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Raversijde-Mariakerke-Wellington west

Wave overtopping and wave forces.
Results from experimental campaigns.

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Abstract

Physical model tests have been carried out at Flanders Hydraulics Research (FHR) and Ghent University to assess wave overtopping and forces on storm return walls foreseen in the Raversijde-Mariakerke-Wellington west area, along the Belgian coast. The experimental campaigns support the design of the sea dike upgrade in the aforementioned area. In particular, the minimum layout (wall height and location) was defined able to guarantee overtopping discharge within the limit of 1 l/s/m. The measurement of wave forces was then requested to finalise the design of the storm walls and their foundations. BFacilities at FHR and UGent were both deployed.

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1 Introduction

1.1 Background

The potential sea level rise and the increased storminess expected for the incoming decades represent great threats for those areas located in low-lying coastal regions such as the Flemish coast. Existing coastal defences are often insufficient to protect against extreme storms, thus it is needed to design protective countermeasures. In Belgium, the Coastal Safety Masterplan (Afdeling Kust, 2011) was approved by the Flemish government in order to realise protections against extreme storm events.

Figure 1 – Location of various cross sections at Raversijde-Mariakerke-Wellington (Profile 097-108). ©Google Earth 2012

The Raversijde-Mariakerke-Wellington West zone is one of the weak links in the Flemish coast, as identified by the master plan. A top view of the area is shown in Figure 1 (Zone 97-108).

The Coastal Safety Masterplan defines the tolerable wave overtopping discharge to be met in order to minimize the risks for a storm with 1000 year return period. The criteria can be summarized as follows:

- The mean wave overtopping discharge rate for a 1000-years storm over paved or armoured promenades or revetment seawalls is less than 1 l/s/m.
- Any erosion or damage to dunes or coastal structures caused by a 1000-years storm event must not result in damage to buildings behind the dune or coastal structure.

To satisfy the wave overtopping criteria of the Coastal Safety Master Plan, construction of storm walls is foreseen along the Raversijde-Mariakerke-Wellington West zone. Prior to the design of these new defences, the loading exerted by the overtopping waves must be quantified.

1.2 Purpose of project

The project is twofold: 1) by means of physical model tests on wave overtopping, the minimum required layout for the storm walls has to be assessed in order to guarantee a mean overtopping discharge smaller than 1 l/s/m for all examined cross sections; 2) once the height and position of the storm walls is decided, the forces to which these walls are exposed, due to the action of the overtopping waves, are measured.

1.3 Project phases

Two different experimental campaigns have been carried out, one at Flanders Hydraulics Research (FHR) and the other at Ghent University (UGent). The tests carried out at FHR were focusing on wave overtopping measurements to define the required minimum layout that could reduce the mean overtopping discharge under the limit of 1 l/s/m. The tests at UGent were carried out to measure the wave loading on storm walls for few selected cross sections in the area object of study. The usage of UGent facilities was due to the temporary unavailability of the wave flume at FHR.

1.4 Report outlines

Chapter 2 briefly describes the methodology that has been followed during the project. Chapter 3 summarises the physical model results from the experimental campaign carried out at Flanders Hydraulics Research on wave overtopping. Chapters 4 and 5 summarizes the physical model results from the experimental campaign carried out at Ghent University on wave forces. Finally the main conclusions are reported in Chapter 6.

2 Methodology

During the preparatory phase of the project, a 5-phases methodology was established (see Projectplan 2016P13_168_3) to optimise the usage of the wave flume at FHR and achieve reliable results with limited cost and in reasonable time. The methodology consisted of general and specific approaches. However, during the execution of the project, the methodology has been amended in agreement with the client. The final work methodology can be summarised as follows.

Preliminary results of wave overtopping have been obtained by means of numerical modelling (IMDC, 2015). A NLSW model, namely SWASH, has been used for the purpose. Twelve cross sections have been identified in total along the Raversijde-Mariakerke-Wellington West area.

For each of these cross sections, a preliminary layout (i.e. wall height and position) has been determined based on SWASH results, in order to guarantee a mean overtopping discharge equal or less than 1 l/s/m. First, wave boundary conditions have been calculated by the numerical model SWASH (Zijlema et al., 2011) in 2D based on the calculated bathymetries by the Durosta model after a 1000-years storm with nourished beach profile (Afdeling Kust, 2011) for 12 sections (099, 103.1, 103.2, 104.1, 104.2, 105.1, 105.2, 105.3, 106.1, 107.1, 108.1, 108.2) used in Suzuki et al. (2013). Figure 2 and Figure 3 show the topography of the cross sections and Table 1 lists the key parameters. It is underlined that the boundary condition described here is not used for the final design.

