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Methodology for safety assessment 2015

Updated methodologies from the report v6.0 to v10.0

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Updated methodologies from the report v6.0 to v10.0

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Abstract

The methodology book version 10.0 (Suzuki et al., 2016) has been published in June 2016 and is the version used in the Safety assessment 2015. Before the publication of the methodology book version 10.0, we intended to use the methodology book version 6.0 (Suzuki et al., 2015) for the safety assessment 2015 as the final version. However, some important questions regarding the XBeach and SWASH models, overtopping method and hydraulic boundary condition were raised after the publication of the methodology book version 6.0 and further investigation of these issues was decided. In this report, the results of those investigations and necessary changes in the methodology are addressed.

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1 Introduction

The methodology book version 10.0 (Suzuki et al., 2016a) has been published in June 2016 and is the version used in the Safety assessment 2015. Before the publication of the methodology book version 10.0, we intended to use the methodology book version 6.0 (Suzuki et al., 2015) for the safety assessment 2015 as the final version. However, some important questions regarding the XBeach and SWASH models, overtopping method and hydraulic boundary condition were raised after the publication of the methodology book version 6.0 and further investigation was decided. Therefore, FHR conducted basic investigations for the 6 topics as shown below.

- Number of the waves
- Toe position
- Overtopping criteria
- Boundary position
- SWASH version
- XB model improvements

In this report, the results of those investigations and necessary changes in the methodology are addressed.

2 Number of waves

2.1 Background

During execution of other projects at FHR (Knokke, project number 15_095, and Raversijde-Mariakerke-Wellington west, project number 13_168), one issue regarding number of waves was raised: Using the number of waves specified in the methodology book version 6.0 can result in a large uncertainty in the obtained overtopping discharges due to the stochastic nature of the data. In general, the larger the number of waves used in one test, the smaller uncertainty / scatter of the result (H_{m0}, $T_{m-1,0}$, setup and q). On the other hand, the larger the number of waves used in one test are more time consuming. As a consequence, the methodology needs to balance between the accuracy and computational time. Therefore number of waves in each simulations for the methodology is re-investigated here.

2.2 Methodology in version 6.0

In the methodology book version 6.0 (Suzuki et al, 2015), the number of the waves were specified as follows.

1) 100 waves for SWASH2D to get incident wave parameters

2) 100 waves for SWASH1D for the calibration

3) 500 waves for SWASH1D for the overtopping calculation.

2.3 Uncertainties in wave parameters and overtopping discharge

2.3.1 Influence of number of waves for overtopping estimated with the semi-empirical equation

FHR conducted SWASH1D calculations (not 2D due to the time restriction for the execution of this test: however the nature of the scatter should be in the same order of magnitude, thus this investigation is meaningful) with changing seed number to get certain statistical information. Three different wave trains have been used, 100, 200 and 500 waves as can be seen in Table 1. Note that the calculations were based on the profile 104. Incident wave height offshore was set H_{m0}=4.75 m, that is why the incident wave height at the toe is higher than the reality (no directional spreading effect).

Table 1 shows that the largest difference in wave height is 3.7 % with 100 waves, which decreases to 1.5 % with 200 and 500 waves. The largest difference in wave period is 11 % with 100 waves, which again decreases to 8.5 % with 200 waves and 5.2% with 500 waves.

Now, the question is how those values (standard deviations of significant wave height and spectral period) influence the wave overtopping discharge. Based on the recent results from Raversijde-Mariakerke-Wellington west (project number 13_168), overtopping discharges have been calculated for the sea dike, without any storm wall or promenade (see Table 2). Using the equations described in the Methodology book v10.0 will now lead to similar conclusions.

A higher wave height (average significant wave height + one standard deviation: 3.7% increase in wave height with 100 waves) gives 22% more average overtopping discharge. In turn, a higher wave period (average spectral period + one standard deviation: 11% increase in wave period with 100 waves) results in

14% higher average overtopping discharges. So, an increase in wave height of only 3.7% has more influence on the overtopping discharges compared to the 11% increase of the wave period, as expected.

If both scatters (wave height and period) coincide time, the overtopping becomes 38% higher with 100 waves and in case of 200 and 500 waves, resp. 20% and 15 % more. an example calculation of the wall height to satisfy the 1 l/s/m overtopping criterion yields an increase of the wall with 5 cm (8 cm wall height instead of 13 cm).

Table 1 - SWASH1D results for incident wave parameters

1D cal	600s-1800s			600s-3000s			600s-6600s		
Seed number	100 waves	error%		200 waves	error%		500 waves	error%	
	Hs at toe	Tmm-1,0 t	SWL toe	Hs at toe	Tmm-1,0 t	SWL toe	Hs at toe	Tmm-1,0 t	SWL toe
0	1,35	37,9		1,40	39,3		1,36	38,7	0,56
1	1,29	38,9	0,54	1,34	38,7	0,54	1,33	36,2	0,55
2	1,39	41,6	0,58	1,36	43,3	0,56	1,37	39,0	0,57
3	1,37	43,8	0,56	1,39	36,4	0,56	1,38	38,8	0,58
4	1,34	39,2	0,55	1,36	46,2	0,57	1,38	40,4	0,56
5	1,42	43,6	0,58	1,36	39,1	0,57	1,33	36,7	0,56
6	1,33	37,8	0,55	1,35	35,9	0,57	1,33	38,0	0,55
7	1,42	41,8	0,56	1,35	39,1	0,57	1,37	38,4	0,56
8	1,30	29,5	0,56	1,31	38,3	0,56	1,37	36,9	0,56
9	1,32	39,0	0,56	1,34	37,3	0,56	1,36	42,7	0,56
ave.	1,35	39,3	0,56	1,36	39,3	0,56	1,36	38,6	0,56
max	1,42	43,8	0,58	1,39	46,2	0,57	1,38	42,7	0,58
min	1,29	29,5	0,54	1,31	35,9	0,54	1,33	36,2	0,55
error(%)	9,5	36,5	8,5	5,4	26,0	4,6	3,7	16,8	4,2
standard div	0,05	4,31	0,01	0,02	3,33	0,01	0,02	2,02	0,01
sigma/ave (%)	3,7	11,0	2,6	1,5	8,5	1,4	1,5	5,2	1,3

<Units> Hs [m], Tm-1,0 [s], SWL [m]

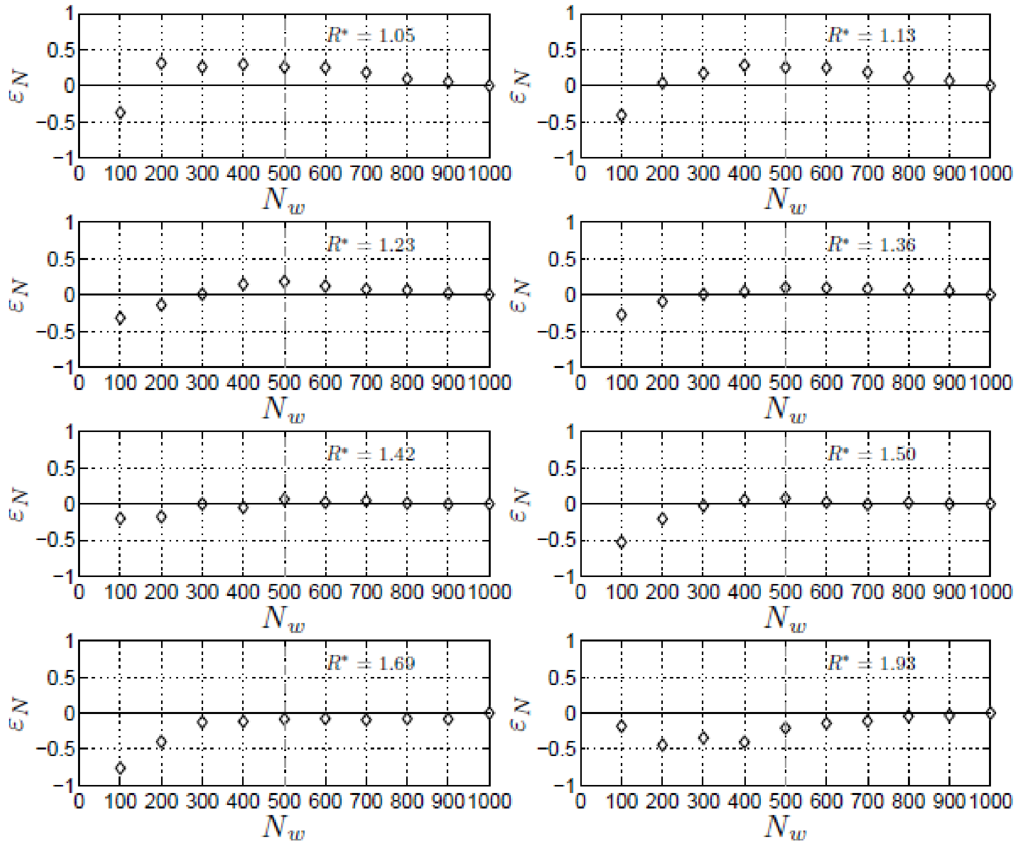
Table 2 - Overtopping calculation by equation used in Raversijde project

	Using Eq. 12																		
	Sectie	δ' H	δ' T	Hm0	Tm-1,0	foreshore	slope	Ac	setup IMDC	Ac without setup	Crest elevation at the seaward edge mTAW	Elevation of the wall toe mTAW	Crest elevation including wall and promenade mTAW	h_toe	hwall [m]	new Rc	Promenade [m]	q new form	
H, 100 waves	104,1	3,7		0,89	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,11	
H, 100 waves	104,1			0,92293	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,35	22%
H, 100 waves	104,1			0,85707	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	0,90	-19%
T, 100 waves	104,1		11	0,89	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,11	
T, 100 waves	104,1			0,89	29,0043	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,26	14%
T, 100 waves	104,1			0,89	23,2557	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	0,96	-13%
H, 200 waves	104,1	1,5		0,89	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,11	
H, 200 waves	104,1			0,90335	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,20	9%
H, 200 waves	104,1			0,87665	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,02	-8%
T, 200 waves	104,1		8,5	0,89	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,11	
T, 200 waves	104,1			0,89	28,35105	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,23	11%
T, 200 waves	104,1			0,89	23,90895	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	0,99	-10%
H, 500 waves	104,1	1,5		0,89	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,11	
H, 500 waves	104,1			0,90335	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,20	9%
H, 500 waves	104,1			0,87665	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,02	-8%
T, 500 waves	104,1		5,2	0,89	26,13	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,11	
T, 500 waves	104,1			0,89	27,48876	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,18	7%
T, 500 waves	104,1			0,89	24,77124	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,04	-6%
H,T 100 waves	104,1	3,7	11	0,92293	29,0043	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,53	38%
H,T 200 waves	104,1	1,5	8,5	0,90335	28,35105	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,33	20%
H,T 500 waves	104,1	1,5	5,2	0,90335	27,48876	50	2,1	2,01	0,1	2,11	9,23	9,23	9,23	0,72	0	2,11	0	1,28	15%
H,T 100 waves	104,1	0	0	0,92293	29,0043	50	2,1	2,01	0,1	2,11	9,23	9,23	9,36	0,72	0,13	2,24	0	1,00	-9%
H,T 200 waves	104,1	0	0	0,90335	28,35105	50	2,1	2,01	0,1	2,11	9,23	9,23	9,32	0,72	0,09	2,20	0	0,98	-11%
H,T 500 waves	104,1	0	11	0,90335	27,48876	50	2,1	2,01	0,1	2,11	9,23	9,23	9,31	0,72	0,08	2,19	0	0,98	-12%

2.3.2 Influence of the number of waves for overtopping, from literature Romano et al. (2015).

This figure below shows the error on wave overtopping by different numbers of waves. Off course the error also depends on the crest freeboard, R_c, but Figure 1 corresponds to our range of applications. It shows that 800 waves can give similar estimation as 1000 waves.

