OVERTOPPING FLOW CHARACTERISTICS AT EMERGED AND OVER-WASHED DIKES

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In the design of coastal defenses and in the estimate of their vulnerability a key aspect is the realistic prediction of the characteristics of the overtopping waves. In fact hydrodynamic forces on landward-side slopes largely depend on the distribution of instantaneous overtopping wave volumes, flow thicknesses and flow velocities. For emerged conditions, flow depths and velocities over the dike crest can be described by existing theoretical models, while for zero freeboard and submerged conditions there are some indications regarding the discharge and the flow characteristics in the literature but not systematically verified. Scope of this contribution is the characterization of the flow over smooth dike crests in case of frequently overtopped and over-washed based on a numerical database of more than 80 tests obtained with the Rans-Vof code (IH-2VOF) developed by the University of Cantabria on a smooth structure varying wave attacks, crest freeboard, seaward and landward slopes.

Keywords: overtopping, overflow, flow depth, flow velocity, smooth dike, numerical modelling

INTRODUCTION

Different types of coastal structures, as earthen levees and dikes, are used throughout the world to protect populations and infrastructure from periodic floods and high water due to storm surges. Ideally, all levees and dikes would have a crown elevation with ample freeboard to prevent flood or storm surge overflow and/or wave overtopping for any conceivable storm scenario. However, economic constraints and environmental and aesthetic impacts often impose more practical levee designs having lower crown elevations with the associated risk that some wave/surge overtopping will occur during extreme events. In addition, the increase of frequency and intensity of storms, combined with the uncertainties related to extreme events and climate change, pose serious challenges in the long term design of defenses from coastal flooding. Therefore it is more likely that in the next future many dikes will operate for longer times at lower crest freeboards, i.e. close to mean sea level or even over-washed. For design purposes, accurate estimates of the statistics of overtopping waves in terms of flow depths, duration and especially velocities for a set of climate conditions are needed and have to be combined with consolidated criteria for identifying tolerable overtopping threshold.

Empirical equations describing wave overtopping processes in terms of incident wave conditions, structure geometry and crest freeboard have been developed based on small- and large-scale physical model tests of common structure geometries. In particular, in literature it is possible to find some theories that describe flow depths and velocities over the dike crest for emerged conditions (Schuttrumpf and Van Gent, 2003; Schuttrumpf and Oumeraci, 2005; Bosman et al. 2008), whereas for zero freeboard and submerged conditions a commonly adopted theoretical approach is not yet available. The information available so far for over-washed dikes can be derived from the (limited) set of tests performed by Hughes and Nadal (2009) and Hughes et al. (2012).

Full-scale testing of dike landward slopes by means of the Wave Overtopping Simulator (WOS) significantly extended the knowledge of landward-side dike resilience (Van der Meer et al., 2009, 2010). However, the WOS requires an accurate estimate of the wave overtopping volumes and related statistics to allow in cascade an accurate reproduction of flow depths and velocities on landward slopes.

To overcome the scale effects intrinsic to physical model tests and the costs of full-scale testing, several numerical modeling approaches have been developed to study wave structure interaction. Among other existing approaches, Nonlinear Shallow Water (NSW), Boussinesq-type or Navier-Stokes (NS) equations models have traditionally been used. Although good results were obtained using NSW equations in terms of averaged magnitudes (Kobayashi et al., 2010), though the energy transfer to higher frequencies occurring before wave breaking cannot be accurately reproduced due to the lack of dispersion. Boussinesq-type models are able to include frequency dispersion, a depth-dependent velocity profile, and they can be applied to both breaking and non-breaking wave conditions (Kirby, 2003). However, this type of models requires setting both the triggering wave breaking mechanism and the subsequent wave energy dissipation due to wave breaking. Moreover, they fail to reproduce the

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strong non-linear shoaling process occurring prior to wave breaking and the higher-order statistics of free-surface elevations and velocities which are thought to be relevant for structure stability.

Models based on 2D Eulerian NS set of equations (Losada et al., 2008; Lara et al., 2008; Guanche et al., 2009; Lara et al., 2011) proved to powerfully address wave-induced processes, providing the user with very refined information on the velocity, pressure and turbulence fields and therefore constituting –thanks to the improved computational capacities- a complementary tool in the design process.

The goal of this paper is to systematically analyze the overtopping process, and specifically the flow depths and velocities over the structure crest, in order to extend the existing theoretical and empirical approaches to low crested and over-washed structures. This analysis was carried out on the basis of a new numerical database derived by running the Rans-Vof code (IH-2VOF) developed by the University of Cantabria in presence of smooth dikes characterized by different slopes and freeboards.

The paper starts with a short description of the model and validation of its accuracy against similar tests carried out in the laboratory on over-washed dikes. Then, the new numerical database is presented and the key parameters are derived from the simulations to verify the new numerical results against experimental data already available in the literature. The evolution of the flow depth and velocity over the dike crest are subsequently investigated on varying of the dike submergence, the wave height and of the sea-ward slope, by keeping constant the other parameters. The numerical results are compared to the existing theoretical approach by Schuttrump and Oumeraci (2005). The values of the overtopping discharge derived from the numerical results are then compared with some of the available formulae representing overtopping and overflow (EurOtop, 2007; Hughes and Nadal, 2009; Van der Meer et al., 2013). Some conclusions on the relevance of the investigated parameters on the flow characteristics and on existing theoretical approaches are finally drawn.

THE NUMERICAL MODEL

The model adopted within this work is the IH-2VOF code, developed by the University of Cantabria (Lara et al., 2011) on the basis of previous versions of the code itself (COBRAS, Liu and Lin, 1997; RIPPLE, Kothe et al., 1991), updated and validated against systematic tests on low-crested structures (Garcia et al., 2004), wave breaking on permeable slopes (Lara et al., 2006), surf zone hydrodynamics on natural beaches (Torres-Freyermuth et al., 2007) and wave overtopping (Losada et al., 2008; Lara et al., 2008).

The code solves the 2DV Reynolds Average Navier–Stokes (RANS) equations, based on the decomposition of the instantaneous velocity and pressure fields, into average and turbulent components. Two extra governing equations for modeling the turbulent kinetic energy (k) and the turbulent dissipation rate (ε) replace simplified closure conditions in case of highly turbulent flow. The Volume of Fluid (VOF) technique is adopted to track the free surface motion.

The Volume-Averaged Reynolds Averaged Navier–Stokes (VARANS) equations, obtained by integrating the RANS equations over a control volume, can be included to represent the flow inside the porous media, where the interfacial forces between fluid and solid phases are modeled through the extended Forchheimer relationship. For the complete mathematical formulation see Liu et al. (1999) and Hsu et al. (2002).

The model accuracy in representing the overtopping process resulting from combined storm surge overflow and wave overtopping effects was tested by Raosa et al. (2012), by comparing the model results with the tests carried out by Hughes and Nadal (2009). In these tests a smooth dike was studied in 1:25 scale in the 0.91-m-wide wave flume at the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL) in Vicksburg, MS.

