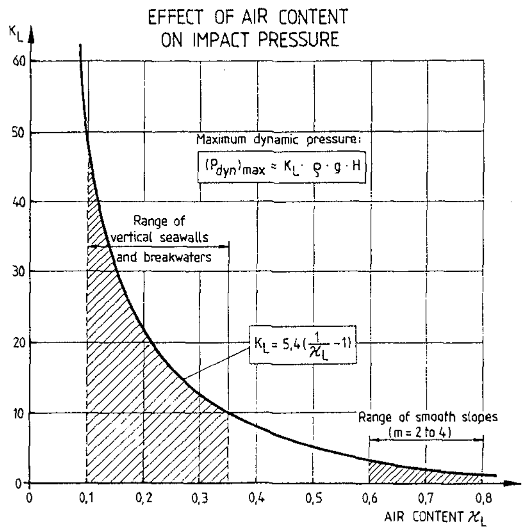

Fig. 9
Effect of air content on impact pressure due to breaking waves

3. Theoretical approach

In order to determine the instantaneous pressure distribution at a vertical wall due to a breaking wave (vertical face assumed), the momentum exchange between the water mass in motion and the rigid assumed structure must be determined. The resulting force per unit width is then given by (Fig. 10):

$$F = m \cdot du/dt = \int_{-h_s}^{h_c} \int_{o}^{t_1} p(z,t) \cdot dt \cdot dz \tag{3}$$

where m = participating water mass,
u = f(z) = velocity distribution under the breaking wave and
t_1 = duration of impact (0.013 s $\leq t_1 \leq$ 0.02 s).

The difficulty is that none of the existing wave theories describes with a sufficient accuracy the velocity field under a breaking wave /10/. A simplified appraoch must therefore be used. For z $\leq$ 0, linear wave theory is applied for $u_1(z)$, whereas for z > 0 an approximation is used for $u_2(z)$ with $u_c \simeq 0.5$ c at the wave crest (Fig. 10). For waves just before breaking, this is in accordance with measurements carried out by LE MEHAUTE and al. /11/, by OCHI and TSAI /12/ and WATANABE, HARA and HORIKAWA /16,17/.

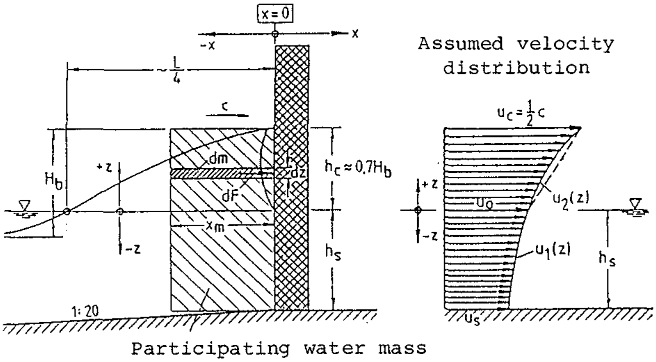

Fig. 10

Definition sketch

It is difficult to determine the exact water mass participating in the momentum exchange during the short duration of the wave impact. At the mean water level, the thickness of the water mass in motion can be approximated to be $x_m \approx 1/2 \cdot L/4 \approx H_B$. Although deeper lying water layers contribute less to the momentum exchange, for reasons of simplification the value of x_m was used over the entire water depth in the following calculations (Fig. 10).

The momentum exchange during the wave impact is of a very short duration with $t_1 \leqq 0.02$ s (Fig. 11).

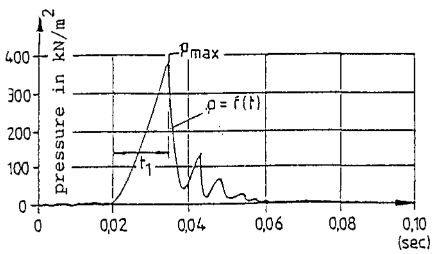

Fig. 11

Increase in pressure during the wave impact at a given location at the wall

Assuming a linear increase in pressure for $0 \leqq t \leqq t_1$ at a given location z on the wall, equation (4) holds:

$$p(z,t) = \frac{p(z)_{max}}{t_1} \cdot t \tag{4}$$

For the maximum pressure at a location z on the wall follows:

$$p(z)_{max} = \frac{2\rho x_m}{t_1} \cdot u(z) \approx \frac{2\rho H_b}{t_1} \cdot u(z) = f(z) \quad . \tag{5}$$