Figure 1 - Mean error ε_N as a function of the considered number of waves per test N_w.



Each panel refers to a different value of the dimensionless freeboard R* (Romano et al., 2015).

2.3.3 Influence on computation time

1) 100 waves for SWASH2D: around 20 min in parallel computation (12 cores). In case of 500 waves can be up to 100 min

2) 100 waves for SWASH1D for wave calibration: less than 10 seconds in parallel computation (12 core)

In case of 500 waves was around 30 s (tested)

3) 500 waves for SWASH1D for overtopping: around 2 min in parallel computation (12 cores).

In case of 1000 waves can be around 4 min

2.4 Methodology in version 10.0

The number of the waves specified in the methodology is based on the investigation above and also on acceptance from the advisors from Coastal division. The updated methodology in the methodology book version 10.0 (Suzuki et al, 2016a) is as follows.

1) 200 waves for SWASH2D to get incident wave parameters

2) 500 waves for SWASH1D for the calibration

3) 500 waves for SWASH1D for the overtopping calculation.

3 Toe position

3.1 Background

During the execution of the Knokke project (project number 15_095), it was found that there are some cases/areas along the Belgian coast in which the position of the toe is located far above SWL. This might be a problem for overtopping calculation by SWASH. Therefore an alternative for determing the position of the toe is explored here.

3.2 Methodology in version 6.0

In the methodology book version 6.0 (Suzuki et al., 2015), it is stated that the position of the toe is at the point where the slope changes into 1/10. However, some cross-sections end up with a toe far above the still water level. In this case the conditions at the toe do not correspond to a wave signal but are like an overtopping signal (only few peaks in the time window).

3.3 Alternative toe position

For a SWASH calculation this position is still possible since there is a calibration process. However, it is not ideal since the evaluation of the waves is done in H_{m0}, $T_{m-1,0}$ and set-up (mean water level increase compared to SWL), which cannot start from an overtopping like signal.

Instead, the position of the toe can be changed to a ‘wet point’. The definition of the wet point is the first grid cell under the input still water level (used as input of SWASH2D, namely still water level from Hydraulic boundary condition book) in the simulation.

One concern is that it is not known that the equation defined by methodology book will properly work in this situation. Therefore, one investigation has been done using the advice report of Knokke (Suzuki et al., 2016b) for this issue. It became clear that the change in toe position changes the ‘dike slope’. This means that the equation is not always applicable, notably for those cases where the dike slope becomes out of the range of the application namely between 1/2 to 1/8.

3.4 Methodology in version 10.0

The position of the toe was specified based on the investigation above and also on acceptance from the advisors from Coastal division. The updated methodology version 10.0 (Suzuki et al, 2016a) is as follows.

The of the position toe is specified at the point where the slope changes to 1/10, unless the specified position is a ‘dry point’ (above SWL). If it is dry point, the position of the toe is changed to the first ‘wet point’ (first grid cell under the input still water level). If the dike slope becomes outside the range of the application, say 1/2 to 1/8, then empirical equation cannot be used.

4 Overtopping criterion

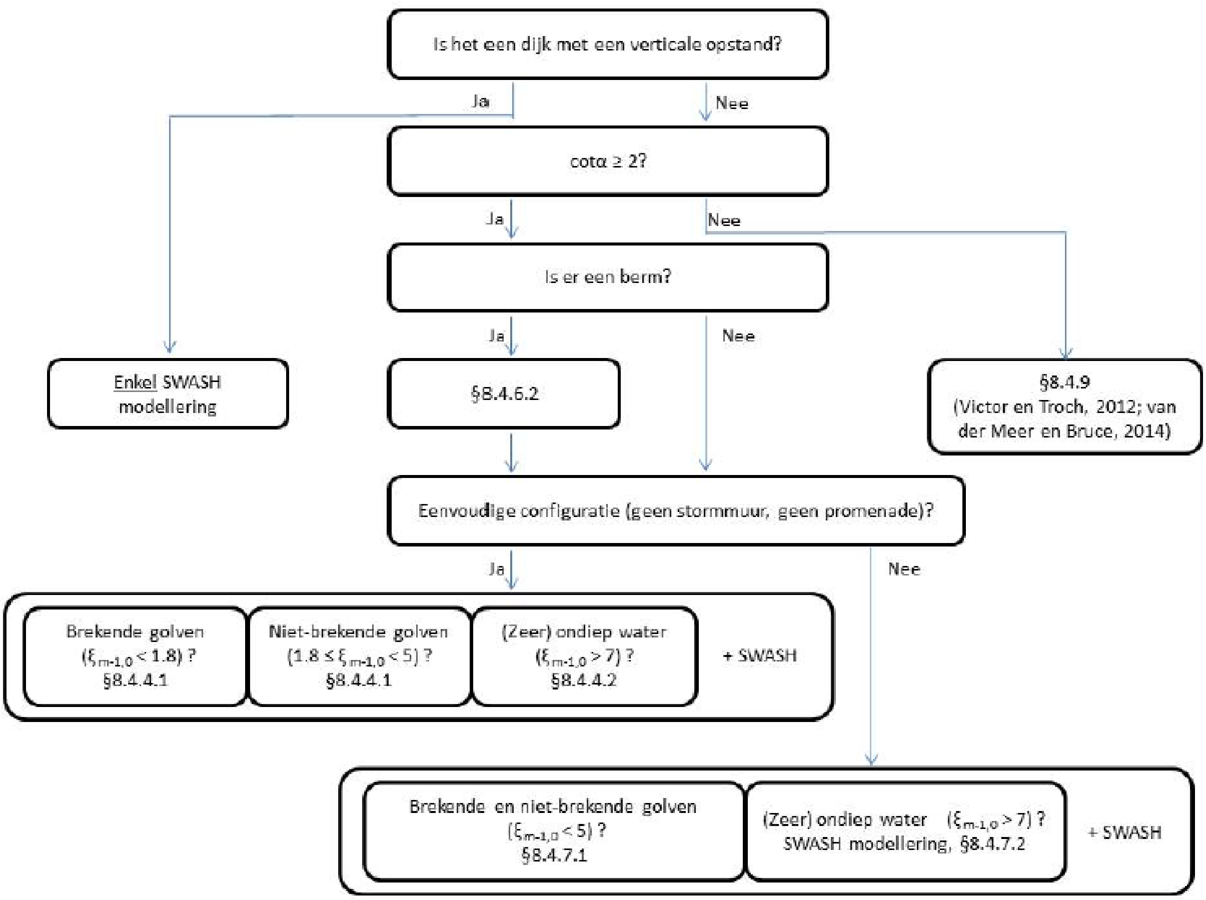
4.1 Background

Recent research has pointed out that the use of the breaker parameter, $\xi_{m-1,0}$, might be not enough as a criterion to define whether or not a (very) shallow foreshore is present. The 2nd edition of the EurOtop Manual (EurOtop, 2016) refers to the use of the wave steepness as an additional criterion. In this chapter, the overtopping criterion for (very) shallow foreshore discussed.

4.2 Methodology in version 6.0

The methodology book version 6.0 specifies overtopping calculation method mainly by xsi value as shown in Figure 2.

Figure 2 - Flowchart in the methodology book version 6.0



4.3 Overtopping criteria for (very) shallow foreshore

Most usual wave conditions have a wave steepness, $s_{m-1,0}$, between 0.01 and 0.06. The largest values correspond to wind waves and the smaller to swell conditions or conditions where the wave height has been reduced due to breaking over a foreshore. However, if the slope of the foreshore is (very) mild and characterized by shallow or very shallow water conditions, the waves may break over a large part of the foreshore and may reduce significantly. The reduction in wave height can achieve up to 20% of the original deep-water wave height, over a shallow foreshore, and even up to 60% in case of severe wave breaking at very shallow foreshores. The wave period drastically changes as well, and low frequencies start to characterize the wave spectrum.

Following the new EurOtop Manual (EurOtop, 2016), a wave steepness of $s_{m-1,0} < 0.01$ usually corresponds to conditions of severe wave breaking, actually occurring in depth-limited conditions (shallow or very shallow waters). As soon as this threshold is exceeded one should realize that the situation probably corresponds to a (very) shallow foreshore with extensive breaking.

If severe wave breaking occurs, the wave height reduces significantly with respect to the offshore wave height and the wave period can become very large. This leads to very low values of the wave steepness [$s_{m-1,0}=2\pi H_{m0}/(gT_{m-1,0}^2)$].

The breaker parameter is defined as $tan(\alpha)/(s_{m-1,0})^{0.5}$, where tan(α) is the dike slope. For the most common values of dike slopes (1:2 to 1:4), a very low steepness means very large values of the breaker parameter. But, for gentle slopes (1:6-1:8), still a small steepness can give relatively small breaker parameters. An example is shown in the next table, where in red the values of $\xi_{m-1,0}>7$ are marked and in green the values of $\xi_{m-1,0}<5$ are marked. An horizontal line in the table marks the threshold of $s_{m-1,0} = 0.01$. A dike is usually defined as having a slope between 1:8 and 1:2. Within this range, the breaker parameter is always less than or equal to 5, independently on the wave steepness. For example, with a slope of 1:5, the breaker parameter is 3.7 for a wave steepness of 0.003, that might correspond to sever breaking in shallow waters.

If we use only the criteria $\xi_{m-1,0}>7$ to identify shallow and very shallow foreshore conditions, we would use the equation for non-breaking waves in this case (eq. 8.2 Methodology Book) and not the equation for shallow foreshores: this might be wrong! Furthermore, it has to be noticed that the criteria $\xi_{m-1,0}>7$ is wrongly associated to the use of Van Gent (1999) equation or similar. In fact, Van Gent (1999) never used such criteria to define shallow water conditions. He mostly refers to the ratio between the water depth at the toe of the dike and the wave height in deep water conditions. Moreover, the tests that Van Gent conducted are rarely characterized by breaker parameter greater than 7, where most common values of 3 to 6 are found. Therefore a formal contradiction exists between the breaker parameter criterion for shallow waters and the use of Van Gent (1999) equation or equations based on Van Gent (like the one proposed in the Methodology Book that introduces the equivalent slope concept).

