CHAPTER 75

WAVE OVERTOPPING ON VERTICAL AND COMPOSITE BREAKWATERS

`L. Franco[1], M. de Gerloni[2] , J.W. van der Meer[3]

Abstract

After an extensive series of 2-D model tests on the overtopping response of various caisson breakwaters, general conceptual design formulae and graphs have been derived which relate the mean discharge with the relative freeboard. The influence of geometrical changes is described by reduction factors with reference to the pure vertical structure. A simple correlation has been made with the overtopping performance of sloping structures. Overtopping volumes per wave were also measured and fitted with a universal probability function; their effects on model persons and cars behind the crownwall were statistically evaluated, thus allowing an upgrading of the existing criteria for the admissible overtopping on breakwaters.

Introduction

Wave overtopping is one of the most important hydraulic responses of a breakwater, since it significantly affects its functional efficiency and to a minor extent even its structural safety (though the latter effect is often negligible for monolithic breakwaters). The overtopping discharge is in fact the main parameter for the design of shape and height of the breakwater crest.

However, little research work had been addressed to this subject in the past, since most attention had been paid to wave forces and breakwater stability. It may be

[1] Prof. Coastal Eng., DIIAR Politecnico di Milano, piazza Leonardo da Vinci 32, 20133 Milano, Italy.
[2] Manager Modelling Unity, ENEL SpA - CRIS, via Ornato 90/14, 20162 Milano, Italy.
[3] Dr., Delft Hydraulics, P.O.Box 152, 8300 AD Emmeloord, The Netherlands.

noted that these aspects are interrelated since the wall crest elevation influences the amount of wave force. Few overtopping studies are available for seawalls (Owen, 1980; Goda, 1985;) and more recently for rubble mound breakwaters (Jensen and Juhl, 1987; Aminti and Franco, 1988; Bradbury and Allsop, 1988; de Waal and van der Meer, 1992). Goda (1985) remains the main reference for the design of vertical and composite structures.

An extensive laboratory investigation on the overtopping performance of modern vertical-face breakwaters has been started in Milano since 1989, with random wave flume model testing. Preliminary results were presented by de Gerloni et al. (1989, 1991).

Additional model tests and a more detailed analysis of the test results have been carried out in 1993 with the support of the UE-MAST 2-MCS (Monolithic Coastal Structures) project funding. Moreover the results obtained by other European hydraulic laboratories from specific studies on similar structures have been incorporated in the analysis to enlarge the data set and improve validation. The first main results have been reported by Franco (1993, 1994) at the MAST workshops.

Experimental setup, test conditions and procedures

Model tests were carried out in the 43 m long, 1.5 m deep random wave flume of ENEL SpA - Center for Hydraulic and Structural Research (CRIS) laboratory in Milano. A special device was used for measuring the overtopping volumes: a tray suspended through a load cell to a supporting beam. The load cell signal reading after each overtopping wave allows the measurement of its individual volume; the number of overtopping waves, the total volume and hence the mean discharge in each test can then be easily computed.

The effects of each overtopping wave were analyzed by placing a few model cars and model persons along the center of the crown slab behind the wall, and by accurately observing the number of displacements and relative distance from the former position after each overtopping event (then repositioning the "targets").

The proper model scaling of the human behaviour (to 1:20) was assessed with a simplified full-scale test procedure: a large bucket and a plastic pipe were used to direct known amounts of water, from an elevation of 4.5 m and without notice, against both a volunteer (the first author of this paper) and a human-size plastic dummy. It was found that the dummy had to be ballasted up to 1500 N (twice the man weight) to have the same falling response of the man.

To improve the statistical validity rather long test durations were used with no less than 1000 waves. Peak periods (T_p) of JONSWAP spectra (bimodal spectra were also generated) varied between 7 and 13 s, significant wave heights (H_s) between 2.5 and 6 m with water depths/wave heights ratios (h/Hso) ranging between 3 to 5 (all figures are expressed in prototype terms for easier engineering interpretation). A total of about 250 tests with non-breaking wave conditions were performed.

Model breakwater configurations are shown in Fig. 1. They include traditional vertical-face caissons, perforated ones (14%, 25%, 40% porosity), shifted sloping parapets and a caisson with rubble mound protection (horizontally composite) with variable elevation and width of the homogeneous porous rock berm (S1 to S6 in Fig.1). All structures were designed for low overtopping conditions (i.e. high freeboard).

