Overtopping at Vagueira sea defence using the SWASH model

Cálculo do galgamento na estrutura de defesa aderente da Vagueira utilizando o modelo SWASH

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RESUMO: A praia da Vagueira está localizada no Município de Vagos, distrito de Aveiro e é protegida por uma estrutura de defesa frontal aderente que foi requalificada em 2015, com alteamento da sua cota de coroamento para + 10 m (ZH), devido à ocorrência de inundações na zona por ela protegida. Para caracterizar o galgamento com a estrutura atual, aplicou-se o modelo numérico SWASH a uma secção desta estrutura, simulando-se os temporais ocorridos entre 1979 e 2018 que, segundo as fórmulas comumente utilizadas para este tipo de estrutura, dariam galgamentos significativos. Assim, nesta comunicação apresenta-se o modelo utilizado, as características da estrutura de defesa aderente e as condições de agitação e nível de mar utilizadas nas simulações. Os resultados obtidos com o SWASH são comparados com os resultados de várias fórmulas empíricas. Para todas as condições testadas, as fórmulas empíricas apresentam sempre caudais médios de galgamento não nulos, enquanto o SWASH apenas origina caudais médios de galgamento superiores a zero quando o espraiamento máximo não atinge o coroamento da estrutura. Consequentemente, a utilização dos resultados das fórmulas empíricas para a emissão de alertas pode conduzir a situações de alarme que não correspondem à realidade.

Palavras-chave: estrutura aderente de enrocamento; espraiamento; perfil barra-fossa; modelo não linear de águas pouco profundas.

ABSTRACT: Vagueira beach is located at Vagos Municipality, Aveiro, and is protected by a longitudinal defence structure. That structure was rehabilitated in 2015 with an increase in its crest level to + 10 m (ZH) to overcome the occurrence of flooding in the area protected by the structure.

To characterize the overtopping over the rehabilitated defence structure, the numerical model SWASH was applied to a cross-section of the structure to simulate the storms that occurred between 1979 and 2018. The input conditions of those storms originated significant overtopping according to the widely used empirical formulae that are applied to this type of structure.

This paper describes the modelling approach, as well as the physical characteristics of the structure and the input wave and sea level conditions that were considered in the numerical simulations. The results obtained with SWASH are compared with the results for several empirical formulae. For all the conditions tested, the empirical formulae always return non-null overtopping discharges. SWASH only gives null overtopping discharges when the maximum run-up does not reach the crest of the structure. Consequently, using results from the empirical formulae to issue alerts can lead to alarming situations that do not correspond to reality.

Keywords: adherent rubble-mound structure; wave run-up; bar-trough profile; nonlinear shallow water model.

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1. INTRODUCTION

To calculate overtopping over a coastal protection structure, formulations are used that, for given conditions (type of structure, bottom profile in front of the structure, wave and water level conditions), allow estimating the mean overtopping discharge. Although they are very little time-consuming, empirical formulations have some limitations. One of the most relevant limitations is the need for the depths to be continuously decreasing towards the structure under study, which in many cases does not correspond to reality. This is the case of the bar-through bottom profiles, where the existing formulations in the literature do not apply, as this is a profile in which depths are decreasing towards the coast.

An alternative that allows considering any bottom profile is the use of numerical models. Although there are several numerical models available in the literature, for a given model to be used in the design phase, it must present a good compromise between calculation time and the accuracy of results. The SWASH numerical model (Zijlema *et al.*, 2011), which solves the nonlinear equations for shallow waters has been applied with good results to the study of overtopping of longitudinal defence structures whose toe is above sea level or at small depths.

To estimate the overtopping over the longitudinal defence structure of Vagueira beach, Rosa (2021) applied empirical formulas and/or neural networks. The formulas used were those presented in Eurotop (2018), Goda (2009), van Gent (1999) modified by Altomare *et al.* (2020), Mase *et al.* (2013), and Masatoshi *et al.* (2019). The neural network used was NN_OOVERTOPPING2 (Coeveld *et al.*, 2005). As the bottom in front of Vagueira beach has a bar-through profile, to overcome this limitation, in Rosa (2021) four bottom profiles were tested that were close to the bar-through profile but ensuring that the depths are always decreasing towards the coastal defence structure. In all the studied alternatives, the bar-through profile was eliminated to comply with the limitation of continuously decreasing depths. From the results obtained with the application of empirical formulas to the approximate bottom profiles tested, Rosa (2021) concluded that the bottom profile had a great influence on the mean overtopping discharge. For all the formulae tested, the profile where the bar was more defined was the one for which the highest overtopping discharges were obtained. Thus, for these types of bottom configurations, if the original profile has to be adapted so that the empirical formulae can be used, care must be taken to ensure that the bar is maintained. This way, the empirical formulae results are safer to be used for the structure design.

This work presents the application of the SWASH model to the study of the wave overtopping over a section of the longitudinal defence structure on Vagueira beach for a bottom profile of the bar-through type.

After this introductory chapter, the study area and the application of the SWASH model are presented, describing the conditions for applying the model and the input data. Finally, the results obtained are presented and discussed.