THE NEW NUMERICAL ANALYSIS

The new numerical database

A series of simulations was performed in 1:10 scale in a numerical flume 52.3 m long and 1.5 m deep. The irregular waves attacks are characterised by different significant wave heights H_s and peak period T_p in the ranges 1÷2 m and 1.74÷4.36 s, respectively. For each wave attack the wave steepness remains constant and close to 2%.

The dimensions of the trapezoidal smooth structure (crest width and height) are kept constant while the seaward α_{off} and landward α_{in} slopes are variable, α_{off} = 1:4 – 1:6 and α_{in} = 1:2 – 1:3. Four different freeboards are considered, ranging from submerged to emerged conditions, by varying the still water depth h from 0.6 m to 1 m (see Fig. 1).

A total of 82 tests resulting from the combination of different structure geometries and wave attacks (synthetized in Tab. 1) were carried out. During the tests, 34 wave gauges were installed on the structure to measure free surface elevations and velocities.

Table 1. Synthesis of the tested conditions for wave attacks and structure geometry

	H_s [m]	T_p [s]	h [m]	R_c/H_s [-]	$cot\alpha_{off}$ [-]	$cot\alpha_{in}$ [-]
min	0.1	1.74	0.6	-1.5	4	3
max	0.2	4.36	1	+0.5	6	4

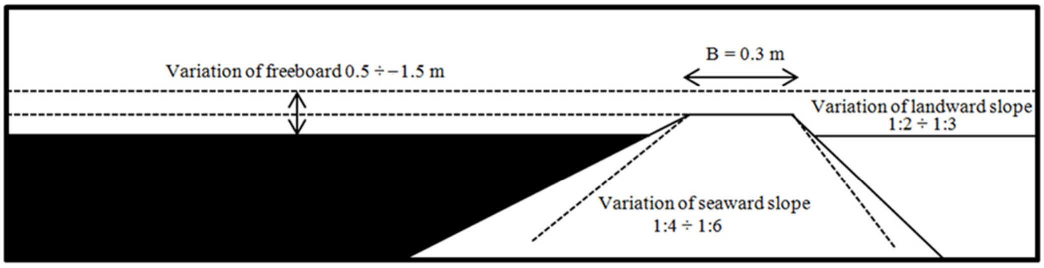


Figure 1. Tested levee cross section (model-scale units).

Verification of the model results

Besides the analysis already performed by Raosa et al. (2012), the wave reflection coefficient K_r is here used to verify the model representation of the hydraulic response of the structure, being the wave reflection process strictly correlated to the wave overtopping, the greater the wave reflection the lower the wave overtopping discharge.

The numerical values of K_r are compared with the experimental data for smooth straight slopes derived from the database by Zanuttigh and Van der Meer (2006). Figure 2 shows the numerical and experimental values of K_r as functions of the breaker parameter $\xi_{0,}$ for the tests characterized by structure slopes of $\cot\alpha_{off} = 3$ and $\cot\alpha_{off} = 4$. It is worthy to remark that the experimental data refer only to the cases of structures in “design conditions”, i.e. relative crest freeboard $R_c/H_s > 0.5$ and peak wave steepness s_{op}>1%.

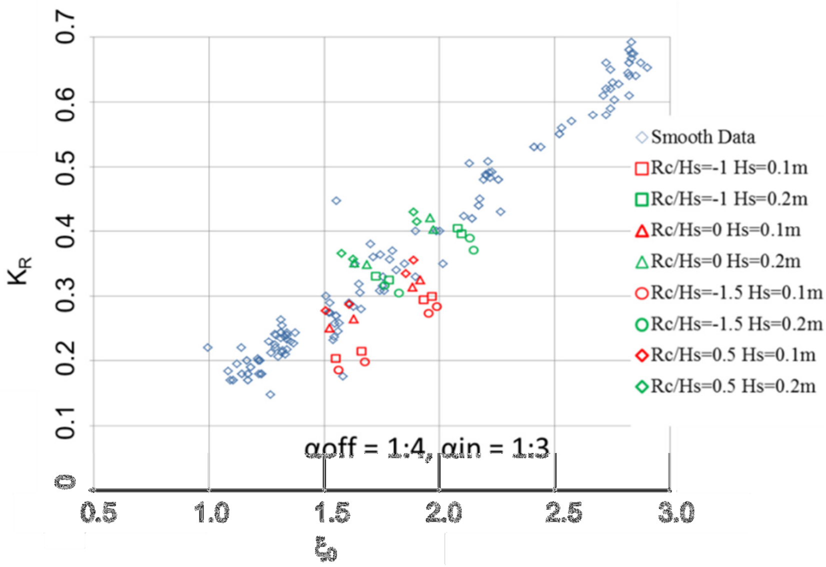


Figure 2. Values of K_r derived from the experimental database for smooth straight slopes (Zanuttigh and Van der Meer, 2006) and from the numerical simulation characterized by α_{off} = 1:4 and α_{in} = 1:3 and different values of crest freeboard.

It can be observed from Fig. 2 that the numerical values of K_r fall perfectly in the range of the experimental values for emerged and zero freeboard cases, while, as expected, submerged conditions produce lower values of K_r, being the points generally below the experimental data. Moreover, all the cases characterized by a lower value of the wave height (i.e. H_s = 0.1 m) induce less reflection, consistently with the lower energy of the wave attack. Then, the numerical results follow the trend of the experimental data in "design conditions" and respect the basic physical concepts, i.e. K_r decreases with increasing H_s and R_c.

ANALYSIS OF FLOW EVOLUTION OVER THE DIKE CREST

The existing semi-empirical approach

Based on physical model investigations and theoretical analysis, Schüttrumpf (2001) developed a simple model to represent the processes of wave interaction with dike slopes. This model was then further refined and tested by Van Gent (2002), Schüttrumpf and Van Gent (2003), Schüttrumpf and Oumeraci (2005), Bosman et al. (2008) to incorporate effects of structure geometries such as the off-shore dike slope.

The flow depth over the dike crest due to frictional losses and consequent overtopping volume deformation is described by the following exponential function:

$$\frac{h_c(x_c)}{h_c(x_c=0)} = exp\left(-\frac{c_3 x_c}{x_B}\right) \tag{1}$$

where x is the horizontal coordinate along the dike crest starting from its off-shore edge (where x_c=0); h_c is the overtopping flow depth on the dike crest at the coordinate x_c; c_3 is the dimensionless coefficient equal to 0.89 for TMA spectra (σ'=0.06) and to 1.11 for natural wave spectra (σ'=0.09) as reported in EurOtop (2007).

The overtopping flow velocities on the dike crest were analysed by using the simplified Navier-Stokes-equations and the following assumptions: i) the dike crest is horizontal; ii) velocities vertical to the dike slope can be neglected; iii) the pressure term is almost constant over the dike crest; iv) the viscous effects along the flow direction are small; v) the bottom friction is constant over the dike crest. The approach was verified by small and large scale model tests (EurOtop, 2007) and led to the function:

$$\frac{v_c(x_c)}{v_c(x_c=0)} = exp\left(-\frac{x_c}{2}\cdot\frac{f}{h_c}\right) \tag{2}$$

where v_c is the overtopping flow velocity on the dike crest and f is the friction coefficient, which was assumed equal to 0.01 for straight smooth slopes base on model tests. Therefore based on Eq. (2) it is clear that the overtopping flow velocity decreases over the dike crest for increasing surface roughness.