Using linear wave theory, the horizontal velocity components under the wave crest are obtained at the mean water level (z = 0):

$$u_o = \frac{\pi \cdot H}{L} \cdot c \tag{6}$$

and at the bottom ($-z = h_s$):

$$u_s \cong \frac{\pi \cdot H}{L} \cdot \frac{c}{\cosh kh_s} \quad . \tag{7}$$

At the wave crest ($z = h_c$), it is assumed (Fig. 10):

$$u_c = \frac{1}{2}\, c = \frac{1}{2}\sqrt{gh_s} \quad . \tag{8}$$

By using the velocity distribution as shown in Figure 10, the instantaneous pressure distribution at the wall is obtained from equation (5). The peak pressure at $z = h_c$ can be determined by the follwing equation:

$$p_c = \frac{\rho \cdot H_b}{t_1}\,(gh_s)^{1/2} \quad . \tag{9}$$

The pressure at z = 0 is obtained by:

$$p_o = \frac{2\pi\rho \cdot H_b^2}{t_1 \cdot L}\,(gh_s)^{1/2} \tag{10}$$

and at the bottom ($-z = h_s$) by:

$$p_s = \frac{2\pi\rho \cdot H_b^2}{t_1 \cdot L \cdot \cosh kh_s}\,(gh_s)^{1/2} \quad . \tag{11}$$

Figure 12 shows the resulting peak-pressure distribution at the wall due to the wave impact as well as the simplifying linear approxi-

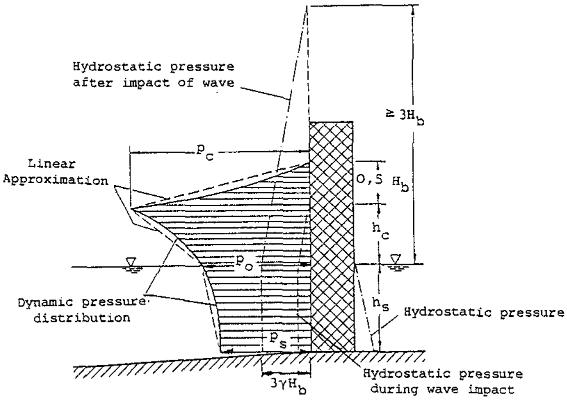

Fig. 12
Resulting peak-pressure distribution

mations. Measurements showed that the uprushing breaker tongue at the wall could reach a height of $3H_b$ or more after the wave impact (Fig. 12).

4. Research program of the Franzius-Institute

The research program of the Franzius-Institute concentrated on the impact of waves breaking against a vertical, solid structure. Portions of the systematic tests were performed in a smaller wave flume with significant wave heights of up to 0.40 m. The most important investigations, however, have been carried out in the Large Wave Channel of the Universities of Hannover and Braunschweig having an overall length of 320 m and a width of 5.0 m. This wave flume is equipped with an hydraulically driven wave maker by means of which monochromatic and random waves can be produced with wave heights up to 2.50 m.

In both flumes, the instantaneous pressure distribution at a vertical wall due to breaking waves was measured by means of a number of high sensitive pressure cells having a natural frequency of 28 kHz with an operating pressure range of up to 5 bar.

In the Large Wave Channel, the vertical wall was composed of seven stop logs, each with a height of 1.0 m and about 25 pressure gauges installed over the face of the wall (Fig. 13).

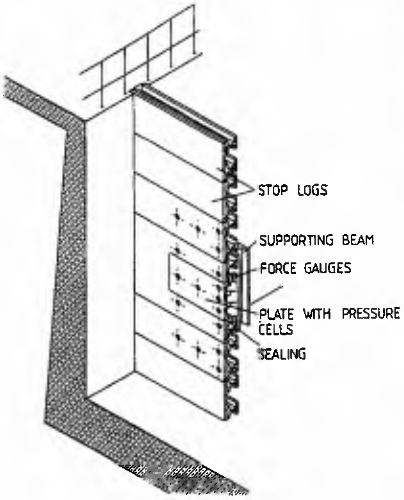

Fig. 13

Installation of vertical wall in the Large Wave Channel

Fig. 14

Wave breaking against vertical wall in the Large Wave Channel

The measurements showed that the maximum dynamic pressure at the wall is exerted at a distance of $h_c \approx 0,7\ H_b$. Above the mean water level, its value depending upon the breaker type and the amount of air enclosed. Figure 15 shows some time series of impact pressures obtained at different measuring points in the Large Wave Channel for a breaking wave height of 1.50 m.

