Flanders
State of
the Art

14_014_4
FHR reports

Safety assessment 2015 – test case

Execution of the test case and evaluation of the contractor’s

DEPARTMENT
**MOBILITY &
PUBLIC
WORKS**

www.flandershydraulicsresearch.be

Safety assessment 2015 – test case

Execution of the test case and evaluation of the contractor's

De Roo, S.; Suzuki, T.; Vanneste, D.; Peeters, P.; Mostaert, F.

Flanders
Hydraulics Research

Flanders
State of the Art

Cover figure © Eddy Baete

Legal notice

Copyright and citation

D/2017/3241/149

This publication should be cited as follows:

De Roo, S.; Suzuki, T.; Vanneste, D.; Peeters, P.; Mostaert, F. (2017). Safety assessment 2015 – test case: Execution of the test case and evaluation of the contractor's. Version 3.0. FHR Reports, 14_014_4. Flanders Hydraulics Research: Antwerp.

Document identification

Customer:	Agentschap Maritieme Dienstverlening en Kust - Afdeling Kust	Ref.:	WL2017R14_014_4
Keywords (3-5):	XBEACH; erosion; SWASH; wave transformation, wave overtopping; coastal safety		
Text (p.):	25	Appendices (p.):	6
Confidentiality:	☐ No	☐ Available online	
	☒ Yes	Released as from:	01/01/2020
		Exception:	☒ The Government of Flanders

Author(s):	De Roo, S.; Suzuki, T

Control

	Name	Signature
Reviser(s):	Vanneste, D.	
Project leader:	Suzuki, T.	

Approval

Coordinator research group:	Peeters, P.	
Head of Division:	Mostaert, F.	

Abstract

A safety assessment is executed around every 6 years in order to evaluate the current status of coastal safety. Since the previous safety assessment (International Marine and Dredging Consultants, 2008), improved knowledge is gained regarding numerical techniques to calculate morphological evolution, wave transformation and wave overtopping. Besides, computational resources have improved significantly (e.g. better computational performance of PC’s and workstations, methods for parallel computation have become available). As a result, the methodology to carry out a safety assessment has been re-evaluated and updated (Suzuki *et al.*, 2016).

Prior to the safety assessment of all coastal sections, a test case was commissioned by Coastal Division to check the interpretation of the methodology by the contractor. This test case comprised the execution of the safety assessment for two coastal sections, i.e. 79: a protected dune section and 81: a dike section.

Flanders Hydraulics Research was asked to execute this test case concurrently with the contractor, in order to compare the results, which might also differ because of different post processing tools and compiler. This report includes the FHR test case results, and the comparison with those of the contractor.

Contents

List of tables

List of figures

1 Introduction

1.1 Background

In 2011, a Master Plan Coastal Safety was framed with the goal to protect the coast against flooding because of severe weather events, which are defined by the normative storm having a return period T = 1000 year (De Roo *et al.*, 2016). The plan stipulated that a coastal safety assessment needs to be carried out every 6 years.

Following the first safety assessment in 2007 (International Marine and Dredging Consultants, 2008), a new assessment is taking place in the course of 2016, using data of 2015; hence, it is named 'Safety Assessment 2015'.

Prior to the safety assessment of all coastal sections, a test case was commissioned by Coastal Division to check the interpretation of the methodology by the contractor. This test case comprised the execution of the safety assessment, applying the methodology as described in Suzuki *et al.*, 2016, for two coastal sections, i.e. 79: a protected dune section and 81: a dike section.

Flanders Hydraulics Research was asked to execute this test case concurrently with the contractor, in order to compare the results.

1.2 Reader's guide through the document

Chapter 2 describes the study area and its hydraulic boundary conditions.

Chapter 3 elaborates on the morphological evolution, modelled by XBEACH. Using these results, i.e. the post-storm bathymetry, wave transformation is modelled using SWASH 2D. Chapter 4 discusses the 2D input, model settings and results, whilst Chapter 5 deals with the calibration of these obtained incident hydraulic boundary conditions at the toe of the dike in SWASH 1D. Chapter 6 outlines the wave overtopping calculation.

Chapter 7 wraps up the outcome of the safety assessment for both tested coastal sections 79 and 81.

2 Study area

Two coastal sections close to one another are selected, i.e. coastal section 79 and 81. These sections are situated between the coastal villages Westende and Middelkerke and characterized by:

- **Section 79**

- Dune with dune foot protection: the beach ends partly in a small dune area before the dike, which acts as a dune foot protection since this hard structure hampers erosion of the dunes behind.

- The safety line is located at the back of the dunes, and defined by (the seaward side of) housing.

- Probable low dune situation: vertical distance between dune crest and water level + decimation height < 6m

- **Section 81**

- Dike: the beach is backed by a dike, with promenade and apartments behind

- The safety line is located at the seaward side of the apartments

Figure 1 Coastal section 79 – a dune with dune foot protection: beach, ‘dike’like dune foot prediction, dunes and houses (source: GISviewer W+B ‘BELA634’)

(a) Beach and dike slope

(b) Dike crest and dunes behind

Figure 2 Coastal section 81 – a typical sea dike: beach, dike, promenade and apartments (source: GISviewer W+B 'BELA634')

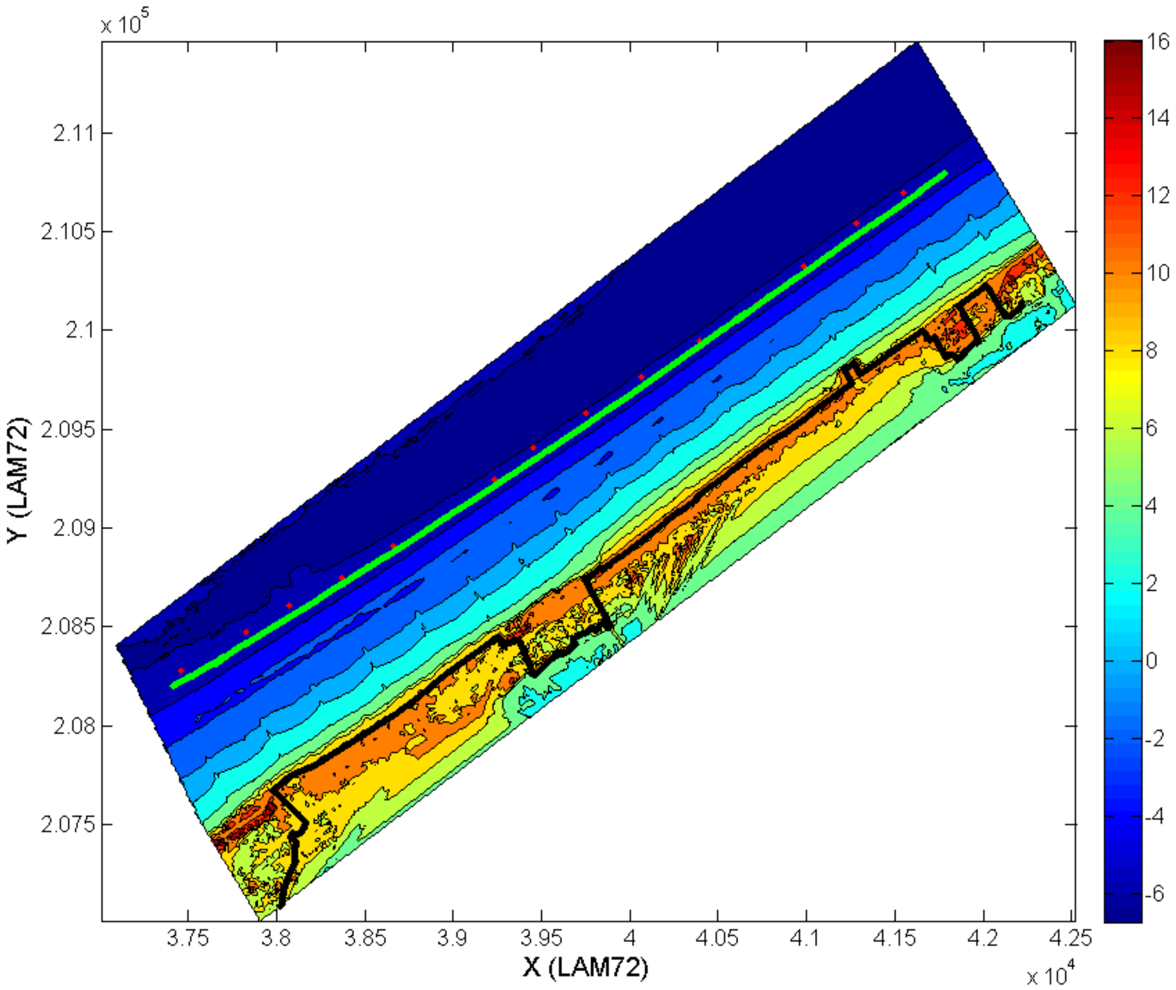
Figure 3 Coastal sections 74-88; zoom on sections 79 and 81, of which the safety will be assessed (green line: safety line, blue line: blue stone line) (source: GISviewer W+B 'BELA634')

2.1 Bathymetry

One of the deliverables of phase 1 in the safety assessment 2015 was a Digital Elevation Model (DEM) of the coastal bathymetry and of the hard structures located along the coast (e.g. sea dike, harbour dams). The input for these models, and the models themselves, are described in Bodde & Cornu, 2016. From these DEMs, the required data is extracted. Figure 4 visualizes the area of interest.

In the safety assessment's model chain, this bathymetry is an input for XBEACH modelling. Subsequently, the post-storm bathymetry (= XBEACH output) is used as input for SWASH modelling. Both coastal sections are part of the same XBEACH model, i.e. model 7, which stretches out from 74 to 88 (cf. Table 4-2 in Bodde & Cornu, 2016). Therefore, the considered 2D bathymetric layout takes into account this entire model area.