Therefore we propose to include an extra criterion in the Methodology Book to identify the conditions as shallow waters: $s_{m-1,0} < 0.01$. This criterion has be added in the flowchart, as well.

Table 3 - Calculation of breaker parameter for different slopes and wave steepnesses

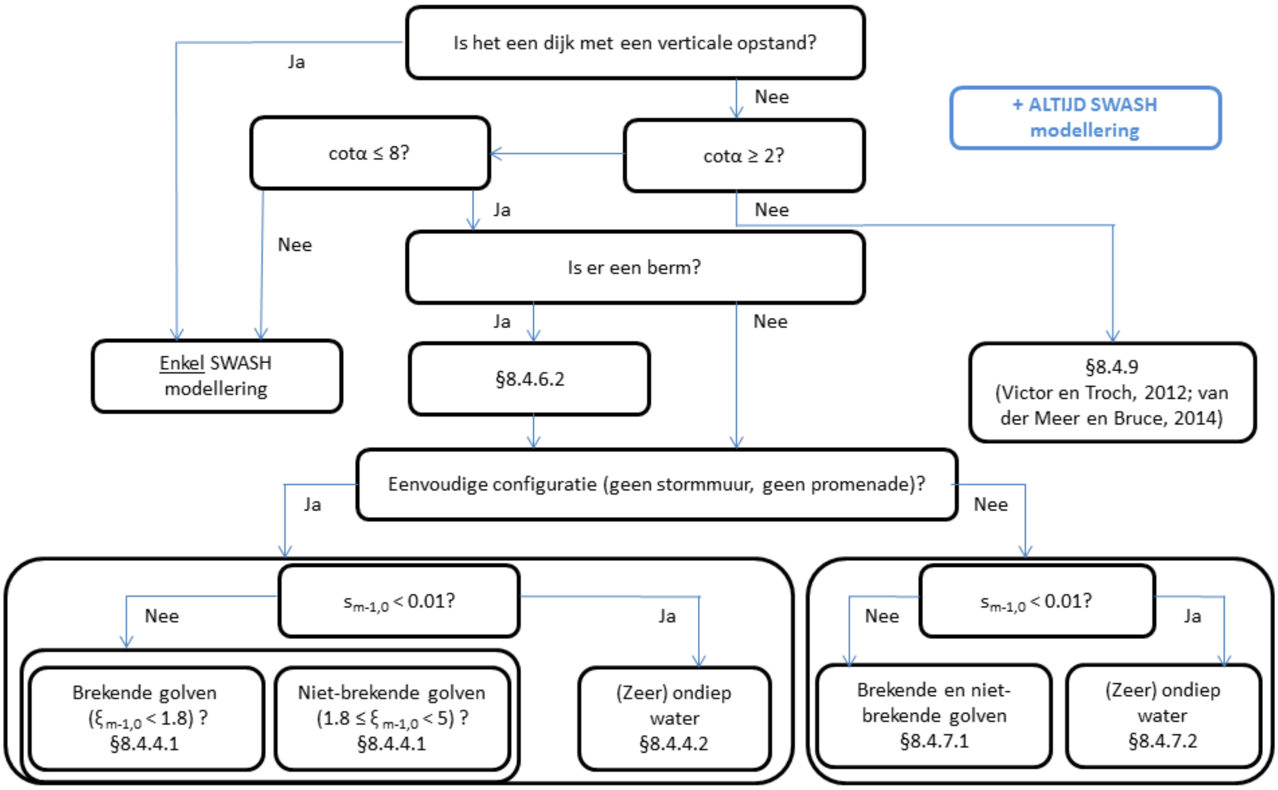
$s_{m-1,0}$	$cot(\alpha)$	$\xi_{m-1,0}$	$cot(\alpha)$	$\xi_{m-1,0}$	$cot(\alpha)$	$\xi_{m-1,0}$	$cot(\alpha)$	$\xi_{m-1,0}$
0.0001	2	50.0	3	33.3	5	20.0	8	12.5
0.0003	2	28.9	3	19.2	5	11.5	8	7.2
0.0005	2	22.4	3	14.9	5	8.9	8	5.6
0.0007	2	18.9	3	12.6	5	7.6	8	4.7
0.0009	2	16.7	3	11.1	5	6.7	8	4.2
0.001	2	15.8	3	10.5	5	6.3	8	4.0
0.002	2	11.2	3	7.5	5	4.5	8	2.8
0.003	2	9.1	3	6.1	5	3.7	8	2.3
0.004	2	7.9	3	5.3	5	3.2	8	2.0
0.005	2	7.1	3	4.7	5	2.8	8	1.8
0.006	2	6.5	3	4.3	5	2.6	8	1.6
0.007	2	6.0	3	4.0	5	2.4	8	1.5
0.008	2	5.6	3	3.7	5	2.2	8	1.4
0.009	2	5.3	3	3.5	5	2.1	8	1.3
0.01	2	5.0	3	3.3	5	2.0	8	1.3
0.011	2	4.8	3	3.2	5	1.9	8	1.2
0.012	2	4.6	3	3.0	5	1.8	8	1.1
0.013	2	4.4	3	2.9	5	1.8	8	1.1
0.014	2	4.2	3	2.8	5	1.7	8	1.1
0.015	2	4.1	3	2.7	5	1.6	8	1.0
0.016	2	4.0	3	2.6	5	1.6	8	1.0
0.017	2	3.8	3	2.6	5	1.5	8	1.0
0.018	2	3.7	3	2.5	5	1.5	8	0.9
0.019	2	3.6	3	2.4	5	1.5	8	0.9
0.02	2	3.5	3	2.4	5	1.4	8	0.9
0.021	2	3.5	3	2.3	5	1.4	8	0.9
0.022	2	3.4	3	2.2	5	1.3	8	0.8
0.023	2	3.3	3	2.2	5	1.3	8	0.8
0.024	2	3.2	3	2.2	5	1.3	8	0.8
0.025	2	3.2	3	2.1	5	1.3	8	0.8
0.026	2	3.1	3	2.1	5	1.2	8	0.8
0.027	2	3.0	3	2.0	5	1.2	8	0.8
0.04	2	2.5	3	1.7	5	1.0	8	0.6
0.05	2	2.2	3	1.5	5	0.9	8	0.6
0.06	2	2.0	3	1.4	5	0.8	8	0.5

Furthermore it is important to include in the flowchart an extra condition on the dike slope, being *cot(α)<8.* In fact, in the Methodology book (as in EurOtop, 2007) a dike is defined having a slope steeper than 1:8. More gentle slopes cannot be classified as a dike. And all the overtopping equations must be applied only for cases where *cot(α)<8.* It might occur, like in the Knokke case, that the final slope, in front of the dike crest, is on average quite gentle. In such a case, only SWASH must be used to determine overtopping. A new proposed flowchart is suggested in Figure 3.

4.4 Methodology in version 10.0

The methodology book version 10.0 specifies overtopping criteria by wave steepness value as shown in Figure 3.

Figure 3 - New flowchart in the methodology book version 10.0



5 Location of the offshore boundary

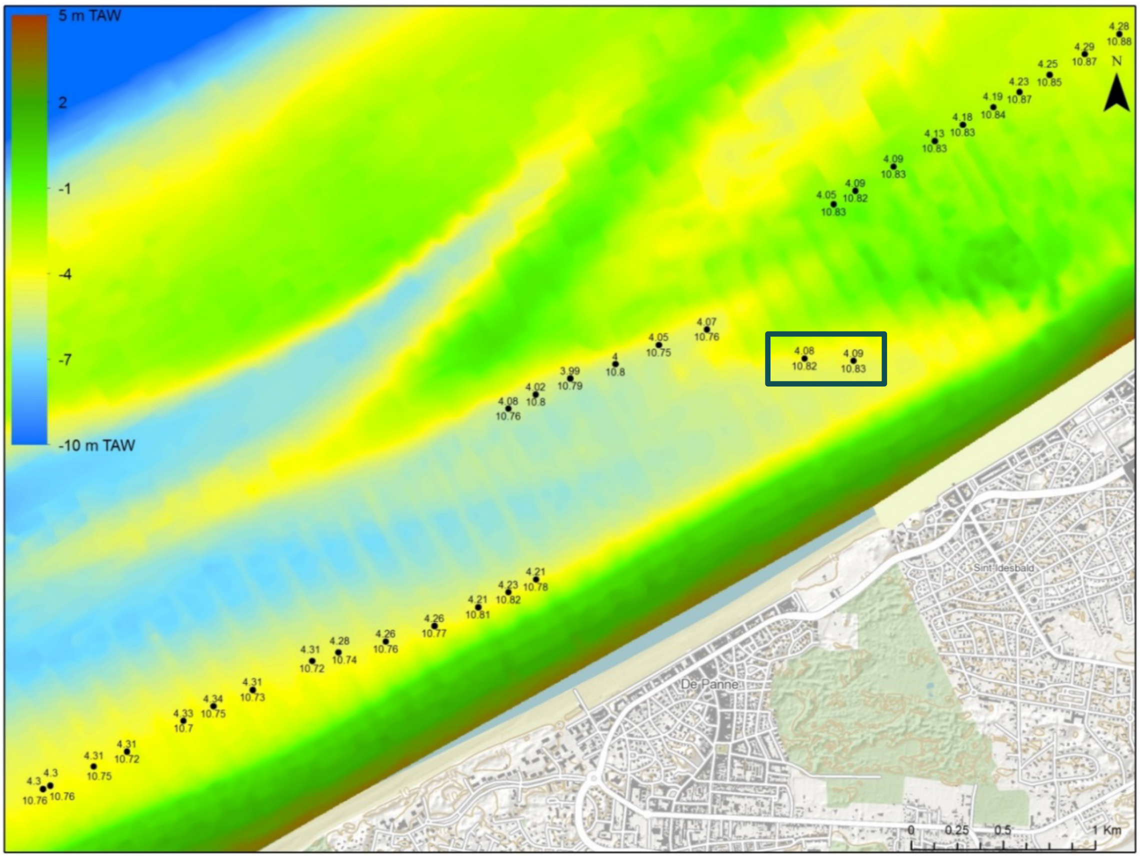
5.1 Background

The offshore hydraulic boundary conditions are input for the XBEACH and SWASH simulations that need to be carried out in the safety assessment, and described in a separate hydraulic boundary condition book (De Roo et al., 2016). These conditions are specified at the first -5 m TAW contour line seaward of the dike or at a normal distance of 1500 m to the dike. The location's choice depends on the near shore bathymetry: 1. in case a depth of -5m TAW is not reached within 1500 m out of the coast, the latter location is opted for and 2. in case sand banks are located near shore, it is decided not to take the -5 m TAW location (if present) but to include this presumed wave breaking area up to 1500 m out of the coast (e.g. coastal section 22).