2. STUDY AREA

Vagueira beach is located in the municipality of Vagos, district of Aveiro. This beach is protected by a longitudinal defence structure that was rehabilitated in 2015, with its crest level being raised to + 10 m (ZH) to reduce overtopping in this area (Figure 1). The sea defence is located at the end of a very shallow foreshore. It is composed of a rubble-mound layer with an angle of 55º. The central cross-section has a toe at +2.89 m (ZH) and a crest level at + 10.76 m (ZH).

In the Vagueira beach area, the feasibility of a detached multifunctional breakwater was recently studied. One of the phenomena to be studied was its influence on the overtopping of the longitudinal sea defence structure (Sancho *et al.*, 2020). In that study, the background profile in front of the structure was defined based on data collected under the COSMO program (COSMO, 2018) and treated by Fortes *et al.* (2020). In Figure 2, a cross-shore profile from the central part of the defence structure is presented, where the bar-through profile of the bottom can be seen.

In the study from Sancho *et al.* (2020) the wave and sea level conditions were defined at a point located at -12 m (ZH) (coordinates 40°34’03.2”N, 8°47’45.9”W) for the period 1979 to 2018 based on the data made available by the European Center for Medium-Term Weather Forecasts (ECMWF) (Fortes *et al.*, 2020).

Between 1979 and 2018, there were several storms in the area, and some of them led to the occurrence of overtopping over the structure, especially in the storms previous to the rehabilitation works of 2015 that increased the level of the crest to up to + 10 m (ZH).

Figure 1. View of the longitudinal frontal defence structure of Vagueira beach (Rosa, 2021).

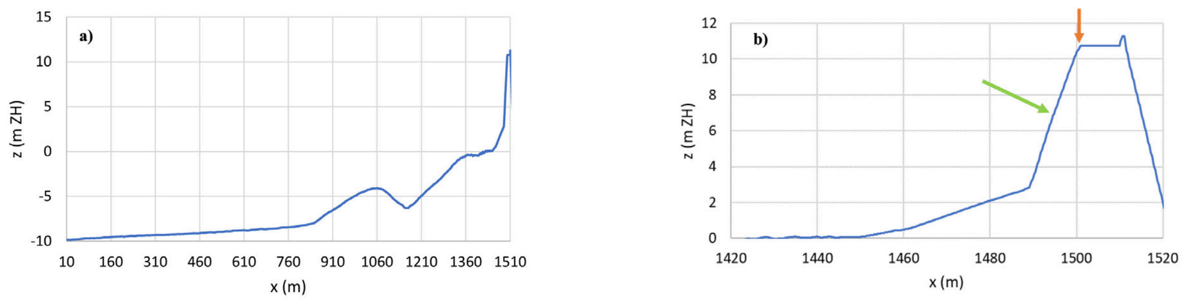

Figure 2. Bottom profile in front of the structure in July 2018. a) entire computational domain, b) near the structure. The green (middle of the structure slope) and orange (crest of the structure) arrows indicate the locations where overtopping was measured.

3. SWASH MODEL SIMULATIONS

3.1 SWASH Model

The SWASH model (Zijlema *et al.*, 2011), solves the non-linear shallow water (NLSW) equations and is relatively efficient in terms of calculation time. As the governing equations are the NLSW equations and include non-hydrostatic pressure, they can describe complex and rapidly changing flows in detailed topo-bathymetries that are often found in coastal flooding events. Therefore, the model can simulate shallow water flows and nearshore processes, including wave propagation, breaking and run-up, wave transmission through structures, non-linear interaction, and wave-induced circulation (Zijlema *et al.*, 2011).
Suzuki *et al.* (2017) tested the model SWASH for estimating overtopping over impermeable coastal structures with shallow foreshores. The authors outlined that the incident wave properties at the toe of the structure need to be accurately reproduced so that reasonable results can be obtained. The estimation of mean overtopping discharges showed good accuracy although the instantaneous wave overtopping was in some cases under-predicted.
Zhang *et al.* (2020) tested the SWASH model in estimating the mean overtopping discharge over a breakwater with an armour layer of Accropode. They compared their results with the ones from the CLASH database (which consists of physical model results). They highlighted the need to properly calibrate the model to obtain the apparent friction coefficient of the armour layer so that the mean overtopping discharges from the CLASH database can be reproduced.
The model has been applied with good results to the study of overtopping over longitudinal defence structures whose toe is above sea level or at small depths, being therefore an appropriate model to apply to the present study case.

3.2 Parametrization and sensitivity analysis

For the case study presented here, simulations were carried out with the one-dimensional version of the numerical model, for a computational domain with a length of 1524 m. The computational domain started at -12 m (ZH) and included the longitudinal defence structure. On the vertical dimension, one layer was used in all simulations.
The simulation time was 1000 waves plus a warm-up period that consisted of 15% of the computational time, as recommended by the SWASH user manual. A Jonswap spectrum with γ=3.3 was imposed at the entrance boundary.