Figure 4 Digital Elevation Model [m TAW] of coastal sections 74-88 (black line: safety line – green line: -5m TAW contour)



2.2 Hydraulic boundary conditions

Depending on the physical process to be calculated, hydraulic boundary conditions are used with/without accounting for the decimation height (Table 1). For SWASH modelling, only the boundary conditions of the coastal sections under study are considered; for XBEACH modelling, all conditions related to coastal sections of model 7 need to be taken into account.

Table 1 Hydraulic boundary conditions of coastal sections 74-88 (De Roo *et al.*, 2016)

Coastal section	SWASH modelling (T = 1000 yr)			XBEACH modelling (T = 1000yr + 2/3 decimation height)		
	Significant wave height H_{m0} [m]	Wave peak period T_p [s]	Water level h [m TAW]	Significant wave height H_{m0} [m]	Wave peak period T_p [s]	Water level h [m TAW]
74				4.95	11.36	7.32
75				4.93	11.32	7.32
76				4.87	11.4	7.32
77				4.94	11.33	7.31
78				4.82	11.23	6.98
79	4.82	11.23	6.98	4.95	11.43	7.31
80				4.8	11.09	6.98
81	4.80	11.09	6.98	4.87	11.4	7.3
82				4.86	11.43	7.3
83				4.87	11.41	7.3
84				4.92	11.41	7.29
85				4.94	11.41	7.29
86				4.9	11.39	7.29
87				4.84	11.38	7.28
88				4.95	11.36	7.32

3 Morphological evolution (XBEACH 2D): sedimentation and erosion

3.1 Input: bathymetry

Appendix C of Suzuki *et al.*, 2016 outlines the bathymetric adaptations to be made in order to set up the XBEACH model domain. In summary:

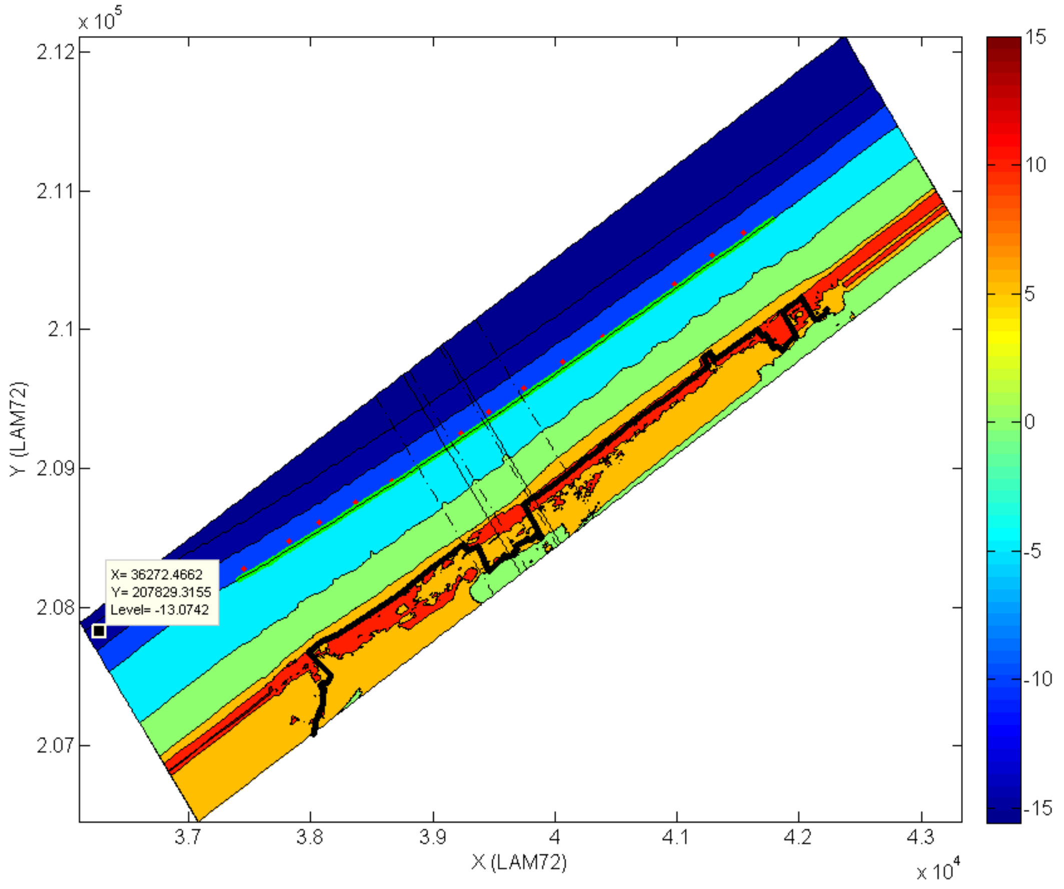
1. The -5m TAW contour line is uniquely defined in coastal sections 74-88. The largest cross shore distance between the safety line and -5 m TAW (~1.3 km) on the one hand, and the alongshore distance between the section transitions 73-74 and 88-89 (~5 km) on the other hand, determine the central part of the model domain.
2. The artificial extension between -5 m TAW and -15 m TAW, having a slope of 1:35, and the adjacent 100 m cross shore horizontal plane at -15 m TAW on the one hand, and the 100 m added bathymetry behind the (most landward point of the) safety line on the other hand, complete the model domain between the coastal sections of interest.
3. To avoid model effects in this area, the model domain is laterally extended (at both sides) with 250 m real and 1000 m artificial bathymetry, of which the bottom profile is identical to the one of the lateral boundary of the real bathymetry.

First, the model domain between the coastal sections of interest is built up by interpolating the bathymetry (of 1. and 2.) on a coarse grid. Second, the cross-shore grid resolution is optimized using the `xb_generate_model' tool, which adapts dx based on bathymetric variations. Third, applying this optimized cross-shore resolution dx (varying between dx = 2-20 m) and the imposed alongshore grid resolution dy (here: dy is uniformly 30 m, no transition zones in (or bordering) 79 and 81), the 'real part' of the XBEACH model domain is established. By gradually enlarging the alongshore resolution up to dy = 100 m, the artificial lateral extensions are added to the XBEACH model domain. This final XBEACH model domain is characterized by 386 cross shore grid cells and 206 alongshore grid cells (input file: bed.dep). Average grain size is $319 \cdot 10^{-6}$ m; $D_{90} = 478 \cdot 10^{-6}$ m.

XBEACH model 7 includes both dike and dune sections, the latter having a dune foot protection. The geometry of these hard structures, being non-erodable bed layers, is accounted for in a separate inputfile (nebed.dep). Note that, since only coastal section 79 is assigned as a dune section and its dune foot protection is characterized as a dike, it is presumed that this hard structure does not fail. Hence, no XBEACH calculation is executed without this dune foot protection (cf. Table 4-2 in Bodde & Cornu, 2016).

Figure 4 shows this adapted bathymetry for XBEACH model 7.

Figure 5 XBEACH bathymetry [m TAW], including coastal sections 74-88 and lateral extensions



3.2 Input: hydraulic boundary conditions

Since the model comprises several coastal sections, spatially varying wave boundary conditions are opted for. By doing so, they match their respective coastal section.

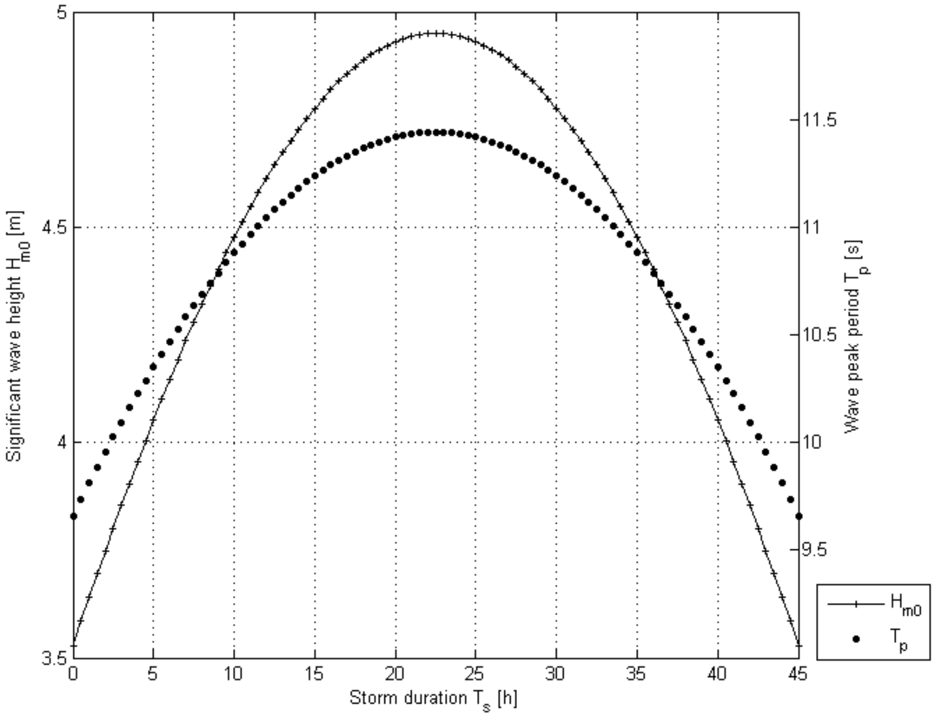
Significant wave height H_s and wave peak period T_p respectively vary over the duration t (= 45h) of the normative storm (T_s = 125h) (Figure 4) as:

$$H_s(t) = H_{s,max} \cos^2\left(\frac{\pi \cdot t}{T_s}\right)$$

$$T_p(t) = T_{p,max} \cos\left(\frac{\pi \cdot t}{T_s}\right)$$

In which $H_{s,max}$ and $T_{p,max}$ are the maximum significant wave height and peak period respectively (cf. Table 1)

Figure 6 Time-varying wave boundary conditions during storm of 45h (example of CS 79)



Wave conditions are applied as a JONSWAP spectrum with peak enhancement factor γ = 3.3; directional spreading is 16° (s-value of 25) (De Roo *et al.*, 2016). The main wave direction, normal to the coast, is 326°. Short wave energy is discretized in 10° directional bins (between the upper and lower limits of the directional grid: ±60°. These varying wave conditions are simulated as 90 wave trains of 30 min (1800 s), every wave train being discretized over constant time intervals of 0.5 s.

