EXTENDING THE H&R WAVE OVERTOPPING MODEL TO VERTICAL STRUCTURES

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Abstract

The H&R wave overtopping model, originally developed for sloping structures, is employed in a number of operational coastal flood forecasting and warning systems. This paper describes the first step in a project to extend the model to encompass vertical structures. Use is made of the CLASH database for the purpose of checking whether it is appropriate to simply extrapolate the existing formulae for the model's empirical coefficients beyond their strict ranges of applicability. The outcome is encouraging. Despite the fact that, in general, the model tended to underestimate the higher wave overtopping rates, it was reasonably accurate in describing the lower overtopping rates which normally trigger flood warnings, suggesting that it may be worthwhile pursuing the task of calibrating and validating the H&R model for vertical structures.

1. Introduction

Since 1998, Royal HaskoningDHV (RHDHV) has been developing coastal flood forecasting and warning systems in the UK for the Environment Agency (EA) and the Scottish Environment Protection Agency (SEPA) and in the Republic of Ireland for Dublin City Council (DCC). The latest development was the new Firths of Forth and Tay coastal flood warning system for SEPA, which has been in operation since November 2012. These systems aim to give those at risk valuable time to protect their families, homes and businesses, by providing advance warning of flooding.

In developing all systems, RHDHV has analysed the suitability of several methodologies/tools for predicting wave overtopping (Lane *et al.*, 2008; Naysmith *et al.*, 2013), such as the AMAZON numerical model (Hu, 2000; Reis *et al.*, 2011), empirical formulae from the EA Manual (Besley, 1999) and the EurOtop Manual (Pullen *et al.*, 2007), and the H&R semiempirical model (Hedges & Reis, 1998, 2004; Reis *et al.*, 2008). The H&R overtopping model, designed especially to predict low wave overtopping rates, proved to be the most robust and reliable, since AMAZON was not as efficient, resulting in higher development costs, and the other empirical methodologies significantly overpredicted low overtopping rates. Overprediction would lead to false warnings being issued and result in a warning system that the public would soon distrust. AMAZON was chosen only for more complicated defence/beach profiles.

Since the H&R model was found to suit the nature of flood forecasting practice, it was recommended by RHDHV to extend this model to vertical structures, not included in its calibration and validation. The CLASH database (Van der Meer *et al.*, 2009) was used for this purpose. It consists of more than 10,000 tests, each described by means of 31 parameters. Many types of coastal structures are included, such as dikes, rubble mound breakwaters, berm breakwaters, caisson structures and combinations. Some of the tests were performed within the CLASH project to cover missing parameter values. Other CLASH tests were undertaken both at prototype and model scales.

This paper describes the first step in extending the H&R model to vertical structures and the further developments which are planned. Following this introduction, Section 2 describes the

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original development of the H&R model. Section 3 deals with the extraction of datasets from the CLASH database for analyzing the current H&R model performance for vertical walls, which is covered in Section 4. The paper ends with the main conclusions drawn so far.

2. The Hedges and Reis (H&R) Model

2.1 Its development

Hedges & Reis (1998) introduced a semiempirical model (the H&R model) based on an overtopping theory for regular waves developed by Kikkawa *et al.* (1968), who had assumed that a seawall acted as a weir whenever the incident water level exceeded the seawall crest level and that the instantaneous discharge was described by the weir formula. The H&R model extended the concept to random waves. It can be written as follows:

$$\begin{cases} q = A\sqrt{gR_{max}^3}\left[1-\dfrac{R_c}{\gamma_f R_{max}}\right]^B & \text{for } 0 \le \left[\dfrac{R_c}{\gamma_f R_{max}}\right] < 1 \\ q = 0 & \text{for } \left[\dfrac{R_c}{\gamma_f R_{max}}\right] \ge 1 \end{cases} \tag{1}$$

where q is the mean wave overtopping discharge per unit length of seawall, A and B are empirical coefficients, g is the gravitational acceleration, R_{max} is the maximum run-up on a smooth slope induced by the random waves during a storm, γ_f is the reduction factor to account for slope roughness/permeability, and R_c is the crest freeboard of the structure. The precise value of R_{max} during any particular storm cannot be known *a priori*: an estimate of its value has to be made. Unless $R_{max} > R_c/\gamma_f$, there is no overtopping apart from wind-blown spray. Figure 1 shows a diagrammatic representation of the model.