Additional results from model studies on similar structures designed in Italy and carried out by other European laboratories were included in the analysis, to enlarge the data set by covering a wider range of geometric and hydraulic conditions ($H_s=2\div8m$ $T_p=6\div15s$ h=9÷18 m). They were performed at Delft Hydraulics (DH) on vertical and shifted caissons and at Danish Hydraulic Institute (DHI) on perforated shifted-wall caissons. Further model test results from a research study on a simple vertical wall were supplied by CEPYC laboratory in Madrid. All these additional model test data typically only refer to the mean overtopping rate.
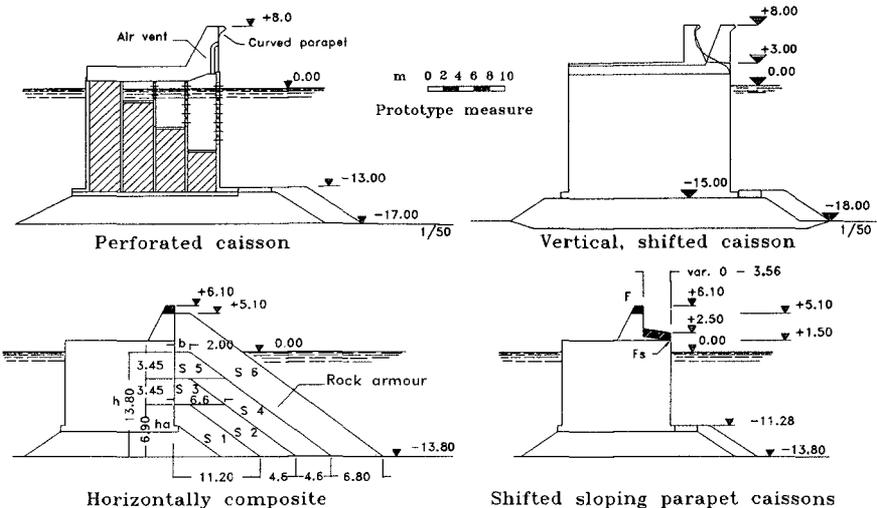

Fig 1 Model test sections of caisson breakwaters.

The influence of onshore winds on the overtopping effects was considered negligible particularly for large water flows above vertical walls. This assumption has been confirmed by a recent laboratory study (de Waal, 1994) which shows that the additional spray transport due to wind never exceeds 3 times the mean rate and is less than 1.4 times in typical deep water conditions similar to the tested ones. This increase is small compared to the much larger variability of overtopping with the wave height.

Admissible overtopping rates

The definition of tolerable limits for overtopping is still an open question, given the high irregularity of the phenomenon and the difficulty of measuring it and its consequences. Many factors, not only technical ones, should be taken into account to define the safety of the increasing number of breakwater users such as the psicology, age and clothing of a person surprised by an overtopping wave.

Still the current admissible rates (expressed in m^3/s per m length) are those proposed by the Japanese guidelines, based on impressions of experts observing prototype overtoppings (Fukuda et al. 1974; Goda, 1985). They are included in CIRIA/CUR-manual (1991), and in British Standards (1991) (Fig. 2). The lower limits of inconvenience to pedestrians may correspond to safe working conditions on the breakwater, while the upper limits of danger to personnel may correspond to safe ship stay at berth.

Obviously the overtopping criteria for design depend upon the function and degree of protection required, and upon the associated risk considerations, taking into account the joint probability of wave heigths and water levels. In fact relatively large overtopping might be allowed during extreme storms (structural design conditions) if transit on the breakwater is then prohibited (functional limit).

The structural safety of the breakwater typically demands less restrictive limits than the safety of its utilization (functional safety). The maximum admissible overtopping discharge for the structural safety of dikes and revetments are shown by Goda (1985). If the features of cast-in-situ concrete superstructures of modern caisson breakwaters are considered, the higher limit given for paved revetments (0.2 $m^3/s/m$) can be assumed.

As far as the functional safety is concerned (e.g. drainage behind seawalls), the figure of 0.01 $m^3/s/m$ is considered as the tolerable discharge for direct

protection of densely populated coastal areas (Goda, 1985). Much lower discharges (0.0001 m^3/s/m) are given for the safe transit of vehicles, such as along a coastal highway. Few data are available on the critical overtopping discharges for the safety of various harbour operations and ship mooring on the breakwater rear side. A value of 0.00042 m^3/s/m in the 50 year design storm is proposed by Sigurdarson and Viggosson (1994) as criterion for damage to equipment and cargo on quay.