This second choice is based on additional wave transformation simulations using SWASH 2D. A typical example of a near shore sandbank to be accounted for is the Broersbank, located 100 m out of the coast of Koksijde. As reported in De Roo et al. (2016), the SWAN model is used for wave transformation from offshore to near shore. This spectral, phase-averaging model is not able to fully calculate the effect of this shallow area, e.g. release of bounded long waves (infragravity waves) because of waves breaking on it, whereas the time-dependent phase-resolving SWASH model can account for these phenomena. Therefore, it is decided to shift the location of the hydraulic boundary conditions offshore, up to 1500 m out of the coast. Note that the locations of the hydraulic boundary conditions are indicated in De Roo et al. (2016).

Below, the influence of the boundary location's choice is investigated for coastal section 22 (including the Broersbank), with boundary locations specified at the first -5m TAW and 1500 m out of the coast.

Figure 4 - Hydraulic boundary conditions in the area De Panne – Koksijde.



'Kustsectie 21' and 'Kustsectie 22' are highlighted in the rectangle (source: De Roo et al., 2015).

5.2 Model settings

The bathymetry of a cross shore profile located in coastal section 22 is shown in Figure 5 and illustrates that seaward of the -5 m TAW contour a shallower area extends. Note that the bathymetry offshore the 1500 m line shows an artificial extension with a slope of 1/35 up to -15m TAW (as specified in the methodology book (Appendix A in Suzuki et al., 2016a).

Hydraulic boundary conditions used in the calculations are shown in Table 4, and reflect the new hydraulic boundary conditions for this cross section (see De Roo et al., 2016).

A simulation of 100 waves was carried out using SWASH 2D.

Figure 5 - Cross shore profile located in coastal section 22, used in a SWASH model

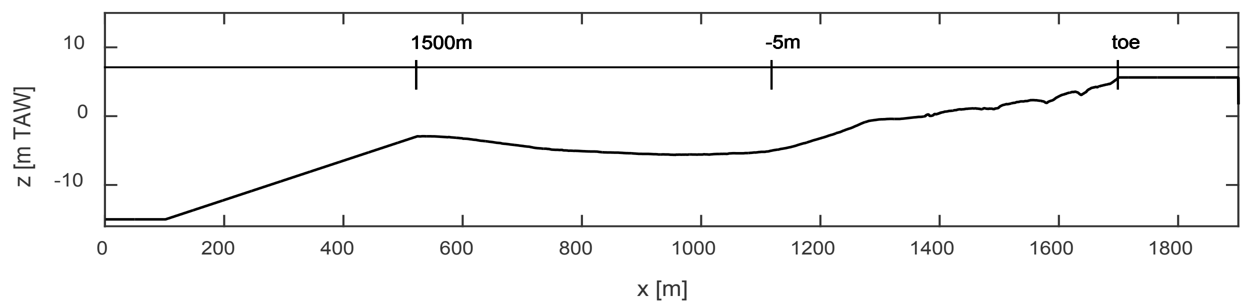


Table 4 - Hydraulic boundary conditions as input for the SWASH calculations

	SWL	Hm0	Tp
Offshore boundary condition	7.10 m TAW	4.1 m	10.8 s

5.3 Result

Table 1 shows the comparison of wave calculation results by SWASH2D using the same bathymetry but different offshore wave boundary points (i.e. one bathymetry is till -5 m TAW= Case '-5m TAW', and the other is till 1500 m= Case '1500m'. On top of that one extra calculation has been conducted to see the effect of the domain size (Case '-5m TAW extended to 1500m'). See detailed results in Figure 6, Figure 7 and Figure 8.

The case '1500m' shows a higher wave height and especially longer wave period at the toe of the dike. This longer period is due to the bump located offshore around x=500 m. The shallow area can cause wave breaking and eventually this can make the wave longer due to the release of the infragravity waves. The extra test result confirms that the domain size does not influence to the result.

Table 5 - Wave parameters at 1500 m, -5 m TAW and at the toe of the dike in each test

Case	Hm0 1500m [m]	Setup 1500m [m]	Tm-1,0 1500m [s]	Hm0 -5m [m]	Setup -5m [m]	Tm-1,0 -5m [s]	Hm0 toe [m]	Setup Toe [m]	Tm-1,0 Toe [s]
-5m TAW	-	-	-	4.10	-0.05	10.5	1.55	0.20	17.5
1500 m	4.10	-0.06	11.0	4.31	-0.05	11.5	1.67	0.19	26.0
-5m TAW ext to 1500m	4.00	-0.03	10.3	4.04	-0.05	10.4	1.55	0.18	16.1

Figure 6 - Case '-5m TAW'

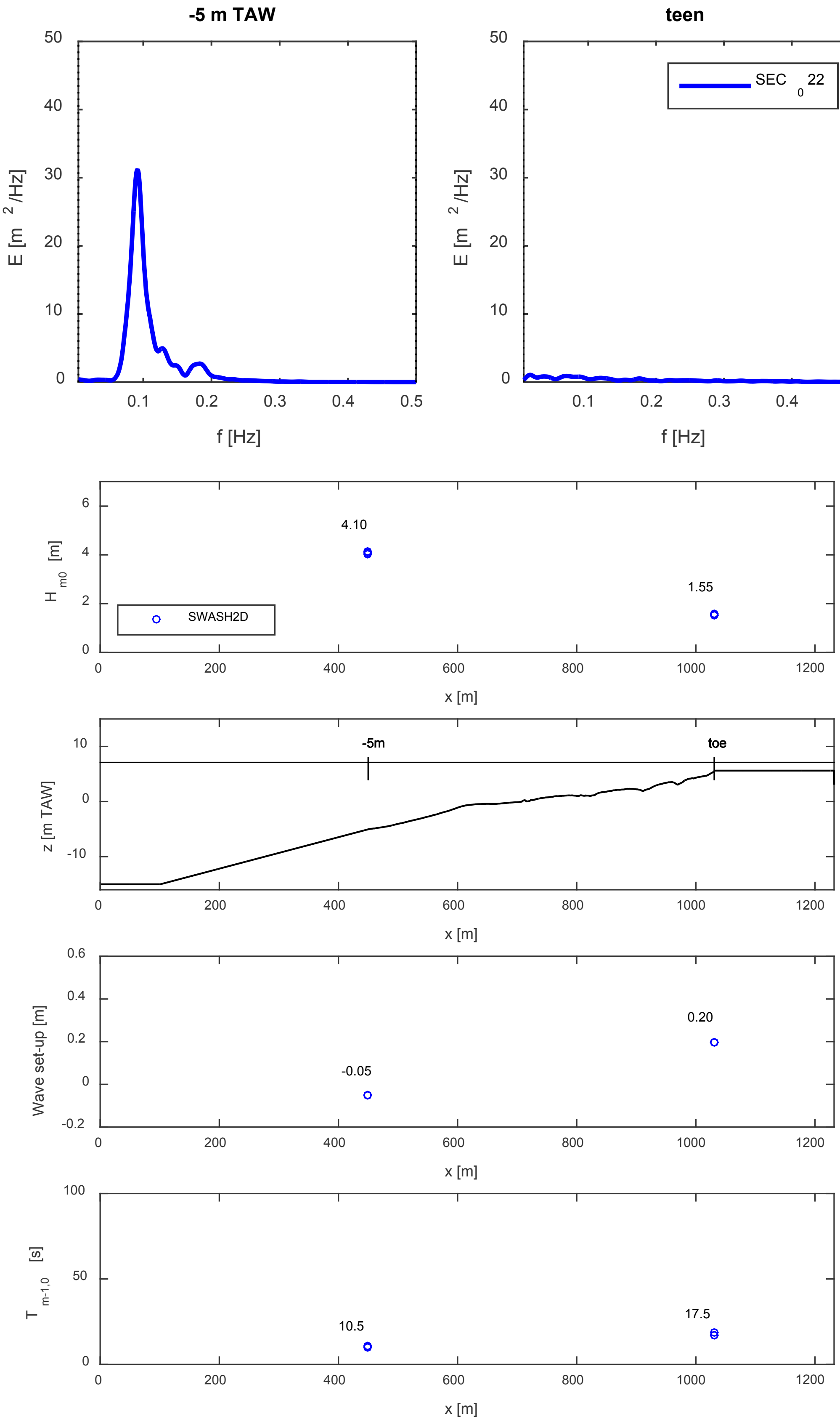


Figure 7 - Case '1500m'