Water level variation *h* during the normative storm of 45h (= T_s) depends on the astronomical tide *AT* and time-varying storm surge *O:*

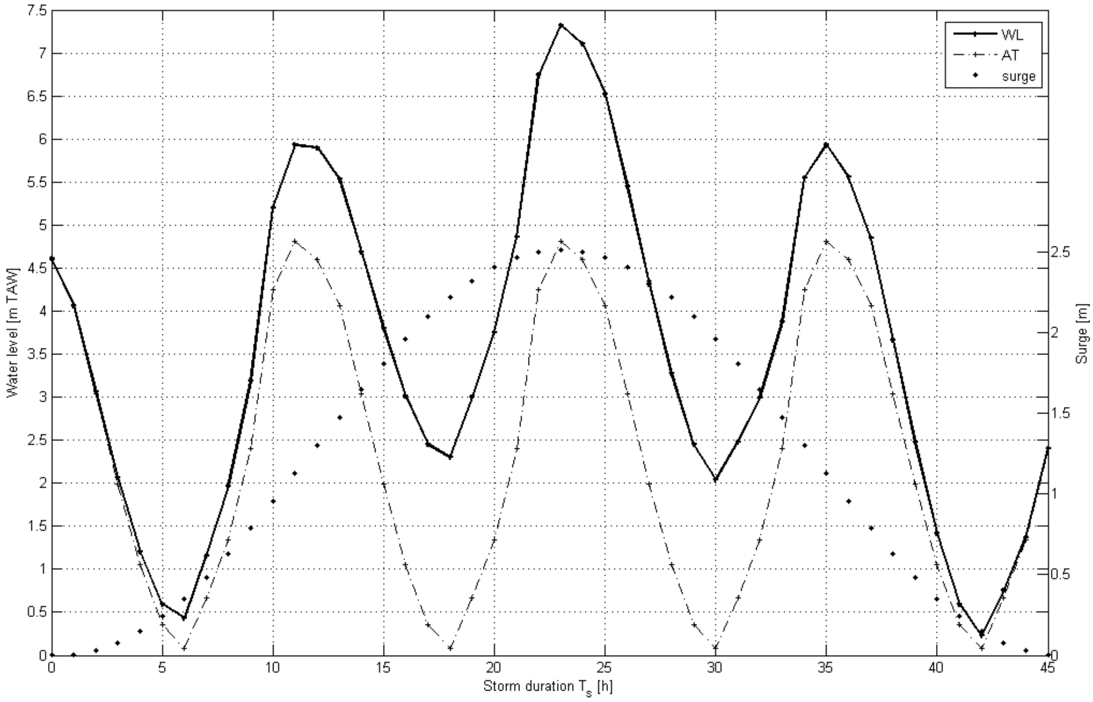
$$O(t) = O_{max}\cos\left(\frac{\pi \cdot t}{T_s}\right)$$

In which O_{max} is the maximum storm surge, being the difference between the maximum water level *h* (cf. Table 1) and the astronomical tide at this specific location (interpolated between the astronomical tides of harbours Oostende and Nieuwpoort, to be found in Vlaamse Hydrografie, 2017).

The varying water level is calculated per hour (3600 s), having a maximum water level h_{max} = +7.32 m TAW (of CS 74-77, being the highest value in the model domain) at the peak of the storm for the entire model.

In the safety assessment, a symmetrical water level variation during the storm event is assumed, implying that the maxima of these hydraulic boundary conditions occur concurrently (cf. Figure 6 and Figure 7).

Figure 7 Time-varying tidal boundary conditions during storm of 45h (valid for entire XBEACH model)



3.3 XBEACH: model settings

The XBEACH input file is added in Appendix A, parameters not listed are used as 'default' values. Input parameters are referred to in the previous subsections. Morphological parameters are specified in Table 6-1 of Suzuki *et al.*, 2016. The keyword 'morstart', indicating the starting time of the morphological processes is set to start immediately, in case some spinup time for the hydrodynamics is taken into account it could also be set to 3600s, i.e. one hour.

To speed up the test case simulation, the 'morfac' (morphological acceleration) keyword was set to 3. This implies that the morphological time scale is accelerated by factor 3 relative to the hydrodynamic time scale. Being linearly correlated with the computational time, the latter is reduced by factor 3. This however results in a 10% (deviating) increase of erosion (in reality) because of the rapidly varying water levels which induce additional alongshore currents (because of influencing the inertia terms). The simulation time is accordingly reduced to obtain an approximate result (choice of 'morfacopt = 1').

The parameters 'jetfac' and 'swrunup', taking into account turbulence production (e.g. scour hole development in front of dike) and short wave runup respectively, are turned on. A known bug, related to 'swrunup', occurs in the Groundhog Day version, causing erosion at the landward side of (dry) dunes. 'Jetfac', the parameter of our interest, does not work without 'swrunup' and hence, the bug interferes with the desired output. Furthermore, the development of the scour hole has been limitedly validated up till now (using 2 lab experiments).

The aforementioned arguments motivate the decision to exclude both parameters from the XBEACH model settings for the safety assessment, see detailed information in Suzuki *et al.*, 2017.

The current simulation in this study took 3 days and 2 hours on 1 processor.

3.4 Output: sedimentation and erosion

Figure 6 indicates the sedimentation/erosion patterns for the coastal sections 74-88. Close to and at the dike/dune foot protection erosion takes place, as well as next to the groynes. At the beach, roughly up to the low water line, sedimentation occurs. More offshore, the bathymetry does not change significantly. Erosion can amount up to 3 m, and is highest for the coastal sections adjacent to a dune area (coastal section 73) or backshore nourished. Sedimentation on the intertidal area can add up to 1 m.

Figure 7 and Figure 8 show the weakest profiles in coastal sections 79 and 81 respectively (see also the black lines in Figure 6). Although no significant erosion took place in these cross sections, they are selected based on their low crest level (of the dike or dune), and the bed level of the sandy beach nearby. Since the crest of the eroded profile is situated lower than 8 m TAW, the surcharge of 25% does not need to be taken into account.

Note that landward of the safety line (alongshore black line in Figure 6) or landward of the dike/dune foot protection (in Figure 7 and Figure 8), the bug's influence can be seen by the artificial sedimentation/erosion patterns that e.g. occur at higher dune areas never reached by the storm surge.

Figure 8 Sedimentation/erosion patterns in coastal sections 74-88 after the 45h storm.
between the dash-dotted lines: assessed coastal sections 79 and 81, weakest profiles: thick black line. The green line marks the -5 m TAW contour, red crosses the locations of the wave boundary conditions, the alongshore black line the safety line.

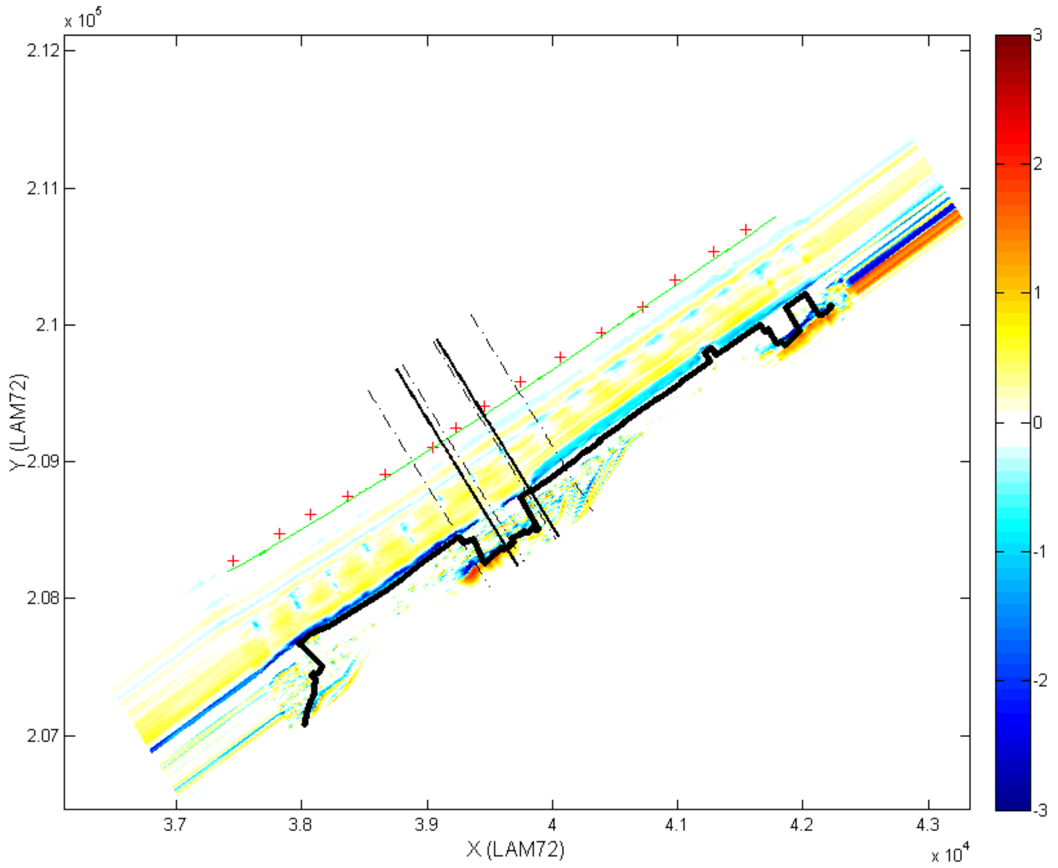


Figure 9 Initial and post-storm bathymetry in the weakest profile of coastal section 79 after the 45h storm (T = 1000yr + dec), with indication of the maximum storm surge and the resulting post-storm sedimentation/erosion pattern.