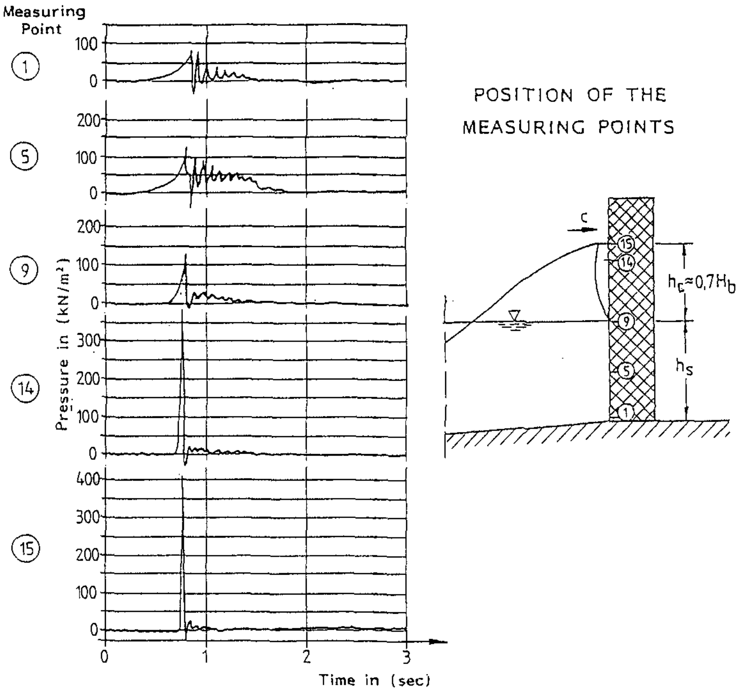

Fig. 15

Time series of impact pressures and location of measuring points

The evaluation of the measurements carried out in the Large Wave Channel is not yet finished. The first results, however, show clearly that the criteria used so far for the design of vertical breakwaters under the impact of breaking waves underestimate considerably the peak pressure and resulting wave forces.

Peak pressures at $z = h_c$ for a breaking wave height of $H_b = 1.50$ m measured in the Large Wave Channel exceeded, for example, the value resulting from the MINIKIN approach /13/ by a factor of 4 (Table 1). Based on the maximum pressure values obtained at different measuring points at the wall, the resulting force per unit width exceeded the corresponding CERC-value by a factor of 7.0 and the overturning moment about the toe of the wall was approximately

12 times greater than that obtained from the current design criteria (Table 1).

Peak pressures measured in the Large Wave Channel for given breaker heights were in good abgreement with the distribution of maximum pressures as shown in Fig. 12 /5/.

In Table 1, a comparison of test results with existing theories and the new approach is made for a breaking wave height of H_b = 1.50 m. Based on the peak pressure values, the calculated wave force per unit width as well as the overturning moment about the toe of the wall were in good agreement with the theoretical values obtained by the new approach (Table 1).

Theoretical approach		Maximum dynamic pressure (kN/m^2)	Force per unit width (kN/m)	Overturning moment (kNm/m)
HIROI	(1903)	23,0	124,6	191,2
NAGAI	(1968)	20,0	45,7	62,6
PLAKIDA	(1970)	23,0	80,3	117,6
C.E.R.C. (Based on MINIKIN, 1950)	(1984)	97,0	63,1	86,7
GODA	(1985)	15,3	100,0	134,0
FRANZIUS-INSTITUTE (Measurements at Large Wave channel)	(1987)	395,0	496,1*	1016,8*
PARTENSCKY	(1987)			
Based on maximum pressure values with t_1 = 0,0167 s:		361,0	541,0	1020,0
Based on maximum pressure values with t_1 = 0,02 s:		301,0	455,0	855,0
Recommended design values with t_m = 0,04 s:		75,2	130,0	230,0

* Calculation based on maximum pressure values

Table 1

Comparison of test results with existing theories and new approach (Example for breaking wave height of H_b = 1.50 m with T = 7.8 s)

5. Recommendations for the design of vertical coastal structures

As can be seen from the time series of impact pressures in Figures 11 and 15, the duration of the peak pressure at the different locations is extremely short (t_1 < 0.02 s). Due to its inertia, the coastal structure will therefore hardly show reactions during the short duration of the momentum exchange.

Prototype measurements at CAISSON-like breakwaters in Japan showed that the natural period of oscillation of these types of coastal structures lies between 0.1 and 0.3 s /14/, which is one order of magnitude higher than the duration of the peak pressure during the wave impact.

In addition, from the recorded pressure time series it can also be seen that the maximum pressures at the different locations along the wall do not occur simultaneously, the phase shifts, however, are small.

The dimensioning of a coastal structure on the basis of the peak pressure distribution of Figure 12 therefore appears to be too conservative. It seems more realistic to replace the resulting peak pressure distribution by an averaged pressure distribution, in which the mean dynamic pressure $p_m = f(z)$ over the total impact duration $t_m = t_1 + t_2$ is applied to each point of the structure (Fig. 16) /15/.