It is worthy to remark that EurOtop (2007) states clearly that "for flow depths larger than about 0.1 m and dike crest widths around 2–3 m, the flow depth and velocity hardly change over the crest." Since these new numerical tests are performed on a 3 m wide smooth structure under H_s=1-2 m at prototype scale we should find decreasing trends for zero freeboard and emerged cases but negligible changes in all tested submerged conditions.

Flow characteristics at the dike off-shore and in-shore edge

Based on the approach outlined above, specific formulae have been developed and checked against experimental data for emerged structures for flow depth and velocity at the off-shore (x=0) edge by Schuttrumpf (2001, 2005) and Van Gent (2002). The authors proposed the same formulae but with different fitting coefficients for depths $c_{A,h}$ and velocities $c_{A,u}$ based on tests carried out on 1:4 and 1:6 off-shore slopes

$$h_{2\%}(x_c=0)=c_{A,h}(R_{u,2\%}-R_c) \tag{3}$$

$$u_{2\%}(x_c=0)=c_{A,u}[g(R_{u,2\%}-R_c)]^{0.5} \tag{4}$$

The coefficients were then synthesized by Bosman including the dependence on the structure offshore slope

$$c_h=0.01/\sin^2\alpha; \quad c_u=0.30/\sin\alpha \tag{5}$$

More recently, based on FlowDike tests in 3D conditions, Van der Meer et al. (2010) proposed to use the same eq.(3) also for the flow depth at the in-shore edge (x_c=B)

$$h_{2\%}(x_c=B)=0.13\cdot(R_{u,2\%}-R_c) \tag{6}$$

and for the off-shore edge flow velocity, eq. (4), proposed the updated coefficient

$$c_u = 0.35 \cdot \cot\alpha \quad (7)$$

Figures 3, 4 and 5 compare the experimental results for flow depths and velocities in case of emerged and zero freeboard structures with eq.s (3), (4) and (6) respectively. In case of flow velocities, both eq.s (4) and (5) are used.

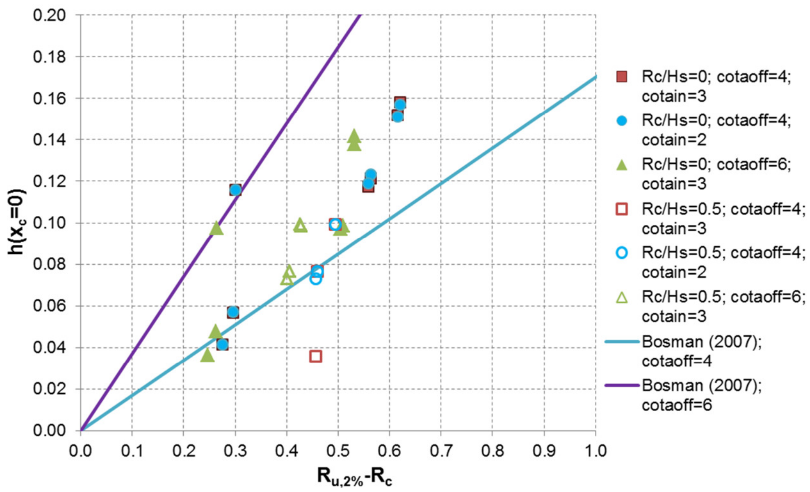


Figure 3. Flow depths at the dike off-shore edge versus the difference between the 2% wave run-up $R_{u,2\%}$ and the crest freeboard R_c. The values of the depths correspond to the 2% of the highest values.

At the off-shore edge the numerical flow depths fall well inside the expected range for the offshore slopes $\cot\alpha_{off}$ =4, 6. Flow velocities instead show much lower values than the predictions: this can be explained considering that the whole theoretical approach is essentially verified (i.e. calibration of the coefficients) based on wave celerity (measurements at the wave gauges over the crest) and not of what we would appropriately call flow velocities (average value of the velocities along the vertical above the structure crest to be measured with laser systems or acoustic profilers). Since the waves are progressive on the crest, the wave celerity is without no doubt greater than the flow velocity. As a confirmation of the different adopted approach, also the wave celerities at the off-shore edge of the dike have been computed and compared to the empirical formulae of Bosman (Eq. (5)), by grouping the values according to R_c and α_{off} (see Fig. 5). From Fig. 5 it is evident that the numerical celerities are better fitted than the flow velocities (Fig. 4), since almost all the data fall within the predictions.

Wave celerities were derived by means of the wave-by-wave procedure developed by Zanuttigh and Lamberti (2006) considering the numerical wave gauges based on the structure crest.

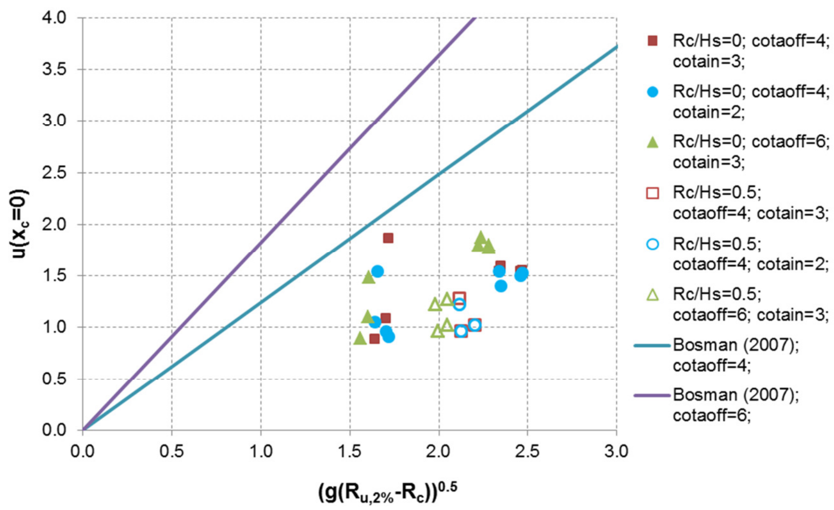


Figure 4. Flow velocities at the dike off-shore edge versus the difference between the 2% wave run-up $R_{u,2\%}$ and the crest freeboard R_c. The values of the velocities correspond to the 2% of the highest values.