    One of the aims of this model study was to assess better criteria in the case of caisson breakwaters. It was then believed that the overtopping volume per wave (V), being actually responsible for the damage on the breakwater crown, was a far better hydraulic parameter than the mean discharge for this analysis, when there is no need of land reclamation drainage.
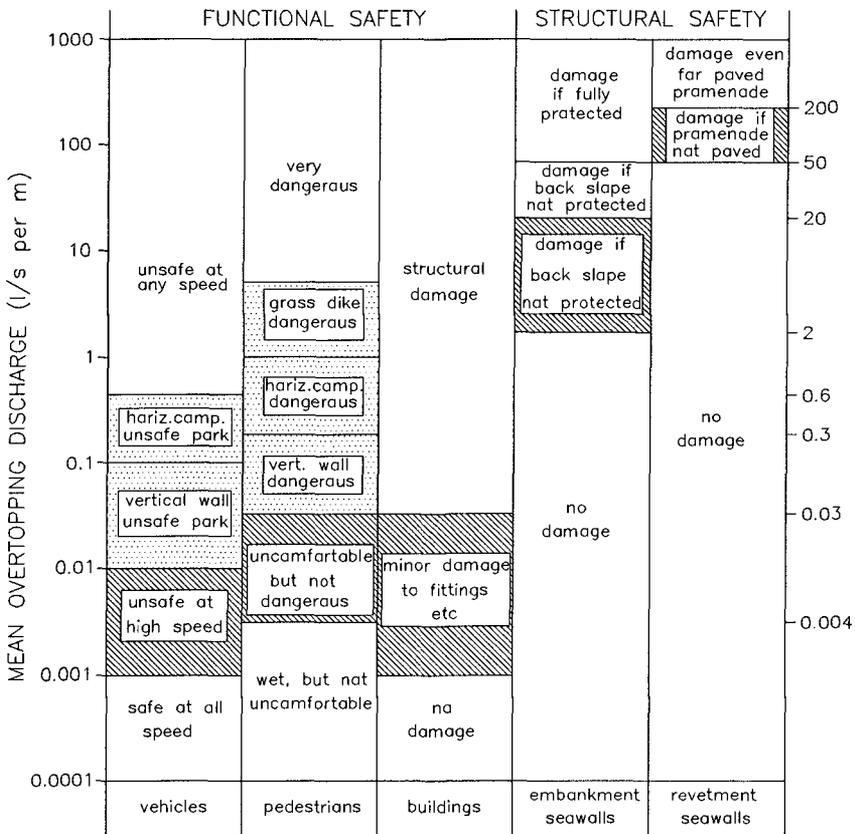
Fig. 2   Critical overtopping discharges of existing guidelines integrated with new safety bands (dotted) for transit on breakwaters.

For each breakwater configuration the individual overtopping volumes recorded in any tests were divided in classes of 0.1 m^3/m and the corresponding effects on model cars and pedestrians were statistically evaluated for each class. Some results obtained for pedestrians are shown in Fig. 3. It is interesting to observe that the effect is dependent on the structure geometry itself. The same overtopping volume is likely to be more dangerous if the breakwater is purely vertical than in the case of perforated or shifted-parapet caissons or horizontally composite ones. This is probably due to the different overflow mechanism which produces a more concentrated and fast water jet falling down from the crest of a vertical wall in comparison with a slower, more aerated, horizontal flow over a sloping structure. It was also observed that pedestrians are slightly more "stable" than vehicles undre the same overtopping wave.
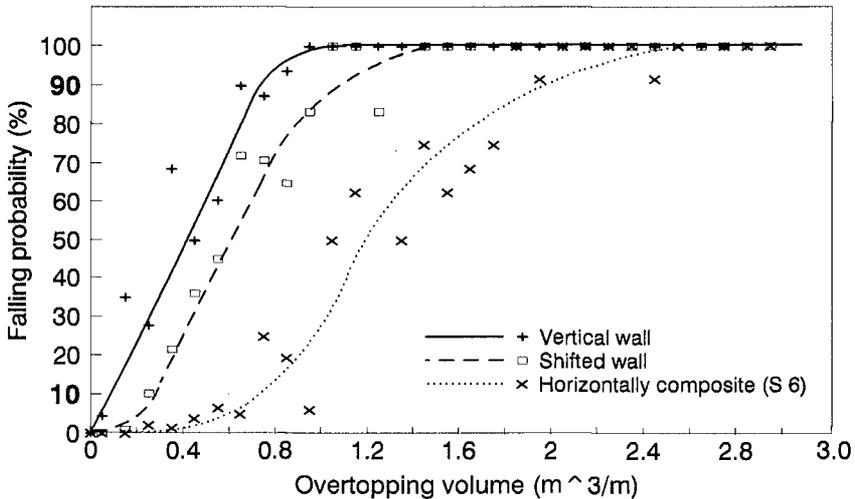
Fig. 3 Risk curves for pedestrians on caisson breakwaters from model tests.