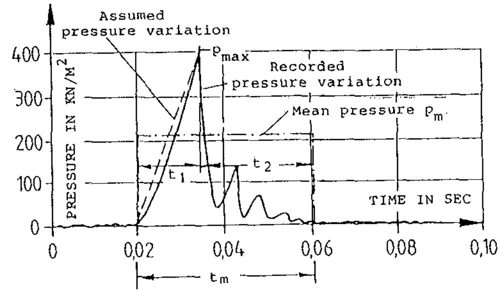

Fig. 16

Mean pressure p_m during the momentum exchange due to the wave impact

With the simplifying assumption of a linear increase and decrease of the pressure during the impact duration t_m, it follows for the mean pressure p_m that:

$$p_m(z) \approx \frac{1}{2}\, p_{max}(z) \tag{12}$$

By using this relationship, the following reduced pressure values result from equations (9), (10), and (11):

$$\text{At } z = h_c: \qquad (p_c)_m = \frac{\rho \cdot H_b}{2\, t_m}\,(g h_s)^{1/2} \tag{13}$$

$$\text{At } z = 0: \qquad (p_o)_m = \frac{\pi \cdot \rho \cdot H_b^2}{t_m \cdot L}\,(g h_s)^{1/2} \tag{14}$$

$$\text{At } -z = h_s: \qquad (p_s)_m = \frac{\pi \cdot \rho \cdot H_b^2}{t_m \cdot L \cdot \cosh k h_s}\,(g h_s)^{1/2} \tag{15}$$

The time series of the impact pressures recorded at the Large Wave Channel show that the impact duration is $t_m \approx 0.04$ s at and above the mean water level, whereas below the mean water level t_m increases with depth (Fig. 15).

Figure 17 shows the recommended distribution of average impact pressures which might serve as a new approach for the future design of vertical coastal structures subject to the impact of breaking waves.

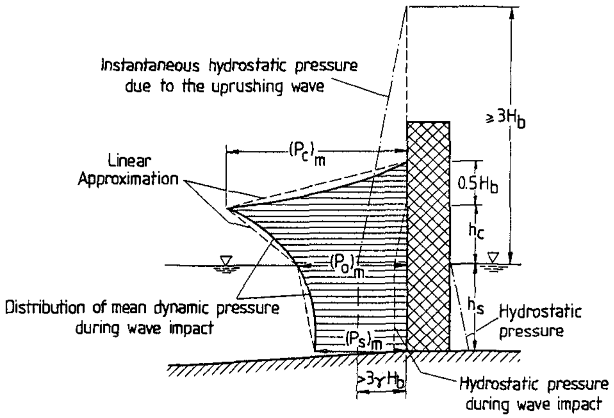

Fig. 17

Recommended pressure distribution for the design of coastal structures subject to the impact of breaking waves

In Figure 18 the dynamic pressure distributions at a vertical structure for a breaking wave height of H_b = 1.50 m are represented as they result from the classical MINIKIN-approach recommended in /13/ and from the new approach.

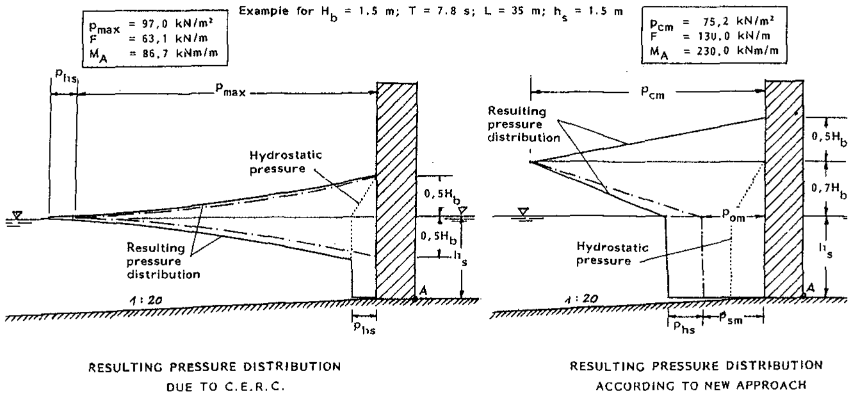

Fig. 18

Comparison of dynamic pressure distributions according to MINIKIN (C.E.R.C.) and the new approach

Although the recommended averaged peak pressure p_{cm} at the wave crest is somewhat smaller than that resulting from MINIKIN's theory, the effective wave force per unit width is twice as high and the overturning moment about the footpoint A of the structure is almost three times greater than that obtained from the classical theory (see also Table 1).

6. Conclusions

The results of the investigations show clearly that the design criteria used so far for the dimensioning of vertical coastal structures such as sea walls, vertical and composite breakwaters underestimate considerably the resulting wave forces.

The proposed pressure distribution of Figure 17 could therefore serve as a more realistic approach for the design of coastal structures under the impact of breaking waves.

To what extent the effect of strucural elasticity as well as the type of foundation must be considered in the design and overall stability of the structure has not yet been investigated. It will, however, be the subject of further studies.

In addition, a numerical analysis of the impact process is planned in order to determine more precisely the time- and space-dependent phases of the momentum exchange at the coastal structure.

7. Acknowledgement

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