CHAPTER 109

DESIGN OF VERTICAL WALLS AGAINST STORM SURGE

A. Kortenhaus[1], C. Miller[2], H. Oumeraci[3]

ABSTRACT

A concept for the design of high water protection (HWP) walls under storm surge conditions has been developed and is applied to the geometry found in the harbour of Hamburg, Germany. However, the design methods used have been generalized so that they may be used for a wide range of cases with similar geometries. Many gaps in standard design formulae have been filled by developing engineering approaches or formulae as reflection by steep berms, new breaker criterion, design formulae for impact breakers, reduction of loads by overtopping and soil pressure distribution in front of the wall.

INTRODUCTION

Interest in protective structures against storm surges is expected to largely increase in the near future, mainly due to the increased rate of storminess in the last decades. In order to be able to react more rapidly and to better protect the coastal estuary zones of high economic, social and environmental value, the performance of existing structures and their design must be reconsidered. Especially in areas with limited space, where the construction of "classic" high water protection (HWP walls) is impossible, the HWP walls as commonly found for instance in Hamburg are very suitable protective structures. Such a study has recently been conducted for the city of Hamburg, reconsidering the safety margin of all vertical wall structures within the harbour area. The motivation for this study was the change of the wave climate and of the rising water level assessments in the Hamburg harbour. To date, no general formulae have been developed to account for (i) the complicated foreland geometries in the harbour area, (ii) the different processes of wave transformation on these types of foreland, (iii) breaking of waves on the foreland and

[1]Dipl.-Ing., Leichtweiß-Institut, TU Braunschweig, Beethovenstr. 51a, 38106 Braunschweig, Germany, e-mail: a.kortenhaus@tu-bs.de

[2]Dr.-Ing., Wirtschaftsbehörde Strom- und Hafenbau, Hamburg, Dalmannstr. 1, 20457 Hamburg, Germany

[3]Prof. Dr.-Ing., Leichtweiß-Institut, TU Braunschweig, Beethovenstr. 51a, 38106 Braunschweig, Germany, e-mail: h.oumeraci@tu-bs.de

(iv) different breaker types occurring at the protective structures. The design of HWP walls is very often considered to be much simpler than other more classical protection works. The relatively small heights of the HWP walls and their "simplicity" constitute the main reason, why this research field was neglected in the past. In fact there are many important peculiarities showing the need to develop this research field. Among the peculiarities there is for instance the size of the areas to be protected by the HWP walls which often extend over considerable lengths (e.g. more than 100 km in Hamburg harbour). Further reasons showing the need for more research will be addressed below.

In the Hamburg harbour area four typical foreland geometries can be identified (Fig. 1) which illustrate the large variety of harbour protection works. Due to these different types, a general design procedure is necessary which (i) accounts for different foreland geometries, (ii) is suitable for engineering practice and therefore must be easy to use and (iii) takes into account the large differences in water levels during a tidal cycle.

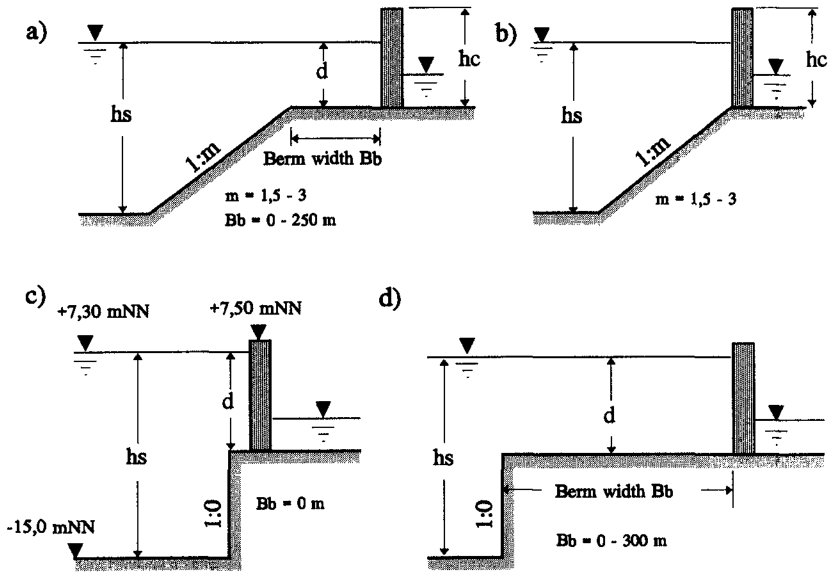

Fig. 1: Typical foreland geometries in the Hamburg harbour area

It is the main purpose of this paper to suggest a new design philosophy for protection works consisting of a variable foreland geometry with a vertical wall on top. In particular, a method is described to evaluate the wave transformation on the foreland and to calculate the most critical forces and moments on the vertical wall with respect to the critical water level and breaker type.