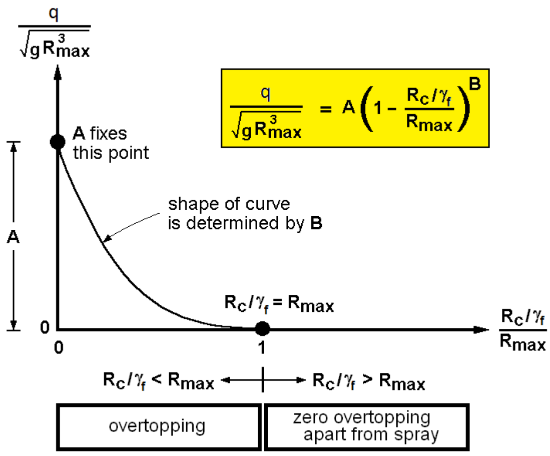

Figure 1. A diagrammatic representation of the H&R model (Hedges & Reis, 2004).

Coefficients A and B have been evaluated using the results of hydraulic model tests. Hedges & Reis (1998) originally used Owen's data for that purpose (Owen, 1980), in which overtopping had been evaluated for runs of 100 waves acting on smooth slopes (γ_f = 1) of 1:1, 1:2 and 1:4. In this case, the most probable maximum run-up during each run (i.e., the value not exceeded in 37% of the cases assuming a Rayleigh distribution of run-ups) was

$$(R_{max})_{37\%,100} = 1.52R_s \tag{2}$$

where R_s is the significant wave run-up, which was initially evaluated using the equations given in the CIRIA/CUR (1991) manual. The value of the maximum run-up not exceeded in 99% of the cases for runs of 100 waves was expected to be described by Equation (3):

$$(R_{max})_{99\%,100} = 2.15R_s \quad (3)$$

More recently, Mase *et al*. (2003) modified and extended the H&R model to account for Japanese data on run-up and overtopping. The new data (referred to later as the Kansai data) covered front slopes as shallow as 1:20. Furthermore, instead of using the expressions given in the CIRIA/CUR (1991) manual to estimate R_s, Mase *et al*. (2003) used a modified version of Hunt's equation that incorporated wave setup:

$$R_s / H_s = \begin{cases} 0.25 + 1.10\xi_p & \text{for } \quad 0 < \xi_p \le 2.2 \\ 3.00 - 0.15\xi_p & \text{for } \; 2.2 < \xi_p \le 9.0 \\ 1.65 & \text{for } \; 9.0 < \xi_p \end{cases} \quad (4)$$

where H_s is the incident significant wave height and $\xi_p = \tan\alpha/(H_s / L_{op})^{0.5}$ is the surf similarity (or breaker) parameter, with α the angle of the seawall front slope measured from the horizontal, $L_{op} = gT_p^2 / 2\pi$ and T_p the wave period corresponding to the peak of the incident wave spectrum. The question of where the incident waves should be specified is dealt with in the next section.

Combining Equations (2) and (4) gives Equations (5).

$$(R_{max})_{37\%,100} / H_s = \begin{cases} 0.38 + 1.67\xi_p & \text{for } \quad 0 < \xi_p \le 2.2 \\ 4.56 - 0.23\xi_p & \text{for } \; 2.2 < \xi_p \le 9.0 \\ 2.51 & \text{for } \; 9.0 < \xi_p \end{cases} \quad (5)$$

Equations (3) and (4) give Equations (6).

$$(R_{max})_{99\%,100} / H_s = \begin{cases} 0.54 + 2.37\xi_p & \text{for } \quad 0 < \xi_p \le 2.2 \\ 6.45 - 0.32\xi_p & \text{for } \; 2.2 < \xi_p \le 9.0 \\ 3.55 & \text{for } \; 9.0 < \xi_p \end{cases} \quad (6)$$

Furthermore, if the Rayleigh distribution applies to run-ups, then:

$$(R_{max})_{37\%,100} = R_{1\%} \quad (7)$$

in which $R_{n\%}$ denotes the value exceeded by n% of all the individual run-ups.

A value such as $(R_{max})_{37\%,100}$ simply provides an estimate of the actual maximum run-up during a storm (i.e., it provides an estimate of the minimum freeboard needed for zero overtopping). However, this estimate should ensure, at the very least, that any overtopping that does occur remains negligible. In this connection, it is worth noting that, in the past, sea defences in continental Europe were often planned with a freeboard equal to $R_{2\%}$ under design wave conditions. $(R_{max})_{37\%,100}$ is about 8.5% greater than $R_{2\%}$ according to the Rayleigh distribution. It is also interesting to note that if $(R_{max})_{37\%,100}$ is a satisfactory estimate of R_{max}, then there should be no evidence of overtopping of seawalls having $R_c > 4.1H_s$, regardless of the front slope or of the incident wave steepness because Equations (5) give $(R_{max})_{37\%,100} = 4.1H_s$ (approximately) as the maximum value. In this regard, Van der Meer & Janssen (1995) record no overtopping discharges for cases in which $R_c / H_s > 4.1$. Even so, for a more conservative approach, designers can choose to estimate R_{max} with $(R_{max})_{99\%,100}$ (= $[R_{max}]_{37\%,10,000} = R_{0.01\%}$). This choice would accept the possibility of overtopping, for certain combinations of slope and wave steepness, whenever $R_c < 5.8H_s$ (approximately). Both the Owen (1980) and Van der Meer & Janssen (1995) models, being of exponential form, predict some degree of overtopping, regardless of how small the waves might be, for all finite values of R_c.