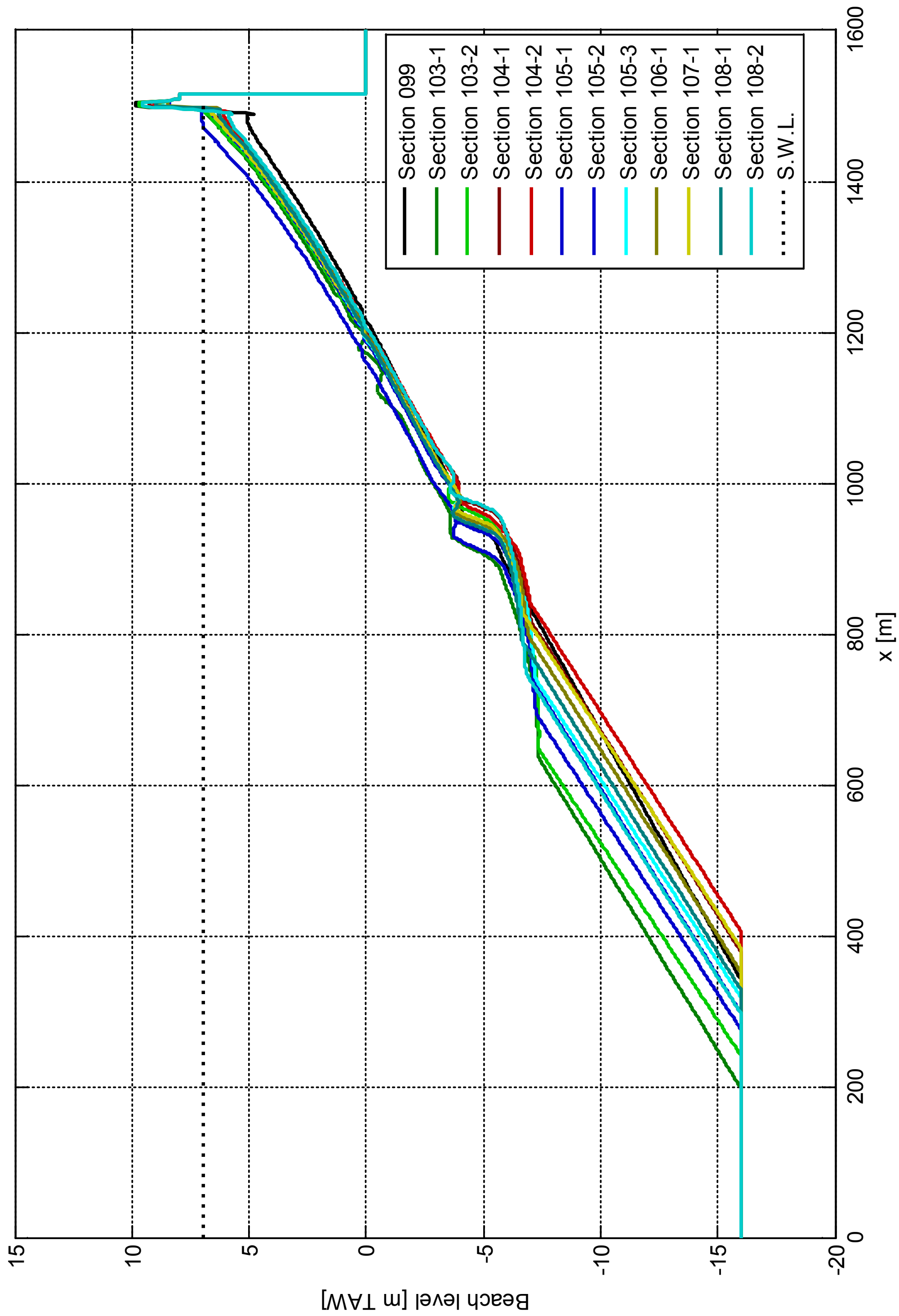
SWASH 2D uses calculated bathymetries up to the toe of the dike and a flat bottom is continued behind the toe of the dike and the Sommerfeld boundary condition is applied at the end of the domain, so that only an incident wave spectrum is obtained at the position of the toe of the dike. Based on the obtained wave spectrum at the toe of the dike from 2D SWASH calculations, 1D SWASH calculations have been conducted to estimate sea wall heights and positions. The reason to use 1D SWASH is that the 1D model is more computationally stable under wet-dry conditions (i.e. wave overtopping calculation), and there will be not so much difference in wave overtopping result between 1D and 2D as long as it is calculated from the toe of the dike (i.e. directional spreading at the toe position is very limited). A firsts preliminary architectural design has been defined based on the SWASH 1D simulation results (rough estimation of the positions and heights of the sea walls). Based on the numerical model results, few representative profiles have been selected for wave overtopping and force measurements.

Three cross sections (corresponding to sections 099, 105.2, 106.1) have been investigated by using each specific characteristics including incident wave calibration. These three cross sections were selected among all to cover all possible ranges of crest freeboard and water depth at the toe. Cross section 099 in particular corresponds to the case with lowest dike toe and highest crest elevation; cross section 105.2 has the lowest crest elevation and the highest toe level. Finally cross section 106.1 represents a case in between the former two and quite representative of the whole investigated area. The calibration of wave generation has been carried out in order to get the target wave conditions at the position corresponding to the toe of the dike (test are conducted without dike to get incident wave height) for the 3 sections. After the calibration of the wave boundary conditions, the 3 sea dikes already built in the physical flume have been used for the overtopping measurements, within the generic approach. For each profile 3 different heights and positions of the storm walls have been tested. The results have been collected and analysed in order to develop a new semi-empirical formula to apply to all cases from the coastal area that is object of study. The formulae have been used to define the final layout of each profile. Note that the final layout is decided based on the calculated bathymetries by the XBeach model after a 1000-years storm with nourished beach profile using the developed formulae.

For the final layout, wave boundary conditions have been calculated by the numerical model SWASH (Zijlema et al., 2011) in 2D based on the calculated bathymetries by the XBeach model after a 1000-years storm with nourished beach profile for 12 sections (099, 103.1, 103.2, 104.1, 104.2, 105.1, 105.2, 105.3, 106.1, 107.1, 108.1, 108.2) provided by the Coastal division and then SWASH 1D calibration is conducted in order to obtain necessary values for overtopping estimation by the developed formulae. Table 2 lists the key parameters from XBeach profile.

After the decision of the final layout, physical experiments have been carried out to assess the wave forces onto the storm return walls for two specific profiles, namely 104.2 and 099.

Figure 2 – Each cross section used in Suzuki et al. (2013).



The original bathymetries calculated by DUROSTA exist down to around -7 m TAW,
but these are extended by using foreshore slope to -16 m TAW for the SWASH calculation.

Figure 3 – Detailed view close to the dike (DUROSTA).

