Application of Overtopping Models to Vertical Walls against Storm Surges

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Abstract

The use of different wave overtopping models has been analysed. These overtopping models were applied to vertical walls with different geometries built for the protection against storm surges in the harbour of Hamburg (Germany). A comparison of existing overtopping test data and results from specific hydraulic model tests for existing geometries in Hamburg was also performed. A simple design diagram to predict both the overtopping rate q and the required freeboard R_c is presented. Furthermore, a simple engineering approach is proposed for the reduction of the horizontal wave load of the harbour walls due to wave overtopping. Finally, conclusions are drawn to come up with a "general overtopping formula" and further research work is outlined.

1. Introduction

The harbour of Hamburg is located on both sides of the river Elbe about 100 km upstream of the river mound (German Bight). The harbour area is divided in many harbour basins with bordering stock areas (Fig. 1). The river Elbe at this location is influenced by the tide resulting in a tidal range of about 2 m in Hamburg. During storm surges the water is pushed upstream the river Elbe from the German Bight and during high floods parts of the harbour are submerged. The cargo areas ("polders") are protected against storm surges by harbour walls with a total length of about 100 km. In Hamburg different geometries are used. Fig. 2 shows a classification of the typical harbour walls.

The local wave conditions during storm surges are as follows: the freeboard is higher than 0.2 m, the significant wave heights vary between 0.1 m and 0.7 m, the

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mean wave periods between 1.0 s and 4.0 s and the wave directions between 0° and 90°. The water depth $h_{\rm S}$ is about 15.0 m for design conditions and the average design wind velocity is about 20 m/s. Short crested wave conditions can be expected during high floods.

From above it can be seen that the design conditions in Hamburg for wave overtopping are very complex and an easy-to-use guideline is wanted to calculate wave overtopping. Within a case study the available reports, published and unpublished data have been collected and adapted for the Hamburg harbours walls. The main objectives of this study were:

- to identify the most reliable model to predict wave overtopping of vertical walls,
- (2) to present an easy-to-use design diagram for the selected model and several applications to different



Figure 1. Port of Hamburg

geometries and wave conditions incl. comparisons to other models,

(3) to develop an engineering approach for the reduction of horizontal forces due to overtopping and



Figure 2. Typical harbour wall geometries

(4) to outline the problems which have to be solved in order to come up with a "general overtopping formula".

2. Influencing Parameters

Wave overtopping is influenced by several parameters which can be identified and classified in the following way (Fig. 2 and 3):

• <u>Structural Parameters:</u>

Structure Type, crest height and width, berm width, height and slope (Fig. 2)

• Wave parameters and water depth:

wave height, period and direction, spectral quantities, water depth in front of the structure

- Wind parameters: Wind velocity and direction
- Scale and model effects
- <u>Measured quantities:</u> average overtopping rate, individual overtopping rate, number of overtopping waves



Figure 3. Definition of angle of wave attack

3. Available Overtopping Models

Various overtopping models have been established over the last decades considering more or less of the aforementioned parameters. The most relevant investigations will be summarized in the following with respect to the geometric conditions in Hamburg. GODA (1985) presents nondimensional diagrams for vertical harbour walls (Type b in Fig. 2) with different foreshore slopes based on small-scale model tests (wave flume) with irregular waves with and without rubble foundation.

DOUGLASS (1984) compares the GODA method with the SPM method (WEGGEL, 1976) and concludes that the two methods show reasonable good agreement for relative water depths $h_S/H_S = 0.4$ and $h_S/H_S = 0.75$. For higher ratios of h_S/H_S the SPM overpredicts GODA. AHRENS and HEIMBAUGH (1989) performed model tests with wave spectra for seven test setups, considering a large variety of structure types. DAEMRICH (1991) performed model tests with wave spectra including wave direction (longcrested waves) for the Hamburg harbour wall type a in Fig. 2 (Fig. 4). MÜHLESTEIN (1992a,b) tested the Hamburg harbour wall type without berm and foreshore (Type c in Fig. 2) in a wave flume and a wave basin using wave spectra and wind (Fig. 5).