Reanalysis of Owen's data and the Kansai data has provided updated values of coefficients A and B in Equations (1). The new analysis has also provided coefficients for the shallower slopes covered by the Kansai data (with slopes as shallow as 1:20). Using $(R_{max})_{37\%,100}$ provided by Equations (5), the values of A and B are now described by the following expressions:

$$A = \begin{cases} 0.0033 + 0.0025\cot\alpha & \text{for} \quad 1 \le \cot\alpha \le 12 \\ 0.0333 & \text{for} \quad 12 < \cot\alpha \le 20 \end{cases}$$
$$B = \begin{cases} 2.8 + 0.65\cot\alpha & \text{for} \quad 1 \le \cot\alpha \le 8 \\ 10.2 - 0.275\cot\alpha & \text{for} \quad 8 < \cot\alpha \le 20 \end{cases} \tag{8}$$

Equally, with $(R_{max})_{99\%,100}$ provided by Equations (6), the values of A and B are described by:

$$A = \begin{cases} 0.0016 + 0.002\cot\alpha & \text{for} \quad 1 \le \cot\alpha \le 10 \\ 0.0216 & \text{for} \quad 10 < \cot\alpha \le 20 \end{cases}$$
$$B = \begin{cases} 5.34 + 1.15\cot\alpha & \text{for} \quad 1 \le \cot\alpha \le 7 \\ 16.61 - 0.46\cot\alpha & \text{for} \quad 7 < \cot\alpha \le 20 \end{cases} \tag{9}$$

Equations (8) and (9) have subsequently been validated using the data of Hawkes *et al*. (1998) for uniform seaward slopes of 1:2 and 1:4 and the SHADOW data (Bay *et al*., 2004) for relatively high discharges over slopes of 1:2, 1:10, and 1:15. Table 1 shows the range of conditions covered by the calibration and validation tests carried out previously. Note that h is the water depth immediately in front of the toe of the structure.

Table 1. Range of conditions covered by the calibration and validation tests (Reis *et al*., 2008).

Seawall Slope	Limits	Tests	ξ_p	R_c/H_s	h/H_s
1:1	Min	calibration	4.71	0.57	1.65
	Max		6.20	2.40	5.06
1:1.333	Min	calibration	3.69	0.06	3.88
	Max		8.25	2.24	21.30
1:2	Min	calibration	2.35	0.57	1.69
	Max		3.13	2.60	5.19
	Min	validation	2.12	0.85	0.93
	Max		6.52	7.65	26.31
1:3	Min	calibration	1.64	0.06	3.88
	Max		3.67	2.24	21.30
1:4	Min	calibration	1.19	0.58	1.65
	Max		1.55	2.42	5.13
	Min	validation	1.13	0.86	0.95
	Max		2.38	2.86	4.29
1:5	Min	calibration	0.98	0.06	3.88
	Max		2.20	2.24	21.30
1:7	Min	calibration	0.70	0.06	3.45
	Max		1.57	2.24	21.30
1:10	Min	calibration	0.55	0.16	3.45
	Max		0.78	1.00	15.50
	Min	validation	0.51	0.69	2.80
	Max		2.55	6.29	14.80
1:15	Min	calibration	0.32	0.06	3.88
	Max		0.73	2.24	21.30
	Min	validation	0.34	0.52	2.93
	Max		1.71	4.63	14.79
1:20	Min	calibration	0.22	0.13	2.76
	Max		0.38	0.75	15.51

2.2 Where should the incident waves be specified?

Clearly, in applying overtopping models, it is important to be aware of where the input wave conditions should be specified. The most common are: offshore, at the toe of the foreshore slope (Owen, 1982; Hedges & Reis, 1998, 2004) and at the toe of the structure itself (Pullen *et al*., 2007).

The input wave conditions are the same at the three locations if the toe of the structure is in deep water.