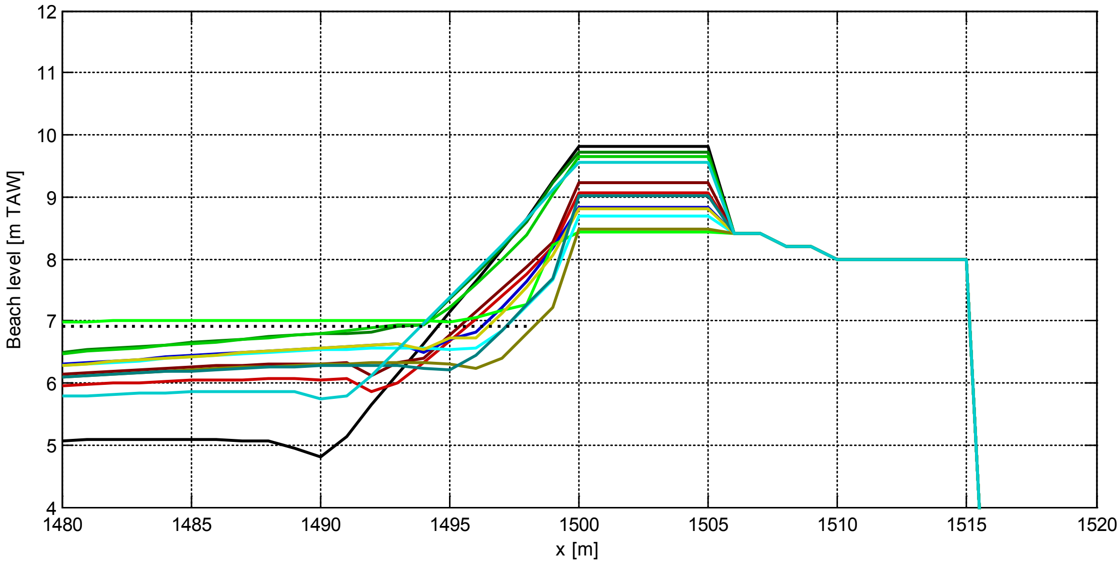


Table 1 – Cross section properties Raversijde-Mariakerke-Wellington (Profile 099-108):

Profile [-]	Dike crest [mTAW]	Toe [mTAW]	SWL [mTAW]	Rc [m]	Toe depth [m]	Dike slope [1/x]	Foreshore slope [1/x]
099	9.81	4.81	6.92	2.89	2.11	2.0	55
103.1	9.73	6.94	6.92	2.81	-0.02	2.2	51
103.2	9.66	6.93	6.92	2.74	-0.01	2.2	47
104.1	9.23	6.40	6.92	2.31	0.52	2.1	49
104.2	9.07	5.85	6.92	2.15	1.07	2.5	48
105.1	8.84	6.49	6.92	1.92	0.43	2.6	50
105.2	8.43	7.26	6.92	1.51	-0.34	1.7	48
105.3	8.7	6.57	6.92	1.78	0.35	1.9	48
106.1	8.47	6.24	6.92	1.55	0.68	1.8	49
107.1	8.8	6.72	6.92	1.88	0.20	1.9	48
108.1	9.01	6.21	6.92	2.09	0.71	1.8	49
108.2	9.07	5.80	6.92	2.15	1.12	2.4	49

After 1000-years storm with nourished beach profile (DUROSTA)

Table 2 – Cross section properties Raversijde-Mariakerke-Wellington (Profile 099-108)

Profile **[-]**	**Dike crest** **[mTAW]**	**Toe** **[mTAW]**	**SWL** **[mTAW]**	**Rc** **[m]**	**Toe depth** **[m]**	**Dike slope*** **[1/x]**	**Foreshore slope** **[1/x]**
099	9.81	6.22	6.93	2.88	0.71	**2.4**	**24.1**
103.1	9.73	6.90	6.92	2.81	0.02	**6.4**	**27.4**
103.2	9.66	6.87	6.92	2.74	0.05	**6.2**	**27.3**
104.1	9.23	6.90	6.92	2.31	0.02	**3.8**	**22.6**
104.2	9.07	6.42	6.92	2.15	0.5	**2.7**	**19.7**
105.1	8.84	6.82	6.92	1.92	0.1	**3.7**	**20.4**
105.2	8.43	6.90	6.92	1.51	0.02	**18.1**	**30.5**
105.3	8.70	6.85	6.92	1.78	0.07	**4.4**	**21.9**
106.1	8.47	6.86	6.91	1.56	0.05	**3.7**	**21.2**
107.1	8.80	6.88	6.91	1.89	0.03	**2.3**	**20.8**
108.1	9.01	6.90	6.91	2.1	0.01	**5.7**	**21.9**
108.2	9.56	6.32	6.91	2.65	0.59	**2.1**	**22.9**

After 1000-years storm with nourished beach profile (XBeach)

*Dike slope is calculated based on the methodology book (Suzuki …), namely using 1/10 slope point or the first wet point connecting to the blue stone point

3 SWASH model

3.1 Preliminary study using SWASH model based on DUROSTA profile (for FHR test)

The preliminary results of SWASH modelling on wave overtopping are reported in IMDC (2015). The layout for each cross section based on overtopping criteria is proposed. All detail information can be found in IMDC (2015). Here, only the main results are summarised for further comparisons. The results of SWASH 1D-2D calibration are reported in Table 3, where H_{m0} and $T_{m-1,0}$ indicate the incident wave height and spectral period at the toe of the dike. The wave overtopping results from IMDC (2015) are summarized in

Table 4. The horizontal distance between the edge of the dike crest and the storm wall location is defined as promenade width. Note that the wall height in

Table 4 for profiles 108.2 and 108.21 existing wall height.

Table 3 – Results of SWASH 1D-2D calibration IMDC (2015).

Profile	$H_{m0,offshore}$ [m]	$T_{p,offshore}$ [s]	$SWL_{offshore}$ [mTAW]	H_{m0} [m]	$T_{m-1,0}$ [s]	SWL_{toe} (setup included) [mTAW]	Depth at the toe [m]
099	1.92	11.7	+7.090	1.24	17.28	+7.13	2.28
103.1	1.85	12.7	+7.260	0.75	34.34	+7.33	0.32
103.2	1.81	11.7	+7.277	0.70	33.01	+7.35	0.35
104.1	1.95	11.7	+7.120	0.89	26.13	+7.22	0.72
104.2	1.83	11.7	+7.110	0.92	22.53	+7.19	1.26
105.1	1.86	12.7	+7.090	0.74	23.66	+7.20	0.60
105.2	1.59	12.7	+7.350	0.57	41.49	+7.49	0.09
105.3	2.06	12.7	+7.080	0.78	27.09	+7.21	0.51
106.1	1.82	11.7	+7.100	0.83	26.16	+7.20	0.86
107.1	1.95	11.7	+7.170	0.80	29.46	+7.27	0.45
108.1	1.88	12.7	+7.090	0.83	25.11	+7.20	0.88
108.2	2.04	12.7	+7.080	1.03	23.00	+7.17	1.28

Table 4 – Wave overtopping results and final layouts from IMDC (2015).