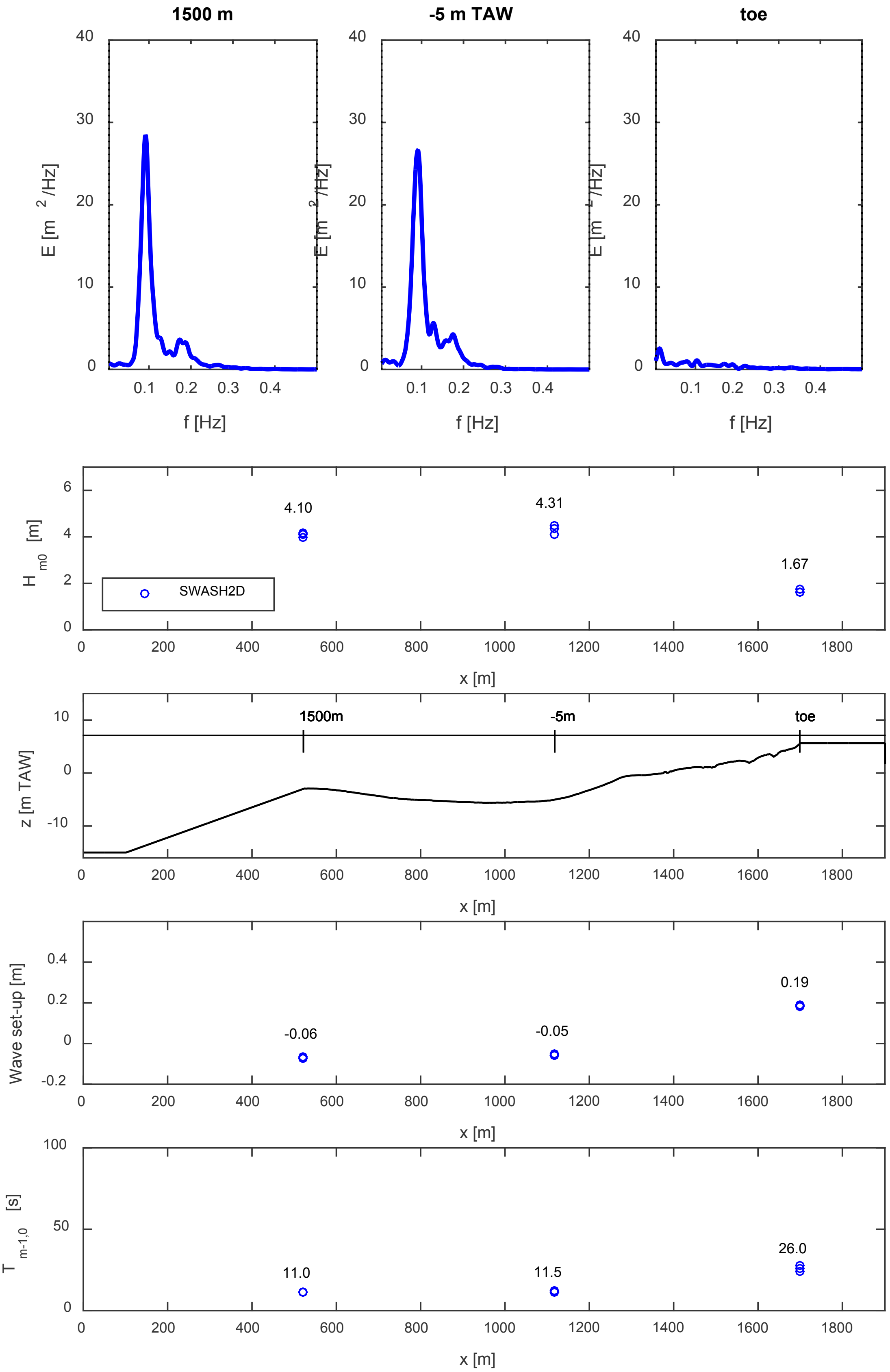
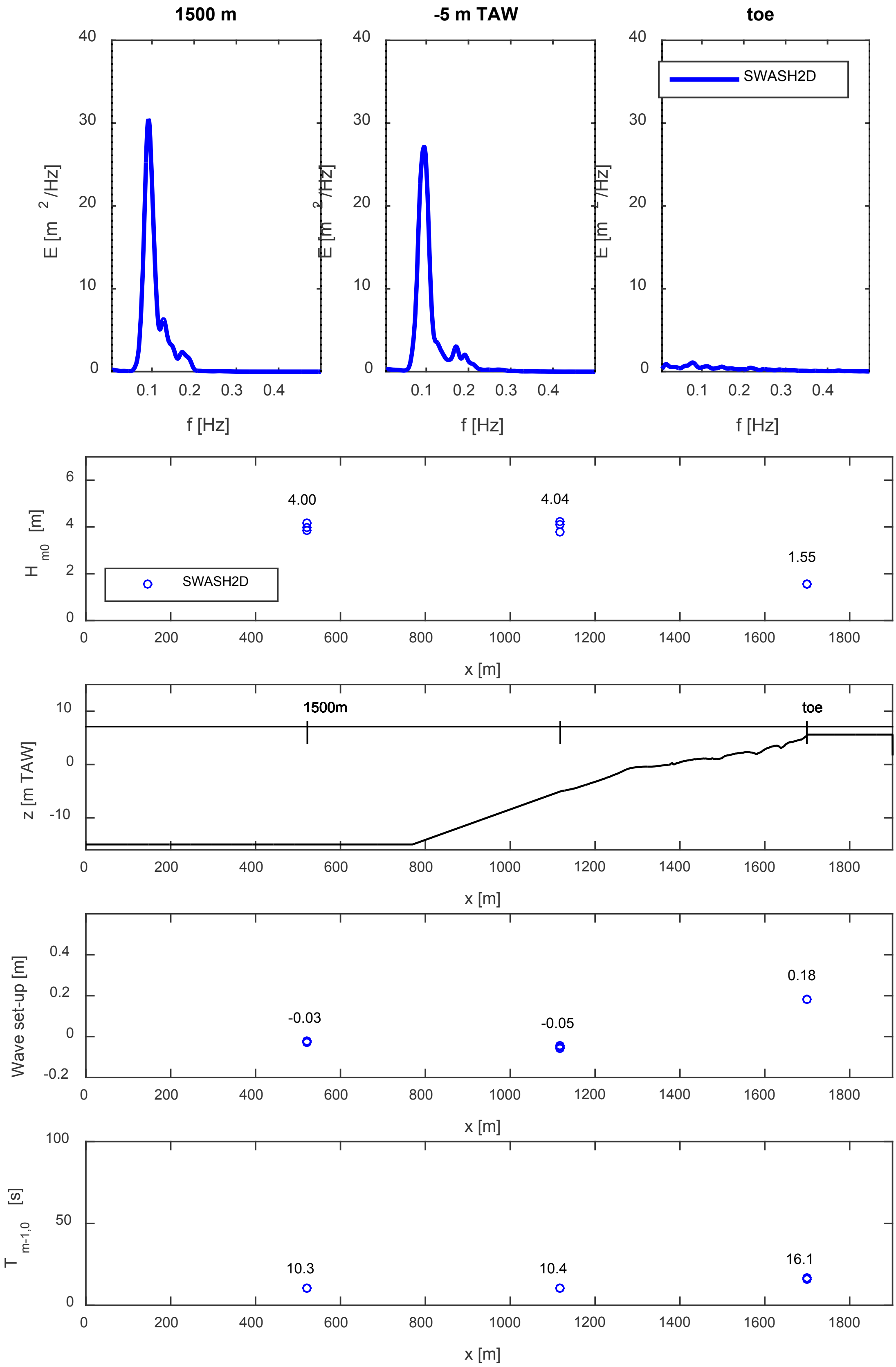


Figure 8 - Case '-5m TAW extended to 1500m'



5.4 Conclusion

The difference of the wave transformation between case 1) offshore boundary condition at the first -5mTAW contour (respective to the dike) and case 2) taking into account the shallow part till 1500 m seaward from the dike is shown. It is not possible to take into account all the sand banks offshore since it is too far, but the effect of a shallower zone within 1500 m should be taken into account for the safety assessment 2015.

It is recommended to check the bathymetry till 1500 m at all the cross sections in which the -5m TAW contour is specified as a boundary point, and if there is a shallower zone than -5m TAW in the domain up till 1500 m, the latter should be used as a boundary point.

6 SWASH version

6.1 Background

During execution of project 13_168, one issue regarding the use of different SWASH versions was raised: “There can be a small difference in the outputs.” Therefore the influence of the SWASH version is investigated here.

6.2 Methodology in version 6.0

The methodology book version 6.0 specifies SWASH version 2.0 or above.

6.3 Influence of the version

Different SWASH versions (i.e. version 2.00, 2.00A, 2.00AB, 3.14) have been tested for SWASH2D calculation.

The calculations have been done using an example cross shore profile located in coastal section 108-2 (from project 13_168). Figure 9 shows the profile 108-2, used as the input for SWASH2D calculation. Table 6 shows the results of significant wave height, spectral period and set-up. After version 2.00AB, the significant wave height and set-up become smaller while spectral wave period stays almost the same. According to developer (personal communication with Dr. Marcel Zijlema), there was a bug in sponge layer setting in 1D calculation before SWASH version 2.00 A. Therefore it is logic to choose the version after 2.00 A. Taking into account the recent update of the SWASH model, it is recommended to use the latest version of the SWASH model.

Figure 9 - Bathymetry of cross section 108-2 used in Raversijde project as SWASH calculation input

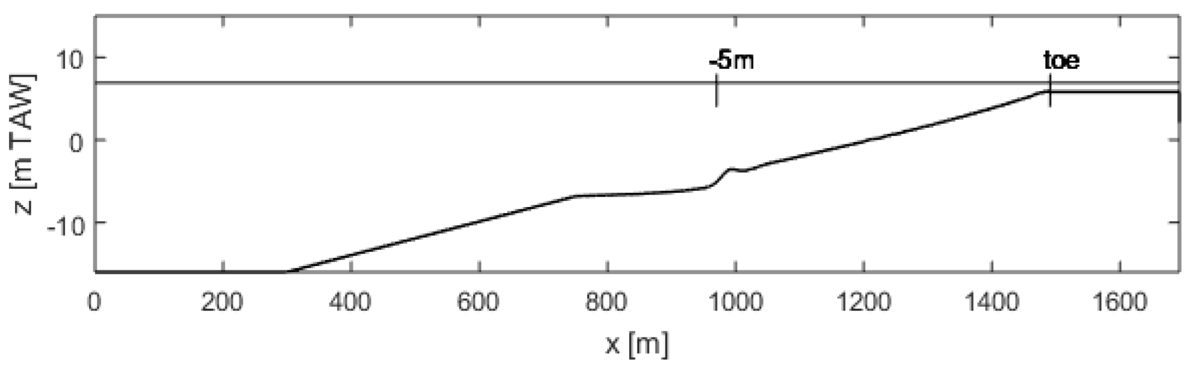


Table 6 - SWASH versions and their output at the toe of the dike

	Number of waves	H_{m0} [m]	$T_{m-1,0}$ [m]	Set-up [m]
SWASH ver2.00	100 waves	1.12	25.4	0.33
SWASH ver2.00	200 waves	1.17	31.9	0.33
SWASH ver2.00A	100 waves	1.12	25.4	0.33
SWASH ver2.00AB	100 waves	1.04	23.6	0.22
SWASH ver3.14	100 waves	1.03	23.8	0.22
SWASH ver3.14	200 waves	1.06	29.4	0.23

6.4 Methodology in version 10.0

The methodology book version 10.0 specifies SWASH version 3.14 should be used.

7 XBEACH model improvements

7.1 Background

In spring 2016, a test case was carried out to verify the contractor's understanding of the methodology (Suzuki *et al.*, 2016a). This test case consisted of the execution of the safety assessment for two coastal sections, i.e. 79: a protected dune section and 81: a dike section (see De Roo *et al.*, 2017).

During this test case, several issues regarding the XBEACH settings arose and were solved, leading to an improved methodology in Suzuki *et al.* (2016a).

7.2 Methodology in version 6.0

7.2.1 XBEACH input

- Artificially extend the 'real' bathymetry alongshore with 1000 m at both sides of the area of interest with a gradually coarser grid size of up to 100 m.
- Use temporally varying offshore boundary conditions, to be varied applying an hourly time step, 3600 s.

7.2.2 XBEACH settings

- Tideloc = 1
- Thetamin/max: ± 90°
- Dtheta: 5°

7.3 XBEACH settings – suggestions & problems

7.3.1 XBEACH input

Previous to the execution of the test case, a technical meeting took place in which following suggestions were made:

- Extend the 'real' bathymetry alongshore with 250 m (= ~2 to 3 wave lengths) at both sides of the area of interest (using the same grid resolution), and then artificially extend it alongshore over 1000 m (at both sides) with a gradually coarser grid size of up to 100 m. By doing so, potentially negative model effects are avoided in the area of interest.
- Use spatially and temporally varying offshore wave boundary conditions.