The application of the model was preceded by a sensitivity study of the model results (Q and Q_{max}, maximum overtopping discharge) to the mesh size ($deltaX$). Mean overtopping discharge, Q, is obtained by summing up the instantaneous overtopping discharge at each time step and dividing it by the computational time.
The sea defence is here treated as an impermeable one and a bottom friction coefficient was used to represent the effect of comprehensive energy dissipation as a consequence of roughness. The Manning coefficient was used to include the friction and roughness of the structure and the foreshore. For the foreshore, the value of 0.019 s/(m$^{1/3}$) was used, as suggested in SWASH's manual.
The sensitivity analysis of the grid size was performed for a storm with an incident significant wave height of 2.61 m, a peak period of 14.88 s, a sea level of +3.79 m (ZH), and a Manning coefficient of the structure of 0.05 s/(m$^{1/3}$). Since no overtopping was obtained for this case, the overtopping was calculated in the middle of the structure slope (green arrow in Figure 2), rather than at the crest of the structure. Meshes with dimensions varying between 0.075 m and 3 m were tested. Figure 3 presents the results of the sensitivity analysis obtained.
Grids of 1.000, 1.500, 2.000 and 3.000 m led to significantly lower discharge values, as they were unable to accurately capture the run-up. The grid with a spacing of 0.200 m was the one that allowed reproducing results with a run-up value variation of around ±4.5% between the 0.100 and 0.750 m grids and a discharge variation of ±2% between the 0.200 and 0.500 m grids. The simulation time was approximately 2h to simulate 1000 waves and was considered acceptable.
For the longitudinal defence, Manning coefficient values between 0.04 to 0.07 s/(m$^{1/3}$) were tested (those values were recommended by Corrado Altomare for this type of structure) for the two storm wave conditions. The grid spacing was 0.2000 m and 1000 waves were simulated. Figure 4 presents the results for one of the storms tested. From the analysis of all the obtained results (Correia, 2023), the Manning coefficient values of 0.05 and 0.06 s/(m$^{1/3}$) were the most consistent ones. After a bibliographic review, it was decided to choose for the rest of the simulations a Manning coefficient of 0.05 s/(m$^{1/3}$).

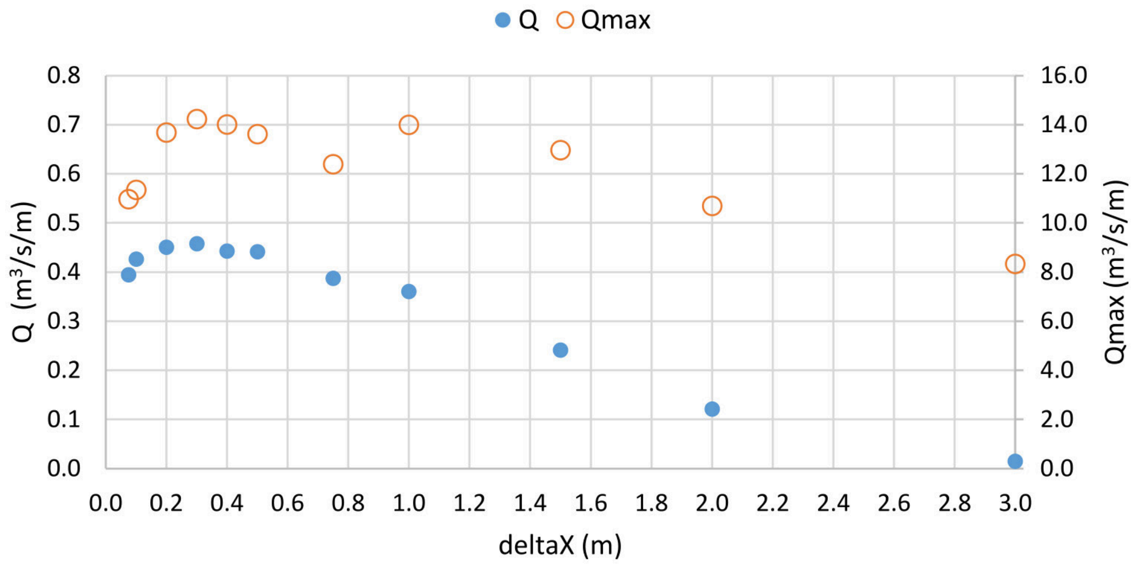

Figure 3. Mean, Q, and maximum overtopping discharge, Q_{max} at the middle of the structure slope obtained for different mesh sizes, *deltaX*, for a storm with an incident significant wave height of 2.61 m, a peak period of 14.88 s, and a sea level of +3.79 m (ZH).