If the wave conditions are specified offshore or at the toe of the foreshore, then the foreshore and structure may be treated as a single entity in estimating overtopping. If the wave conditions are specified at the toe of the structure, then it is important to be aware of the influence of wave breaking over the foreshore, particularly as wave breaking will induce a set-up of the water level in front of the structure. Wave breaking becomes important if the ratio of the depth of water, h, to the incident significant wave height, H_s, is less than about 3.

Finally, note that Owen's and the Kansai datasets, against which the H&R model was calibrated, related overtopping rates to wave conditions at the toe of a foreshore sloping uniformly seaward to a depth unaffected by wave breaking. The relative water depths at the toes of the model embankments were in the range $1.65 < h / H_s < 5.20$ (see Table 1). Consequently, use of the H&R model is currently limited to the specification of wave conditions at the toe of the foreshore within the above range of relative water depths.

2.3 Extension to vertical structures

Extending the H&R overtopping model to vertical structures is planned to encompass:

1. A thorough review of the data for vertical walls (and other steep slopes) in the CLASH database (Van der Meer *et al*., 2009) to check if: i) the H&R model needs further calibration; and ii) if there are sufficient data for both calibration and validation of the model;
2. A reflection on any additional wave conditions and/or structure geometries, beyond those already in the CLASH database, which might need testing;
3. Composite (numerical and physical) modeling of additional wave conditions and structure geometries;
4. Final H&R model calibration.

The next sections report on the work carried out within the scope of step 1i) and on its outcome.

3. Extraction of Datasets from the CLASH Database

3.1 Datasets

As noted earlier, the CLASH database includes various types of coastal structures, from simple vertical walls to complex rubble-mound breakwaters. Datasets were extracted only for vertical structures with smooth, impermeable faces. Excluded from these selected datasets were data for vertical structures with toes, berms, re-curved walls and broad crests, as well as data for oblique wave incidence. Data for zero overtopping rates were used in the analysis.

The extraction provided 602 overtopping results from 9 datasets, all obtained from small-scale tests. Table 2 lists the dataset names and, for each dataset, the number of tests and the ranges of some of the parameters considered. In this table, $T_{m-1,0\ deep}$ is the deep water wave period obtained from the 0 and the -1 moments of the spectrum (m_0 and m_{-1}); $T_{p\ deep}$ is the peak wave period in deep water; $T_{p\ toe}$ is the peak wave period at the toe of the structure; $H_{mo\ deep}$ is the significant wave height determined at deep water from spectral analysis; $H_{mo\ toe}$ is the significant wave height determined at the toe of the structure from spectral analysis; $m = \cot\theta$ is the cotangent of the foreshore slope, where m = 1000 represents a test in a flume with no foreshore; h_{deep} is the deep water depth; and, as mentioned earlier, h is the water depth in front of the structure toe.

Table 2. Characteristics of extracted CLASH datasets for vertical walls.

Dataset		028	106	107	224	225	351	402	502	802
Number of tests		173	30	56	35	18	2	32	47	209
$T_{m-1,0\ deep}$ (s)	Min	1.102	0.972	1.070	1.291	1.273	1.311	0.690	0.965	1.613
	Max	2.313	1.962	7.500	3.000	2.373	1.695	1.069	1.517	2.656
$T_{p\ deep}$ (s)	Min	1.212	1.069	1.020	1.420	1.400	1.442	0.759	1.045	1.774
	Max	2.544	2.158	5.114	3.300	2.610	1.864	1.176	1.707	2.922
$T_{p\ toe}$ (s)	Min	1.222	1.069	1.020	1.420	1.419	1.442	0.759	1.045	0.000
	Max	2.607	2.158	5.114	3.300	2.637	1.864	1.176	1.707	2.971
$H_{mo\ deep}$ (m)	Min	0.050	0.054	0.047	0.125	0.114	0.084	0.030	0.050	0.051
	Max	0.210	0.262	0.247	0.201	0.194	0.115	0.091	0.148	0.179
$H_{mo\ toe}$ (m)	Min	0.030	0.054	0.047	0.119	0.114	0.084	0.030	0.031	0.000
	Max	0.166	0.262	0.247	0.197	0.173	0.115	0.091	0.099	0.168
m=cotθ	(-)	10, 30, 100	1000	1000	50	20	1000	1000	10, 50	10, 30
h_{deep} (m)	Min	0.700	0.700	0.600	0.670	0.567	0.276	0.700	0.700	0.580
	Max	0.700	0.775	0.800	0.970	0.809	0.286	0.700	0.700	0.580
h (m)	Min	0.050	0.700	0.600	0.200	0.167	0.276	0.700	0.090	-0.100
	Max	0.200	0.775	0.800	0.500	0.409	0.286	0.700	0.247	0.225
R_c (m)	Min	0.035	0.025	0.000	0.130	0.165	0.117	0.070	0.085	0.065
	Max	0.200	0.200	0.200	0.400	0.400	0.127	0.150	0.150	0.264

3.2 Wave conditions

As noted earlier, wave conditions should be specified at the toe of the foreshore when using the H&R model. However, wave conditions in the CLASH database are specified in deep water and at the toe of the structure, only. For the 9 datasets extracted from the CLASH database, wave conditions in the database were obtained from measurements ("Meas") or calculated *a posteriori* ("Calc"), as shown in Table 3.