Profile	Crest elevation including storm wall [m TAW]	Depth at the toe [m]	Freeboard [m]	Promenade width [m]	Dike level at the edge [m TAW]	Dike level at the wall location [m TAW]	Wall height [m]	Overtopping [l/(m s)]
099	+10.11	2.28	2.98	6.70	+9.81	+9.95	0.16	1.00
0991 (variant)	+10.11	2.28	2.98	4.00	+9.81	+9.89	0.22	1.00
0992 (variant)	+10.05	2.28	2.92	0.00	+9.81	+9.81	0.25	1.00
103.1	+9.73	0.32	2.4	0.00	+9.73	+9.73	0.00	0.14
103.2	+9.66	0.35	2.31	7.30	+9.66	+9.66	0.00	0.05
103.21 (variant)	+9.66	0.35	2.31	0.00	+9.66	+9.66	0.00	0.01
104.1	+9.23	0.72	2.01	7.40	+9.23	+9.23	0.00	0.49
104.11 (variant)	+9.23	0.72	2.01	0.00	+9.23	+9.23	0.00	0.40
104.2	+9.16	1.26	1.97	0.00	+9.07	+9.07	0.09	1.00
105.1	+8.99	0.6	1.79	0.00	+8.84	+8.84	0.15	1.00
105.2	+8.67	0.09	1.18	0.00	+8.43	+8.43	0.24	1.00
105.3	+9.06	0.51	1.85	0.00	+8.70	+8.70	0.36	1.00
106.1	+8.86	0.86	1.66	12.50	+8.47	+8.72	0.14	1.00
106.11 (variant)	+8.86	0.86	1.66	0.00	+8.47	+8.47	0.39	1.00
107.1	+9.05	0.45	1.78	12.46	+8.80	+9.05	0.00	0.96
107.11 (variant)	+8.99	0.45	1.72	0.00	+8.80	+8.80	0.19	1.00
108.1	+9.27	0.88	2.07	12.67	+9.01	+9.27	0.00	0.43
108.11 (variant)	+9.01	0.88	1.81	0.00	+9.01	+9.01	0.00	0.87
108.2	+9.92	1.28	2.75	10.14	+9.07	+9.19	0.73	0.004
108.21 (variant)	+9.99	1.28	2.82	14.80	+9.08	+9.26	0.73	0.00002
108.22 (variant)	9.40	1.28	2.23	0.00	+9.07	+9.07	0.33	1.00

3.2 Final SWASH model based on XBeach (for UG test and the final design)

SWASH 2D model results are shown in Table 5 while 1D-2D calibration are reported in Table 6. According to the methodology book (Suzuki et al., 2015), the wave height H_{m0} must be within ± 3% of the SWASH 2D results, the spectral wave period $T_{m-1.0}$ within ± 5% of the SWASH 2D results, and the average water level (SWL) within ± 5 cm (in prototype scale). Some of the results shown in Table 6 do not satisfy the criteria but those are accepted after some attempts. Note that the influence of those difference will be limited.

The bold values in Table 6 will be used for the final design.

Table 5 – Results of SWASH 2D based on XBeach profile

Profile	$H_{m0,toe}$ [m]	$T_{m-1,0, toe}$ [s]	Set-up [m]	SWL_{toe} (setup included) [mTAW]
099	1.00	23.2	0.25	7.18
103.1	0.73	31.6	0.38	7.30
103.2	0.73	30.8	0.38	7.30
104.1	0.79	29.3	0.37	7.29
104.2	0.97	23.3	0.26	7.18
105.1	0.84	27.2	0.34	7.26
105.2	0.69	33.9	0.38	7.30
105.3	0.80	30.5	0.35	7.27
106.1	0.82	31.0	0.37	7.28
107.1	0.82	31.3	0.38	7.29
108.1	0.81	31.3	0.39	7.30
108.2	1.01	22.6	0.26	7.17

Table 6 – Results of SWASH 1D calibration based on SWASH 2D calculation

Profile	$H_{m0,offshore}$ [m]	$T_{p,offshore}$ [s]	$SWL_{offshore}$ [mTAW]	$H_{m0, toe}$ [m]	Ratio (Calibrated H/Target H)	$T_{m-1,0, toe}$ [s]	Ratio (Calibrated T/Target T)	SWL_{toe} (setup included) [mTAW]	Difference (Calibrated level-Target level)
099	1.90	11.7	**7.15**	**1.01**	1.01	**24.3**	1.05	7.22	0.04
103.1	1.67	11.7	**7.24**	**0.71**	0.98	**33.1**	1.05	7.35	0.04
103.2	1.65	11.7	**7.24**	**0.72**	0.98	**32.3**	1.05	7.34	0.05
104.1	1.70	11.7	**7.22**	**0.76**	0.97	**30.9**	1.05	7.33	0.03
104.2	1.64	11.7	**7.19**	**0.94**	0.97	**24.5**	1.05	7.24	0.07
105.1	1.70	11.7	**7.22**	**0.82**	0.97	**28.3**	1.04	7.31	0.05
105.2	1.67	11.7	**7.24**	**0.67**	0.97	**35.7**	1.05	7.35	0.05
105.3	1.70	11.7	**7.22**	**0.78**	0.97	**31.5**	1.03	7.32	0.04
106.1	1.80	11.7	**7.22**	**0.79**	0.97	**30.3**	0.97	7.32	0.05
107.1	1.70	11.7	**7.22**	**0.80**	0.97	**30.4**	0.97	7.32	0.03
108.1	1.78	11.7	**7.24**	**0.77**	0.96	**33.1**	1.06	7.35	0.05
108.2	1.79	11.7	**7.17**	**0.98**	0.97	**23.7**	1.05	7.23	0.06

4 Experimental campaign at Flanders Hydraulics Research

4.1 Flume setup and instrumentation

4.1.1 Wave flume in FHR

The wave flume at FHR is 70 m long, 4.0 m wide and 1.45 m deep (Figure 4). The facility is equipped with a piston-type wave generator with a stroke length of 0.6 m, which can generate regular and random waves with pre-assigned spectrum. The wave generator is equipped with an Active Wave Absorption System (AWAS). The flume at FHR has been split in four parts for this project as shown in Figure 5. One part (without dike) is used for measuring incident wave boundary conditions. The other 3 parts are used for overtopping measurements.

Figure 4 – Wave flume at Flanders Hydraulics Research.