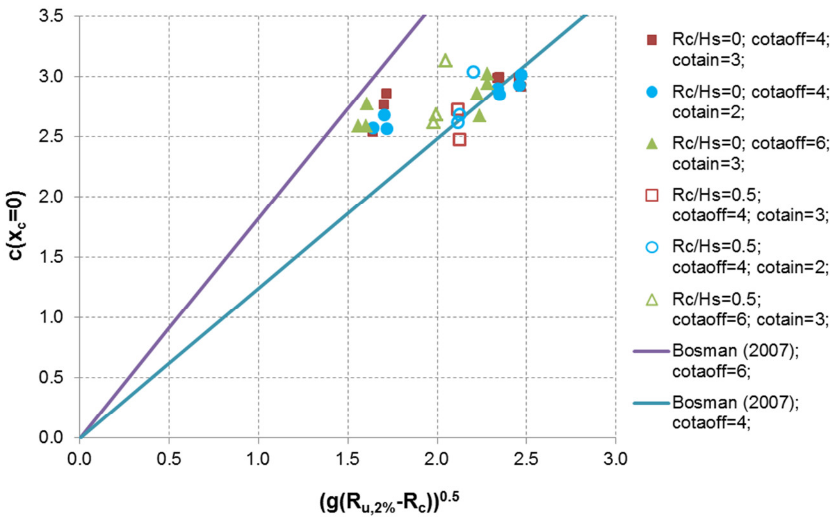


Figure 5. Wave celerities at the dike off-shore edge versus the difference between the 2% wave run-up R_u and the crest freeboard R_c. The values of the celerities correspond to the 2% of the highest values.

Overall, the milder the slope ($\cot\alpha_{off}$ = 6, represented in Fig.s 3, 4 and 5 by the triangles), the lower the wave run-up and the lower the velocities, celerities and flow depths, as indicated by Eq. (5). As expected, the inner slope ($\cot\alpha_{in}$) does not produce any significant effect. The decrease of the numerical values with R_c/H_s (void symbols) is more evident for flow depths than for flow velocities and celerities. In case of R_c/H_s=0 (filled-in symbols) the numerical values are clearly divided into two groups depending on the different values of H_s.

At the inshore edge (see Fig. 6) the prediction of flow depths is more accurate for R_c/H_s>0, while it tends to underestimate the values for R_c/H_s=0, as it is expected since the lower the R_c/H_s the greater the run-up while the coefficient in eq. (6) was calibrated for emerged cases only.

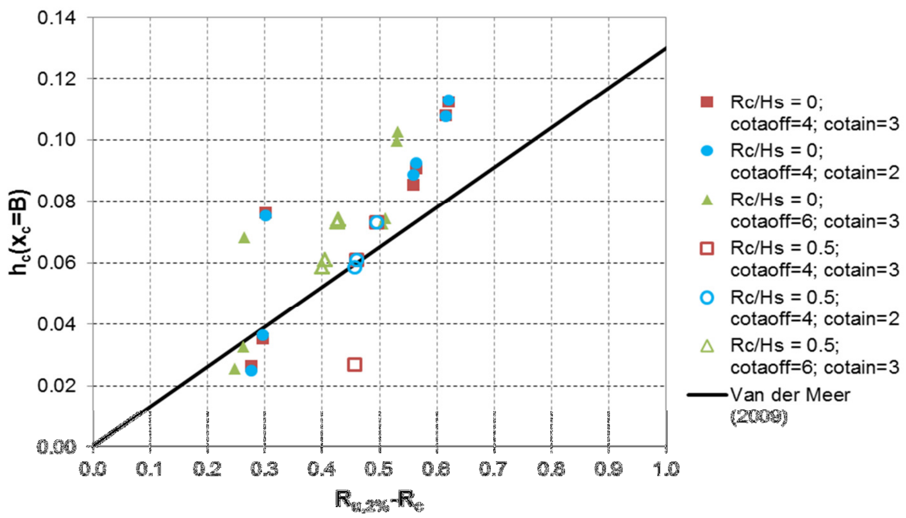
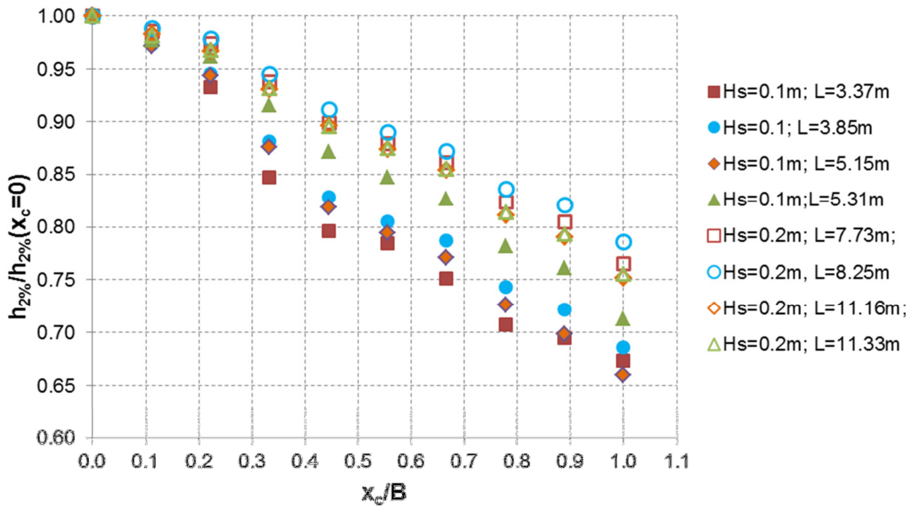


Figure 6. Flow depths at the dike in-shore edge versus the difference between the 2% wave run-up R_u and the crest freeboard R_c. The values of the depths correspond to the 2% of the highest values.

Trends of the flow characteristics over the dike crest

As a general remark, the flow depth over the dike crest tends to decrease as well as already predicted by the existing model described above while the flow velocity tends to increase instead (Fig. 7). This numerical result is indeed in agreement with the continuity equation and was actually already suggested by a note of the EurOtop (2007).



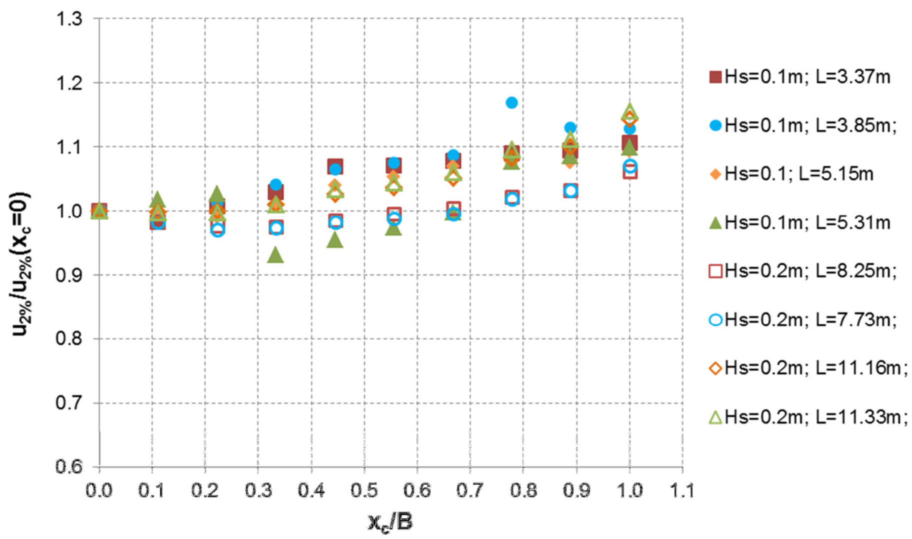


Figure 7. Flow depths (top) and velocities (bottom) along the dike crest for R_c=0 and cotα_{off}=4. The values of both depths and velocities are the 2% of the highest values at a given dike crest section and are non-dimensionalized with the value at the dike off-shore crest. All the tests with different wave height H_s=0.1 m and 0.2 m are included (respectively, filled-in and void symbols); of course the different wave height lead essentially to a scale effect, being all the other parameters constant.