The first suggestion was immediately agreed upon; the latter suggestion was to be tested within the framework of the test case (De Roo *et al.*, 2017).

7.3.2 XBEACH settings

- Tideloc = 1

'tideloc' needs to be applied in case of a time-varying water level. Setting the keyword to 1 indicates that one time-varying water level is applied to the offshore boundary and a fixed value to the backshore boundary. This however results in a varying water level at the backshore, landward boundaries too, and hence, induces erosion and slumping at higher elevations which impossibly can be related to storm surge wave attack.

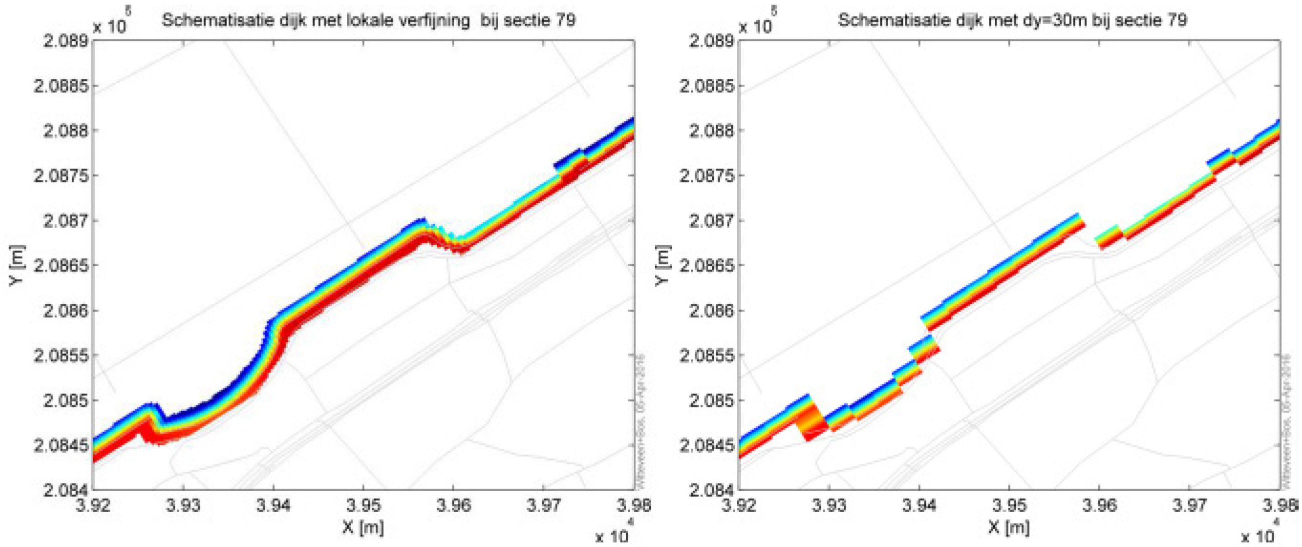
- Jetfaq = 1

'jetfaq' turned on adds turbulence production (and dissipation) to the simulation and hence, the development of a scour hole in front of a hard structure, which might be of interest to verify whether the dike's toe remains covered by sand during the storm event (and thus, dike stability is maintained). Yet, it was not known at the time of writing the methodology book that 'jetfaq' needs to be applied together with the keyword 'swrnup' = 1, which computes short wave runup and consecutive avalanching. The latter setting however includes a bug in the Groundhog Day XBEACH version, to be used in the safety assessment since the WTI settings are only validated for that version.

- Very long computation time (thetamin/max – dtheta)

While executing the test case, the simulation's duration appeared to last very long (> 1 month), much longer than expected. One reason for this issue is the further refinement (dy = 30 m -> 5 m) of the alongshore grid resolution in the vicinity of dike curvatures. In the technical meeting of 08/03/2016 this appeared to be a necessity for accurate modelling (Figure 10).

Figure 10 - Example of refinement of alongshore grid resolution because of a dike curvature.



(left: dy=5 m, right: dy=30 m) (source: Bodde *et al.*, 2016)

Several tests were done to end up with a more acceptable run time.

7.4 XBEACH settings – solution

Table 7 gives an overview of the executed XBEACH simulations. Run 'fin' corresponds to the simulation with settings according to the updated methodology.

Except for 'fast v4-5,7' & 'v8-10', where a reduced bathymetry including 3 cross shore profiles was used, all simulations were executing for the entire model domain.

Note that all simulations were carried out using 1 processor on cluster Stevin; hence without parallel computation.

Table 7 - Overview of the XBEACH simulations carried out to optimise the methodology

XB run	simulation time	# coastal sections	HBC dt	HBC	tideloc	morfac	jf sw	thetamin/max	dtheta
v1	29 d.	74-88	3600 s.	varying	1	1	on (= 1)	± 90°	5°
v4	14 d.	74-88	1800 s.	varying	2	1	off (= 0)	± 60°	5°
v5	14.5 d.	74-88	1800 s.	varying	2	1	on	± 60°	5°
fin	8.5 d.	74-88	1800 s.	varying	2	1	off	± 60°	10°
fast v3	5.5 d.	74-88	1800 s.	varying	1	3	on	± 60°	10°
fast v4	54 min	79 (77-79)	1800 s.	fixed	1	3	on	± 60°	10°
fast v5	57 min	79 (77-79)	1800 s.	fixed	1	3	1 - 0	± 60°	10°
fast v7	57 min	79 (77-79)	1800 s.	fixed	1	3	off	± 60°	10°
fast v8	59 min	79 (83-85)	1800 s.	fixed	1	3	on	± 60°	10°
fast v9	59 min	79 (83-85)	1800 s.	fixed	1	3	1 - 0	± 60°	10°
fast v10	56 min	79 (83-85)	1800 s.	fixed	1	3	off	± 60°	10°
fast v6	5.5 d.	74-88	1800 s.	varying	1	3	off	± 60°	10°
fast v11	5.5 d.	74-88	1800 s.	fixed	1	3	on	± 60°	10°
fast v12	57 min	79 (83-85)	1800 s.	fixed	2	3	on	± 60°	10°
fast v14	59 min	79 (83-85)	1800 s.	fixed	2	3	off	± 60°	10°
fast v15	2.5 d.	74-88	1800 s.	varying	2	3	off	± 60°	10°
fast v16	3 d.	74-88	1800 s.	varying	2	3	on	± 60°	10°

7.4.1 XBEACH input

- Varying hydraulic boundary conditions

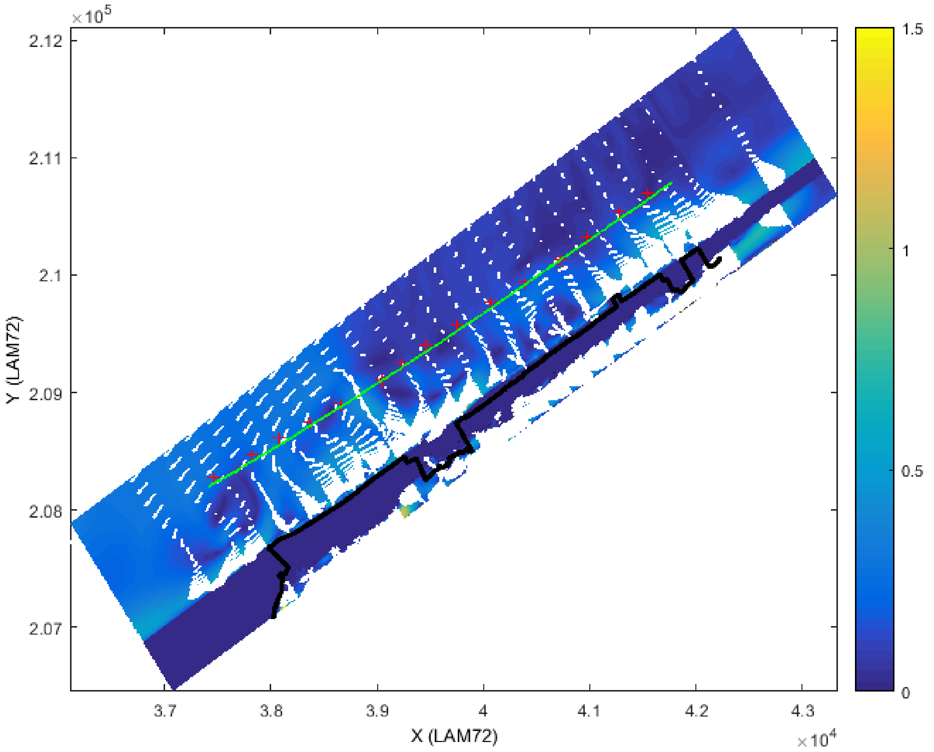
Simulations were executed applying:

A. a time-varying hydraulic boundary condition offshore equal to the maximum significant wave height (+ decimation height) over coastal sections 74-88
B. time- and spatially varying hydraulic boundary conditions, every condition being representative for its respective coastal section.

Figure 11 and Figure 11 show the depth-averaged GLM velocity (= Stokes drift + Eulerian velocity) averaged over the hour around the peak of the storm (between *t*=22 and 23h) for scenario A and B respectively. Both plots are very alike, both in magnitude and direction of the velocities. Note that the result of the 'tideloc' set to 1 is visible by the velocity patterns landward of the safety line.

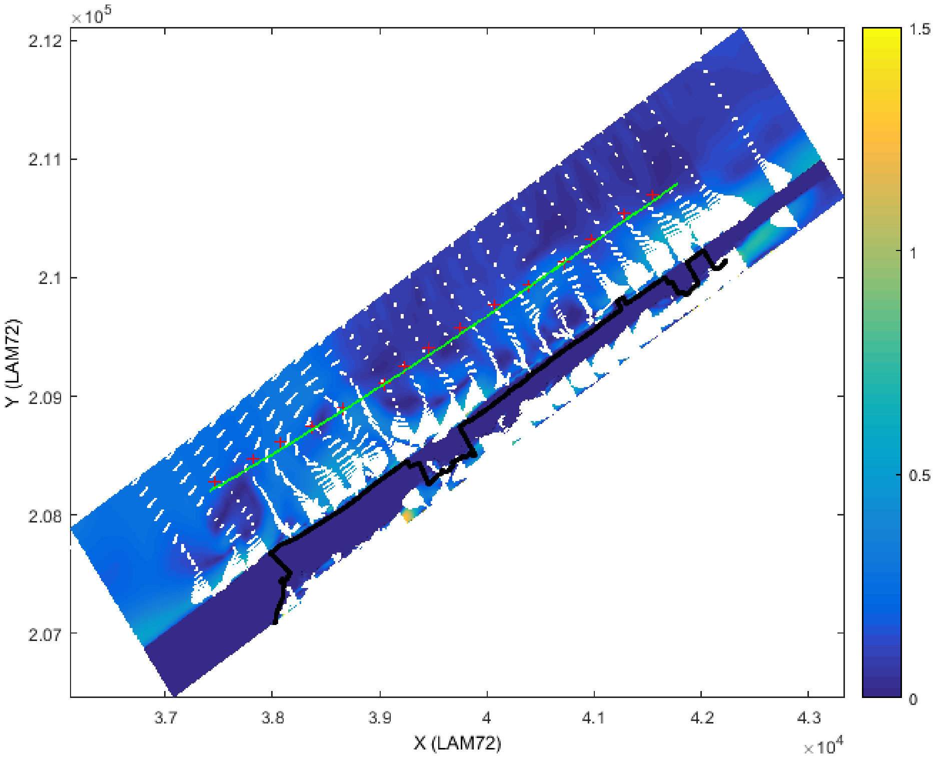
Figure 13, being the difference plot between scenarios A and B, confirms this observation.