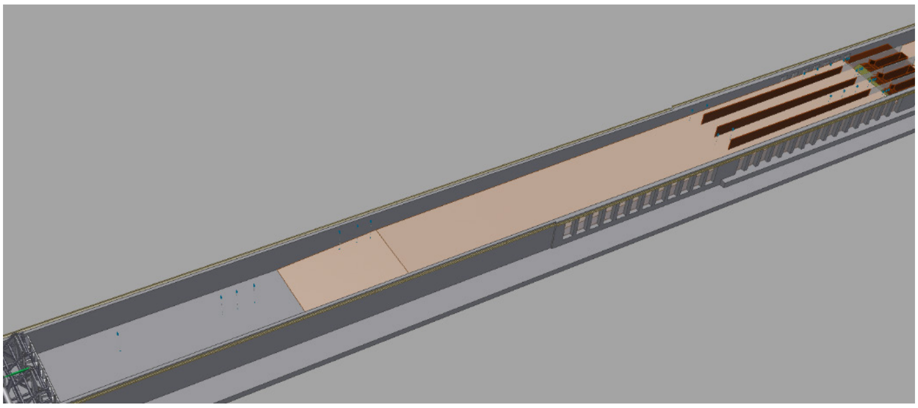
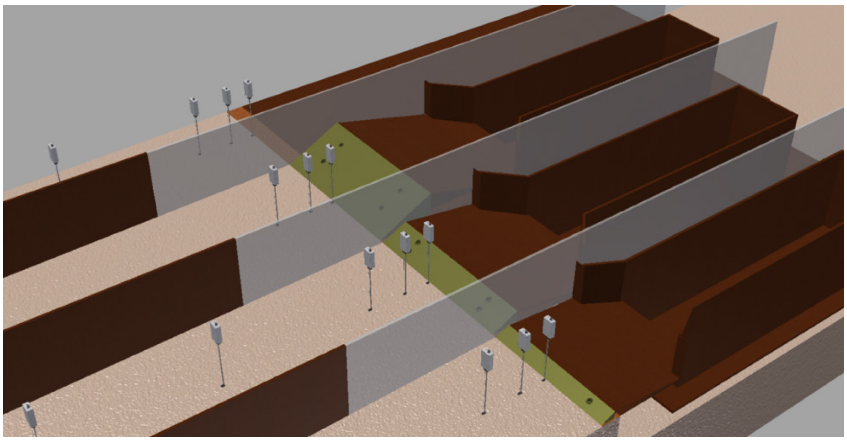


Figure 5 – Detailed view of the wave flume at Flanders Hydraulics Research.



4.1.2 Instrumentation

Wave height measurement

Measurements of water surface elevation were obtained with resistance-type wave gauges installed at the 29 locations illustrated in Figure 6, providing information on wave height transformation on the foreshore and wave characteristics at the toe of the dike. Analysis of wave height measurements from wave gauges is made in both the time and frequency domain using WaveLab 3.66, software developed at Aalborg University. To characterize the wave heights of incident waves, the significant wave height from spectral analysis ($H_{m0} = 4\sqrt{m0}$, where m0 is the zeroth moment of the wave spectrum) is used. Wave periods are defined as the average period obtained from spectral analysis ($T_{m-1,0}$) defined by m_{-1}/m_0, and the peak period (T_p) obtained from time domain analysis.

Classical reflection analysis methods (e.g. Mansard and Funke, 1980) using two or three wave gauges are not suitable in shallow water conditions because they are usually based on linear wave theory, while non-linear effects are dominating in the very shallow foreshore case (Van Gent, 1999). Instead, the measurements of wave height and period have been conducted using wave gauges at the location of the dike toe (WG19, WG20, WG21), but instead of having the sea dike, a horizontal platform was inserted just after the foreshore and damping material has been located after the platform. The aim was to reduce the reflection as much as possible in order to measure only incident wave characteristics. The waves running over the foreshore dissipate a large part of their energy for wave breaking and the water that reaches the location of the dike toe flows easily toward the back of the flume without any kind of reflection.

A cut-off frequency of 0.005 Hz (prototype scale, 200 s) has been used for the wave analysis at the toe, as prescribed in Suzuki et al. (2015). An example of wave gauge is shown in Figure 7

The presence of possible cross waves was investigated due to the particular setup of the wave flume, with four different split parts. The difference in wave height across the flume before the separation walls resulted negligible (see Appendix A).

Figure 6 – Position of the wave gauges

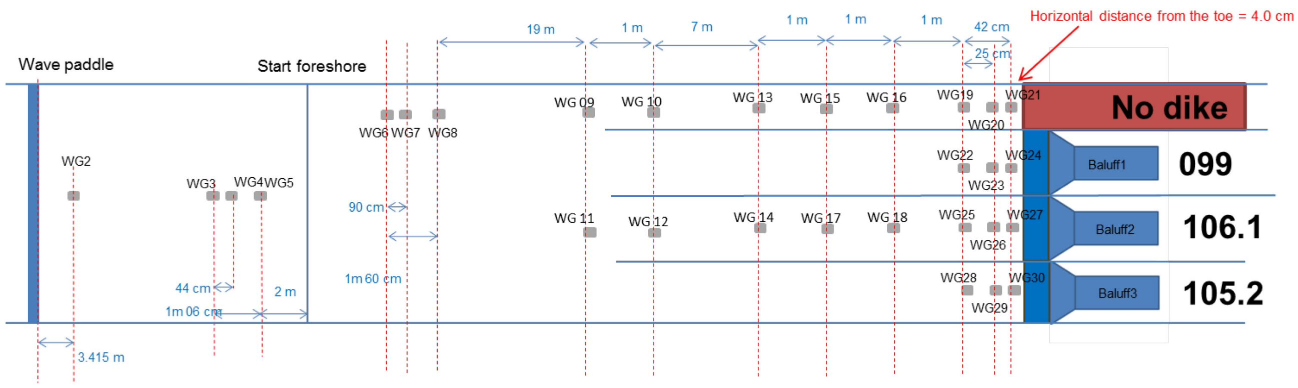


Figure 7 – Wave gauge (left image) and micropulse transducer (right image)

Wave overtopping measurement

A wave overtopping discharge was measured by collecting the water that overtopped the crest in overtopping boxes located downstream of the sea dikes. A short ramp sloped from the end of the dike (or from the rear side of the storm wall, if installed) into the overtopping box. The overtopping box dimensions were 0.5 m wide by 2.0 m long and 0.30 m deep.