From Fig. 3 it can be seen that the "critical bands" of overtopping volume (being dangerous above the upper limit and safe below the lower one) lie between 0.2 and 2.0 m^3/m (but a concentrated jet of 0.05 m^3/m on the upper body can be enough to make a person fall down as shown by the full scale calibration tests)

These results may be translated in terms of traditional mean rates for easier comparison with the previous criteria, by using the graph of Fig. 4, where the correlation between the mean discharge and the maximum (one in 1000 waves) volume is shown for some structural configurations. The critical band volumes

within the range of 10% to 90% falling probability in Fig. 3 are assumed as $V_{max}=V_{0.1\%}$ in a conservative approach to enter in Fig. 4.

The new proposed critical discharges, which are at least 10 times higher than the previous ones, are illustrated in Fig. 2, which also includes the results of recent large scale tests (Smith et al., 1994) showing that the dangerous discharge for a man standing on a dike is in the range $(1\div10)\cdot10^{-3}$ m^3/s/m. From Fig. 4 it can be observed that this discharge actually corresponds to maximum volumes of 0.5$\div$2 m^3/m while in the same range of V_{max} the average discharge over a vertical wall is $(0.1\div0.5)\cdot10^{-3}$ m^3/s/m. The ratio V_{max}/q can in fact vary between 100, for large mean discharges and percentage of overtopping waves, and 10,000, for small mean discharges and for vertical structures.

Thus it is confirmed that the significant parameter for the breakwater functional safety is the overtopping volume rather than the mean discharge. A relationship exists between the two parameters but it varies with the structure geometry and wave conditions.
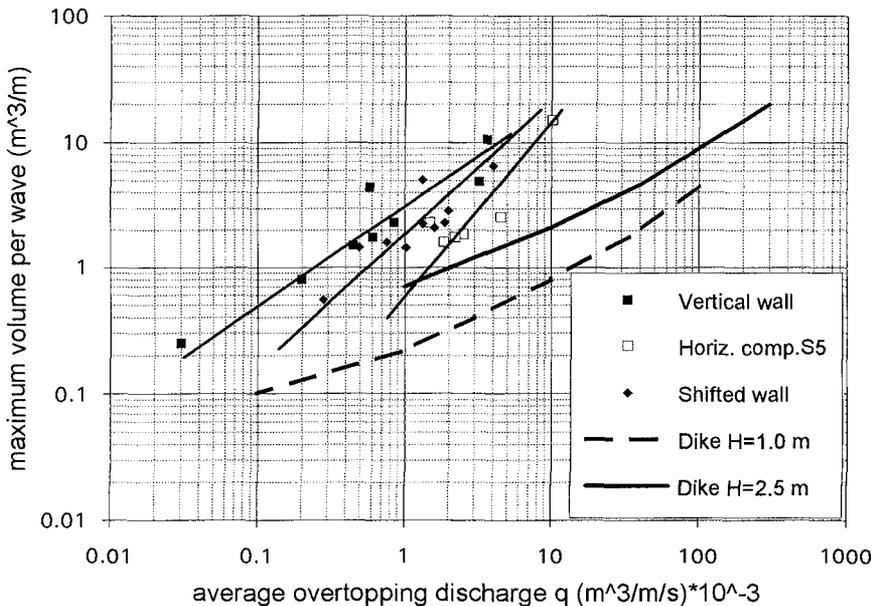

Fig. 4 Relation between mean discharge and maximum overtopping volume.

Probability distribution of individual overtopping waves

Overtopping events occur unevenly both in time and amount, often just a few waves overtopping among the thousands. The measurement of the individual overtopping volumes carried out during the model tests allowed the definition of their probability distribution. Considering the tests with a number of overtopping waves $N_{ow}>30$ (for obvious reasons of statistical significance) the exceedance probability of each overtopping volume P_v was calculated and a 3-parameter Weibull distribution function gave best fit to the data as shown in the example of Fig. 5:

$$P_v = 1 - \frac{i}{N_w + 1} = C \cdot \exp\left(-\frac{V}{A}\right)^B = \exp\left(-\frac{V-C}{A}\right)^B \qquad (1)$$

where N_w is the number of waves in the test, V is a volume in the ith rank and A,B,C are fitting constants.