DESIGN STRATEGY

A set of five parameters is defined which describes all typical wall and foreshore geometries in the harbour area (Fig. 1). The walls are built as sheet walls or concrete wall structures with free heights of up to three meters. The design strategy for these conditions can be principally summarized in Fig. 2. The variable water level in this figure requires an iteration procedure until the design water level is reached which yields the most critical load at the wall. Breaking wave criteria, wave overtopping and wave load forces are evaluated by using the most updated design formulae/diagrams. It can be seen from Fig. 2 that three different loading cases may be distinguished:

- Standing waves: standing waves are very rare under prototype conditions for irregular wave trains but may occur during high water levels. A modification of the MICHE-RUNDGREN procedure for standing or almost standing waves was found to give reasonable results (*SPM, 1984*).
- Broken waves: broken waves at the wall represent the most frequent loads in the harbour area and may carry floating bodies (empty containers) hitting the wall. The standard CERC procedure for broken waves will be used to predict these loads (*SPM, 1984*).
- Plunging breakers: plunging breakers are relatively rare and will only occur under particular storm surge conditions but represent the most dangerous situation for the protection works since breaking waves cause very high impact loads. Results of hydraulic model tests which have been performed under the Marine Science and Technology Programme (MAST III) of the European Communities and formulae most recently developed to account for these type of loads (*Klammer et al., 1996*) will be used.

The variety and complexity of the foreland geometry makes it more difficult to define the design wave in front of the wall. The reason for this is that there are no reliable tools to describe the wave transformation and breaking criteria for most of the conditions shown in Fig. 1. The same reason is also valid for the lack of general design formulae to calculate the wave load and overtopping under these conditions.

The problem becomes more difficult by the complexity of the dynamic interaction of the "wave-structure-soil"-system under the impact load of breaking waves. Very often it is impossible to simplify the problem so that a static design method can be used. Moreover, the design water levels, wave parameters, loading and overtopping conditions do vary along the structure implying that there is not the same safety along the HWP walls at equal wall height.

The four design steps shown in Fig. 2 may be briefly described in the following sections.

(a) Step 1: Determination of Input Parameters

First, all important input parameters have to be defined or determinated, respectively. The relevant water levels and the geometry of the foreland as well as the construction can be summarized by five parameters: water depth at the toe of the berm hs, slope of the berm m, width of the berm Bb, water depth at the wall d and the free wall height hc (Fig. 1).

The significant wave height $H_0 = H_s$, the peak-period T_p and the wave direction θ are needed as deep water input parameters. These parameters can be determined from