The mean overtopping discharge was obtained by dividing the total volume of water collected during a test by the total duration of the test (usually between 35-40 min, in model scale, i.e. 2.9-3.3 h in prototype scale, corresponding to 1000 times the spectral period in deep water conditions ≈ 1000 waves).

The instantaneous overtopping discharge was measured by the installation of two Baluff “Micropulse” water level sensors inside the overtopping boxes (Figure 7). The instantaneous water level reading from the sensor could be converted to volume by measuring the difference in water level per overtopping wave and multiplying this number by the dimensions of the overtopping box. The Baluff sensors were finally implemented in this project to measure the average overtopping discharge, being related to the difference in water level before and after each test. The wave-by-wave overtopping was not analysed in detail, but only for some numerical model calibration, i.e. SWASH, that has been carried out later on and the results of which are not included here, since they are out of scope of the project.

4.2 Model geometry

4.2.1 Dike crest level

Three different dikes have been constructed, one for each flume partition. The last partition has no dike to measure the incident wave conditions. The dike height s (distance from the toe to the crest) correspond to the prototype values of profiles 099, 105.2 and 106.1 (respectively 5.00 m, 1.17 m and 2.23 m, in prototype scale). The dikes are constructed into the flume in a way that the toe of each dike is located at the same elevation (= 93.0 cm) with respect to the horizontal bottom of the flume. A sketch of the wave flume with the indication of the dike location and extension of the foreshore is depicted in Figure 8. The plan view and cross section of the three dike models are depicted in Figure 9 to Figure 11. The overtopping boxes are located just after the dikes. The figures correspond to the case with maximum promenade width (0.50 m, model scale). Each promenade has a 2% slope directed seawards.

Figure 8 – Sketch of wave flume with dike location

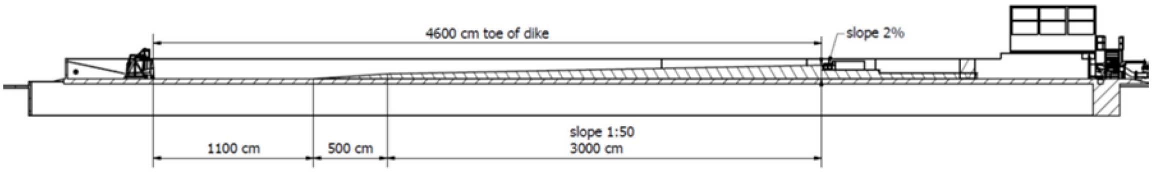


Figure 9 – Plan view and cross section of dike model corresponding to the geometry of the 099 profile

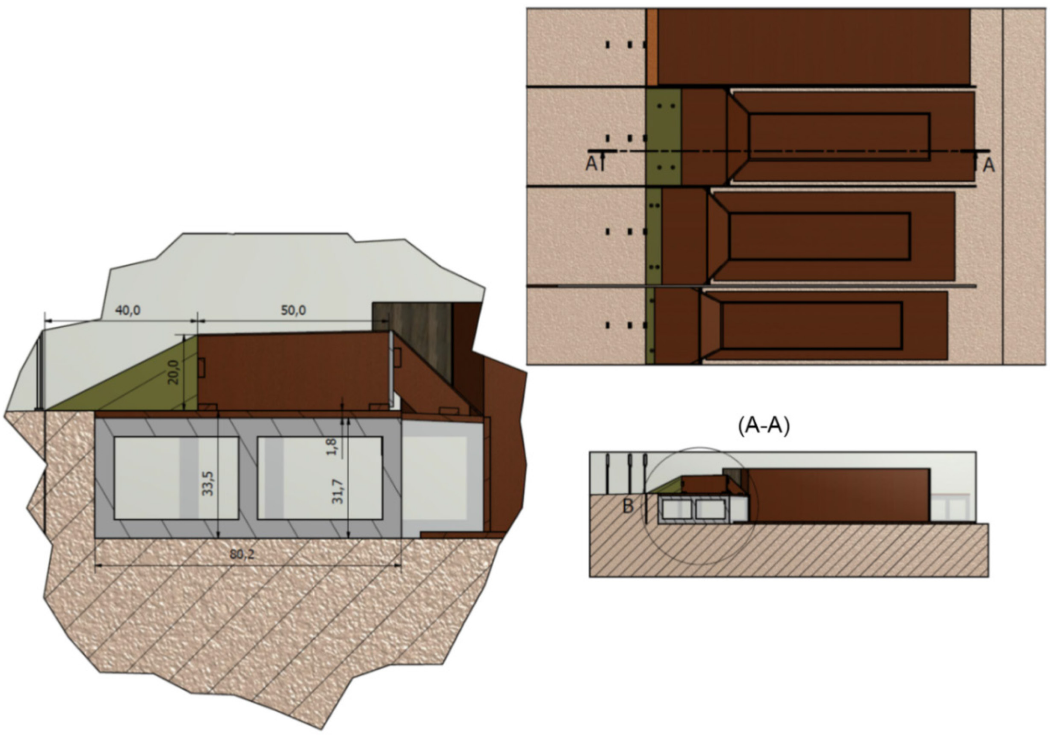


Figure 10 – Plan view and cross section of dike model corresponding to the geometry of the 106.1 profile