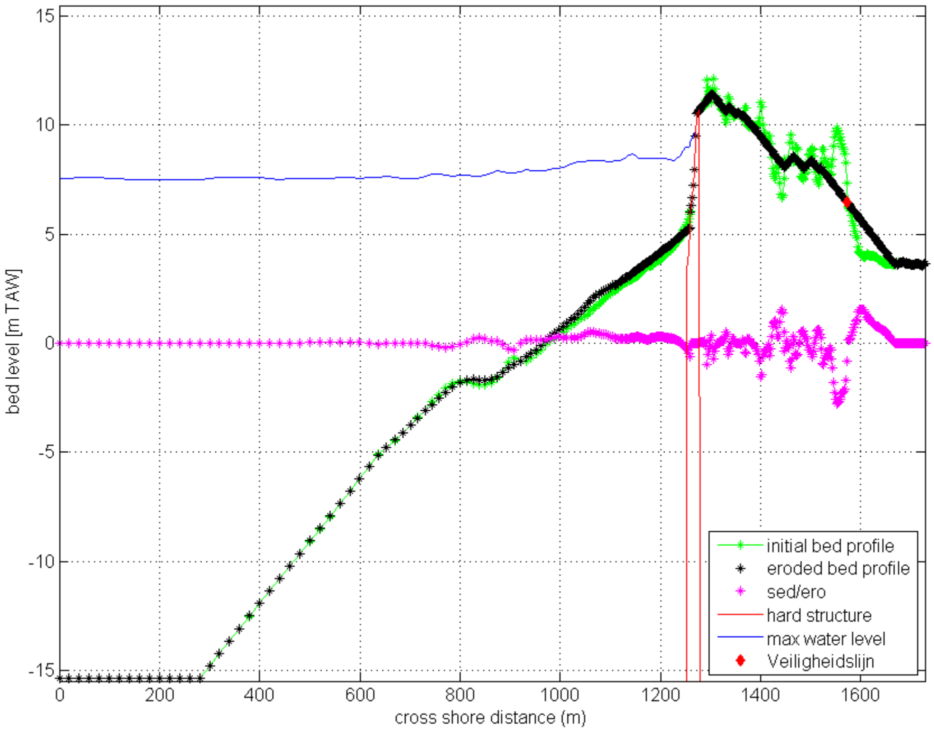
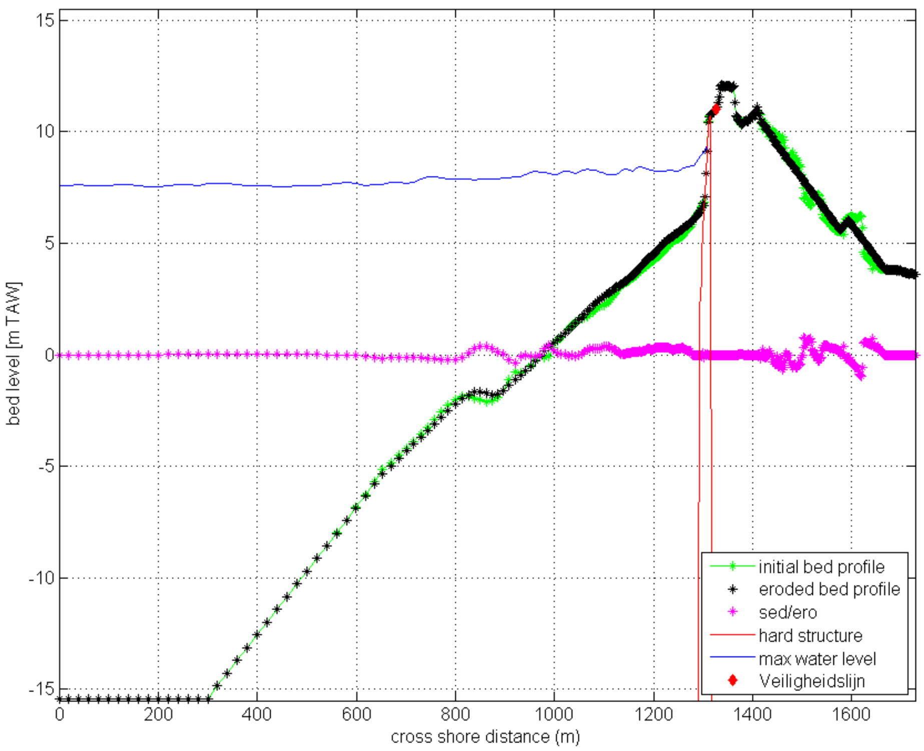


Figure 10 Initial and post-storm bathymetry in the weakest profile of coastal section 81 after the 45h storm (T = 1000yr + dec), with indication of the maximum storm surge and the resulting post-storm sedimentation/erosion pattern.



3.5 XBEACH: comparison FHR – W+B results

3.5.1 Bathymetry: morphological change

Figure 11 indicates the differences between the FHR and W+B bathymetries (where the W+B bathymetric values are interpolated on the FHR grid). Marked differences are not observed, although below and above the -5 m TAW contour the FHR bathymetry has a bed level somewhat lower and higher compared to W+B's respectively.

These differences result from:

- Difference in alongshore grid resolution *dy*:

  dy_{FHR} = 30 m, since no transition zones need to be taken into account -> ny = 206

  dy_{W+B} varies between 5 and 30 m, finer grid resolution in transition areas (*dy* = 10 m) and around dike curvatures (*dy* = 5 m, optional, based on the 2D geometry instead of assumed 1D) -> ny = 468.

- Difference in main grid orientation (related to main wave direction, normal to the coast):

  FHR: 326° - W+B: 328°

  error in FHR grid (thetamin/thetamax) of 10° (has no implications on results)

- Different end cross section of real bathymetry results in different lateral extension into the artificial bathymetry
- Different start- and ending point of the artificial 1:35 slope seaward of the -5 m TAW contour:

  FHR: starts at first real bed level value deeper than -5\,m TAW and extends up to first real value deeper than -15 m TAW

  W+B: cuts of at -5 m TAW and hence, extends up to exactly -15 m TAW

Sedimentation/erosion patterns of both simulations are very similar; Figure 12 indicates the differences in morphological change. Although the bathymetries (mainly) differ in the seaward part of the model, almost no difference in sedimentation/erosion patterns is observed at that location (which is logic since the closure depth is located at a higher bed level). In general, differences between both model results are small, i.e. in the range of ±0.2 m, and primarily owing to the differences in alongshore grid resolution and grid orientation.

Figure 11 Difference [m] between the FHR and W+B XBEACH bathymetries

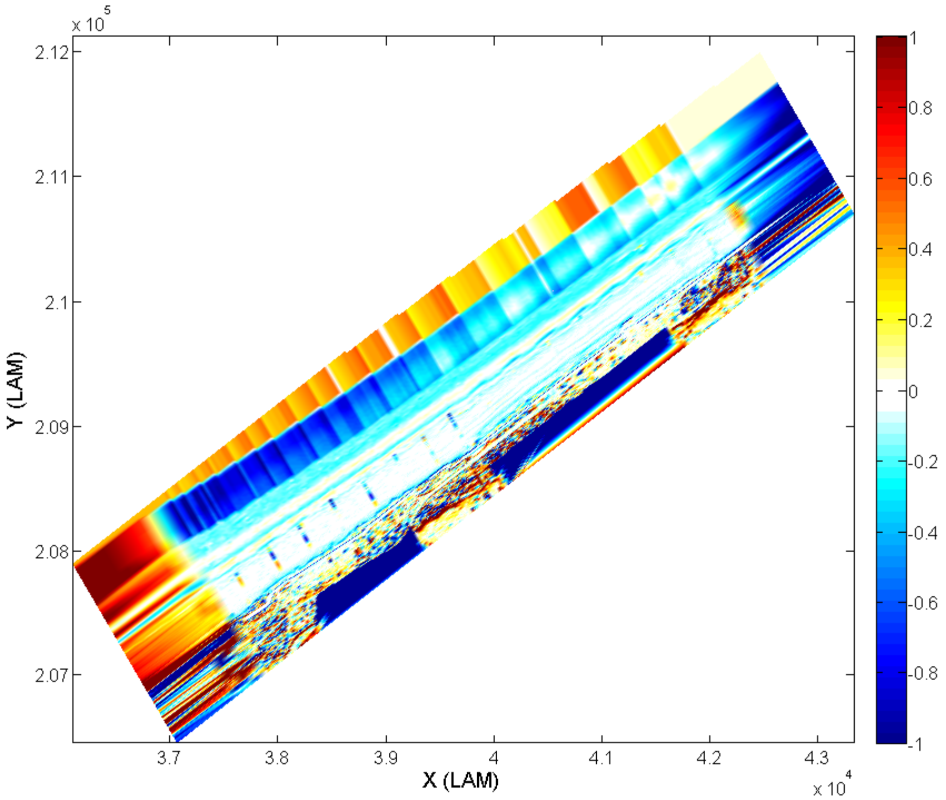
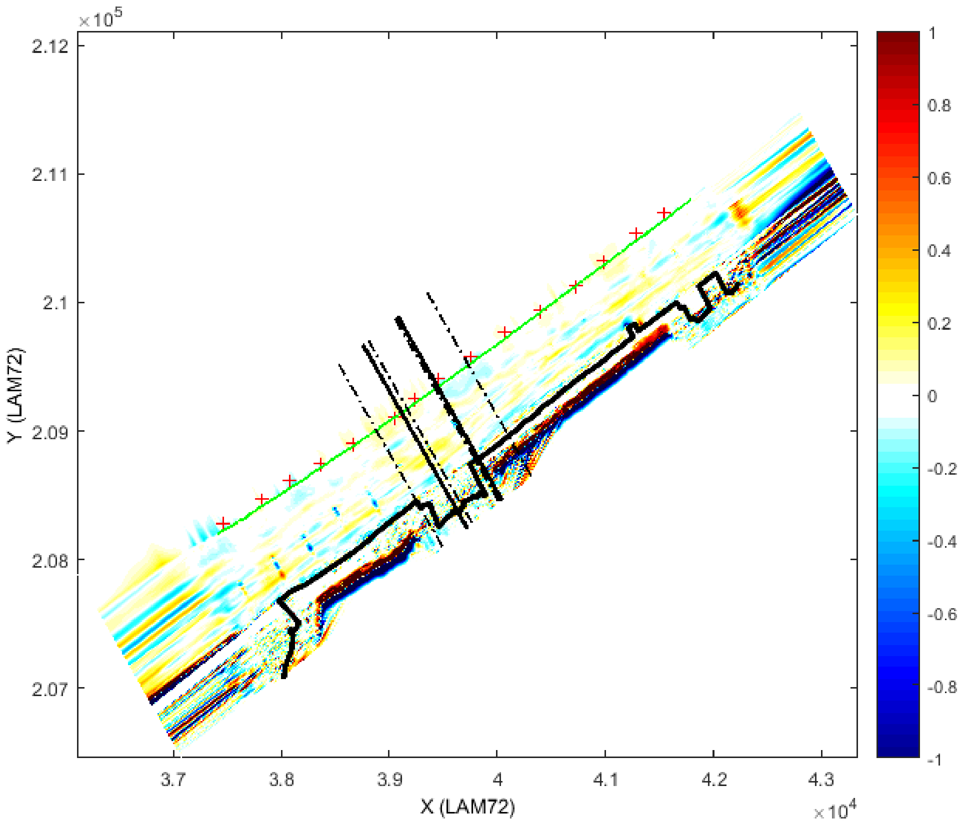