It would also be possible to use in Fig. 5 on the horizontal axis the probability related to the number of overtopping waves (N_{ow}) instead of the incident waves (N_w). In that case the graph would start at 100% and C in eq. 1 would become 1.0.

It can be demonstrated with some mathematics that the scale parameter A can be defined by the equation:

$$A = 0.84 \cdot \overline{V} = \frac{0.84 \cdot T_m \cdot q}{\dfrac{N_{ow}}{N_w}} \qquad (2)$$
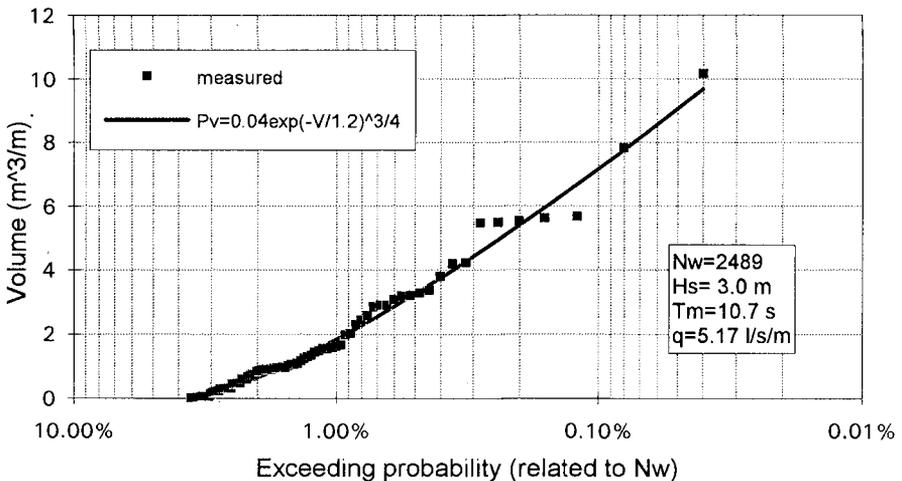

Fig. 5 Example of probability distribution of overtopping volume per wave: vertical wall caisson, test 21.

where T_m is the mean wave period, q is the average overtopping discharge and $\overline{V}$ is the average overtopping volume per wave. Hence the exceedance probability of a given volume is related to the mean discharge and to the overtopping probability. Even V_{max} can be related to the other two overtopping parameters from eq. 1 as:

$$V_{max} = A \cdot \ln\left(N_{ow}\right)^{4/3}$$

(3)

The shape parameter B was found to have a little variability around a mean value of 0.75, which is the same found by Van der Meer and Janssen (1994) for dikes. Then B=0.75 is assumed to be constant.

The "set-off" coefficient C, being the intercept with the x axis (Fig. 5), represents the percentage (probability) of overtopping waves (N_{ow}/N_w), which is assumed to be Rayleigh distributed, and can be expressed by the following equation (Fig. 6):

$$C = \frac{N_{ow}}{N_w} = \exp\left(-\frac{1}{k}\frac{R_c}{H_s}\right)^2$$

(4)

where best fitting gives k=0.91 for caisson breakwaters and R_c/H_s=R is the relative freeboard, R_c being the wall crest height above the sea level.

The assumption that C represents the overtopping probability was confirmed experimentally as shown in Fig. 7, which presents the comparison between measured ratios $C=N_{ow}/N_w$ and the corresponding values calculated from data fitting in eq. 1; the experimental verification of eq. 2 for coefficient A is also shown.
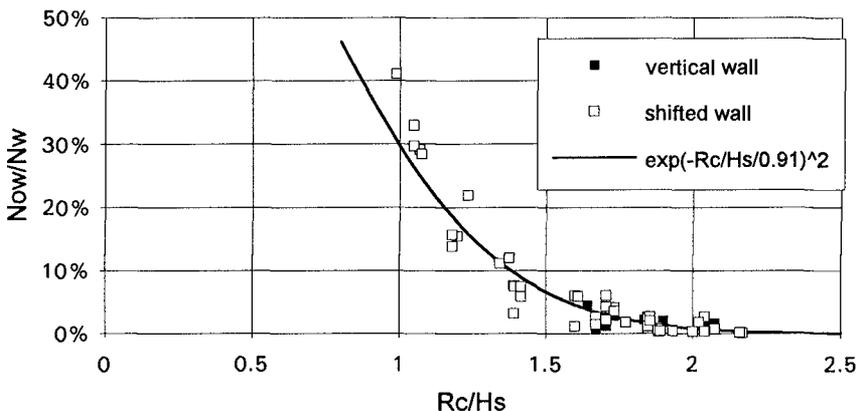