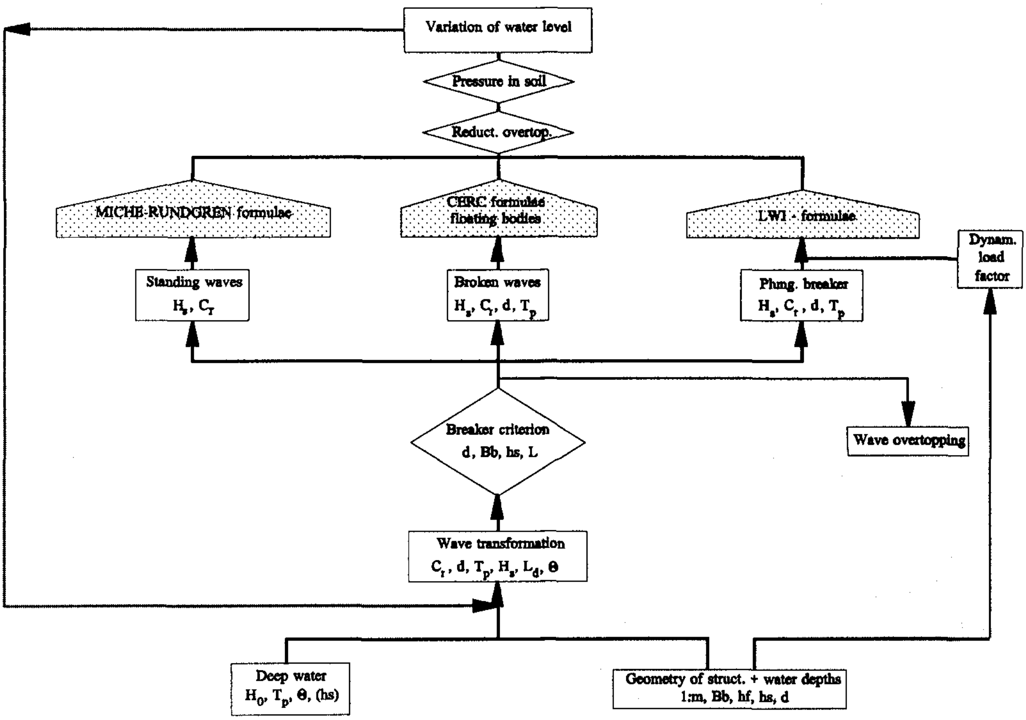

Fig. 2: Design procedure for high water protection works

measurements or by numerical wave forecast (*van Vledder, 1995*). Furthermore, particular boundary conditions provided by harbour authorities must also be taken into account. In Hamburg "Strom- und Hafenbau" is responsible for the protection works used to provide shelter against high storm surges. These authorities generally provide information regarding the design of the load by floating bodies (empty containers etc.) as well as detailed data about the geometry of the HWP wall segments and their alignment relative to the wind direction. The latter is important as the design of HWP walls on the lee side can be performed by considering only the highest possible hydrostatic pressure (water level at the top of the wall); i.e. without any wave load (*Kortenhaus and Oumeraci, 1996*).

(b) Step 2: Wave Transformation and Breaker Criteria

Incident waves approaching from deep water will be transformed on the berm by shoaling, refraction and reflection; i.e. the wave height H_d in the water depth d (at the wall) can be derived by:

$$H_d = \kappa_S \cdot \kappa_R \cdot k_X \cdot H_{max} \qquad (1)$$

H_{max} is the maximum wave height in deep water (Rayleigh distribution) which can be derived from $H_{max} = 1.86\ H_s$ (H_s from step 1).

κ_s, κ_R and k_x are the shoaling, the refraction and the reflection factor, respectively (determined by linear wave theory). The shoaling factor κ_s and the refraction factor κ_R can be calculated as follows:

$$\kappa_S = \frac{k_{s,W}}{k_{s,0}} \tag{2}$$

$$\kappa_R = \frac{k_{R,W}}{k_{R,0}} \tag{3}$$

where $k_{s,\,0}$ and $k_{s,\,W}$ are the shoaling coefficients in deep water (subscript 0) and at the wall (subscript W) and $k_{R,\,0}$ and $k_{R,\,W}$ are the refraction coefficients. The reflection factor k_x describes the reduction of wave energy by the reflection due to the steep berm and can be calculated as follows:

$$k_x = \sqrt{1 - \varkappa_B^2} \tag{4}$$

The total reflection coefficient of the wall-berm-system $\varkappa_B$ can be estimated as follows:

$$\varkappa_B = \sqrt{\frac{E_{01}}{E_0} \cdot \varkappa_{01}^2} \tag{5}$$

where $\varkappa_{01}$ is the reflection coefficient at the berm, E_0 is the wave energy in deep water calculated from linear wave theory and E_{01} is the part of the wave energy in front of the berm which can also be estimated by linear wave theory. The reflection coefficient at the berm $\varkappa_{01}$ can be estimated as follows (ξ is the Iribarren number):

$$\varkappa_{01} = 1 - \exp\left(-\frac{1}{7}\xi^2\right) \tag{6}$$

Fig. 3 shows the reflection factor k_x plotted against the relative water depth d/L_0 according to Eqs. (4)-(6). It can be seen that changes in effective reflection coefficients up to 25% may result for design water levels ($d \approx 2$ m; $L_0 \approx 20$-25 m) in the Hamburg harbour area.

To determine the relevant loading case under storm surge conditions breaking criteria are needed which account for the reflection properties of the HWP wall and for the foreland geometry.

Assuming the reflection coefficient to be 0.9 for vertical HWP walls (*Kondo et al., 1986*) the wave height of the breaking wave H_b can be determined by the following formula (*Oumeraci et al., 1993*):

$$H_b \approx 0.10\, L_0 \left[\tanh\left(2\pi \frac{d}{L_b}\right)\right]^2 \tag{7}$$

L_0 is the wave length in deep water and L_b is the wave length at the breaking point (water depth d) which can be calculated from linear wave theory as follows:

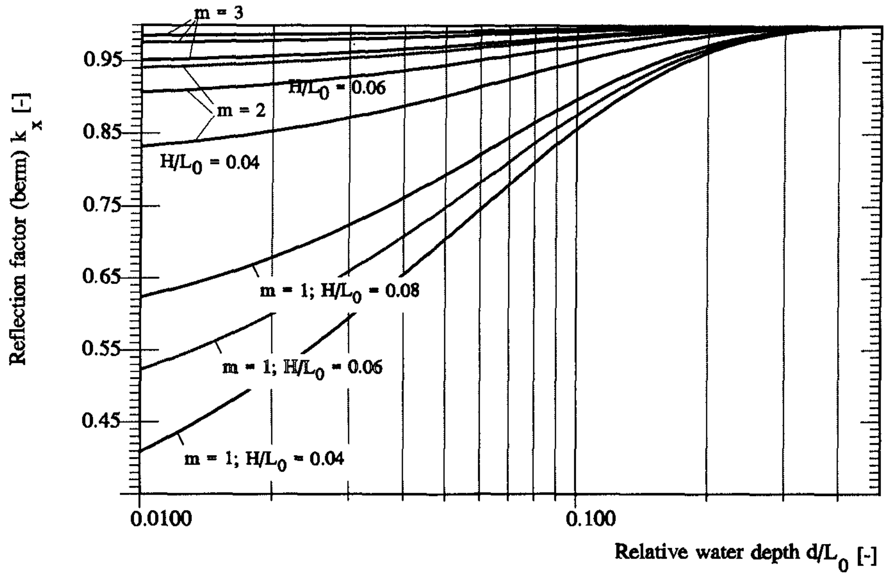

Fig. 3: Reflection nomogram for a vertical wall with a foreshore sloping berm

$$L_b \approx L_0 \left[\tanh \left(2\pi \frac{d}{L_0} \right)^{3/4} \right]^{2/3} \tag{8}$$

The comparison of the breaking wave height H_b from Eq. (7) and the wave height H_d from Eq. (1) allows a first distinction into two loading cases. If H_b is smaller than H_d the waves will not break and the 'Standing wave' loading case can be applied.

If H_b is larger than H_d loading cases 'plunging breaker' or 'broken wave' can be applied. To check for 'plunging breaker' conditions a 'breaker type nomogram' in Fig. 4 has been developed on the basis of existing experimental data (*Takahashi et al., 1993*). Comparison to results obtained in large-scale model tests showed that these breaking criteria give acceptable results (*Oumeraci and Kortenhaus, 1996*). A further comparison to another method recently published by *Allsop et al. (1996)* shows only slight differences. The relative berm width Bb/L_b, the relative berm height h_r = (hs - d)/hs and the relative wave height H_d/d are needed as input parameters for the nomogram.

The 'plunging breaker' loading case occurs when the point is inside the corresponding H_d/d area. If the point is outside the loading case "Broken Wave" can be assumed. Two examples in Fig. 4 demonstrate the use of the breaker type nomogram.

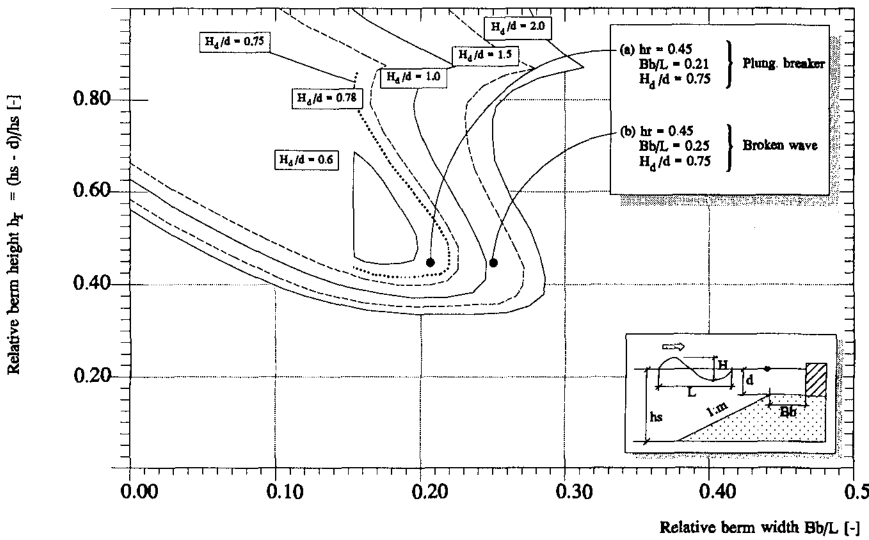

Fig. 4: Nomogram for identification of loading case 'plunging breaker'

(c) Step 3: Calculation of Loading Cases

The wave pressure distribution at the wall, wave forces and moments for each of the loading cases described before have to be determined. For all cases a load reduction due to overtopping and the pressure distribution in the soil in front of the wall will be given. These calculations are described below in the third section of this paper.

(d) Step 4: Variation of Water Level

During storm surge conditions the water level will vary significantly in front of the wall. Therefore it is necessary to perform step 3 for a stepwise reduction of the water level (down to d = 0). As a result a critical water level and a corresponding load for each section of the wall along the HWP line are obtained.