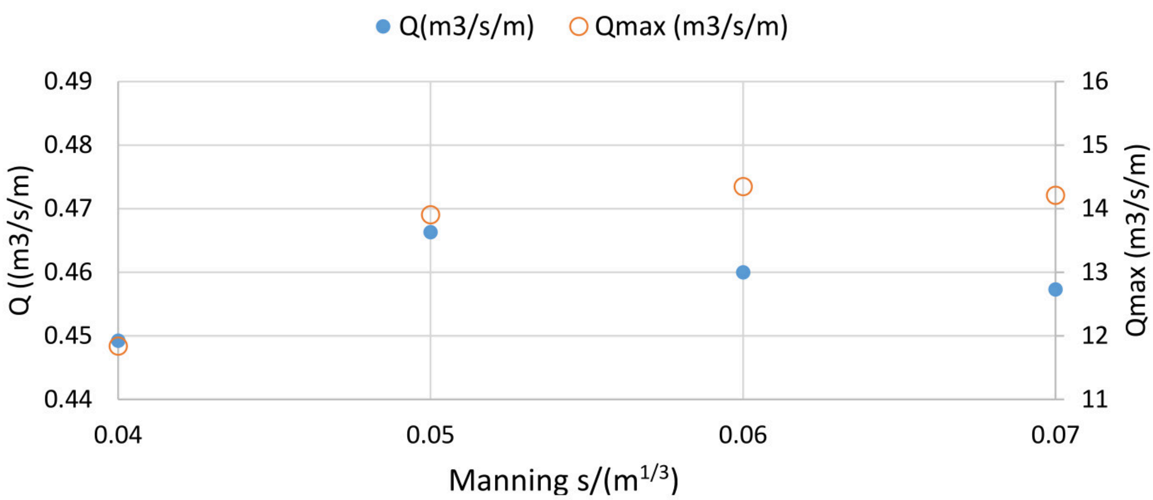

Figure 4. Q and Q_{max}, obtained at the middle of the structure slope for different values of Manning coefficient, for a storm with an incident significant wave height of 2.61 m, a peak period of 14.88 s, and a sea level of +3.79 m (ZH).

3.3 Numerical Simulations

To analyse mean overtopping discharge, Q, after the rehabilitation of the defence structure, numerical simulations were carried out for a cross-section located in the central zone of the structure for wave conditions that occurred between 1979 and 2018. The selected conditions (Table 1) corresponded to the ones that originated the highest values of Q through the various empirical formulations applied by Rosa (2021) to simplified bottom profiles. Table 1 summarizes the wave and sea level characteristics at -12 m (ZH) during the selected events (named hereafter Storm 1 and Storm 2), where Hs is the significant wave height, T_p is the peak period, Dir is the wave direction and SWL is the sea level. Each storm is composed of several wave conditions, whose characteristics are given every 3h.

Table 1. Wave and sea level conditions of the simulated storms at -12 m (ZH) (conditions for which no null overtopping results were obtained are highlighted in blue).

Storm (-)	Hs (m)	T_p (s)	Dir (º)	SWL (m ZH)
	2,26	12,09	269	3,37
	2,84	14,88	271	3,51
	3,48	16,50	275	3,47
1	3,74	16,50	275	3,72
	3,50	14,88	275	2,99
	3,46	14,88	275	3,41
	3,54	14,88	275	2,91
	3,76	12,09	291	3,21
	5,51	14,88	289	3,47
	6,26	16,50	293	3,19
2	6,17	18,31	293	3,62
	6,54	18,31	295	2,63
	5,81	18,31	293	3,38
	5,18	16,50	295	2,63

4. OVERTOPPING RESULTS

4.1 SWASH results

SWASH was applied for the cases presented with the grid dimension and Manning coefficient selected during the sensitivity analysis tests (3.2). For each simulation, the time series of run-up was obtained and, in case of occurrence of overtopping, also the mean overtopping discharge. Table 2 presents the values of Q, Q_{max}, and of maximum run-up, Ru_{max}, obtained for each wave/sea level condition of the two storms in analysis.

Table 2. SWASH model results. Q, Q_{max}, and $RU_{2\%}$, for the simulated storms (conditions for which no null overtopping results were obtained are highlighted in blue).

Storms (-)	Q (l/s/m)	Q_{max} (l/s/m)	Ru_{max} (m)
	0.0000	0.0000	4.54
	0.0000	0.0000	5.32
	0.0000	0.0000	5.66
1	0.0002	8.3073	6.40
	0.0000	0.0000	5.09
	0.0000	0.0000	5.56
	0.0000	0.0000	4.96
	0.0000	0.0000	4.90
	0.0000	0.0000	6.40
	0.0000	0.0000	6.19
2	0.0122	468.20	6.00
	0.0000	0.0000	5.29
	0.0016	89.68	6.14
	0.0000	0.0000	5.30

As can be seen from Table 2, there was no null overtopping only for one wave/sea level condition of Storm 1 and for two wave/sea level conditions of Storm 2 (those conditions are highlighted in blue in Table 2). For Storm 1, the wave condition that led to overtopping presented the highest Hs and SWL values (Figure 5) of the storm. However, Q_{max} is rather large, more than 8 l/s/m. For Storm 2 (Figure 6), a different trend is found, with the highest SWL leading to the highest overtopping conditions, but with a Hs smaller than the highest. The other condition with no null Q corresponds to high values of Hs and SWL, although they are not the highest, they are associated with the highest Tp. In this storm, Q_{max} values are both much higher than Q_{max} for Storm 1. SWL in both storms are similar, but Storm 2 has much higher values of Hs and Tp. Figure 7 and Figure 8 show the free surface elevation at the time step where overtopping occurred in the simulation for Storm 1 and Storm 2, respectively. As can be seen in the figures, the wave overtopping is higher for Storm 2 events, as aforementioned.