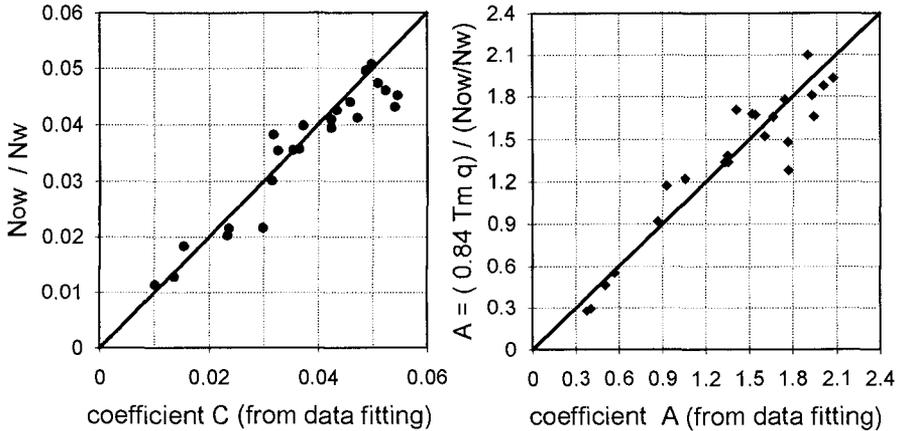Fig. 6 Correlation between the percentage of overtopping waves and the relative freeboard.

Fig. 7 Verification of functional relationship of coefficients C and A.

Conceptual design formulae

The method of analysis proposed by van der Meer and de Waal (1992) to derive a general design formula was applied to the tests results (restricted to a wave steepness range of 0.018-0.038) in terms of mean overtopping discharge allowing a direct comparison with the above admissible limits and an easy evaluation of the overtopping volumes per wave with eq. 1 and 2.

Consistent curves have been fitted with the least square method to the experimental data representing the dimensionless mean overtopping discharge $Q = q / \sqrt{g \cdot H_s^3}$ against the relative freeboard R_c/H_s, which is the most important parameter. Since an exponential relationship is assumed according to Owen (1980), the data should give a straight line on a log-linear plot:

$$Q = a \cdot \exp\left(-\frac{bR_c}{H_s}\right) \qquad (5)$$

It was found (Fig. 8) that for vertical-face breakwaters b=4.3 and a=0.192, which is close to the one found by van der Meer and Janssen (1994) for sloping structures (a=0.2); the value a=0.2 was then kept constant for the successive regressions with different geometries which generally showed a high correlation coefficient (Fig. 9). The physical interpretation of "a" is the dimensionless mean discharge when the freeboard is set at the mean water level. It may be observed in Fig. 9 that the overtopping rates predicted by eq. 5 are
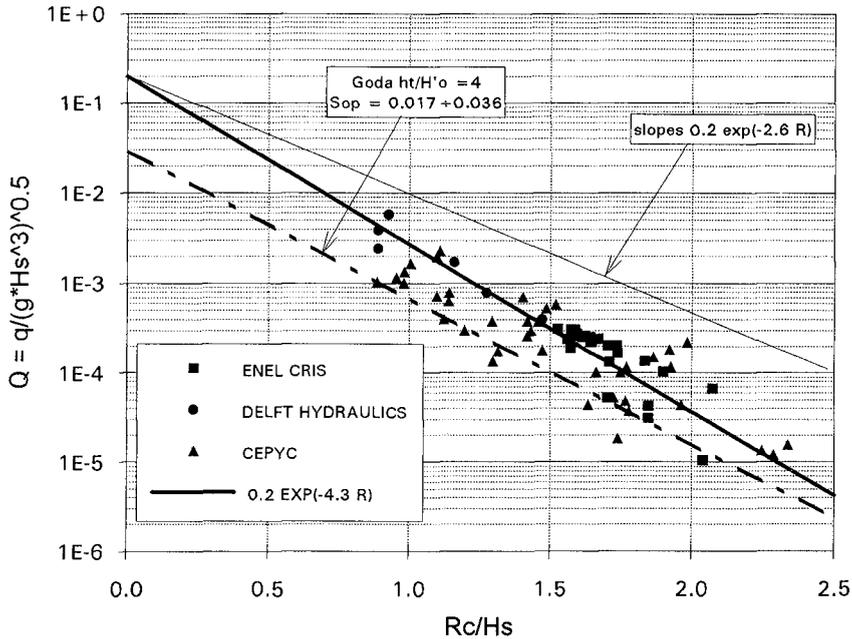
Fig. 8 Regression of wave overtopping data for vertical wall breakwaters.