This procedure can be used for the identification of the most critical spots along the HWP walls in the harbour area where impact loading may occur under particular water level and sea state conditions.

DETERMINATION OF LOADING CASES

This chapter summarizes the methods to estimate the pressure distributions, forces and moments due to the wave action in front of the structure for the three loading cases shown in Fig. 2. Due to the limited space for this paper, related references will be given in all cases where standard procedures have been used.

(a) Standing Waves

For the loading induced by standing waves the method of *Sainflou (1928)*, the method by *Rundgren (1958)*, modified by Miche and summarized in *SPM (1984)* and a more recent method based on a crest elevation proposed by *Goda (1985)* have been compared. As a result the method of Miche-Rundgren has been selected for the design, because of its simplicity in engineering use and because it accounts for both terms of higher order as well as for the reflection coefficient of the HWP wall (see design diagrams in *SPM, 1984*).

It is proposed to use a constant reflection coefficient for the Miche-Rundgren method. It is well known that a reflection coefficient of 1.0 is too conservative but to be on the safe side a constant reflection coefficient of $C_r = 0.9$ is proposed. In applying the method to the harbour of Hamburg this value was used.

(b) Broken Waves

This loading case is assumed to be the predominant loading case for HWP walls under storm surge conditions and geometric boundary conditions as for instance found in Hamburg harbour. The fast change from deep water conditions in front of the berm to shallow water conditions in front of the wall will induce wave breaking. A plunging breaker at the wall will occur under very special geometric conditions (relative berm height and width) as already shown in Fig. 4. Therefore broken waves are more likely to be expected. The design for this loading case may be combined with a load induced by floating bodies as it is most likely that floating bodies will be transported by broken waves. For the assessment of design load, the method by CERC (*SPM, 1984*) is proposed.

The input parameters for the SPM method are:

- the wave depth at the wall d
- the wave height of the breaking wave H_b according to Eq. (7)

(c) Plunging Breakers

For plunging breakers at the wall a new method was developed to calculate pressure distribution and forces at the vertical wall (*Klammer et al., 1996*). The latter formula takes into account the total duration and the rise time of the load.

From experiments on a large-scale breakwater model the following formula for the impact force has been derived for the design of the HWP walls in Hamburg:

$$F_{h,max} = 8.0 \cdot \rho g H_b^2 \tag{9}$$

$F_{h,max}$ is the maximum horizontal wave force, H_b is the height of the breaking wave in front of the structure and ρ is the density of the water. From statistical analysis it has been found that the non exceedance probability of the relative horizontal force $F_{h,max}/\rho g H_b^2$ in Eq. (9) is about 10 to 15%. The point of application of the force is close to the height of the design water level. The pressure distribution at the time of the maximum horizontal force and further design details can be taken from *Klammer et al., 1996.*

To account for the dynamic behaviour of the system the load has to be multiplied by a dynamic load factor D. Dynamic load factors for caisson structures were principally investigated by *Oumeraci and Kortenhaus (1994)*. In the example case of the Hamburg harbour dynamic load factors were determined from prototype measurements at the protection walls in the Hamburg harbour area where D was found to be in the range from 0.85

to 1.2 (*Kruppe, 1996*). A conservative value for D of 1.5 can be used for each section along the HWP walls where no detailed information on the dynamic characteristics of the structure-soil-system are available. The force increased by the dynamic load factor has to be used for a static design approach instead of the values calculated by the method described before:

$$F_h = D \cdot \left(8{,}0 \cdot \rho g H_b^2\right) \tag{10}$$

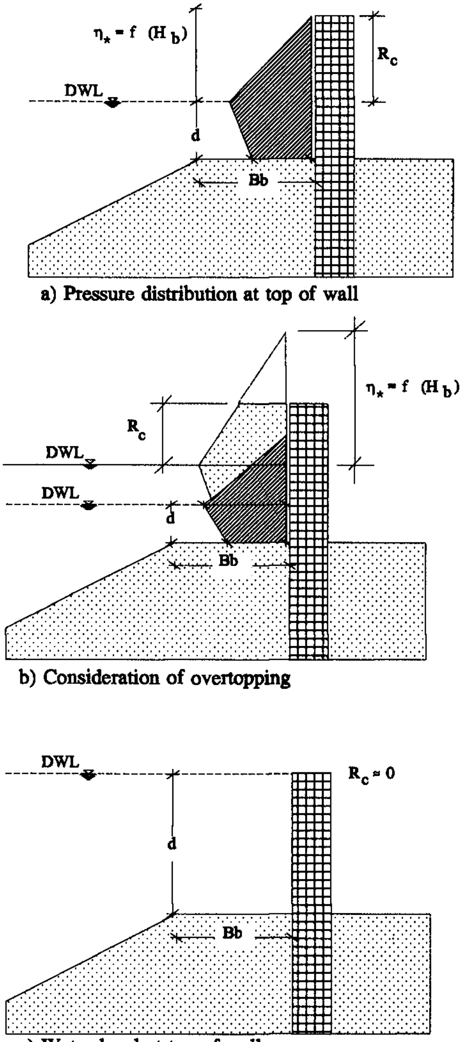

Fig. 5: Comparison of pressure distributions with and without overtopping

Reduction of wave impact at the wall due to oblique wave attack and short-crestedness may also be taken into account by using the results of extensive 3D hydraulic model tests (*Franco et al., 1995*). For short-crested waves almost no reduction of horizontal forces can be found whereas for long-crested waves, reductions in horizontal forces can be found only for higher wave obliquity.

OVERTOPPING

Overtopping will lead to a reduction of the horizontal loading. This phenomenon has not yet been addressed in detail in the literature. Generally, in the case of overtopping the pressure figure is cut at the top of the wall (Fig. 5b). The pressure ordinate at the top of the wall is then calculated by an interpolation between the ordinate at the height of the design water level (DWL) and the point above the water level where the pressure would be zero if the wall were high enough. However, this method particularly fails for higher DWL. Therefore an additional approach is suggested below.

In Fig. 5a the wave and the pressure distribution just reach the top of the wall whereas in Fig. 5c the design water level

(DWL) has reached the top of the wall. In the latter case the dynamic pressure induced by wave motion is relatively small as compared to the hydrostatic head. Hence, the governing load is the hydrostatic head related to the water depth at the wall.

Therefore the reduction of the loading on the wall due to overtopping is about zero at the top of the wall. Especially for impact loading this is not the case when the pressure distribution is simply cut off at the top of the wall. Contrarily this procedure would result in a significantly high pressure at the top of the wall.

Hence, a factor k_{Fh} has been introduced to reduce the loading by more than a simple 'cutting' of the pressure distribution. This factor accounts for the fact that the pressure distribution and the force in Fig. 5c has to be zero ($R_c \approx 0$) and has its maximum for an infinitely high wall (Fig. 5a):

$$F_{h,ov} = k_{F,h} \cdot F_h \tag{11}$$

with

$$k_{F,h} = 1 \qquad \text{für } \eta_* \le R_c$$

$$k_{F,h} = \sqrt[3]{\frac{R_c}{\eta_*}} \qquad \text{für } \eta_* > R_c \tag{12}$$

$F_{h,ov}$ is the reduced force, F_h is the horizontal force according to the method valid for each loading case, η_* is the distance of the highest point of the pressure distribution to the design water level and R_c is the freeboard of the wall. Applying this procedure for the three loading cases shown will result in the lower curve related to the respective loading case in Fig. 6.

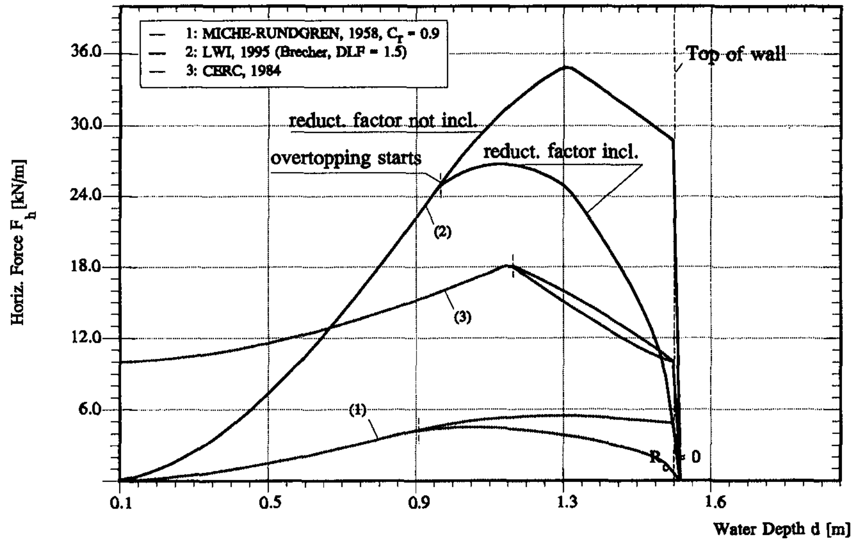

Fig. 6: Reduction of horizontal force by reduction factor k_{Fh}

SOIL PRESSURE IN FRONT OF THE WALL

(a) Standing Waves

For standing waves where the loading changes with the wave period T_p the pressure in the height of the berm p_s for a water depth d can be calculated by:

$$p_s = \rho_W \, g \left(1 + C_r\right) \frac{H}{2} \frac{1}{\cosh \frac{2\pi d}{L}} \tag{13}$$