Figure 12 Difference [m] in morphological change between the FHR and W+B XBEACH simulations



3.5.2 Weakest profiles in coastal sections 79 and 81

Figure 13 and Figure 14 illustrate the difference in sedimentation/erosion patterns on the cross-shore weakest profile of coastal section 79 and 81 respectively. To verify the simulation's accuracy, the FHR weakest profiles are used as base for comparison. For both cross shore profiles, the results are very close to one another.

Note that the W+B weakest profile of coastal section 79 differs from FHR's because of the different alongshore grid resolution and the coastal section being less uniform in geometry. Their profile is located close to coastal section 78 while FHR's to 80, which results in a +10.35 m TAW and +12.12 m TAW level of the crest of the dune respectively. Nonetheless, both assessment results for this coastal section are identical (see Sections 6.5 and 7.1).

Figure 13 Coastal section 79: difference in sedimentation/erosion patterns on the weakest cross shore profile

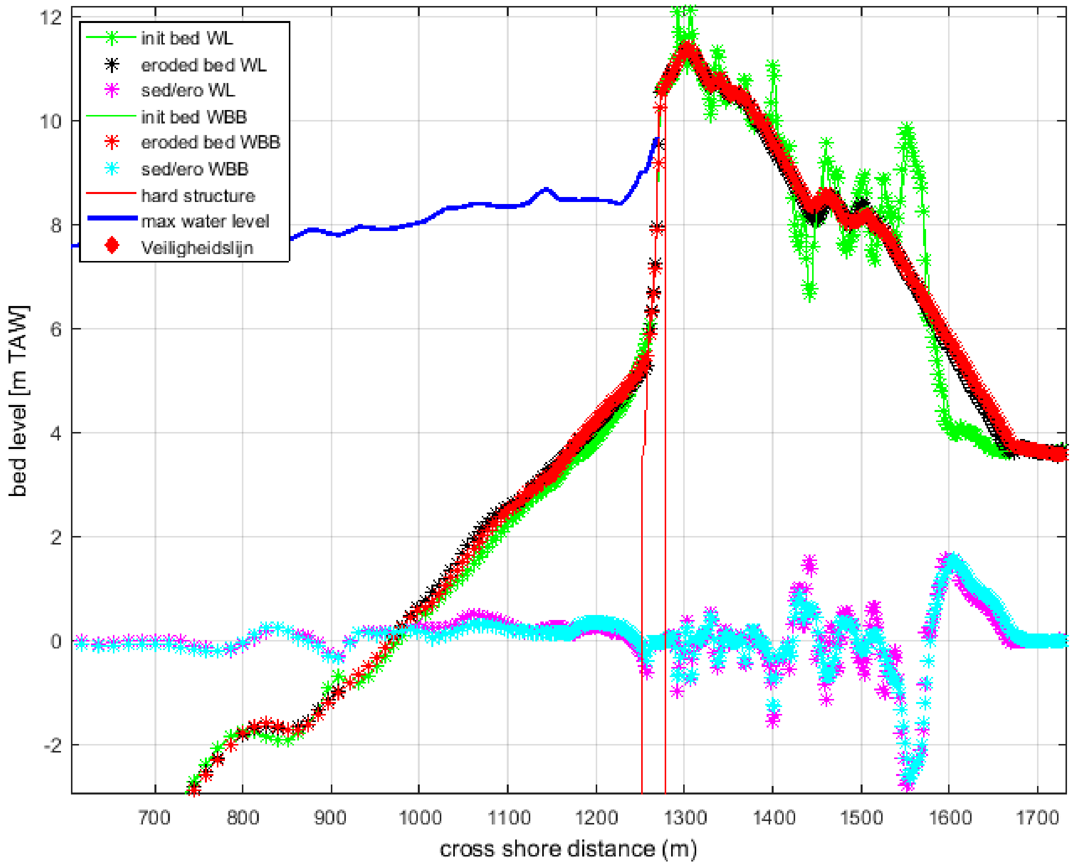
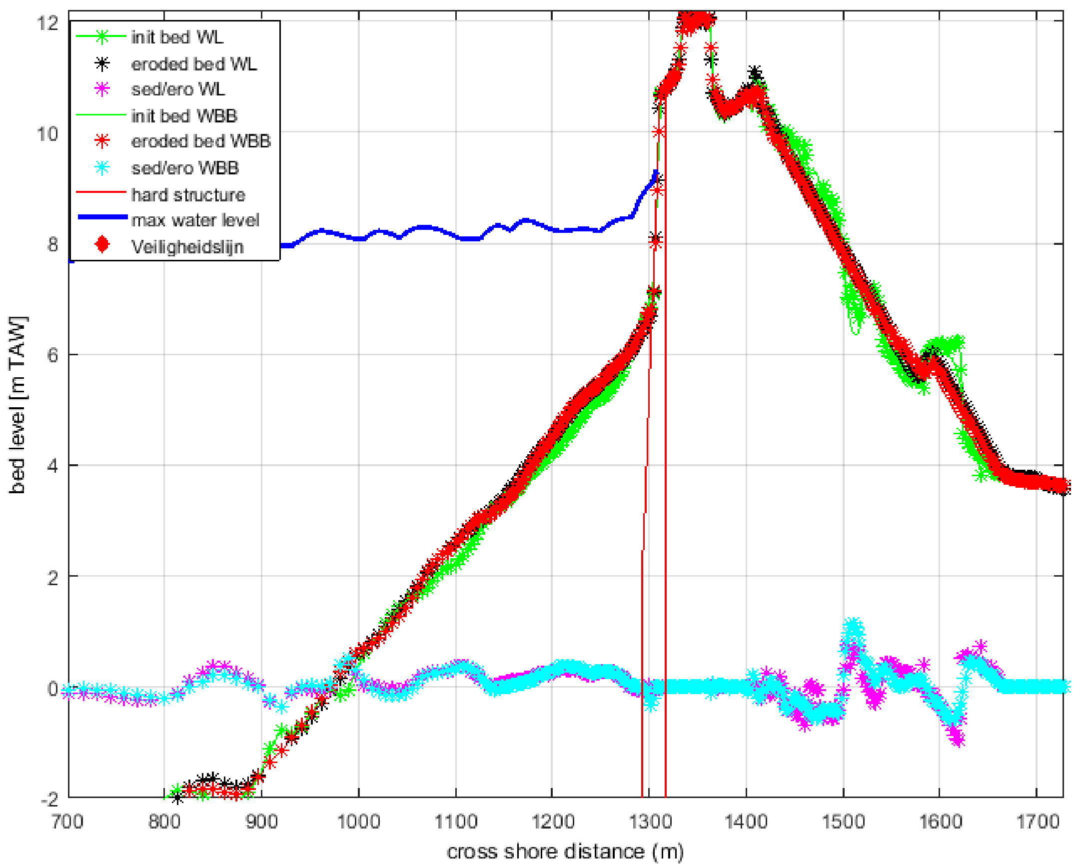


Figure 14 Coastal section 81: difference in sedimentation/erosion patterns on the weakest cross shore profile



4 Wave transformation (SWASH 2D)

4.1 Input: bathymetry

The weakest profiles of coastal sections 79 and 81 (Figure 8 and Figure 9 resp.) are the input for the wave transformation calculation. Appendix A of Suzuki *et al.*, 2016 outlines the bathymetric adaptations (and model settings) to be made in order to set up the SWASH 2D model domain. In summary:

1. Seaward of -5m TAW: Artificial extension between -5 m TAW and -15 m TAW, having a slope of 1:35, and adjacent a 100 m cross shore horizontal plane at -15 m TAW
2. Landward of the toe of the dike (defined as the transition between a milder and steeper slope than 1/10): Bathymetry is truncated at the level of the toe, and extended horizontally over 200 m.
3. To incorporate 2D wave effects, this cross-sectional profile is uniformly extended laterally over 400 m.

By doing so, incident hydraulic boundary conditions at the toe can be obtained.

The grid size in the x- and y-direction was set to 2.0 m and 4.0 m respectively. The model was run with one layer in the vertical direction since the kd value was less than 1 in all cases, indicating that the estimated phase velocity error is insignificant (where k is the wave number and d the water depth).