slightly larger than those predicted by Goda (1985) for deep water vertical walls and with similar wave steepness range, besides the small differences in seabed slope and toe geometry. A similar underprediction of Goda's curves was also found by De Waal (1994).

Then the influence of structural modifications with reference to the vertical-face breakwater can be described by suitable freeboard reduction factors (γ), which are the ratios between the reference value b=4.3 and the various b coefficients fitted by eq. 5 as given in Fig. 9. Even the sloping structures (under non-breaking conditions) can be easily compared with the vertical ones considering the ratio γ_s=4.3/2.6=1.66, 2.6 being the fitting coefficient obtained by van der Meer and Janssen (1994) as also shown in Fig. 8. All the data can be plotted together (Fig. 10) after correction of the Rc/Hs values for each geometry with the corresponding γ, the general equation thus becoming:

$$Q = 0.2 \cdot \exp\left(-\frac{4.3}{\gamma} \frac{R_c}{H_s}\right) \tag{6}$$
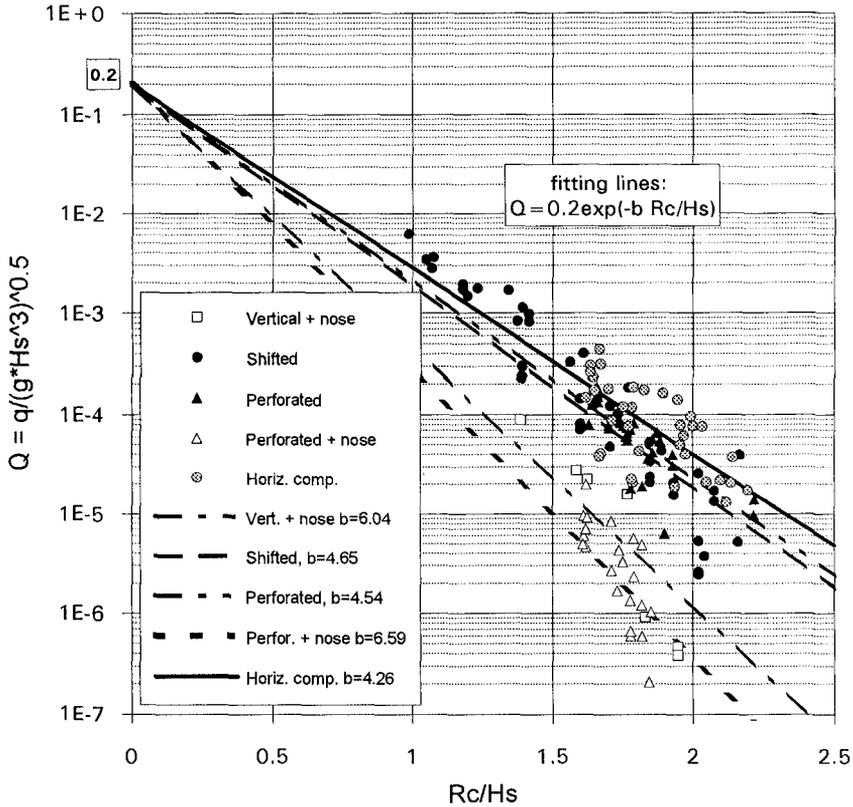
Fig. 9 Wave overtopping data for various types of caisson breakwaters.

which can be effectively used for the preliminary design of vertical breakwaters. The reliability of the formula 6 can be given by taking the coefficient 4.3 as a normally distributed stochastic variable with a standard deviation $\sigma = 0.3$.