Figure 4. Model Tests by DAEMRICH (1991)

Figure 5. Model Tests by MÜHLESTEIN (1992 a,b)

FRANCO et al. (1995) compiled model data (2D and 3D) from several European laboratories within the MAST II MCS-project (MAS2-CT 92-0047) and added own data (Type c in Fig. 2) from multidirectional wave tests. He proposed the following exponential relationship to calculate the average overtopping rate (in the following referred to as MCS-formula):

$$\frac{q}{\sqrt{g H_S^3}} = a \exp\left(-b \frac{R_C}{H_S} \frac{1}{\gamma_i}\right)$$
(1)

with: q = average overtopping rate $[m^3/(s \cdot m)]$

 H_{S} = significant wave height [m]

 R_{C} = freeboard [m] (measured with respect to SWL)

 γ_i = non dimensional reduction coefficient [-]

a, b = nondimensional coefficients (recommended: a=0.082; b=3.0 for normal wave attack and no directional spreading)

For oblique wave attack FRANCO recommends $\gamma_{\theta} = \cos\theta$ (for $\theta < 37^{\circ}$) and $\gamma_{\theta} = 0.79$ (for $\theta > 37^{\circ}$) for longcrested waves and $\gamma_{\theta} = 0.83$ (for $\theta \le 20^{\circ}$) and

 $\gamma_{\theta} = \cos(\theta - 20^{\circ})$ (for $\theta > 20^{\circ}$) for shortcrested waves.

The influence of wind on wave overtopping is considered by HAYAMI et al. (1966), IWAGAKI et al. (1966) and DE WAAL (1996) for vertical walls. They found that firstly wind has an influence on wave overtopping by affecting the wave profile which again influences the breaker type (breaking occurs earlier, breaker number becomes smaller) and the breaking point moves seaward. Secondly basic spray is transported landward from the sea and thirdly the so called "green water" overtopping increases. Model tests are available for "green water" overtopping resulting in the following findings:

- For most experimental conditions wave overtopping increases due to wind except for very small relative water depths $(h_S/L_0 \le 0)$.
- A quantitative description of the wind influence is still not available due to the scaling problems of wind (drops, drop transport capacity, shear stress between wind and water).

Therefore, further investigations are necessary to check the influence of wind on wave overtopping by large-scale model tests and field measurements.

None of the investigations has yet considered the width of the berm (Type a and Type d in Fig. 2). It is questionable whether the approach by VAN DER MEER et al. (1998) for smooth slopes can be used in the same way or has to be adapted for vertical walls. VAN DER MEER et al. (1998) considered the influence of a berm by introducing a reduction factor γ_h which is defined as:

$$\gamma_{b} = 1 - \frac{B_{b}}{L_{berm}} \left(1 - 0.5 \left(\frac{d}{H_{S}} \right)^{2} \right)$$
(2)

with: d = distance between the middle of the berm and still water level (SWL)

 B_b = width of berm [m]

 L_{berm} = horizontal distance between the two points which are 1.0 ·H_S below and above the middle of the berm on the structure [m]

 H_S = significant wave height [m]

Table 1 shows a comparison of the geometric wall configurations in Hamburg with the model setups of the main investigations used in this paper.

4. Comparison of Model Data to Overtopping Models

Chapter 3 has shown that only the approach by FRANCO considers short- and longcrested waves. Therefore, this approach is compared to the model test results from MÜHLESTEIN (1992a, b) and DAEMRICH (1991) with longcrested waves. These studies were performed to test the geometrical conditions in Hamburg so that comparisons between model data and MCS-formula will show the applicability of the formula for different conditions. Subsequently, the model tests by DE WAAL (1996) for shallow water are introduced in the analysis (s. Table 1).

Parameter	Hamburg	MÜHLE- STEIN	DAEMRICH	GÓDA	DE WAAL.	FRANCO
Berm	yes	no	yes (1:1.7)	no	no	yes (1:3)
H _S /L ₀ [-]	0.01÷0.1	0.007÷0.017	0.043÷0.057	0.012÷0.036	0.01÷0.04	0.02÷0.04
R _C /H _S [-]	>0.43	0.32÷5.56	0÷1.5	0.45÷1.73	1.0÷2.5	1.18; 1.5; 1.63
θ [°]	0÷90	0, 20, 45	0, 20, 40	0	0	0÷60
VWind [m/s]	20	0 and 20	0	0	0	0
h _S /H _S [-]	21÷150	7.45÷25.92	7.0÷16.7	0.45÷1.73	1.0÷3.0	4.7÷4.8
1m [•]	1;∞	1:∞	1:10	1:10; 1:30	1:50	1:∞
spectra	short crested	long crested	long crested	wave flume	wave flume	short- and longcrested
model scale	1:1	1:10	1:20	no info.	no info.	1:30
Type (Pig. 2)	a-d	c	b	c	c	a, b, c