The cross-sectional profiles, used in the 2D wave transformation, are shown in Figure 15 (FHR) and Figure 16 (W+B). Table 2 lists the levels of the defined toe of the dike.

Bearing in mind the difference in the selected weakest profile for coastal section 79, resulting in a different level of the defined toe of the dike, it is expected to have some difference in wave properties at the toe as well. Besides, not only the toe level but also the bathymetry up to that point influences the wave transformation and hence, resulting wave properties at the toe of the dike.

Figure 15: Post-processed 1000 year storm bathymetry calculated in XBEACH for section 79 (upper panel) and 81 (lower panel): FHR

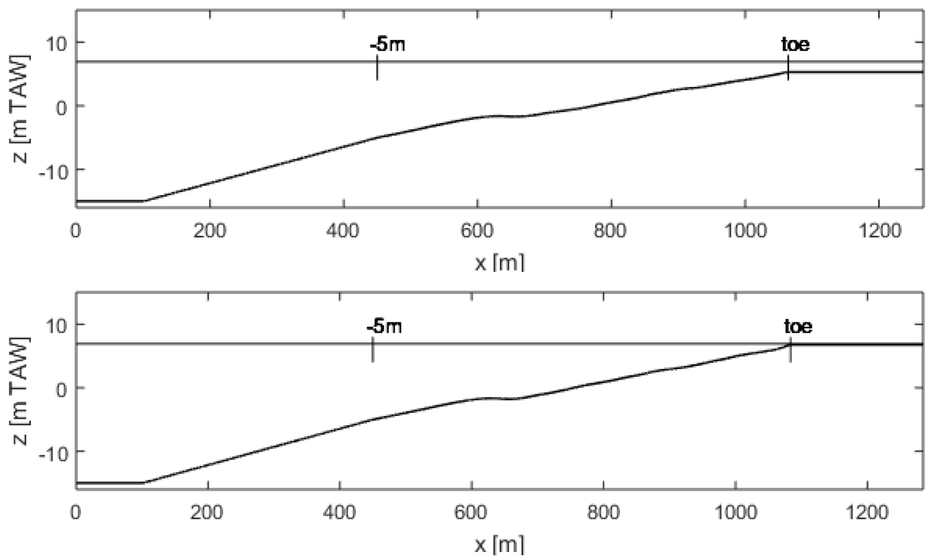


Figure 16: Post-processed 1000 year storm bathymetry calculated in XBEACH for section 79 (upper panel) and 81 (lower panel): W+B

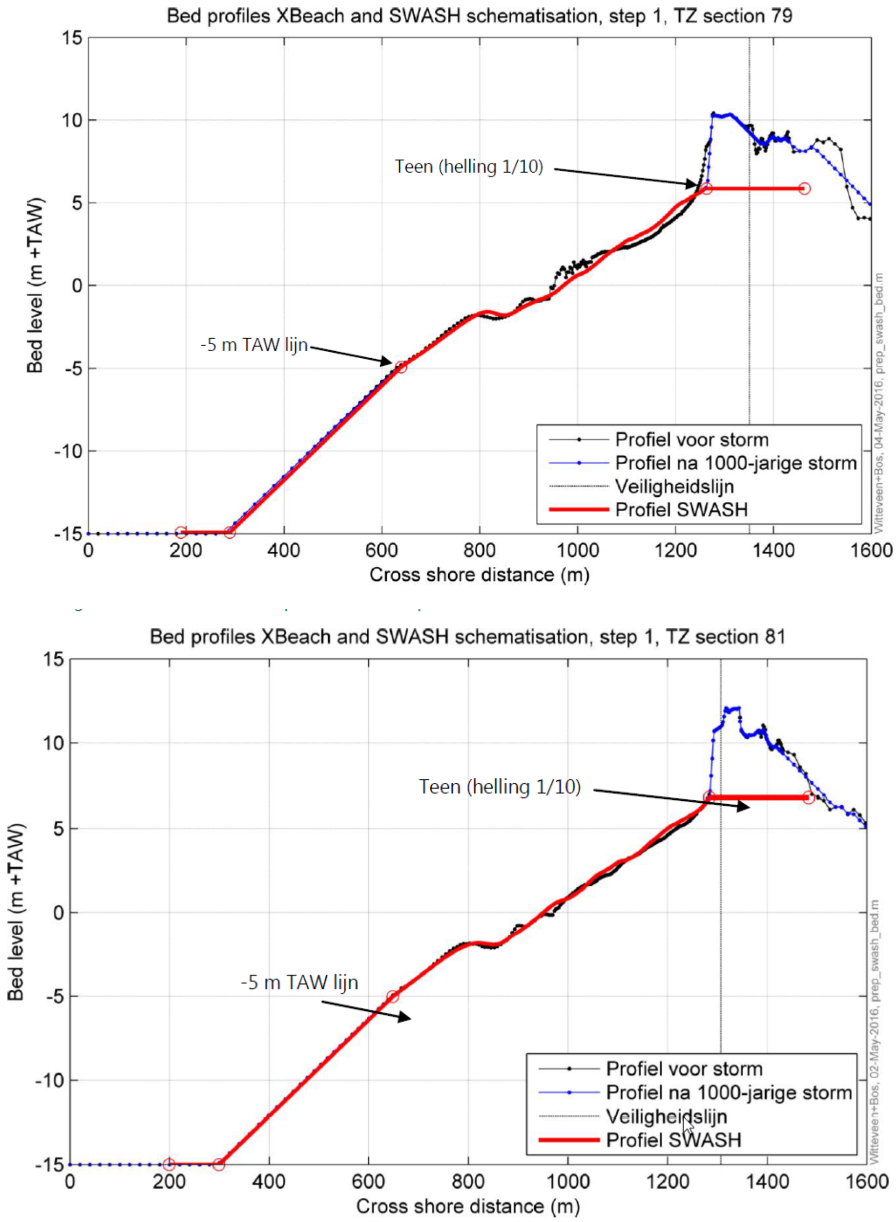


Table 2 Level of the defined toe of the dike for coastal section 79 and 81

Coastal section	Toe level (FHR)	Toe level (W+B)
79	+5.28 m TAW	+5.86 m TAW
81	+6.75 m TAW	+6.80 m TAW

4.2 Input: hydraulic boundary conditions

The hydraulic boundary conditions (cf. Table 1, and wave directional spreading of 16°) are imposed at the offshore boundary with a weakly reflective boundary condition. A Sommerfeld radiation condition was applied at the landward end of the numerical domain in order to minimize wave reflection.

4.3 SWASH: model settings

Numerical simulations were carried out with SWASH version 3.14; the SWASH 2D input file is attached in Appendix B. Appendix A of Suzuki *et al.*, 2016 gives an overview of the model settings to be applied.

Time duration of the numerical simulations was 40 min in prototype scale, which corresponds to about 200 waves. The numerical time step is automatically changed during SWASH calculations to satisfy the Courant–Friedrichs–Lewy (CFL) condition.

A Manning coefficient of n=0.019 s/m$^{1/3}$ was adopted since the wave actions are on the sandy beach. The breaking parameters (see details in Smit *et al.,* 2013) are fixed as default values (i.e. no tuning for wave transformation and overtopping by choosing alternative wave breaking parameters).

The non-hydrostatic pressure term was applied with a Keller-box scheme, which has a significant influence on wave transformation. Explicit time integration was used with a default time step restriction.

4.4 Output: incident hydraulic boundary conditions

Table 3 summarizes the offshore significant wave heights, i.e. at the -5 m TAW contour line, which need to be calibrated to 101% of the significant wave height H_{m0} of the respective coastal section (to be found in De Roo *et al.*, 2016). After calibration, the difference between the target (= 101% H_{m0}) and modelled wave height at that location needs to be smaller than ±3%. Wave peak period T_p, wave direction and directional spreading are assumed not to vary between the model domain boundary and the -5 m TAW contour line.

Table 4 lists the incident hydraulic boundary conditions at the toe of the dike. As commented in Section 4.1, the different toe level in coastal section 79 results in different incident hydraulic boundary conditions at that location. The deeper toe position in the FHR test consequently leads to larger wave properties. With regard to coastal section 81, similar results are obtained.

4.4.1 Post processing of SWASH output

The post-processing method might influence the wave parameters, resulting from SWASH simulations.

SWASH output files were post-processed using in house MATLAB scripts. Time series data was transformed into wave spectra by Fast Fourier Transform (FFT) algorithms and wave parameters such as H_{m0}, T_p and $T_{m-1,0}$ were calculated.

A Hanning window was applied for visualization of the calculated wave spectra, but no smoothing filter was applied to calculated wave parameters. A cut-off frequency of 0.005 Hz was applied to exclude the (very) low frequency waves.

On the other hand, W+B used the WAFO tool box (http://www.maths.lth.se/matstat/wafo/). It gives slightly different results for the spectral wave period $T_{m-1,0}$ because it is sensitive to the number of frequency bins used in the calculation (sensitivity more pronounced for shallow water cases). In order to minimize the difference, W+B will use at Parzen window size L=300 for the wave analysis.

Table 3: Significant wave height H_{m0} (average of 3 wave gauges) at -5 m TAW calculated by SWASH2D

Section	Executer	H_{m0} input (cf. Table 1)	Target H_{m0} [m]	Modelled H_{m0} [m]	Error to target
79	FHR	4.82	4.87	4.81	-1.3%
79	W+B	4.82	4.87	4.81	-1.3%
81	FHR	4.80	4.85	4.67	-4%
81	W+B	4.80	4.85	4.73	-2.5%

Table 4: Incident hydraulic boundary conditions at the toe of the dike (average of 3 wave gauges)

Section	Executer	H_{m0} [m]	$T_{m-1,0}$ [s]	Mean WL [m TAW]
79	FHR	1.22	17.1	7.19 (0.21+6.98)
79	W+B	0.99	20.1	7.22 (0.24+6.98)
81	FHR	0.74	30.5	7.30 (0.32+6.98)
81	W+B	0.77	31.8	7.32 (0.34+6.98)

5 Wave calibration (SWASH1D)

5.1 Input: bathymetry

The cross shore profiles, used for the 2D wave transformation (Figure 10), are again used, but now without lateral extension.