Table 3. Determination of wave characteristics of extracted CLASH datasets.

Dataset	Number of tests	m = cotθ	$H_{mo\ deep}$ (m)	$T_{p\ deep}$ (s)	$T_{m\ deep}$ (s)	$T_{m-1,0\ deep}$ (s)	$H_{mo\ toe}$ (m)	$T_{p\ toe}$ (s)	$T_{m\ toe}$ (s)	$T_{m-1,0\ toe}$ (s)
28	173	10, 30, 100	Meas	Calc	Meas	Calc	Calc	Calc	Calc	Calc
106	30	1000	Calc (equal to conditions at the toe)				Meas	Meas	Meas	Meas
107	56	1000	Calc (equal to conditions at the toe)				Meas	Meas	Meas	Meas
224	35	50	Meas	Meas	Calc	Calc	Meas	Meas	Calc	Calc
225	18	20	Meas	Meas	Calc	Calc	Calc	Calc	Calc	Calc
351	2	1000	Calc (equal to conditions at the toe)				Meas	Meas	Calc	Calc
402	32	1000	Meas	Calc	Meas	Calc	Meas	Calc	Meas	Calc
502	47	10,50	Meas	Meas	Meas	Meas	Meas	Meas	Meas	Meas
802	209	10, 30	Meas	Calc	Calc	Calc	Calc	Calc	Calc	Calc

In this table, $T_{m\ deep}$ and $T_{m\ toe}$ are the mean wave periods obtained either from spectral analysis or from time-domain analysis in deep water and at the toe of the structure, respectively, and $T_{m-1,0\ toe}$ is the wave period at the structure toe obtained from m_0 and m_{-1}.

Details of the methodologies used for calculating parameter values ("Calc") can be found in Van der Meer *et al*. (2009). As noted by these authors, if wave characteristics were only measured at the toe of the structure and not in deep water, then in the case of relatively deep water at the toe of the structure, it was assumed that the wave characteristics in deep water were the same as at the toe. In this case, it would also be reasonable to assume that the wave characteristics at any foreshore toe would be the same as the values at the toe of the structure and in deep water. Obviously, if there was no foreshore slope (denoted m = 1000 in the database), the wave conditions at the toe of the foreshore were the same as those at the toe of the structure. For the spectral wave period, $T_{m-1,0}$, a fixed relationship between T_p and $T_{m-1,0}$ was accepted for single-peaked spectra: $T_{m-1,0} = T_p/1.1$. It was also assumed that $T_p = 1.2T_m$.

In this study, it was assumed that the wave conditions at the toe of the foreshore are not very different from those in deep water. Wave transformation between deep water and the structure toe was checked for breaking and shoaling: i) if $h/H_{mo\ deep} > 3$, no significant breaking is expected (Hedges & Reis, 2004); and ii) if $h/L_{op\ deep} > 0.05$, no substantial changes in the significant wave height are anticipated due to shoaling (Goda, 2000). If these conditions are met at the structure, then this will also be the case at the foreshore.

Table 4 presents the range of values for $h/H_{mo\ deep}$ and $h/L_{op\ deep}$ for the 9 datasets considered. Since both Owen's and the Kansai calibration data were used with $h/H_s > 1.65$ in the original development of the H&R model (see Table 1), the condition $1.65 < h/H_{mo\ deep} < 3$ was also considered here. As a result, the data have been split into bands in this study, according to the following conditions:

- Condition I: $h/H_{mo\ deep} > 3$ and $h/L_{op\ deep} > 0.05$;
- Condition II: $1.65 < h/H_{mo\ deep} < 3$ and $h/L_{op\ deep} > 0.05$;
- Condition III: $h/H_{mo\ deep} > 3$ and $h/L_{op\ deep} < 0.05$;
- Condition IV: $1.65 < h/H_{mo\ deep} < 3$ and $h/L_{op\ deep} < 0.05$.

Table 4. Range of values for $h/H_{mo\ deep}$ and for $h/L_{op\ deep}$ for the extracted CLASH datasets.