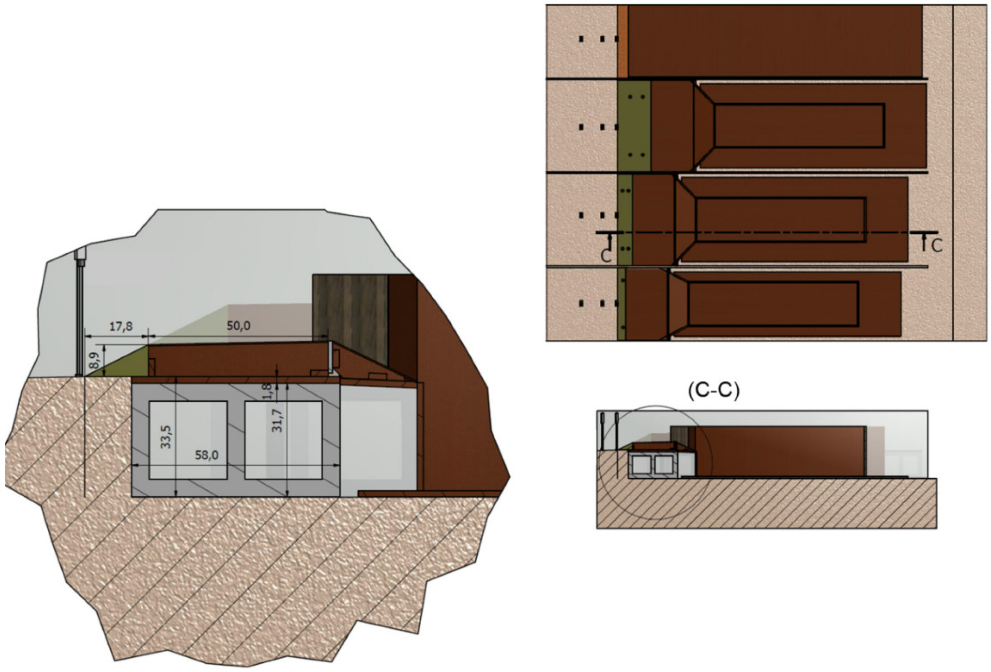
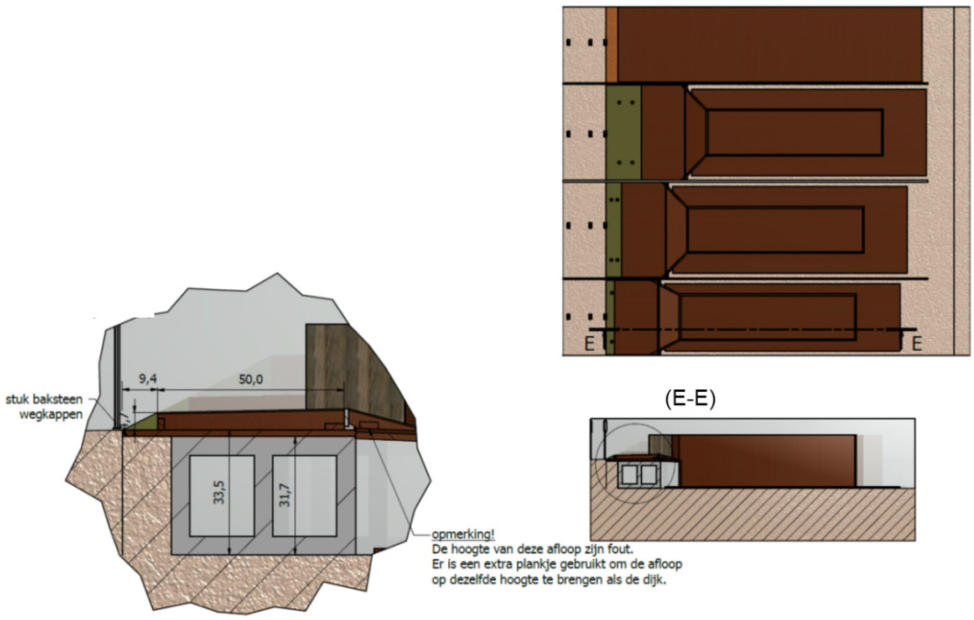


Figure 11 – Plan view and cross section of dike model corresponding to the geometry of the 105.2 profile



4.2.2 Dike and foreshore slopes

The dike slope to be adopted in the scale model is 1:2. The foreshore slope is 1:50 with a 5 m long transitional slope of 1:15.

4.2.3 Wave return wall designs

The sea dike and foreshore are the same for all experimental configurations in every single partition of the wave flume. Different configurations of wave return walls on top of the dike are tested as part of the experimental program. The walls are made of PVC (TROVIDUR EC PVC XT RAL7011). The overtopping is measured for the position of the wall at 0.00 m, 6.75 m and 12.5 m (prototype) from the edge of the dike.

The final geometry (in model scale) for the model is summarised below:

- Model scale = 1:25
- Foreshore slope = 1:50
- Elevation of the toe of dike with respect to the horizontal bottom of the flume = 0.93 m
- Dike slope = 1:2
- Dike crest levels = 0.982m; 1.024m; 1.135m
- Promenade slope = 2%
- Height storm return wall = 1.07cm; 2.07cm; 3.00cm (due to model construction, the two smallest walls finally results 0.07cm higher than foreseen, that mean 0.75cm in 1:1 scale)
- Promenade width = 0.00m; 0.27m; 0.5m

4.3 Test program

4.3.1 Ranges of hydraulic and physical parameters

In total 271 tests were performed of which:

- in 111 tests, the measured wave overtopping was equal to 0 l/s/m.
- 160 tests have non-zero overtopping.
- 42 tests with non-zero overtopping correspond to cases without storm walls and with a promenade width equal to 0 m (i.e. only smooth dike).

The range of significant wave height, peak period and water depth at the wave generator (expressed as *offshore* values) are reported in Table 7, already expressed in prototype scale.

Table 7 – Range of the input hydraulics parameters at the wave generation (prototype scale).

$H_{m0,offshore}$ [m]	$T_{p,offshore}$ [s]	Water depth at the wave generation [m]
1.00-3.75	11.30-16.00	22.50-25.43

The range of hydraulics and physical parameters used during the experimental campaign and referred to the toe of the structure is reported in Table 8 (prototype values).

Table 8 – Range of hydraulics and physical parameters as a result of the experimental campaign.

$H_{m0, toe}$ [m]	$T_{m-1,0, toe}$ [s]	A_c [m]*	Freeboard [m]	Water depth at the toe of the dike [m]**	Wall height [m]	Promenade width [m]
0.30-1.81	11.38-61.50	-0.11*-3.00	0.38-6.00	-1.00**-2.2	0.00, 0.27, 0.52, 0.75	0.00, 6.75, 12.50

*Negative values of Ac correspond to SWL above the dike crest. Set-up is not included.
**Negative values of the water depth correspond to emergent toes (e.g. -1.00 means that the toe is at 1.00m toe above the SWL). Set-up is not included

Where A_c is equal to the difference between the elevation of the dike edge and the still water level, so it doesn't consider the presence of wall and promenade. The freeboard is the difference between the total dike elevation (including eventual wall and promenade) and the still water level. The still water level is the water level measured at time=0 s of each physical model test and doesn't include the wave setup.