It has to be noticed that Figure 7 includes all the tested waves for the same structure geometry and crest freeboard, i.e. H_s=0.1 m and H_s=0.2 m. Given that all the other parameters are kept constant the effect of H_s along the crest is essentially a scale effect, i.e. the higher the wave height the higher the flow depth and the velocity at the same cross-section over the dike crest.

The most significant effects on the trend of flow depths and velocities along the crest are synthetized in Fig.s. 8 and 9

- change of the degree of submergence by keeping constant the wave attacks and the structure geometry (see Fig.s 8 and 9). By increasing the submergence, the decay rate of the flow depth decrease till it completely disappears when R_c/H_S=-1.5. The velocity growth rate is indeed modest and tends to decrease with decreasing R_c/H_S, becoming the flow essentially a weir-like flow rather than breaking dominated;
- change of the structure seaward slope by keeping constant the wave attack (Fig. 9). The effect of the seaward slope is modest and appreciable only for R_c/H_S>0 as it could have been expected since it directly affects the wave run-up process. Hence in these cases the milder the seaward slope the milder is the flow depth decay. Irrespectively of the submergence, the seaward slope does not significantly affect the evolution of the flow velocity.

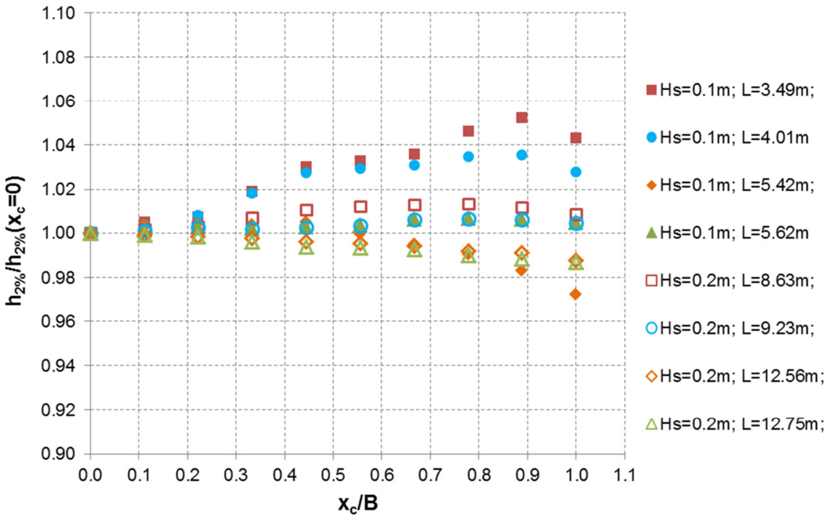


Figure 8. Flow depths along the dike crest for R_c/H_s=-1.0, cotα$_{off}$=4; the filled-in and the void symbols correspond respectively to H_s=0.1 m and H_s=0.2 m. The depth values are the 2% of the highest values at a given dike crest section and are non-dimensionalised with the depth value at the dike off-shore edge.

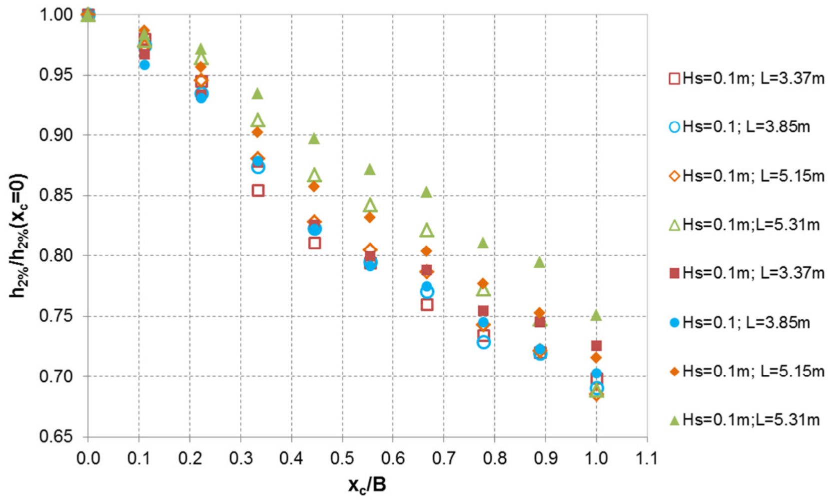


Figure 9. Flow depths along the dike crest for R_c/H_s=0, cotα$_{in}$=3, H_s=0.1 m. The filled-in and the void symbols correspond respectively to cotα$_{off}$=4 and cotα$_{off}$=6. The depth values are the 2% of the highest values at a given dike crest section and are non-dimensionalised with the depth value at the dike off-shore crest.

Fitting the numerical trends

The trends of the numerical flow heights and velocities over the dike crest have been compared to the semi-empirical formulation proposed respectively by Eq.s (1) and (2). Based on the very similar trends observed for the 7 groups of tests with same R_c/H_s and H_s, common average values of the coefficients c_3 and f were computed for each group. Table 3 reports the values of c_3 and f and the corresponding average values of the standard deviation (σ) and the coefficient of determination (R^2):

$$\sigma=\frac{1}{N_t}\sum_{k=1}^{N_t}\left(\sqrt{\frac{\sum_{i=(x_c=0)}^{x_c=B}(\widehat{y}_i-y_i)^2}{N_x}}\right)_k \qquad (8)$$

$$R^2=\frac{1}{N_t}\sum_{k=1}^{N_t}\left(1-\frac{\sum_{i=(x_c=0)}^{x_c=B}(\widehat{y}_i-y_i)^2}{\sum_{i=(x_c=0)}^{x_c=B}(\widehat{y}_i-\bar{y})^2}\right)_k, \qquad (9)$$

where $\widehat{y}_i$ is the numerical value of the flow depth at the abscissa $x_c = i$, i ranging from x_c=0 (off-shore edge) to x_c=B (inshore edge), y_i is the corresponding value derived from eq. (1), $\bar{y}$ is the mean of y_i –

values, N_x is the number of points on the crest dike among x_c=0 and x_c=B (corresponding to the number of wave gauges situated on the crest, i.e. 10 in this case) and N_t is the number of tests grouped and fitted by the same value of c_3. The two error indexes are both adopted to provide a quantitative evaluation of the agreement among exponential fittings and numerical results, but they are supposed to display two different aspects. R^2 is a normalized index ranging from 0 to 1, being 1 the perfect correspondence, and therefore it is not affected by the values of the data to fit, while σ represents the dispersion of the numerical values around the fitting and its value consequently depends by the magnitude of the data.