From the influence factors of the various caisson geometries, as compared to the plain vertical wall some useful engineering conclusions can be distilled:
• the greatest overtopping reduction can be achieved by introducing a recurved parapet (nose) at the crest of a vertical front wall: the corresponding $\gamma_n = 0.7$ means a 30% crest elevation reduction to get the same overtopping rate; this may however be limited to relatively small discharges;
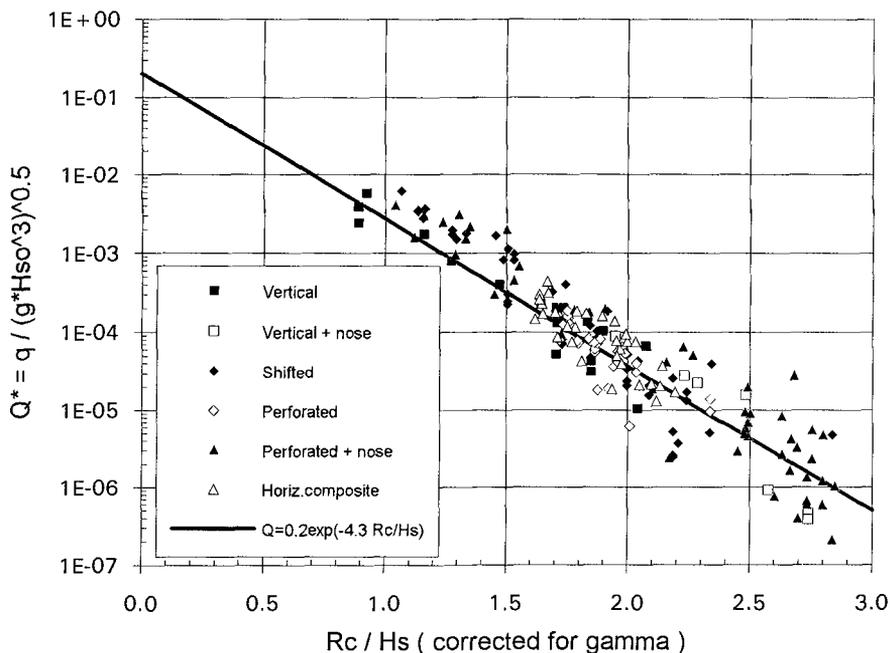• for simply perforated or shifted caissons the freeboard saving is only 5÷10%;

Fig. 10 Wave overtopping on vertical and composite breakwaters: conceptual design graph.

- if a nose is adopted at the crest of a perforated caisson, then the combined reduction factor can achieve 0.65, while its effect on a shifted parapet is negligible;
- the overtopping of horizontally composite breakwaters is influenced by porosity, slope, width and elevation of the mound. Overtopping increases if the armour crest is below or at mean sea level (max $\gamma_{ss}=1.15$).

Conclusions and further work

The results of an extensive 2D model test investigation on the overtopping performance of caisson breakwaters have been analysed to produce updated criteria for their functional safety and a new comprehensive conceptual design method.

The following remarks, valid for structures designed for relatively small overtopping, may be outlined:

- the proposed admissible overtopping discharges (q) for the safety of people and vehicles on breakwaters are

larger than those presently recommended in the manuals and are dependent on the structure geometry;

- the overtopping volume (V) is however a better parameter for allowable criteria;
- overtopping discharges on deepwater vertical walls are slightly larger than those predicted by Goda (1985) and quite smaller than those for an equivalent sloping structure (dike);
- the same exponential relationship between Q and Rc/Hs applies for both structure types (the reduction factor for vertical walls being 0.6), whereas very different and large ratios V_{max}/q can be observed;
- the combination of a perforated wall with a recurved crest (nose) on the front wall produces the largest overtopping reduction, whereas a rock protection in front of the caisson up to the sea level can increase overtopping;
- the probability distribution of overtopping volumes per wave is well defined by a Weibull distribution with a shape factor of 0.75 and a scale factor dependent on q and on the percentage of overtopping waves.

Further work is necessary to take into account the effects of wave obliquity and directional spreading. Actually a 3D caisson model study has just been carried out at Delft Hydraulics within the same European MAST-MCS project and results will be published soon. Additional analysis should also be performed to verify the influence of other less important structural and hydraulic parameters. The critical overtopping for the structural integrity of the caisson foundation system as well as for other facilities on and behind the breakwater should be also evaluated.

Finally, large scale model studies and prototype measurements of wave overtopping on real breakwaters are recommended to verify the present guidelines.

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Photo 1 Assessing first author's stability under water jets; a ballasted manikin (on the left) is waiting for its turn.

Photo 1 Effects of wave overtopping on model cars and model persons on a caisson breakwater.