The grid size in the x-direction was set to 2.0 m. SWASH 1D was executed with one layer in the vertical direction.

5.2 Input: hydraulic boundary conditions

The still water level and significant wave height are varied at the offshore model boundary until the incident hydraulic boundary conditions match the SWASH 2D results within the specified error band: H_{m0} ±3%, $T_{m-1,0}$ ±5% and set-up ±0.05 m.

5.3 SWASH: model settings

Numerical simulations were carried out with SWASH version 3.14. Appendix B.1 of Suzuki *et al.*, 2016 gives an overview of the model settings to be applied.

The time duration of the numerical simulations was 110 min in prototype scale, which corresponds to about 500 waves.

A Manning coefficient of n=0.000 s/m$^{1/3}$ was adopted in order to get a slightly conservative overtopping value.

The breaking parameters are fixed as default values. The non-hydrostatic pressure term was applied with a Keller-box scheme and explicit time integration was used with a default time step restriction.

5.4 Output: offshore hydraulic conditions for wave overtopping calculation

Table 5 and Table 6 summarize the calibration results for coastal section 79 and 81 respectively. Note that for coastal section 81 a different seed number was used because the target wave could not be obtained after four trials.

Only the FHR calibration result is shown here since the W+B result is reported in Bodde *et al.* (2016).

Table 5: Calibration for coastal section 79 (FHR)

Target			Input			Result		
H_{m0} [m]	$T_{m-1,0}$ [s]	SWL [m]	Seed #	H_{m0} [m]	WL [m]	H_{m0} [m]	$T_{m-1,0}$ [s]	WL [m]
1.22	17.1	7.19	Default	1.80	7.18	1.27 (+4%)	19.5 (+14%)	7.21 (+2 cm)
1.22	17.1	7.19	Default	1.60	7.18	1.23 (+1%)	17.7 (+4%)	7.20 (+1 cm)

Table 6: Calibration for coastal section 81 (FHR)

Target			Input			Result		
H_{m0} [m]	$T_{m-1,0}$ [s]	SWL [m]	Seed #	H_{m0} [m]	WL [m]	H_{m0} [m]	$T_{m-1,0}$ [s]	WL [m]
0.74	30.5	7.30	Default	1.60	7.24	0.70 (-5%)	33.9 (+11%)	7.32 (+2 cm)
0.74	30.5	7.30	Default	1.40	7.28	0.69 (-7%)	30.0 (-2%)	7.35 (+5 cm)
0.74	30.5	7.30	Default	1.55	7.28	0.71 (-4%)	32.3 (+6%)	7.36 (+6 cm)
0.74	30.5	7.30	Default	1.72	7.26	0.73 (-1%)	35.6 (+17%)	7.35 (+5 cm)
0.74	30.5	7.30	111111	1.60	7.24	0.72 (-2%)	29.0 (-5%)	7.34 (+4 cm)

6 Wave overtopping (SWASH1D and empirical equation)

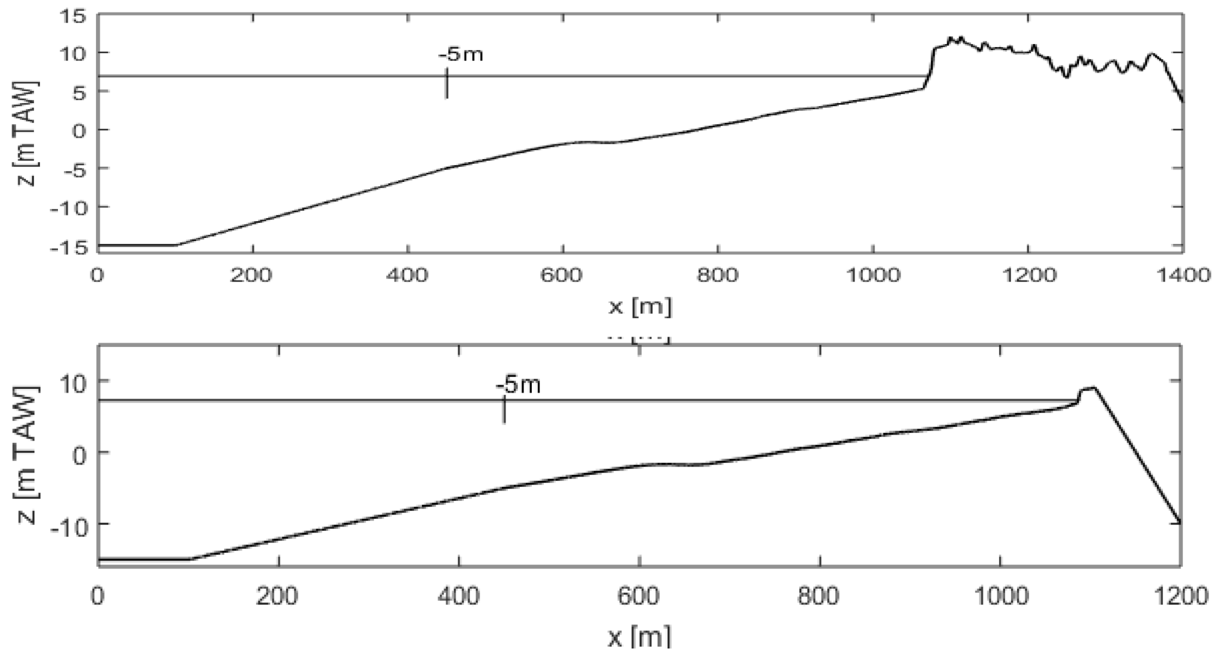
6.1 Input: bathymetry

Figure 17 illustrates the cross shore weakest profiles used in the wave overtopping calculation, in which the real bathymetry is now added above the level of the toe of the dike (cf. Figure 9 and Figure 10). Appendix B.2 of Suzuki *et al.*, 2016 outlines the bathymetric adaptations to be made in order to set up the SWASH 1D model domain.

The crest level of the dike in coastal section 81 equals +10.99 m TAW and thus, no or a very small wave overtopping discharge is expected. Therefore, the crest level is lowered to +9.0 m TAW in order to get at least some wave overtopping discharge close to 1 l/s/m.

The grid size in the x-direction was set to 2.0 m. SWASH 1D was executed with one layer in the vertical direction.

Figure 17: Cross shore weakest profiles used for overtopping calculation for the section 79 (upper panel) and 81 (lower panel), FHR



6.2 Input: hydraulic boundary conditions

The calibrated hydraulic boundary conditions (Table 5 and Table 6) are imposed at the offshore boundary with a weakly reflective boundary condition. A Sommerfeld radiation condition was applied at the landward end of the numerical domain in order to minimize wave reflection.

6.3 SWASH: model settings

Numerical simulations were carried out with SWASH version 3.14; the SWASH 1D input file is attached in Appendix A. Appendix B.2.2. of Suzuki *et al.*, 2016 gives an overview of the model settings to be applied.

The time duration of the numerical simulations was 110 min in prototype scale, which corresponds to about 500 waves. A Manning coefficient of n=0.000 s/m$^{1/3}$ was adopted in order to get a slightly conservative overtopping value. The breaking parameters are fixed as default values. The non-hydrostatic pressure term was applied with a Keller-box scheme and explicit time integration was used with a default time step restriction.

6.4 Empirical equation

Only for the 'dike' coastal section 81 an empirical equation is applicable besides SWASH to calculate wave overtopping. The empirical equation to estimate wave overtopping on a shallow foreshore, equation 8.6 in Suzuki *et al.* (2016), is applied together with the equations to calculate the surf similarity parameter (eq. 8.7) and the average slope (eq. 8.8).

6.5 Output: average wave overtopping discharge

Table 7 indicates the average wave overtopping discharge, obtained using the SWASH model and the empirical equation.

With regard to coastal section 79, the average wave overtopping discharge calculated by FHR is not really differing from the one by W+B (using SWASH 1D). However, both results are obtained using different weakest profiles (cf. Section 3.5.2) and thus, cannot be compared directly.

With regard to coastal section 81, the FHR and W+B average wave overtopping discharges calculated by the empirical equation are similar. The difference is acceptable (W+B result is 1.6 times bigger than FHR result) taking into account the varying bathymetries, the toe levels, the different calibration results and postprocessing tools. The FHR and W+B average wave overtopping discharges calculated by SWASH 1D vary more (W+B result is 2.7 times bigger than FHR result).

These differing results between SWASH 1D and empirical equation are however normal because the empirical equation gives a deterministic value (i.e. one standard deviation is added to the average value, see Allsop *et al.*, 2016) whereas SWASH 1D outputs a random deterministic result, defined by the chosen seed number and simulation time.

Thus, a number of SWASH calculations, defined by different seed numbers and an identical simulation time, result in an average wave overtopping result. To this result, a standard deviation needs to be added to obtain a similar deterministic result as the above described, related to the use of an empirical equation (see project 16_011 'Uncertainty in wave overtopping calculation using SWASH') .

Table 7: Average wave overtopping discharge q for coastal sections 79 and 81, both SWASH1D and empirical equation

Coastal section	q_{SWASH} (FHR) [l/s/m]	q_{SWASH} (W+B) [l/s/m]	$q_{emp\ eq}$ (FHR) [l/s/m]	$q_{emp\ eq}$ (W+B) [l/s/m]
79	0.0	0.1	Not applicable	Not applicable
81	0.6	1.6	2.2	3.6

7 Safety assessment

7.1 Coastal section 79

A ‘protected dune’ section, where the safety line is determined by (the seaward side of) housing, needs to be assessed as to prevent damage to these buildings because of the normative storm. A minimum distance of 5 m to the building, followed by a 1:1.5 limit line (between the crest of the eroded profile and the water level (maximum of T = 1000yr + dec)) which cannot be intersected by the post-storm profile, needs to be maintained at any time (cf. Figure 6-5 in Chapter 6 of Suzuki *et al.*, 2016) .