The numerical results show that as well as in the semi-empirical formulation, the flow depth over the crest tends to exponentially decrease (see, e.g., Fig. 10, top, which reports the values of the flow depths for R_c/H_s = 0). The higher the wave height the lower the decay due to scale effects (since B is constant), as it can be appreciated from Fig. 9 (top), by comparing the two average trends for H_s=0.1 m (continuous line) and H_s =0.2 m (dashed line). For submerged conditions –in agreement with EurOtop (2007)- the decay becomes negligible (see Tab. 3, cases of R_c/H_s = -1 and -1.5), and the trend seems to invert, as it is derived from the negative values of the c_3 coefficients derived for the cases of modest wave heights (H_s =0.1 m). It is worthy to note that of course eq. (1) in this case is applied outside its range of validity (emerged structures only).

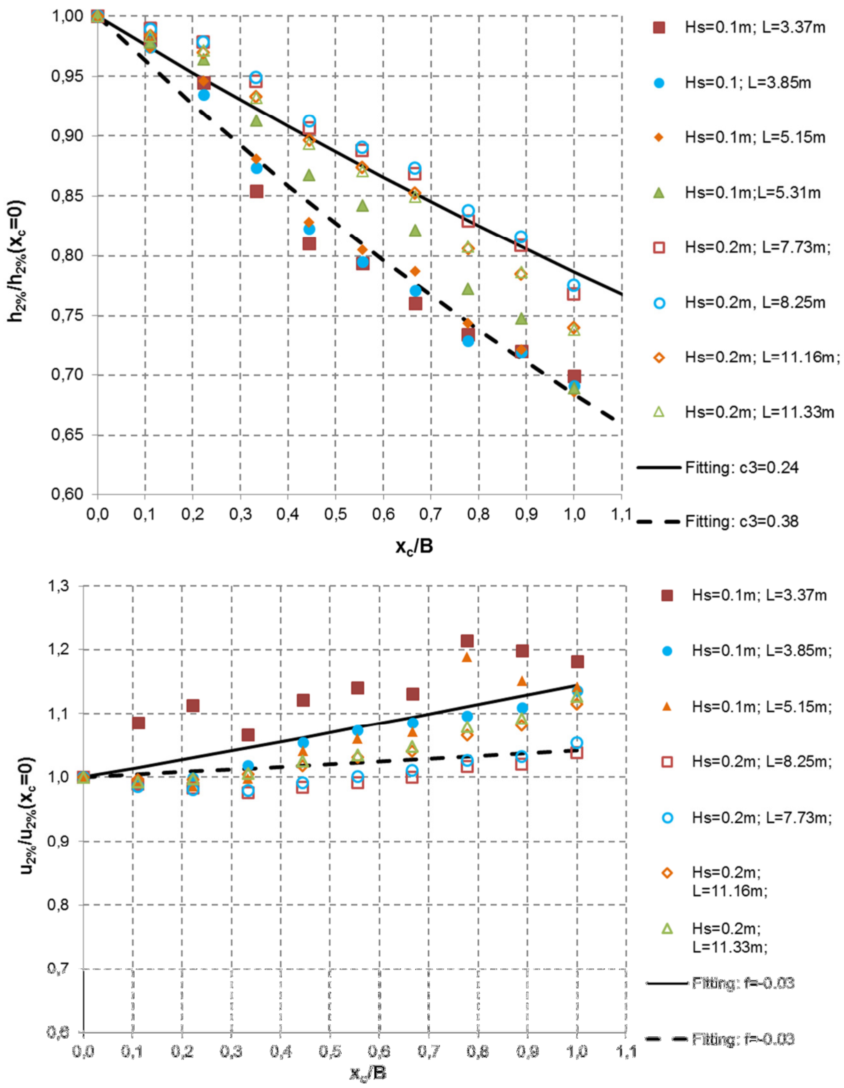


Figure 10. Flow depths (top) and velocities (bottom) along the dike crest for R_c/H_s =0 and $cot\alpha_{off}$=4. The values of both depths and velocities are the 2% of the highest values at a given dike crest section and are non-dimensionalized with the values at the dike off-shore edge. In both figures all the tests with different wave height H_s=0.1 m and 0.2 m are included (respectively, filled-in and void symbols). The two fitting curves represent the results obtained by fitting the data for H_s=0.1 m (solid line) and 0.2 m (dashed line) with the eq. (1) for the flow depths and with eq. (2) for flow velocity.

As for the flow depths, also the flow velocities are found to be dependent on the submergence and on the wave height, but they tend to slightly increase over the dike crest (see, e.g., Fig. 10, bottom, which reports the values of the velocities for for R_c/H_s = 0). This trend can still be approximated with an exponential function like eq. (2), but all the coefficients for f are found to be negative (see Tab. 3) with the exception of the emerged tests (R_c/H_s =0.5). Being f a physical coefficient representing a friction factor, the negative values shown in Tab. 3 are supposed to be considered as absolute values and the negative sign to substitute the exponential decay proposed by eq. (2) with an exponential increase.

Table 3. Synthesis of the coefficients for the exponential fitting decay of the flow depth (c_3) and velocity (f) over the dike crest, respectly referring to eq. (1) and eq. (2).

Depths	R_c/H_s=-1.5			R_c/H_s=-1			R_c/H_s=0			R_c/H_s=0.5		
	c_3	σ	R^2	c_3	σ	R^2	c_3	σ	R^2	c_3	σ	R^2
H_s=0.1m	-0.035	0.02	0.97	-0.035	0.008	0.69	0.38	0.05	0.98	0.34	0.03	0.98
H_s=0.2m	0.005	0.01	0.89	0.005	0.13	0.84	0.24	0.12	0.99	-	-	-
Velocities	R_c/H_s=-1.5			R_c/H_s=-1			R_c/H_s=0			R_c/H_s=0.5		
	f	σ	R^2	f	σ	R^2	f	σ	R^2	f	σ	R^2
H_s=0.1m	-0.15	0.03	0.89	-0.10	0.05	0.72	-0.03	0.12	0.85	0.1	0.22	0.84
H_s=0.2m	-0.20	0.04	0.99	0.10	0.10	0.96	-0.03	0.03	0.83	-	-	-

ANALYSIS OF THE OVERTOPPING DISCHARGE

While consolidated formulae do exist for wave overtopping in case of emerged dikes and rare overtopping (EurOtop, 2007; Van der Meer et al., 2013), the overtopping discharge in case of low-crested breakwaters and over-washed dikes is still a subject of research.

For the case of steady water overflow over a dike due to a storm surge higher than the crest freeboard, a subcritical flow is established a the off-shore side of the dike that then becomes critical somewhere along the crest if this is sufficiently wide to ensure that the pressure distribution is still hydrostatic. The flow along the inshore slope is then super-critical in almost all cases due to the slope steepness. The discharge per unit length can be then calculated as for a broad-crested weir, assuming the friction losses to be negligible.

In presence of storm surge and waves the discharge is then given both by the effects of the storm surge $q_{overflow}$ and on the incoming waves $q_{overtop}$. In this case it is indeed rather difficult to distinguish and accurately reproduce by means of theoretical approaches the contribution of wave overtopping and weir-like flow over the structure crest, also because of the discharge flowing back off-shore over the crests (Zanuttigh et al., 2008) in deeply submerged cases.