Table 1. Parameter Used by the Different Investigations (Notations s. Figs. 2-4)

A comparison to the MCS-formula is shown in Fig. 6 for tests by MÜHLESTEIN (1992a,b). From Fig. 6 a very good agreement between MCS-formula and model tests can be obscrved for overtopping rates higher than $q \approx 0.1 l/(s \cdot m)$. For lower overtopping rates there is a considerable scatter in the results which is common in many model tests. A similar scatter can be found in model tests by OWEN (1980) and VAN DER MEER et al. (1998).

The same conclusions can be drawn when the MCS-formula is plotted against data from DAEMRICH (1991) (Fig. 7). The difference between calculated and measured overtopping rates increases with a decrease in the overtopping rate (below 5 $V(s \cdot m)$). It can be concluded that the MCS-formula results in fairly smaller overtopping rates for small overtopping rates especially for oblique wave attack ($\theta = 40^\circ$). For high overtopping rates (q>100 $I/(s \cdot m)$) the MCS-formula overestimates the overtopping rate. This fact probably results from missing overtopping tests for $R_C=0$ by FRANCO. Using the model results by DAEMRICH the range of applicability of the MCS-formula is extended for low relative freeboards, higher wave steepnesses and a small steep berm. In Fig. 8 a comparison between the MCS-formula and the overtopping tests by DE WAAL (1993) is shown. In this case the range of applicability of the MCS-formula is extended for high freeboards (small overtopping rates). The scatter of the data is considerable.



Figure 6. MÜHLESTEIN data versus MCS-formula



Figure 7. DAEMRICH data versus MCS-formula



Figure 8. DE WAAL data versus MCS-formula

5. Design Diagram for Wave Overtopping

The comparison between different model data sets and the MCS-formula shows that FRANCOs formula is a reasonable good approach to predict wave overtopping for many different vertical wall geometries. Therefore, this equation has been plotted in a diagram which has been first presented by MÜHLESTEIN (1992) and which is represented in a modified version in Fig. 9 by using Eq. (1).

The overtopping rate can be calculated by entering the diagram with the significant wave height H_S and the freeboard R_C . On the other hand, the requested freeboard can be calculated by using the significant wave height H_S and a critical average overtopping rate q.

The diagram is valid for longcrested waves. For shortcrested waves the freeboard R_C must be divided by γ_{θ} =0.83 (for θ ≤20°) or γ_{θ} =cos(θ -20°) (for θ >20°) and only the line for θ =0° in the left diagram can be used.

From Table 1 and chapter 4 the range of validity of this diagram is obvious. It can be used for relative freeboard heights $R_C/H_S=0.55$, wave steepnesses $H_S/L_0=0.007\div0.057$ and relative water depths between $h_S/H_S=4.7$ ÷deepwater. Furthermore reasonable scatter has to be expected for overtopping ratios lower than 5.0 l/(s·m). Oblique wave attack has to be treated very carefully, because of higher possible overtopping rates.

6. Influence of Wave Overtopping on Wave Forces

The influence of the wave overtopping rate on the wave loading of the wall has not yet been investigated. Many investigations have been performed concerning wave pressures at the front of vertical walls due to waves (e.g. GODA, 1985; KORTENHAUS et al., 1996) but to the authors knowledge the reduction of pressures due to wave overtopping still remains an unsolved problem. Two main aspects can be distinguished which are dependent on the quantity of wave overtopping:

- indirect reduction of wave forces due to reduction in wave reflection,
- direct reduction of wave forces due to wave overtopping



Figure 9. Easy-to-use design diagram for long-crested waves

6.1. Indirect Reduction of Wave Forces due to Reduction of Wave Reflection

DAEMRICH (1991) published data on the reflection coefficient influenced by wave overtopping. The reflection coefficient is defined as:

$$C_{r} = \frac{H_{S,r}}{H_{S,i}}$$
(3)

A reanalysis of DAEMRICH's data (1991) shows a decreasing trend of the reflection coefficient with increasing overtopping rate. This trend is obvious but the scatter of the data is considerable. Unfortunately, model tests on the influence of wave overtopping on wave reflection are still lacking, so that further comparison is not possible.