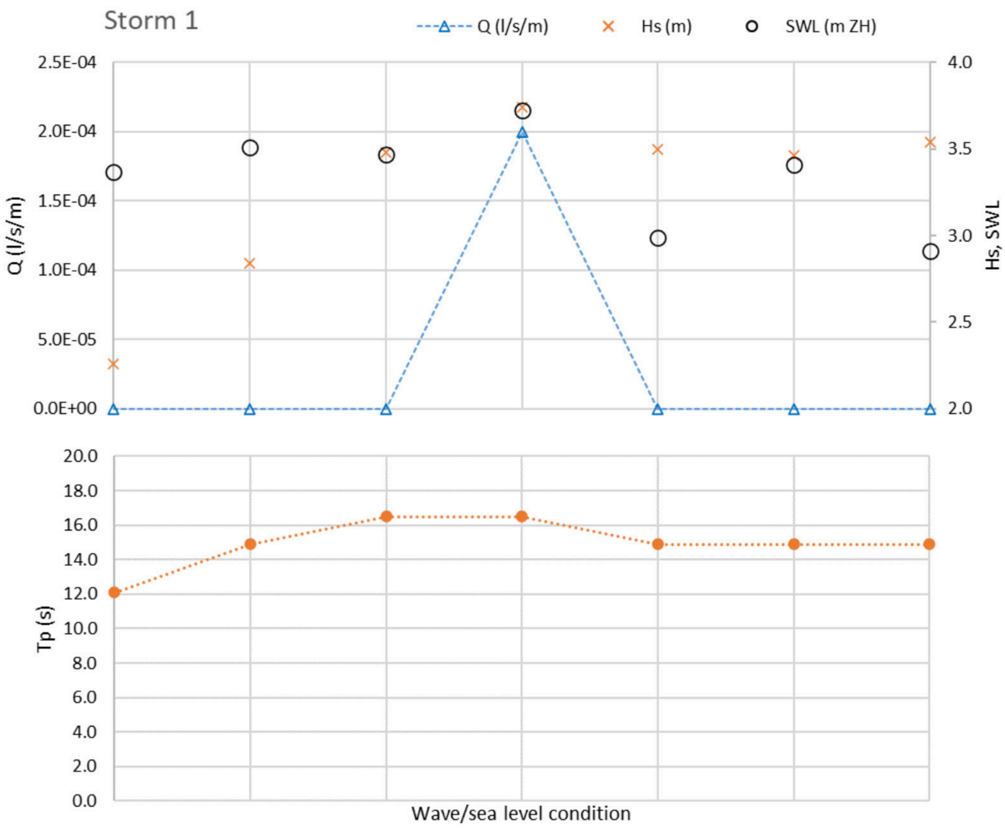

Figure 5. SWASH model results and wave/sea level conditions for Storm 1 (Top: Q values and significant wave height and sea level, Bottom: peak period).