Dataset		028	106	107	224	225	351	402	502	802
$h/H_{mo\ deep}$ (-)	Min	0.357	2.703	2.490	1.005	1.006	2.482	7.692	0.809	-0.752
	Max	3.725	14.352	17.021	3.023	3.103	3.266	23.333	3.285	2.941
$h/L_{op\ deep}$ (-)	Min	0.005	0.107	0.018	0.015	0.019	0.053	0.324	0.021	-0.020
	Max	0.087	0.434	0.462	0.124	0.123	0.085	0.778	0.145	0.046

4. Performance of the Current H&R Model for Vertical Walls

Figures 2 to 7 compare the values of measured mean overtopping discharges provided in the CLASH database for the 9 datasets considered, q_{CLASH}, with the values estimated by the H&R model, $q_{H\&R}$, by extrapolating the formulae for A and B (Equations (8) and (9)) outside their strict ranges of applicability, taking $\cot\alpha = 0$ (for a vertical structure). Note that $q_{H\&R(37\%,100)}$ implies the use of Equations (5) and (8) with Equation (1), whilst $q_{H\&R(99\%,100)}$ implies the use of Equations (6) and (9). The figures on the left, with logarithmic scales, show most of the data but cannot display zero overtopping rates. These are presented in the right-hand figures, using linear scales. The 9 different datasets are colour-coded for identification.

The figures for Condition I show that the H&R model with values of A and B extrapolated to $\cot\alpha = 0$ underestimates the higher overtopping rates by a factor of about 4; but in the range of very small degrees of overtopping, the extended model is reasonably accurate. The model using $(R_{max})_{37\%,100}$ is marginally better than that employing $(R_{max})_{99\%,100}$.

For Condition II, the under-prediction at higher overtopping rates is less than that for Condition I but the scatter is generally greater.

There are few results for Condition III (not shown here) and those for Condition 4 are generally unsatisfactory, an unsurprising outcome given the fact that the values of $h/L_{op\ deep}$ imply the possibility of significant changes in the wave conditions due to shoaling between the deep water values assumed and those at the toe of the foreshore.

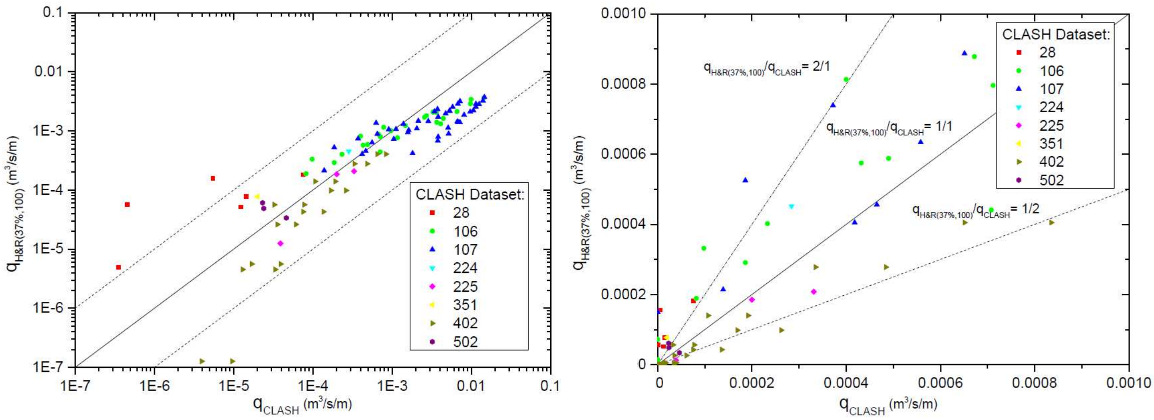

Figure 2. Relationship between q_{CLASH} and $q_{H\&R(37\%,100)}$ for $h/H_{mo\ deep} > 3$ and $h/L_{op\ deep} > 0.05$ (Condition I).