6.2. Direct Reduction of Wave Forces by Wave Overtopping

Overtopping will lead to a reduction of the horizontal load on the wall. To date this phenomenon has given very little attention in literature and to the authors

knowledge only very few hydraulic model tests have been conducted with a relatively low crest height where considerable amount of overtopping occurred.

The present working assumption for design is to cut the pressure figure at the top of the wall if any overtopping occurs. A typical example for a comparison of cases with and without overtopping is given in Fig. 10. Fig. 10b shows how the pressure distribution is cut at the top of the wall. The pressure ordinate at the top of the wall can be calculated from 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 was high enough.

However, this method does not result in a significant decrease of the load. Therefore, an additional approach is proposed. Two boundary conditions are defined: in Fig. 10a the calculated pressure distribution just reach the top of the wall (Case I) whereas in Fig. 10c the design water level (DWL) has reached the top of the wall (Case II). In the latter case wave induced hydro-dynamic pressures cannot exist any more. So this case can be designed by using simple hydrostatic approach. Between cases I and II further reduction of wave pressures and forces should be considered (Fig. 10b).

In Case II the wave-induced loading has to be zero at the top of the wall. Especially for impact breakers this is not the case when the pressure figure is simply cut off at the top of the wall (a pressure head of p_1 in the height of the DWL is still calculated by any design formulae). Therefore, this procedure will result in a too high pressure at the top of the wall.

A factor k_{Fh} is introduced that significantly reduces the loading. This factor accounts for the fact that the pressure distribution and the force in Fig. 10c (Case II) has to be zero and has its maximum at an infinite high wall (Fig. 10a). A reduction of horizontal forces F_h can then be obtained by:

$$F_{h,ov} = k_{F,h} \cdot F_h \tag{4}$$

and for the moment M_h the reduction is given by:

$$M_{ov} = k_{F,h} \cdot M_h \tag{5}$$

In Eq.(4) $F_{h,ov}$ is the reduced force, F_h is the horizontal force according to the design method used and $k_{F,h}$ is the reduction factor for overtopping as given by Eq.(6).

$$\begin{aligned} \mathbf{k}_{F,h} &= 1 & \text{für } \eta_* \leq \mathbf{R}_c \\ \mathbf{k}_{F,h} &= \sqrt[3]{\frac{\mathbf{R}_c}{\eta_*}} & \text{für } \eta_* > \mathbf{R}_c \end{aligned} \tag{6}$$



Figure 10. Comparison of Pressure Distributions With and Without Overtopping

In Eq.(6) 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 (see Fig. 10). Cutting the pressure distribution at the top of the wall is independent from this approach and will be taken into account in all cases. The k_{Fh} factor results in the lower curves shown in Fig. 11 for three different loading cases (1:standing waves; 2:impact waves; 3:broken waves).



Figure 11. Reduction of horizontal force by reduction factor k_{Fh}

These curves are assumed to be closer to reality than the common method of cutting the pressure distribution. However, for more accurate methods hydraulic model tests should be performed where the reduction of horizontal forces due to overtopping can be measured.

7. Summary and Recommendations for Future Research Work

Many research work has been performed for wave overtopping on vertical walls during the last years. Available papers and data have been reviewed with respect to the various geometric conditions in Hamburg. The most universal overtopping model has been selected and presented in a diagram for practical engineering use. Problems and restrictions of this model have been outlined. Finally an engineering approach for the reduction of wave forces by wave overtopping was shown. The results presented in this paper are in use for the design of about 100km vertical wall length in Hamburg.

From this study the following problems using available model tests and overtopping data have been encountered:

- A variety of overtopping models are available. It is however important to consider both reliability and simplicity when recommend any overtopping model for design.
- The influence of the different berm widths on wave overtopping is insecure. Therefore, more research work conducting model tests is necessary, especially for long berms.
- The influence of wave overtopping on the structure load has not yet been investigated. Consequently, new research work has to concentrate on the direct and indirect influence of wave overtopping. As a first working assumption it is proposed to use the approach presented in this paper.
- The influence of wind on wave overtopping is still under discussion and further research work is needed here.

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