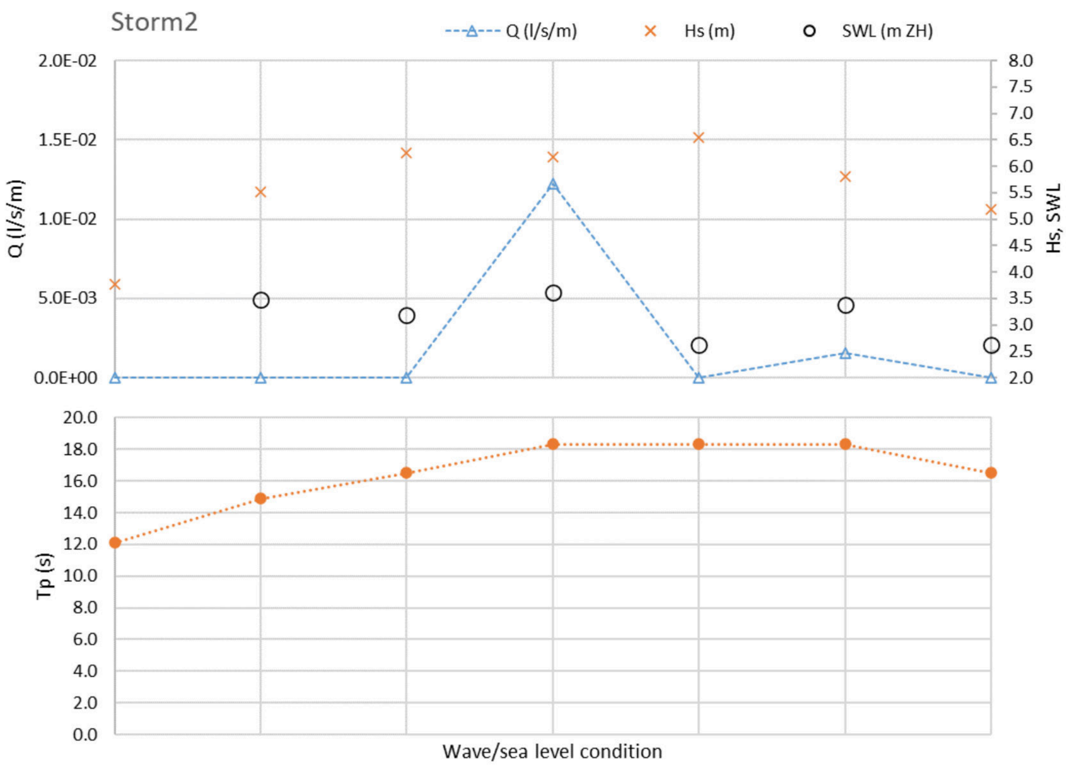

Figure 6. SWASH model results and wave/sea level conditions for Storm 2 (Top: Q values and significant wave height and sea level, Bottom: peak period).

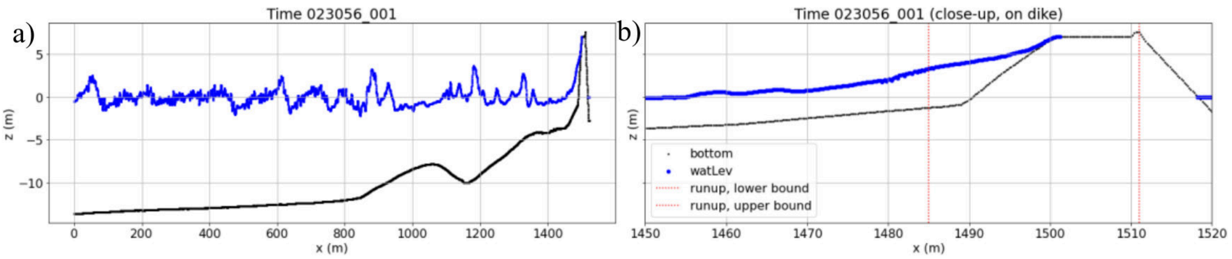

Figure 7. Free surface elevation results for a storm with an incident significant wave height of 3.74 m, a peak period of 16.5 s, and a sea level of +3.72 m (ZH) at the time step that corresponded to the maximum run-up: a) all the computational domain, b) near the structure.

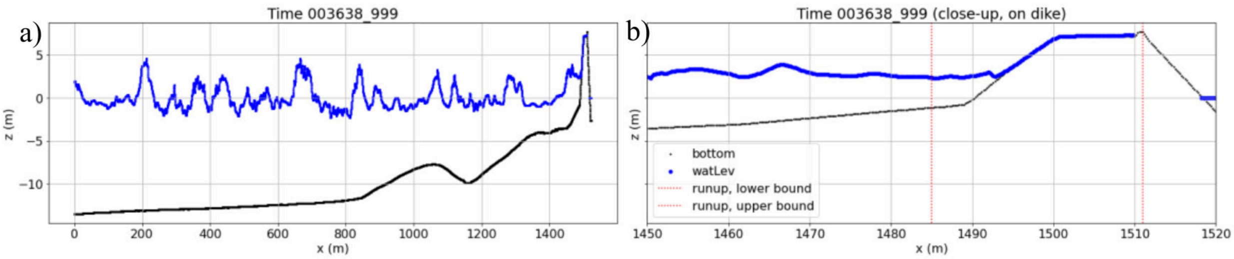

Figure 8. Free surface elevation results for a storm with an incident significant wave height of 6.17 m, a peak period of 18.31 s, and a sea level of +3.62 m (ZH) at the time step that corresponded to the maximum run-up: a) all the computational domain, b) near the structure.

4.2 SWASH and empirical formulae results

The mean overtopping discharges obtained with SWASH were compared with the ones obtained with the following empirical formulae: Eurotop (2018), Goda (2009), van Gent (1999), Goda (2009) modified by Altomare *et al.* (2020) and van Gent (1999) modified by Altomare *et al.* (2020). Mase *et al.* (2013) and Masatoshi *et al.* (2019) are not presented since they give null overtopping discharge for all these events. The latter two formulae calculate the mean overtopping discharge only if the maximum run-up calculated by the formula is higher than the crest level, whereas the other formulae mentioned above calculate directly the value of Q, which will always be greater than zero even in cases where no overtopping is expected. Table 3 and Table 4 present the results for Storm 1 and Storm 2 events, respectively, with the maximum values obtained highlighted in blue. In Figure 10 and Figure 11, the graphical representation of the Q values is presented in logarithmic scale for the two storms.
As mentioned before, the empirical formulae were applied considering a simplified foreshore, where the through was eliminated (see Figure 9), while with the SWASH model, the correctly defined foreshore was used.