EurOtop (2007) proposed as first approximation the following equations

$$q=q_{overflow}+q_{overtop} \tag{10}$$

where

$$q_{overflow}=0.6\sqrt{g\cdot|-R_c^3|} \tag{11}$$

and

$$\begin{cases} q_{overtop}=0.0537\cdot\xi_{m-1,0}\cdot\sqrt{gH_{m0}^3}\ ,\ for\ \xi_{m-1,0}<2 & (12a) \\ q_{overtop}=\left(0.136-\frac{0.226}{\xi_{m-1,0}^3}\right)\cdot\sqrt{gH_{m0}^3},\ for\ \xi_{m-1,0}\geq 2 & (12b) \end{cases}$$

Equation (10) is considered to be a rough approximation of the total discharge q. Equations (12a,b) are provided for zero freeboard conditions only.

More recently Van der Meer et al. (2013) proposed to modify the formula for the overtopping discharge $q_{overtop}$ at emerged structures to better approximate also the zero freeboard conditions

$$q_{overtop}=\frac{0.026}{\sqrt{\tan\alpha}}\cdot\xi_{m-1,0}\cdot exp\left(-\left(2.7\frac{0.226}{\xi_{m-1,0}^3\cdot H_{m0}\cdot\gamma_b\cdot\gamma_f\gamma_v\cdot\gamma_\beta}\right)^{1.3}\right)\cdot\sqrt{gH_{m0}^3} \tag{13a}$$

with a maximum of

$$q_{overtop}=0.09\cdot exp\left(-\left(1.5\frac{R_c}{\xi_{m-1,0}^3\cdot H_{m0}\cdot\gamma_f\gamma_\beta}\right)^{1.3}\right)\cdot\sqrt{gH_{m0}^3} \tag{13b}$$

Therefore Eq.s (13a,b) should replace Eq.s (12a,b).

Hughes and Nadal (2009) also proposed to split and sum the contributions and adopted the following equation

$$q=0.0336\cdot\sqrt{gH_{m0}^{3}}+0.973\cdot q_{overflow}\cdot\left(-\frac{R_c}{H_{m0}}\right)^{0.083} \tag{14a}$$

with

$$q_{overflow}=0.5443\cdot\sqrt{g}\cdot|R_c|^{\frac{1}{3}} \tag{14b}$$

Equations (12a,b) were developed for R_c<0 and fitted against the experimental dataset of the authors. It is however worthy to remark that the tested conditions were very close to zero freeboard.

Figures 11 and 12 show the relative errors $(q-q_{num})/q$ derived by comparing the numerical results and the formulae: EurOtop (2007), i.e. eq.s (10), (11) and (12a, b); Hughes and Nadal (2009), i.e. eq.s (14a) and (14b); Van der Meer et al. (2013), i.e. eq. (13a,b).

In Figure 11 the comparison is carried out considering the whole set of data against EurOtop (2007) as an example. It can be appreciated that the values of $(q-q_{num})/q$ increase with decreasing R_c/H_S, as it is expected considering that the empirical overflow is merely represented through the value of the crest freeboard and it does not account for the return flow over the crest (Zanuttigh et al., 2008) and therefore tends to a systematic overestimation. For $R_c/H_S \geq 0$ the better approximation of the numerical results both in terms of values and scatter is obtained with the formula by Van der Meer et al. (2013), see Fig. 12 (void symbols). The error for R_c/H_S <0 does not improve if the formula by Hughes and Nadal (2009), Fig. 12 (filled-in symbols), is used instead of EurOtop (2007).

Similarly to the flow characteristics at the dike off-shore edge, also the computation of the overtopping discharge has been carried out with both velocities and celerities. In this latter case, the discharge is calculated as the sum of the wave-by-wave volumes per unit of dike length divided by the duration of the simulation, which is based on approximately 300 waves analysis. The volumes are given by the wave-by-wave products of wave celerities and overtopping flow depths, which are derived from two numerical gauges placed over the crest.

Fig.s 13 and 14, which respectively correspond to Fig.s 11 and 12, show the numerical results of q_{num} derived from the wave celerities. The error for R_c/H_S <0 sensibly decreases for both the EurOtop formula (Fig. 13) and the Hughes and Nadal formula (Fig. 14). The EurOtop one generally shows a lower underestimation of q: the improved representation of the overtopping process can be probably explained by the adoption of the breaking parameter $\xi_{m-1,0}$ (see Eq.s (12a,b)), accounting also for the wave period, a key-parameter which is instead not considered by Hughes and Nadal.

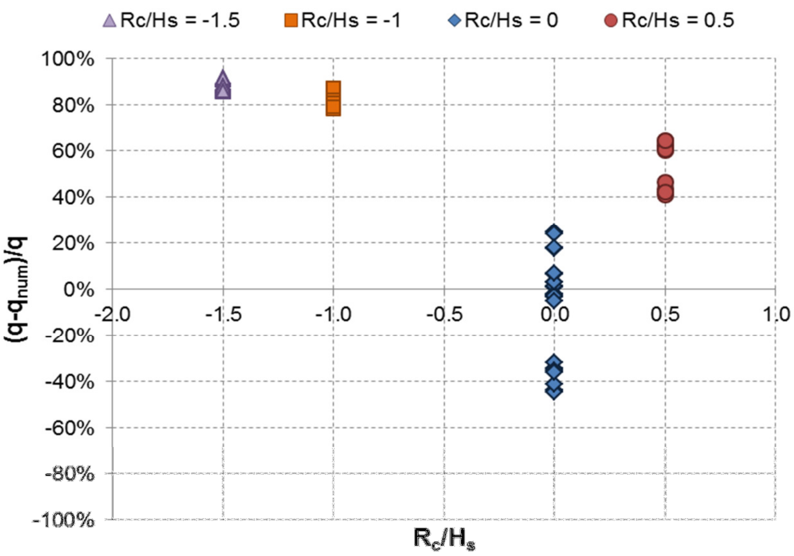


Figure 11. Discharge over the structure crest predicted by the EurOtop (2007), *q*, and obtained by the numerical results of the flow velocities, q_{num}. The relative error is shown for all tested conditions.

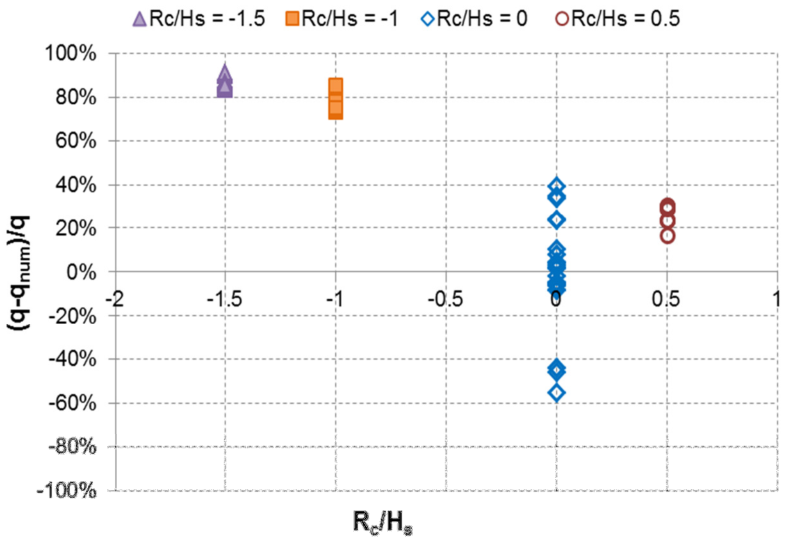


Figure 12. Discharge over the structure crest predicted by the formulae, q, and obtained by the numerical results of the flow velocities, qnum, . The values of qnum are compared with the values of q as predicted by Hughes and Nadal (2009) For submerged cases and by Van der Meer et al. (2013) for emerged cases.