Taking into account that the horizontal distance between the eroded profile and the safety line is 314 m (Figure 7), which is much larger than the aforementioned minimum prerequisites, coastal section 79 is assessed and considered <u>safe</u>. The same assessment result was obtained by W+B.

However, since the vertical distance between the crest of the dune (12.12 m TAW) and the water level (+7.32 m TAW) is smaller than 6 m, wave overtopping might be of importance and needs to be calculated using SWASH. No wave overtopping however resulted.
Aside, the zero maximum cross shore overtopping discharge q_x (output of XBEACH) also indicates no wave overtopping.

The stability of the revetment of the ‘dike’like dune foot protection is not evaluated (which might be a prerequisite for the safety assessment of this coastal section when indispensable for its safety).

7.2 Coastal section 81

A ‘dike’ section, where the safety line is determined by (the seaward side of) housing, needs to be assessed as to prevent damage to these buildings and the dike itself because of the normative storm. Therefore, the maximum allowed wave overtopping is 1 l/s/m, and the dike revetment needs to be adequate. The latter’s stability is here not evaluated.

The crest of the dike (+10.74 m TAW) of the selected weakest profile is high enough to prevent waves overtopping its crest. Hence, coastal section 81 is also considered <u>safe</u>. The same assessment result was obtained by W+B.

As the purpose of this testcase is to verify the contractor’s understanding of the methodology, it was decided to lower the crest of the dike to +9 m TAW in order to allow for wave overtopping. These results are already discussed in Section 6.5.

8 References

Allsop, N.W..; Bruce, T.; De Rouck, J.; Kortenhaus, A.; PUllen, T.; Schüttrumpf, H.; Troch, P.; van der Meer, J.W.; Zannutigh, B. (2016). EurOtop. Manual on wave overtopping of sea defences and related structures. 2nd editio.

Bodde, W.; Cornu, T. (2016). Toetsing van de zeewering: Data inventarisatie en fotorapportage. Witteveen+Bos: Antwerpen

Bodde, W.; Cornu, T.; van Tol, P. (2016). Toetsing van de zeewering - Rapportage testcases

De Roo, S.; Trouw, K.; Ruiz Parrado, I.; Suzuki, T.; Verwaest, T.; Mostaert, F. (2016). Het Hydraulisch randvoorwaardenboek (2014). *WL Rapporten*, 14_014. Waterbouwkundig Laboratorium/Fides Engineering: Antwerpen

International Marine and Dredging Consultants. (2008). Veiligheid Vlaamse Kust. Deel I - toetsing. IMDC: Antwerpen

Smit, P.; Zijlema, M.; Stelling, G. (2013). Depth-induced wave breaking in a non-hydrostatic, near-shore wave model. *Coast. Eng. 76*: 1–16. doi:10.1016/j.coastaleng.2013.01.008

Suzuki, T.; De Roo, S.; Altomare, C.; Peeters, P.; Mostaert, F. (2017). Methodology for Safety Assessment 2015. Updated methodologies from the report v6.0 to v10.0. *WL Rapporten*, 14_014. Flanders Hydraulics Research

Suzuki, T.; De Roo, S.; Altomare, C.; Zhao, G.; Kolokythas, G.K.; Willems, M.; Verwaest, T.; Mostaert, F. (2016). Toetsing kustveiligheid-2015 - Methodologie: toetsingsmethodologie voor dijken en duinenVersie 10. *WL Rapporten*, 14_014. Waterbouwkundig Laboratorium: Antwerpen. Versie 10.

Vlaamse Hydrografie. (2017). Getijtafels 2017 voor Nieuwpoort, Oostende, Blankenberge, Vlissingen, Prosperpolder, Antwerpen en Wintam. Vlaamse Hydrografie

Appendix A: XBEACH input file

```
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%% XBeach parameter settings input file                             %%%
%%%                                                        %%%
%%% date:     20-Jan-2015 12:42:52                                 %%%
%%% function: xb_write_params                                      %%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%% Flow parameters %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
cf       = 0.001000
%%% General %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
D50      = 0.000319
D90      = 0.000478
jetfac   = 1
swrunup  = 1
swave    = 1
lwave    = 1
flow     = 1
sedtrans = 1
morphology = 1
nonh     = 0
gwflow   = 0
q3d      = 0
tidelen  = 46
%random = 0
%cmax = 10.0
%%% Grid parameters %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
depfile  = bed.dep
posdwn   = 0
nx       = 383
ny       = 205
alfa     = 0.0
vardx    = 1
xfile    = x.grd
yfile    = y.grd
xori     = 0
yori     = 0
```

thetamin = -106

thetamax = 14

dtheta = 10

thetanaut = 0

%%% Initial conditions %%%

%zs0 = 0

%%% Model time %%%

%%% Morphology parameters %%

wetslp = 0.260000

struct = 1

ne_layer = nebed.dep

%%% Roller parameters %%

beta = 0.138000

%%% Sediment transport parameters %%

facSk = 0.375000

facAs = 0.123000

%%% Tide boundary conditions %%

%front = 1

%back = 2

%left = 0

%right = 0

zs0file = WL_s.txt

tideloc = 2

paulrevere = 0

epsi = -1

%%% Wave boundary condition parameters %%%

instat = 41

%%% Wave-spectrum boundary condition parameters %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

bcfile = loclist.txt

wbcversion=3

nspectrumloc=15

%%% Wave breaking parameters %%

gamma = 0.541000

alpha = 1.262000

gammax = 2.364000

fw = 0

%%% Wave-spectrum boundary condition parameters %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

Morphological calculation options

morfacopt= 1

morfac = 3

morstart = 00

%%% Output variables %%%

tstart = 000

tintg = 3600

tintp = 3600

tintm = 3600

tstop = 162000

taper = 0

----

nglobalvar = 12

zb

zs

H

u

v

thetamean

qx

qy

sedero

alfau

alfav

alfaz

nmeanvar = 12

zb

zs

H

u

v

thetamean

qx

qy

sedero

alfau

alfav

alfaz

Appendix B: SWASH 2D and 1D input files

SWASH2D input for wave transformation:

```
$*************HEADING****************************************
$
PROJ 'TES_081' '01'
$
$*************MODEL INPUT*************************************
$
MODE NONST TWOD
SET LEVEL  6.98
$
CGRID   0.0   0.0   0.0 1284.0 400.0   642   100 REPEATING Y
VERT 1
$
INPGRID BOTTOM   0.0   0.0   0.0   2568   1  0.5  400.00
READINP BOTTOM -1. 'TES_081.bot' 1 0 FREE
$
INIT zero
$
BOU SHAP JON 3.3 DSPR DEGR
BOU SIDE W BTYPE WEAK HYPER ADDBoundwave CON SPECT   4.85  11.09   0.00  16.0
BOU SIDE E CCW BTYPE RADIATION
SPON RI   100.0
$
FRIC MANN   0.019
Break
VISC Horizontal CON
$
NONHYDROSTATIC
DISCRET UPW MOM
TIMEI
$
$************* OUTPUT ***************************************
$
POINTS 'GAUGE' FILE 'TES_081.wvg'
```

```
TABLE 'GAUGE' NOHEAD 'TES_081.tbl' TSEC DIST BOTL WATL OUTPUT 000000.000 0.20 SEC
BLOCK 'COMPGRID' NOHEAD 'TES_081.mat' HSIG SETUP
QUANT HSIG  dur 40 min
QUANT SETUP dur 40 min
$
TEST 1 0
COMPUTE 000000.000  0.050 SEC 005010.000
STOP
$
```

SWASH1D input file for wave overtopping:

```
$*************HEADING****************************************
$
PROJ 'TES_081' '01'
$
$*************MODEL INPUT************************************
$
MODE NONST ONED
SET LEVEL  7.25
SET SEED 111111
$
CGRID   0.0   0.0   0.0 1284.0 0.0   2568   0
VERT 1
$
INPGRID BOTTOM   0.0   0.0   0.0   2568   1  0.5  400.00
READINP BOTTOM -1. 'TES_081.bot' 1 0 FREE
$
INIT zero
$
BOU SHAP JON 3.3
BOU SIDE W BTYPE WEAK HYPER ADDBoundwave CON SPECT   1.70  11.09
BOU SIDE E CCW BTYPE RADIATION
SPON RI   100.0
$
FRIC MANN   0.000
Break
$ VISC Horizontal CON
$
NONHYDROSTATIC
```

```
DISCRET UPW MOM
TIMEI
$
$************* OUTPUT ***************************************
$
POINTS 'GAUGE' FILE 'TES_081.wvg'
TABLE 'GAUGE' NOHEAD 'TES_081.tbl' TSEC DIST BOTL WATL OUTPUT 000000.000 0.20 SEC
POINTS 'LAYER' FILE 'TES_081.lay'
TABLE 'LAYER' NOHEAD 'TES_081.ltn' TSEC DIST BOTL WATL OUTPUT 000000.000 0.20 SEC
TABLE 'LAYER' NOHEAD 'TES_081.lsp' TSEC DIST BOTL VEL  OUTPUT 000000.000 0.20 SEC
BLOCK 'COMPGRID' NOHEAD 'TES_081.mat' HSIG SETUP
QUANT HSIG  dur 100 min
QUANT SETUP dur 100 min
$
TEST 1 0
COMPUTE 000000.000  0.100 SEC 015010.000
STOP
$
```

DEPARTMENT **MOBILITY & PUBLIC WORKS**
Flanders hydraulics Research

Berchemlei 115, 2140 Antwerp
T +32 (0)3 224 60 35
F +32 (0)3 224 60 36
waterbouwkundiglabo@vlaanderen.be
www.flandershydraulicsresearch.be