Figure 11 - Fixed hydraulic boundary conditions + tideloc = 1



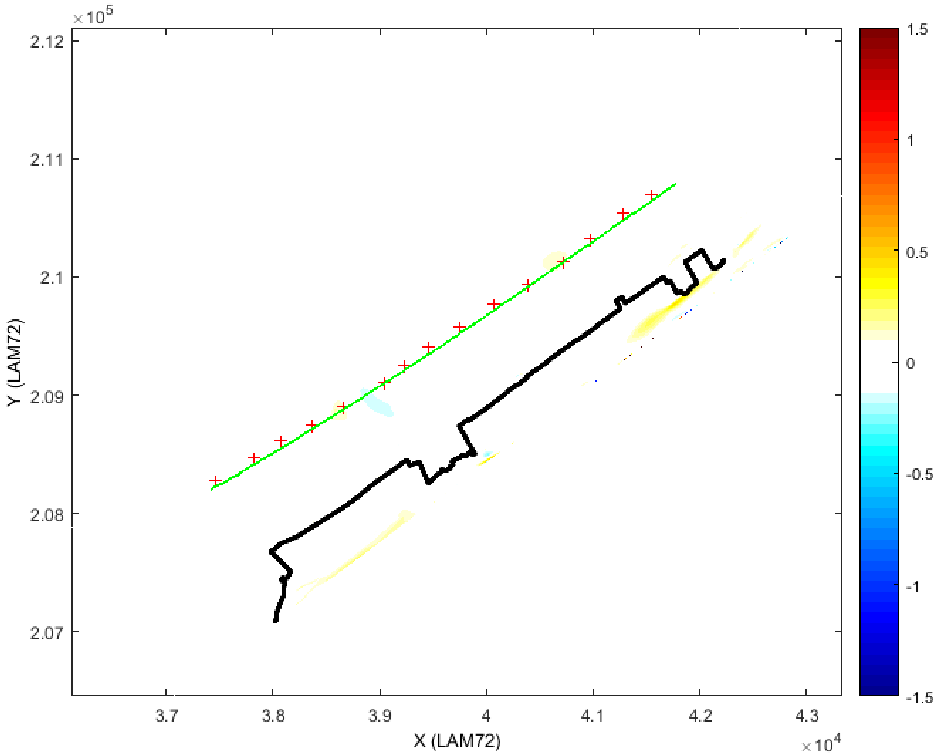
(Average) velocity patterns in coastal sections 74-88 during the peak of the storm (t = 23h).

Figure 12 - Varying hydraulic boundary conditions + tideloc = 1



(Average) velocity patterns in coastal sections 74-88 during the peak of the storm (t = 23h).

Figure 13 - Varying-fixed hydraulic boundary conditions: difference



Difference in average velocity patterns during the peak of the storm (t=23h)

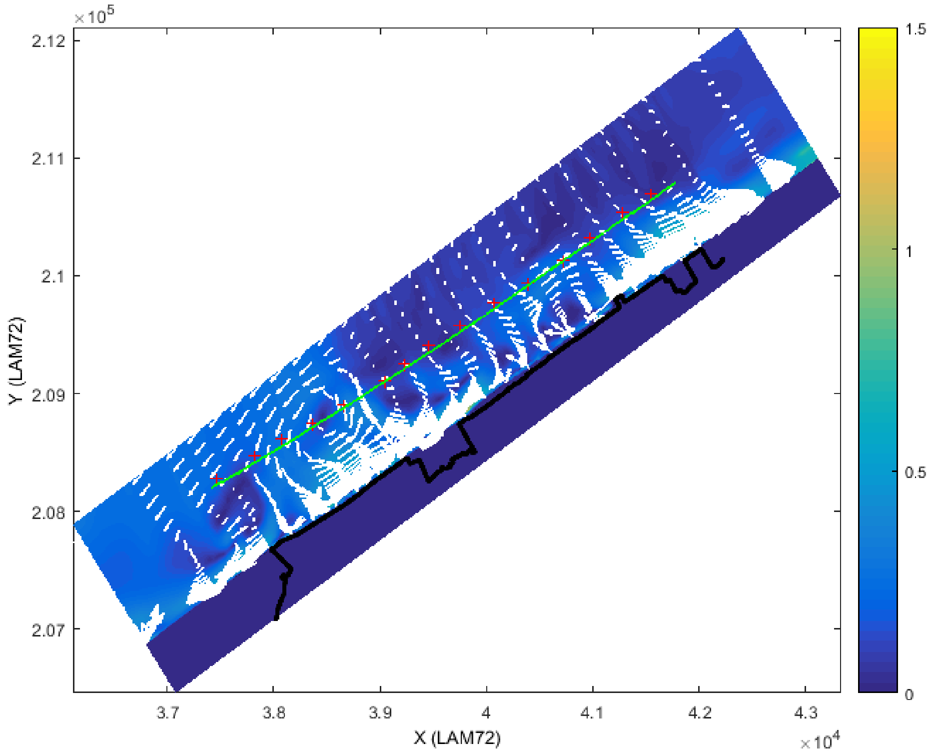
7.4.2 XBEACH settings

- Tideloc = 1 → 2

Changing the keyword 'tideloc' to 2 and adding an extra column = 0 to the 'zs0file' solves the issue (cf. Figure 11). The keyword 'paulrevere' is set to 0 since tide is now specified both at a sea and land boundary.

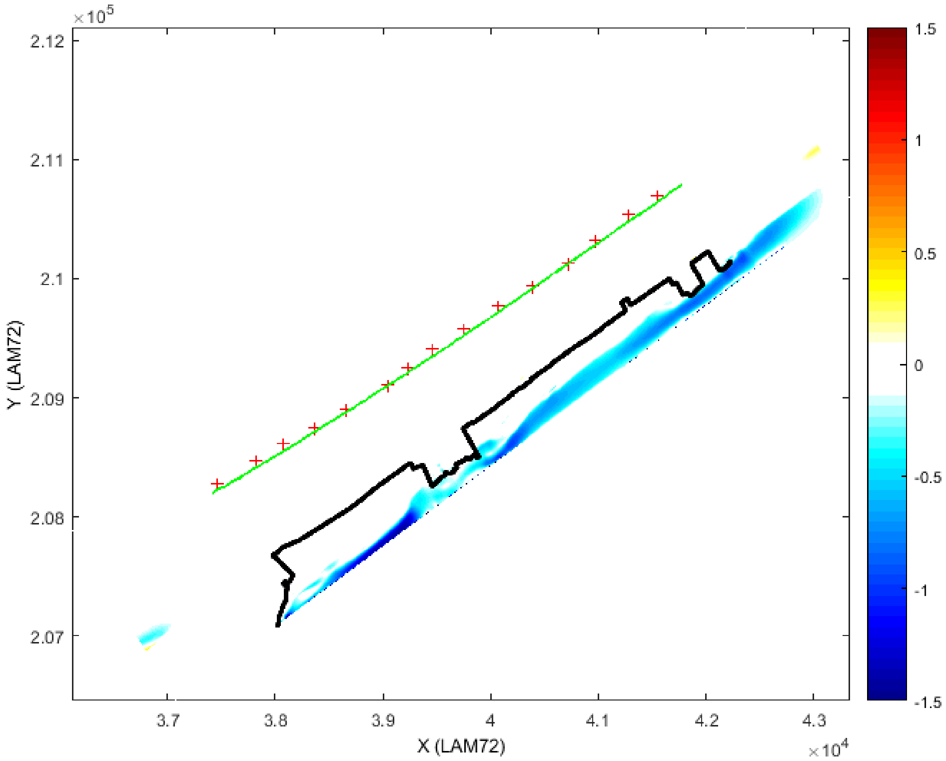
Figure 15, being the difference plot of the average velocity patterns when 'tideloc' is set equal to 1 and 2, indicates that velocity patterns are preserved (being identical) seaward of the coastal structure. Landward, the changed setting results in the desired, dry backshore boundary.

Figure 14 - Varying hydraulic boundary conditions + tideloc 2



(Average) velocity patterns in coastal sections 74-88 during the peak of the storm (t = 23h).

Figure 15 - Varying hydraulic boundary conditions (tideloc 1-2): difference



Difference in average velocity patterns during the peak of the storm (t=23h)

- Jetfaq & swrunup = 0

The combination of these keywords is only important to some coastal sections (located at De Panne and Raversijde) whereas 'jetfaq' might be of interest for all dike sections.