The representation of emerged cases is more accurate when compared with Van der Meer et al. (2013), see Fig. 14, rather than with EurOtop (2007), see Fig. 13. The case $R_c/H_S = 0$ is the one showing more scatter, with a systematic tendency to overestimation of the numerical values by Eq. (13a,b).

It can be appreciated that the difference among the values of $(q\text{-}q_{num})/q$ derived either from celerities or velocities significantly reduces with increasing R_c/H_s, as expected, since the overtopping flow over an emergent crest is highly non-linear (breaking dominated) and therefore the flow velocity almost equals the wave celerity.

Overall, the predictions by Eq.s (13a,b) better fit the emerged structures, while for submerged cases only the discharges obtained from wave celerities can be compared with the existing formulae and in this case the EurOtop (2007) approach provides more accurate results.

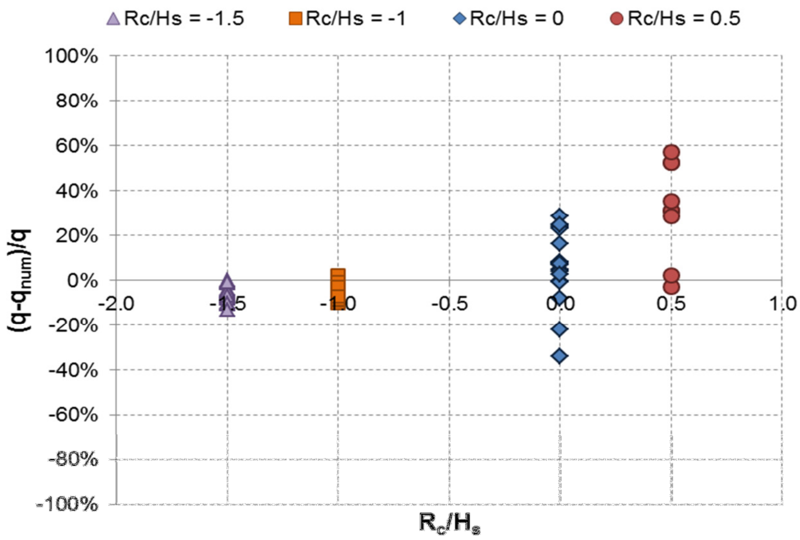


Figure 13. Discharge over the structure crest predicted by the EurOtop (2007), *q*, and obtained by the numerical results of the flow celerities, q_{num}. The relative error is shown for all tested conditions.

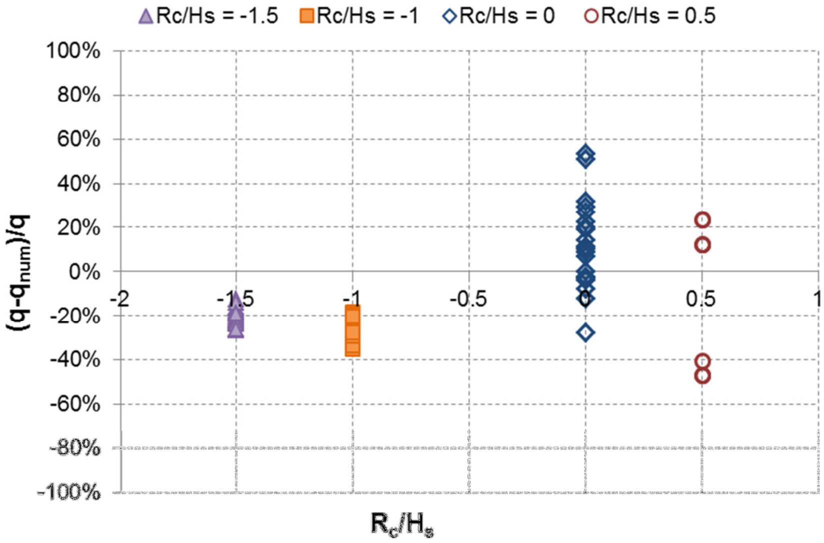


Figure 14. Discharge over the structure crest predicted by the the formulae, q, and obtained by the numerical results, of the wave celerities q_{num}. The values of q_{num} are compared with the values of q as predicted by Hughes and Nadal (2009) For submerged cases and by Van der Meer et al. (2013) for emerged cases.

CONCLUSIONS

Numerical simulations with the IH-2VOF model developed by the University of Cantabria were carried out in order to analyse the flow characteristics (velocities and depths) on the dike crest.

The numerical data, derived by this analysis, allowed to perform a systematic investigation, which may be useful to extend the existing theoretical approach (Schuttrumpf and Oumeraci, 2005; EurOtop, 2007).

The effects of the structure design parameters (i.e. seaward and landward slopes and submergence) on the trend of both depths and velocities over the crest were examined.

As regards the influence of the submergence we can observe that:

- by increasing the submergence, the wave height decay decreases and it tends to disappear when $R_c/H_s \geq -1.0$;
- the overtopping flow velocities tend to increase over the crest, with decreasing growth rate by increasing the relative submergence;
- by keeping constant R_c/H_s, the wave height decay tends to decrease when the significant wave height increases; this is indeed a scale effect since the crest width is constant in all the tested conditions.

These results are therefore perfectly aligned with EurOtop (2007): flow depths tend to decrease, flow velocities on the contrary as already suspected by the EurOtop tend to increase and these effect disappear with increasing structure submergence.

As regards the influence of the seaward/landward slope:

- in case of $R_c/H_S \geq 0$, the values of the flow depths and velocities over the crest appear to be affected by the seaward slope; the milder the slope the greater the depth decay and the velocity increase; when the dike is submerged, there is no appreciable effect induced by the seaward slope;
- in all tested conditions, the results are independent from the landward slope.

The curves fitting the variation of the wave height and velocity over the dike crest are the same as proposed by Schuttrumpf and Oumeraci (2005). Different (lower) decay coefficients were determined on the basis of the submergence and of the significant wave height.

The present theoretical approach (EurOtop, 2007) for the estimation of the overtopping discharges is based on the measurement of wave celerites – instead of what are properly called “velocities”. This approach leads to a good agreement with the numerical results in case of emerged structures (Van der Meer et al., 2013), due to the non-linearity of the flow over the crest and the substantial equivalence among wave celerity and flow velocity, but tends to overestimate the discharges for submerged dikes (EurOtop 2007; Hughes and Nadal 2009), when the velocities are sensibly lower than the celerities. Therefore, further research is still needed for to provide a reasonable estimate of the discharge in case of submerged structures.

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