For a berm made of *rubble material* it can be assumed that the decrease of pressure p(z) in the soil still can be described by linear theory. Increasing depth in the soil z will result in:

$$p(z) = p_S \frac{\cosh\left[2\pi\left(t_B + z\right)\right]}{\cosh\left(\frac{2\pi d_b}{L}\right)} \tag{14}$$

For a berm made of *finer soil material* like sand the method of *Moshagen and Tφrum (1975)* can be used. For $k_x = k_z$ (same permeability in horizontal and vertical direction) the method of *Moshagen and Tφrum (1975)* can be simplified to:

$$p(z) = p_S \frac{\cosh\left[\mu \, (z + t_B)\right]}{\cosh\left[\mu \, t_B\right]} \tag{15}$$

n is the porosity of the soil material ($n \approx 0{,}4$); k_x is the permeability of the soil material in horizontal direction ($k_x \approx 10^{-4}$ m/s for sand); T is the wave period in s; L is the wave length in the water depth d in m and μ can be given by:

$$\mu = \left[\left(\frac{2\pi}{L}\right)^4 + \left(\frac{\frac{2\pi}{T} n \, \rho_W \, g}{k_x E_F}\right)^2\right]^{\frac{1}{4}} \tag{16}$$

Since *Moshagen and Tφrum (1975)* assumed a total saturation of the soil E_F is set equal to E_{water}. This is not completely true as air and water will fill the pores of the soil. Therefore the following approach for E_F will be used in Eq. (16):

$$E_F = \frac{1}{\beta'} \tag{17}$$

In this ß' is the compressibility of the pore fluid which can be calculated from:

$$\beta' = \beta + \frac{1 - s}{p_a} \quad \text{für } (1 - s) \ll 1 \tag{18}$$

ß is the compressibility of the water ($\beta = 4.2 \; 10^{-7}$ m²/kN); s is the saturation of the soil; p_a is given by $p_a = p_{atm} + p_{hydrostat.}$; p_{atm} is equal to 101.325 kN/m² and p_{hyd} is the hydrostatic pressure which is given by $p_{hyd} = \rho_W \, g \, d$.

The application of Eq. (18) to three typical water depths in the Hamburg harbour area is given in Tab. 1.

Tab. 1: Compressibility ß' of the pore fluid as a function of the saturation of the soil s and the water depth d

Saturation s [-]	Compressibility of the pore fluid ß' [m^2/kN]		
	d = 0.5 m	d = 1.0 m	d = 2.0 m
1.00	ß' = ß = 4.2 10^{-7}		
0.999	9.83 10^{-6}	9.42 10^{-6}	8.96 10^{-6}
0.99	9.45 10^{-5}	9.04 10^{-5}	8.31 10^{-5}
0.98	1.89 10^{-4}	1.80 10^{-4}	1.66 10^{-4}
...	...	...	...
0.95	4.71 10^{-4}	4.50 10^{-4}	4.14 10^{-4}

(b) Broken Waves and Impact Breakers

For impact loads and broken waves no method is yet available to assess the damping in fine soil material. For rubble material the respective pressure distribution can be extended in the soil. For fine soil materials (sand) it is proposed to neglect the pressure distribution in the soil due to the strong damping (highly frequent loading).

CONCLUDING REMARKS AND FUTURE RESEARCH WORK

A concept for the design of high water protection (HWP) walls under storm surge conditions has been developed. The concept was initially developed for the harbour of Hamburg (Germany) but the design methods have been generalized so that they may be used for a wide range of cases with similar geometries.

In the future however a more elaborated concept for the safe and economic design of this kind of protective structures is needed which requires both an integrative procedure by accounting for hydraulic, structural and soil mechanical aspects as well as for all possible failure modes and their probability of occurrence. Therefore it is necessary to use dynamic and probabilistic design methods where the 3D-character of the problem has to be taken into consideration.

This task has been undertaken in the "Marine Science and Technology"-programme (MAST III) of the European Union. This research project of 23 European institutes out of different disciplines (hydrodynamics, coastal engineering, soil mechanics, structural mechanics, applied mathematics, etc.) is coordinated by Leichtweiss-Institut, Braunschweig (*Oumeraci, 1995*) and is aiming for providing a basis for the design of vertical breakwaters under probabilistic aspects.