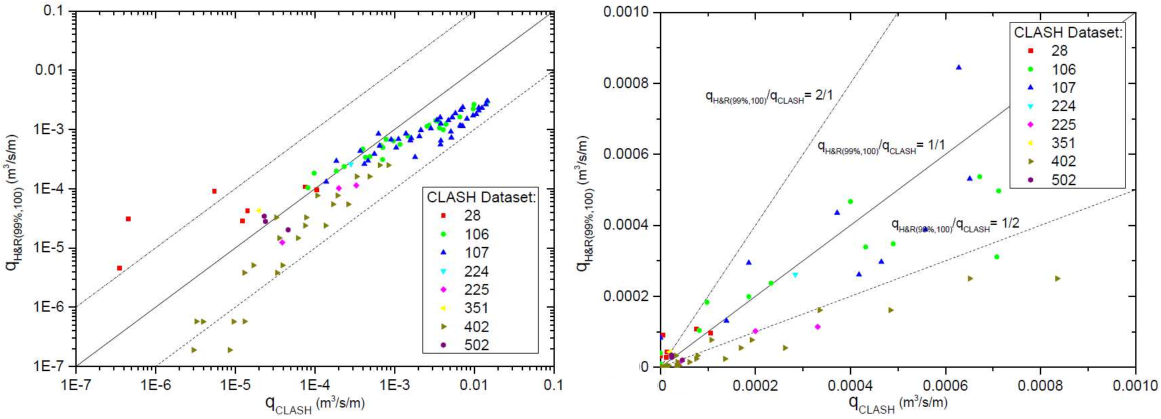

Figure 3. Relationship between q_{CLASH} and $q_{H\&R(99\%,100)}$ for $h/H_{mo\ deep} > 3$ and $h/L_{op\ deep} > 0.05$ (Condition I).

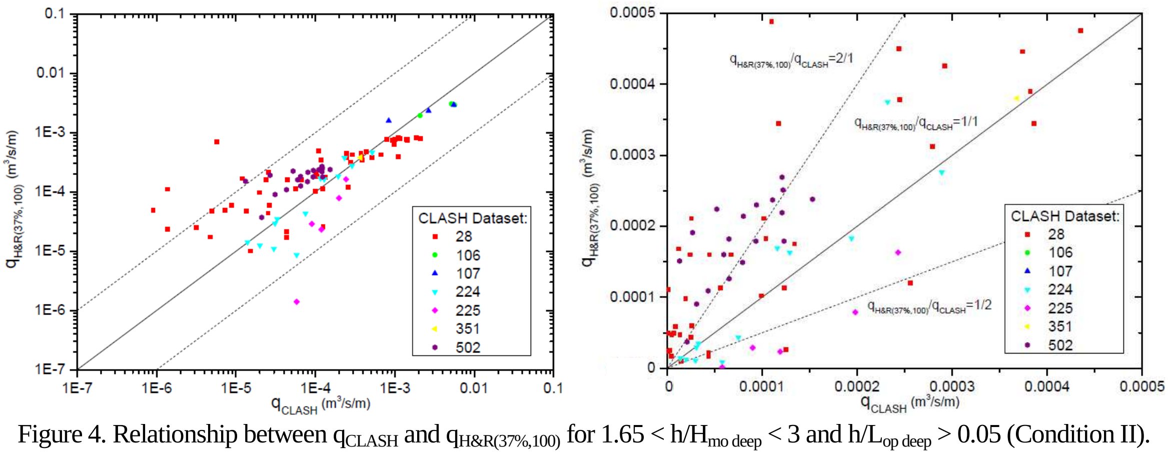

Figure 4. Relationship between q_{CLASH} and $q_{H\&R(37\%,100)}$ for $1.65 < h/H_{mo\ deep} < 3$ and $h/L_{op\ deep} > 0.05$ (Condition II).

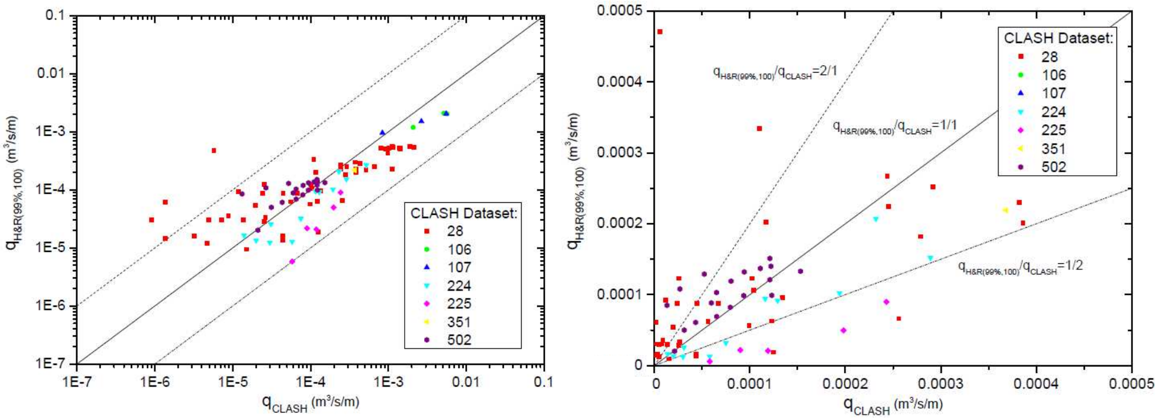

Figure 5. Relationship between q_{CLASH} and $q_{H\&R(99\%,100)}$ for $1.65 < h/H_{mo\ deep} < 3$ and $h/L_{op\ deep} > 0.05$ (Condition II).