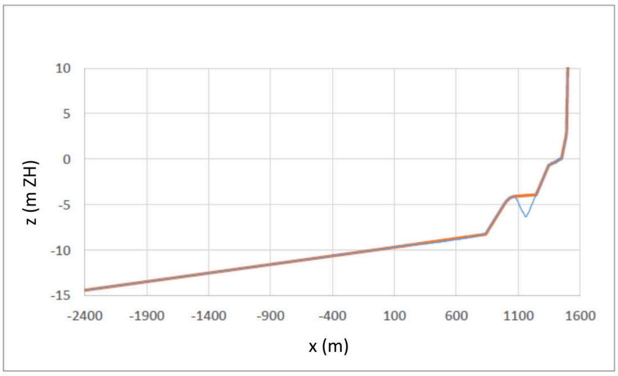

Figure 9. Approximate bathymetric profile tested by Rosa (2021) where the bar was better maintained. The blue line indicates the original profile and the orange line indicates the adapted profile.

One of the main differences between the results obtained with SWASH with the ones obtained with the empirical formulae is that the formulae never give null overtopping whereas SWASH gives null overtopping when the maximum run-up does not reach the crest of the structure.
As can be seen from the analysis of Table 3, Table 4, Figure 10, and Figure 11, the highest values of Q are obtained for the same event in both storms, regardless of the method used to estimate it

(empirical formulae or SWASH). However, the value estimated for Q varies among the different methods. For Storm 1, Goda (2009) modified by Altomare *et al.* (2020) and Goda (2009) gave almost the same Q values and corresponded to the highest ones. They were, in fact, significantly higher than the ones obtained with SWASH and the other formulae. The only formula that gave lower Q values than SWASH was the one from Eurotop (2018), and the one from van Gent (1999) and van Gent (1999) modified by Altomare (2020) estimated Q values similar to SWASH.

Table 3. Q values obtained with the SWASH model and with the empirical formulae for Storm 1.

Q (m³/s/m)					
SWASH	Eurotop (2018)	Goda (2009)	van Gent (1999)	Goda (2009) modified by Altomare et al. (2020)	van Gent (1999) modified by Altomare et al. (2020)
0.00E+00	2.43E-14	2.66E-06	4.32E-11	2.50E-06	5.78E-11
0.00E+00	2.66E-13	9.23E-05	1.12E-08	8.65E-05	9.92E-09
0.00E+00	4.73E-11	4.22E-04	7.99E-08	3.96E-04	7.74E-08
2.00E-07	1.44E-09	1.13E-03	1.61E-06	1.06E-03	9.78E-07
0.00E+00	2.52E-16	1.26E-04	6.41E-12	1.18E-04	2.86E-11
0.00E+00	2.78E-13	1.81E-04	1.22E-08	1.70E-04	1.37E-08
0.00E+00	6.46E-17	1.84E-04	9.72E-13	1.72E-04	6.24E-12

In Storm 2, for some events, the sea level does not reach the toe of the structure and the empirical formulae could not be applied. For this storm, Goda (2009) and Goda (2009) modified by Altomare *et al.* (2020) gave the highest Q values, which were significantly higher than the ones obtained with SWASH and the other formulae, as occurred for Storm 1. As for Storm 1, van Gent (1999) and van Gent (1999) modified by Altomare (2020) estimated Q values higher than SWASH, and Eurotop (2018) gave lower Q values than SWASH.

Table 4. Q values obtained with the SWASH model and with the empirical formulae for Storm 2.

Q (m³/s/m)					
SWASH	Eurotop (2018)	Goda (2009)	van Gent (1999)	Goda (2009) modified by Altomare et al. (2020)	van Gent (1999) modified by Altomare et al. (2020)
0.00E+00	1.14E-19	3.75E-05	6.56E-11	3.52E-05	1.39E-10
0.00E+00	9.36E-08	2.04E-03	7.71E-07	1.91E-03	7.44E-07
0.00E+00	1.34E-10	5.45E-03	4.00E-07	5.11E-03	6.57E-07
1.22E-05	1.80E-07	1.11E-02	2.54E-05	1.04E-02	1.95E-05
0.00E+00	2.54E-19				
1.55E-06	2.75E-08	7.17E-03	3.37E-06	6.72E-03	3.76E-06
0.00E+00	7.41E-14				

Summing up, for these events the empirical formulae that give similar values to SWASH are van Gent (1999) and van Gent (1999) modified by Altomare (2020). However, even when giving smaller Q, the formulae never predict null overtopping.