ACKNOWLEDGEMENTS

This study has been supported by the Hamburg harbour authority "Strom- und Hafenbau". The support of basic research studies by the German Research Council (DFG) on monolithic breakwaters and by the European Union (contract no. MAS2-CT95-0041) on vertical structures is also gratefully acknowledged. Without the latter basic research studies the results described in this paper would not have been achieved.

REFERENCES

ALLSOP, N.W.H.; VICINANZA, D.; CALABRESE, M; MCKENNA; J.E. (1996): Wave forces on vertical and composite breakwaters. *Strategic Research Report. Hydraulics Research Station, Wallingford,* SR 443, Wallingford, U.K., 94 pp.

FRANCO, L.; DE GERLONI, M.; VAN DER MEER, J.W. (1995): Wave overtopping on vertical and composite breakwaters. In: *Final Proceedings, MAST II, MCS-Project: Monolithic (Vertical) Coastal Structures,* Paper 4.10, 16 pp.

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

KLAMMER, P.; KORTENHAUS, A.; OUMERACI, H. (1996): Wave impact loading of vertical face structures for dynamic stability analysis - prediction formulae. *Proceedings International Conference Coastal Engineering (ICCE),* ASCE, Orlando, Florida, USA, vol. 25. In these Proceedings.

KONDO, K.; AKAMA, M.; ISOBE, M. (1986): Measurement of reflection coefficient of seawall in Omura Bay. *Proceedings International Conference Coastal Engineering (ICCE),* ASCE, Taipei, Taiwan, vol. 20, Part 2, Chapter 146, pp. 1987-2001.

KORTENHAUS, A.; OUMERACI, H. (1996): Hochwasserschutzwände im Hamburger Hafen - Lastansätze für Wellendruck. *HTG-Sprechtag "Hochwasserschutz in Häfen / neue Bemessungsansätze", Oktober 1996,* HtG, Hamburg, Germany, 21 S. In German.

KRUPPE, J. (1996): Berücksichtigung dynamischer Einflußfaktoren bei Wellendruckschlag. *HTG-Sprechtag "Hochwasserschutz in Häfen / neue Bemessungsansätze", Oktober 1996,* HtG, Hamburg, Germany, 9 S. In German.

MOSHAGEN, H.; TORUM, A. (1975): Wave induced pressures in permeable seabeds. *Journal of Waterways, Harbors and Coastal Engineering Division,* ASCE, vol. 101, no. WW1, pp. 49-57.

OUMERACI, H.; KLAMMER, P.; PARTENSCKY, H.-W. (1993): Classification of breaking wave loads on vertical structures. *Journal of Waterway, Port, Coastal and Ocean Engineering,* ASCE, vol. 119, no. WW4, pp. 381-397.

OUMERACI, H.; KORTENHAUS, A. (1994): Analysis of dynamic response of caisson breakwaters. *Coastal Engineering, Special Issue on 'Vertical Breakwaters',* Oumeraci, H. et al., Amsterdam, The Netherlands: Elsevier Science Publishers B.V., vol. 22, nos. 1/2, pp. 159-183.

OUMERACI, H. (1995): Multi-disciplinary research experience on vertical breakwaters: the MCS-Project. In: Commission of the European Communities (ed.): *MAST Days and EUROMAR Market. Project Reports.*, Sorrento, Italy, 26 pp.

OUMERACI, H.; KORTENHAUS, A. (1996): Bemessung von Hochwasserschutzwänden im Hamburger Hafen. *Berichte Leichtweiß-Institut für Wasserbau, Technische Universität Braunschweig,* LWI, Braunschweig, Germany, Nr. 799, 44 S.; 9 Teilberichte. In German.

RUNDGREN, L. (1958): Water wave forces - a theoretical and laboratory study. *Kungliga Tekniska Hoegskolans handlingar*, Stockholm, Sweden, no. 122, 123 pp.

SAINFLOU, G. (1928): Essai sur les diques maritimes verticales. *Annales des Ponts et Chaussées*, vol. 98, no. 4, pp. 5-48. In French.

SPM (1984): Shore protection manual. Chapters 6 through 8; appendices A through D, Vicksburg, Mississippi, USA, 4th edition, 800 pp.

TAKAHASHI, S.; TANIMOTO, K.; SHIMOSAKO, K.-I. (1993): Experimental study of impulsive pressures on composite breakwaters. *Report. Port and Harbour Research Institute (PHRI)*, Tokyo, Japan, vol. 31, no. 5, pp. 33-72.

VAN VLEDDER, G.PH. (1996): Wave climate during the design storm surge. *HTG-Sprechtag "Hochwasserschutz in Häfen / neue Bemessungsansätze", Oktober 1996*, HtG, Hamburg, Germany, 9 pp., 4 figs. In German.