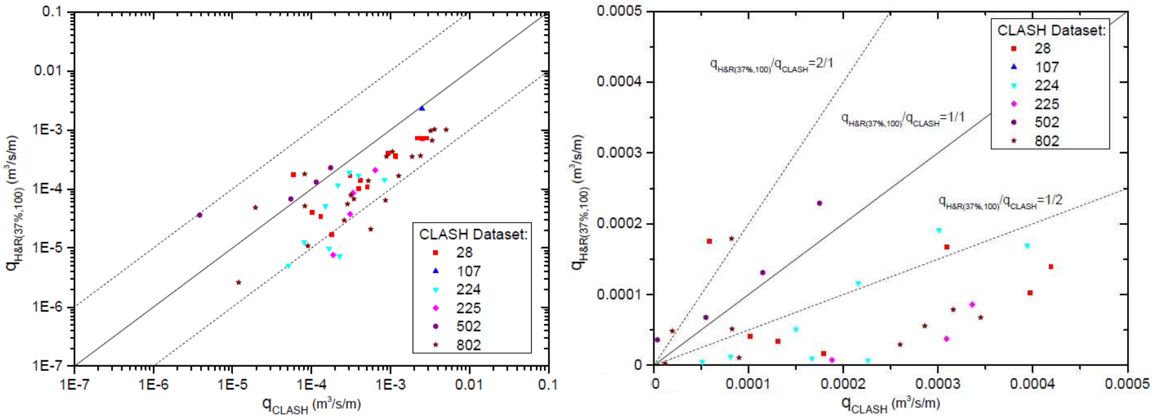

Figure 6. Relationship between q_{CLASH} and $q_{H\&R(37\%,100)}$ for $1.65 < h/H_{mo\ deep} < 3$ and $h/L_{op\ deep} < 0.05$ (Condition IV).

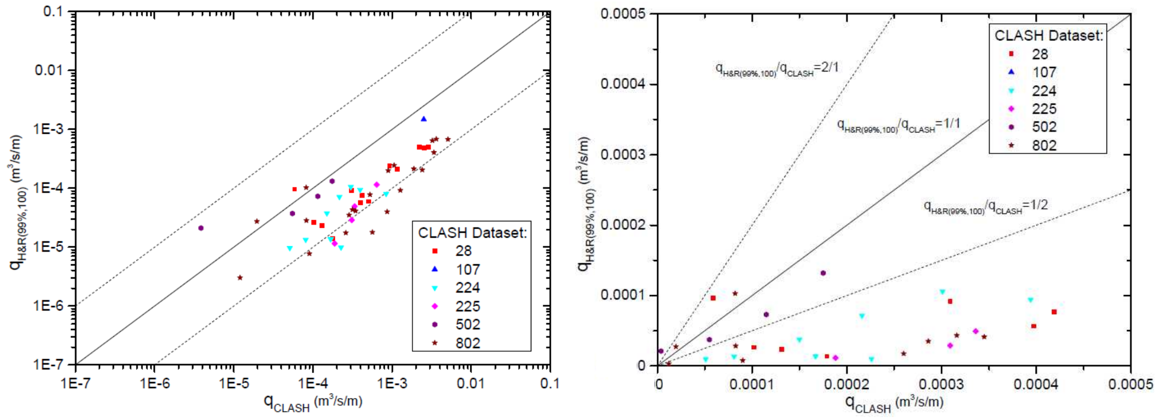

Figure 7. Relationship between q_{CLASH} and $q_{H\&R(99\%,100)}$ for $1.65 < h/H_{mo\ deep} < 3$ and $h/L_{op\ deep} < 0.05$ (Condition IV).

5. Conclusions

The H&R wave overtopping model, originally developed for sloping structures, is employed in a number of operational coastal flood forecasting and warning systems. This paper describes the first step in extending the H&R wave overtopping model to vertical structures. For this purpose, use was made of the CLASH database for simple, smooth, impermeable, vertical walls acted on by waves approaching normally. The main aim was to assess the validity of extending the H&R model by simply extrapolating the values of its empirical coefficients outside the range for which they had previously been established.

In general, the model tended to underestimate the higher overtopping rates. However, it was reasonably accurate in predicting the lower overtopping rates, suggesting that it may be worthwhile pursuing the task of calibrating and validating the model for vertical structures, as it is the lower overtopping rates which normally trigger flood warnings.

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