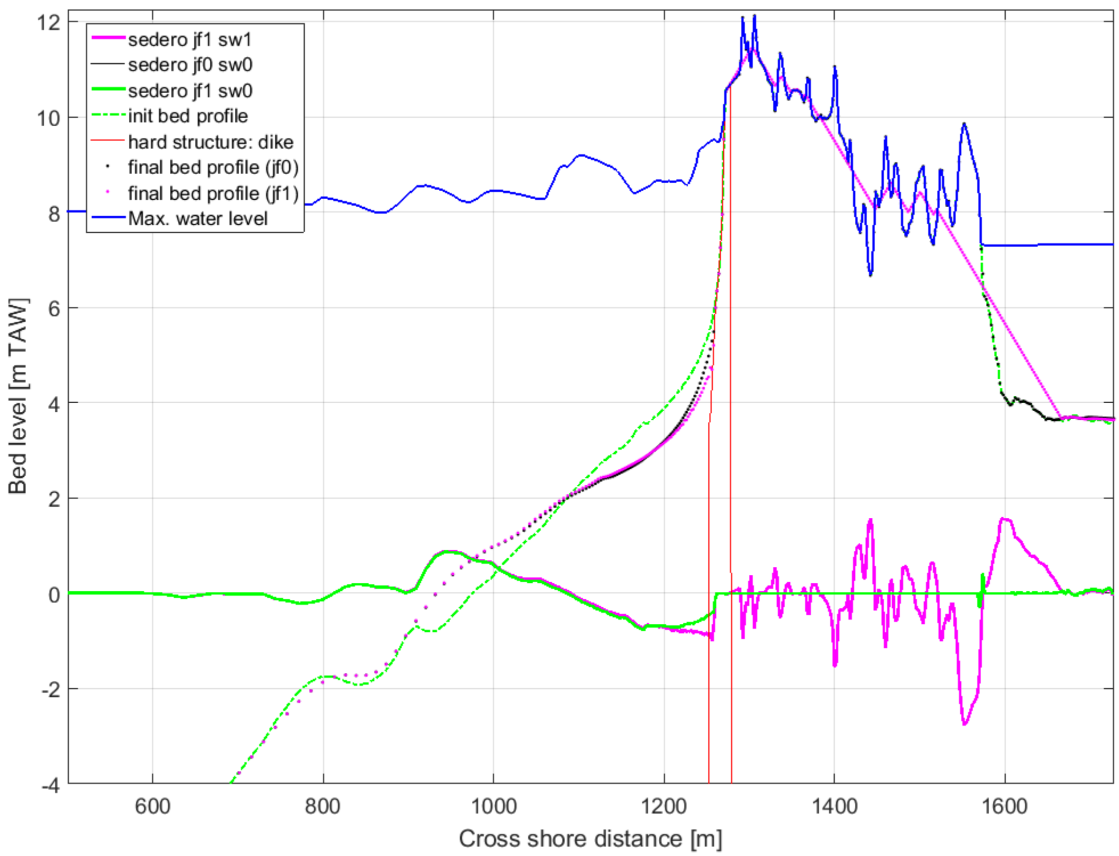
Van Thiel de Vries, 2012 discussed that scour hole development is underestimated in case short wave runup occurs and induces erosion above the revetment. He concluded that using XBEACH 1D and validating the model for 3 laboratory experiments. Van Geer *et al.*, 2014 further adapted the XBEACH 1D model to account for scour hole development (without short wave runup). Based on 3 laboratory experiments (having a fixed water level, identical wave conditions and sediment characteristics of the bed), they confirmed the improved capability of XBEACH 1D to deal with it but also pointed out the need for further validation against e.g. varying hydraulic boundary conditions, and in 2D mode.

Because of 1.) these results, indicating a further need to improve XBEACH (2D) for these processes, and 2.) XBEACH GHD being validated for the WTI settings without these physical processes, it is decided to exclude these keywords from the XBEACH simulations within the framework of the safety assessment 2015.

Figure 16 indicates the difference in sedimentation-erosion patterns turning the keywords 'jetfac' and 'swrunup' on or off. Besides the bug, clearly visible at the landward side of the dike in the cross shore profile, (scour) erosion is slightly increased at the toe of the dike when the keywords are turned on. The difference is however not significant and does not result in dike instability (because of undermining). More seaward, erosion and sedimentation processes are alike.

Note that the different keyword combinations were tested using tideloc = 1, as is noticeable by the backshore water level.

Figure 16 - Coastal section 81: sedimentation/erosion after the 45h storm on a cross shore profile



– keywords 'jetfac' and 'swrunup' on/off

- Very long computation time (thetamin/max – dtheta)

Simulation time can be reduced by adjusting some keyword values:

- Thetamin/thetamax

'thetamin' and 'thetamax' are two keywords linked to the directional limits of the grid for short waves and rollers. First, 'thetamin' and 'thetamax' were set to ±90° with respect to the main wave direction, being normal to the coastline. Given that all wave energy is still captured when narrowing the directional grid up till ±60°, the latter settings are opted for.

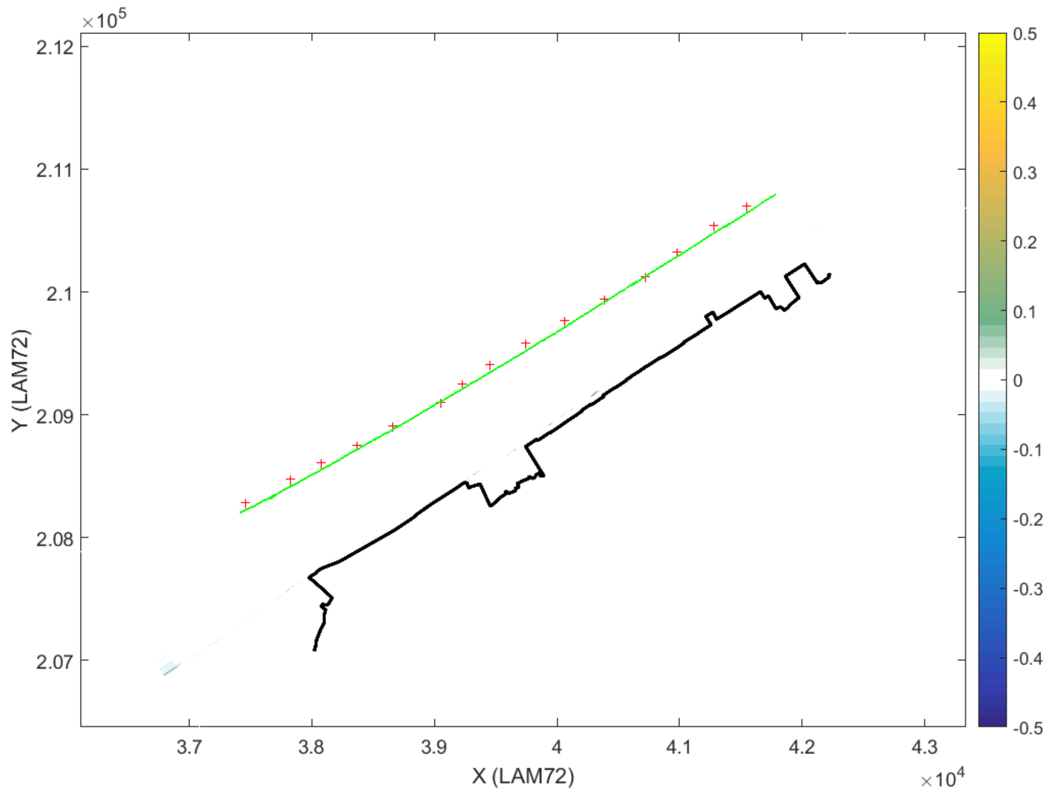
- dtheta

'dtheta' defines the directional bin size. The directional resolution is coarsened to 10° (instead of 5°), i.e. 12 bins. This causes slightly more diffusion for long waves since it reduces the longshore groupiness forced by the radiation stress gradient.

Figure 17 illustrates that there is no difference in sedimentation and erosion pattern applying the different dtheta settings; hence, the coarsening can be applied to speed up simulation time.

Aside, to further speed up simulation time, the wave boundary conditions are imposed with a half hourly time step (1800 s) instead of 3600 s.

Figure 17 - Difference in sedimentation/erosion pattern at the end of the storm (t = 45h) using settings dtheta=10° or =5°



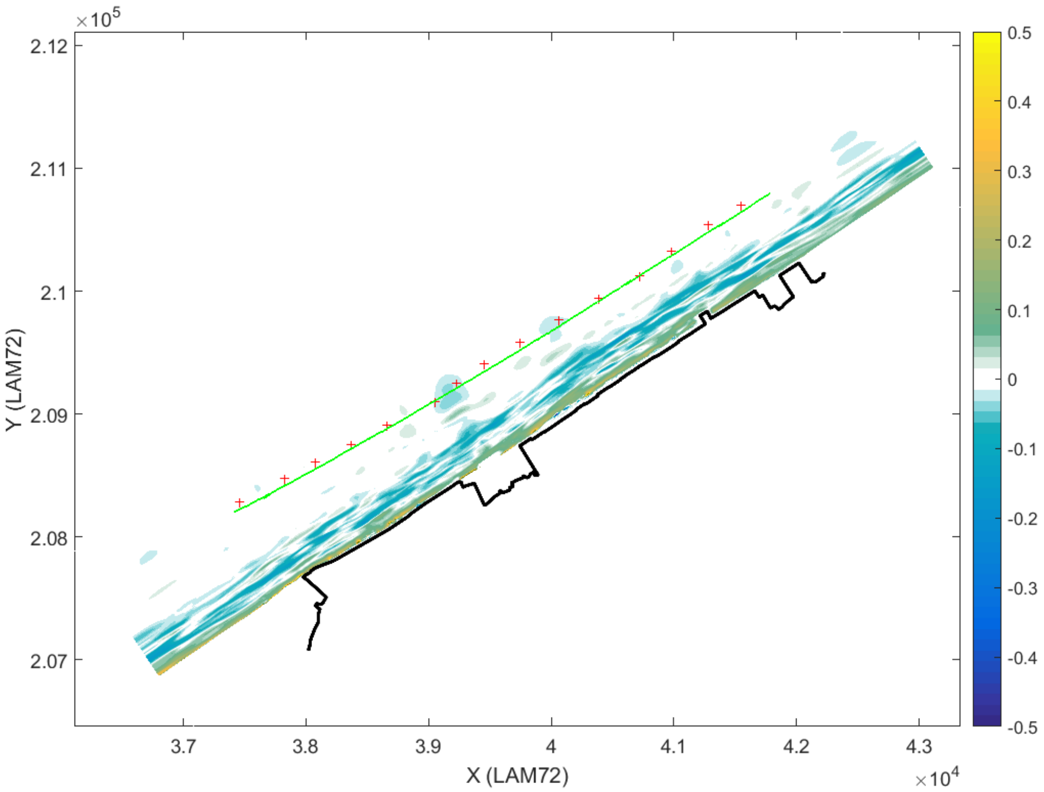
- morfac

In the test case, a morphological acceleration factor (keyword 'morfac') of 3 was applied (cf. Section 3.3 in De Roo *et al.*, 2017). This measure speeds up simulation time but introduces also an erratic increase of erosion/sedimentation (compared to reality).

Figure 18 indicates the differences in sedimentation and erosion patterns between a 'morfac' setting of 1 and 3. Close to the dike, morphological acceleration (times 3 relative to the hydrodynamic time scale) causes increased erosion, resulting in a complementary increase in sedimentation in the intertidal area

This morphological deviation is too high (compared to reality) for the purpose of the coastal safety assessment; hence, the keyword 'morfac' is set to 1.

Figure 18 - Difference in sedimentation/erosion pattern at the end of the storm (t = 45h) using settings morfac=1 or =3



7.5 Methodology in version 10.0

7.5.1 XBEACH input

- Extend the 'real' bathymetry alongshore with 250 m (= ~2 to 3 wave lengths) at both sides of the area of interest (using the same grid resolution), and then artificially extend it alongshore over 1000 m (at both sides) with a gradually coarser grid size of up to 100 m.
- Use spatially and temporally varying offshore wave boundary conditions, to be varied applying a 30 min hourly time step, 3600 s.

7.5.2 XBEACH settings

- Tideloc = 2
- Thetamin/max: ± 60° with respect to main wave direction
- Dtheta: 10°

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