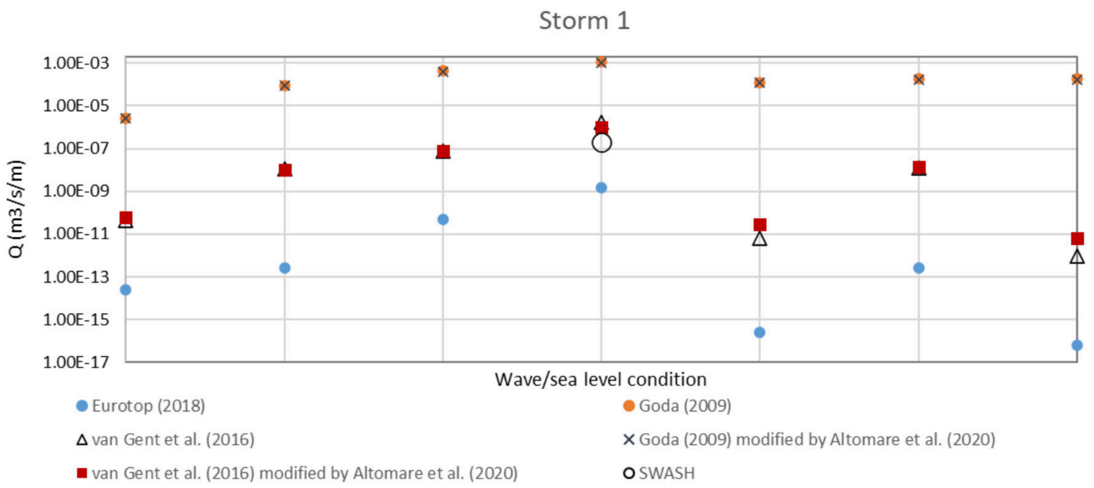

Figure 10. Graphical representation in logarithmic scale of the Q values obtained with SWASH model and with the empirical formulae for Storm 1.

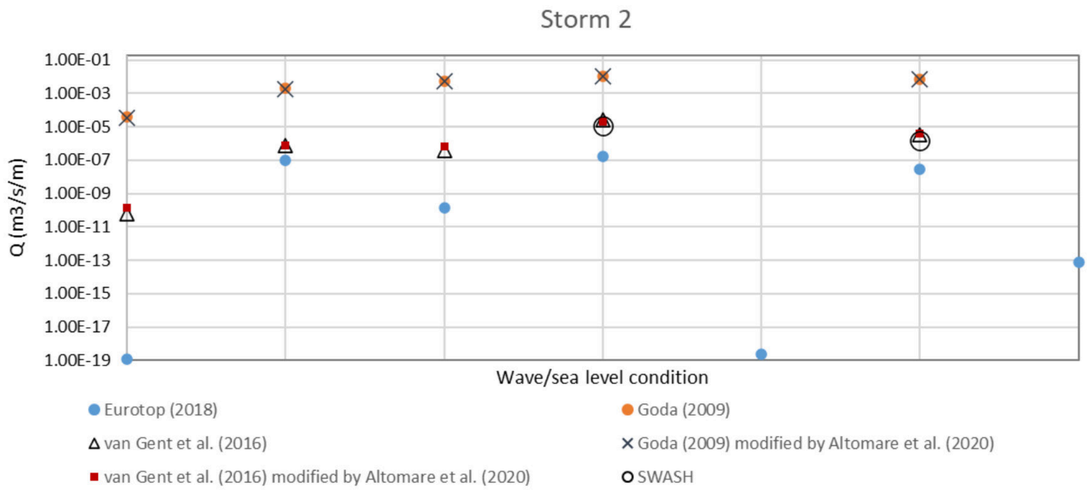

Figure 11. Graphical representation in logarithmic scale of the Q values obtained with SWASH model and with the empirical formulae for Storm 2.

5. FINAL REMARKS

SWASH model was applied to the longitudinal defence structure of Vagueira beach for incident wave conditions and sea levels of two groups of events that occurred between 1979 and 2018, named storms 1 and 2. The longitudinal defence structure was rehabilitated in 2015, with its crest level being raised to + 10 m (ZH) to reduce wave overtopping.

SWASH results only conducted no null overtopping discharges in one event for Storm 1 and two events for Storm 2. The new configuration of the structure appears to be successful in reducing wave overtopping events, as confirmed by the small number of events in which overtopping occurred after the rehabilitation.

The wave/sea level conditions that led to overtopping presented the highest Hs, SWL, and Tp values for Storm 1 and the combination of the highest SWL with the highest Tp for Storm 2. So, these two latter variables were the ones that mainly influenced the occurrence or not of wave overtopping in these two cases.

Although Q values obtained were rather small, Q_{max} values were considerably high. This shows that caution must be taken when designing a structure as individual overtopping can put people at risk or cause severe damage, even though the mean values are not that high.

The results obtained with SWASH were compared with the results for several empirical formulae, in terms of the mean overtopping discharge values. As the bottom in front of Vagueira beach has a bar-through profile and like for the application of the formulae one has to ensure that the depths are always decreasing towards the coastal defence structure, following the results of Rosa (2021), an approximate profile that kept the presence of the bar was used. For all the incident wave and sea level conditions tested, the empirical formulae always return non-null overtopping discharges, whereas SWASH gives null overtopping discharges when the maximum run-up does not reach the crest of the structure. Although they are very little time-consuming, this can be a drawback for the application of empirical formulae. For instance, if the objective is to have Q values to be used within an alert system, and if the thresholds defined by Eurotop (2018) are used, it is possible to issue alerts for the most limiting activities (such as pedestrian use) that may be too conservative and alarming, and not correspond to reality.

However, it is considered that more tests of the SWASH model are needed in prototype situations to assess its performance, especially for cases for which there is data regarding overtopping events.

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