4.3.2 Calibration of wave boundary conditions

The scope of the calibration phase in the wave flume is that the wave boundary conditions at the toe of the dike are approximately equal to the wave conditions in SWASH 2DH model. The wave height H_{m0} must be within ± 3% of the SWASH 2HD results, the spectral wave period $T_{m-1.0}$ within ± 5% of the SWASH 2HD results, and the average water level (SWL) within ± 5 cm (in prototype scale). To achieve this goal, the wave height and peak period at the wave generator and the water level in the wave flume need to be adjusted.

4.3.3 Repeatability

The results of the calibration are shown in Table 9 for the cross sections 099, 105.2 and 106.1. Three different tests (with the same wave time series) were conducted for each profile in order to check the repeatability in term of wave conditions. The results shown in Table 9 demonstrate that the tests were repeatable and similar wave height, period and water level were achieved for three different tests with the same wave generation inputs. The wave conditions used at the wave generation are reported in Table 10 and compared with IMDC (2015). It must be noticed that one test from profile 099 has an initial water level 8 cm smaller than the other two tests: this difference can cause partially the differences in the values measured at the toe (Table 9).

4.3.4 Comparison with SWASH

For profiles 105.2 and 106.1 the wave conditions were successfully calibrated to within the specified tolerances. The differences in water level for profile 106.1 is 6 cm or 7 cm, hence they are larger than the tolerance of 5 cm (prototype), however the results are still considered acceptable due to the wave gauge accuracy (≈ 1 mm corresponding to 2.5 cm in prototype). A remarkable difference is noticed for the 099 profile: the calibration started using the same boundary conditions indicated in IMDC (2015) for SWASH 1DH modelling, however it resulted very difficult from the beginning to achieve similar wave conditions at the toe; then the wave height at the wave generation was drastically reduced and the wave period increased, but still the similar setups were not achieved. After several attempts, it was not possible to get all three values of wave height, wave period and wave setup at the toe of the dike as in SWASH 2DH. There could be several possible explanations, related to numerical and physical model inaccuracies.

Table 9 – Results of FHR vs SWASH calibration.

		SWASH 2D Wave conditions at the toe			*FHR Wave conditions at the toe*			*Difference SWASH2D-FHR(toe)*		
Profile	**#test rep.**	**H_{m0} [m]**	**$T_{m-1,0}$ [s]**	**SWL [m TAW]**	**H_{m0} [m]**	**$T_{m-1,0}$ [s]**	**SWL* [m TAW]**	**$\varepsilon_{\_Hm0}$ [%]**	**$\varepsilon_{\_Tm-1,0}$ [%]**	**$SWL^*_{(FHR)}$-$SWL^*_{(SWASH2D)}$ [m]**
099	1	1.26	16.46	+7.10	1.34	12.27	6.91	6	-25	-0.19
099	2	1.26	16.46	+7.10	1.28	12.60	6.84	1	-23	-0.26
099	3	1.26	16.46	+7.10	1.31	11.75	6.91	4	-29	-0.19
105.2	1	0.58	39.74	+7.45	0.60	41.59	7.45	3	5	0.00
105.2	2	0.58	39.74	+7.45	0.59	41.60	7.45	3	4	0.00
105.2	3	0.58	39.74	+7.45	0.60	41.57	7.46	3	5	0.01
106.1	1	0.85	25.27	+7.16	0.84	24.53	7.09	-1	-3	-0.07
106.1	2	0.85	25.27	+7.16	0.84	23.91	7.09	-1	-5	-0.07
106.1	3	0.85	25.27	+7.16	0.85	23.97	7.10	0	-5	-0.06

*set-up is included

Table 10 – Offshore wave conditions in SWASH1D and FHR tests.

		SWASH 1D Wave conditions offshore			*FHR Wave conditions offshore*		
Profile	**#test rep.**	**$H_{m0,offshore}$ [m]**	**$T_{p,offshore}$ [s]**	**SWL [m TAW]**	**$H_{m0,offshore}$ [m]**	**$T_{p,offshore}$ [s]**	**SWL [m TAW]**
099	1	1.92	11.70	+7.09	1.02	14.13	6.31
099	2				0.98	14.63	6.23
099	3				1.03	13.22	6.31
105.2	1	1.59	12.70	+7.35	1.28	12.80	7.29
105.2	2				1.28	12.80	7.29
105.2	3				1.28	12.80	7.29
106.1	1	1.82	11.70	+7.10	1.21	10.78	7.04
106.1	2				1.24	11.38	7.04
106.1	3				1.25	11.38	7.04

Therefore, a further analysis using SWASH has been carried out: the numerical flume has been set up to mimic the physical flume (same dimensions, foreshore layout, dike position, etc.) and the wave time series coming from the physical model experiments have been used as wave boundary conditions for SWASH 1DH. In this way it has been possible to judge whether or not the numerical results are in agreement with the physical ones.

Firstly, a case from section 105.2 (Test Id Rav7I) has been used to validate SWASH 1DH versus physical model results. The results are depicted in Figure 12 to Figure 14, where the second image from the top in each figure shows the bathymetry. The spectral analysis to determine the incident significant wave height and spectral period at the toe of the dike has been conducted using an ad-hoc matlab script. The same analysis has been conducted using WaveLab 3.66 showing a difference of only 1.5% that is assumed negligible. The use of SWASH 1DH for section 105.2 confirms the physical model results: the wave height in SWASH is only 0.01 m smaller than the experimental one, the wave setup is 0.01m higher and the wave period, that shows the biggest difference, is 13% smaller. Furthermore, qualitatively, the wave spectra from offshore to the toe of the dike show similar shapes between numerical and physical modelling. It has to be considered that the main difference between SWASH 1DH and the physical wave flume is the absorption of the reflected waves at the wave generation. This can cause the small differences, that are still observed and that can be considered minor differences.

Once it has been verified that for a very shallow water case, physical and numerical modelling give similar results, SWASH 1DH has been applied to simulate 4 different cases representative of cross section 099, which do no present very shallow water conditions at the toe. The results are depicted in Figure 15, Figure 16, Figure 17 and Figure 18 for tests Rav1, Rav1B, Rav1E and Rav_extra4 respectively: the main difference among these tests is the wave height at the wave generation, being the biggest one for test Rav1 and the smallest one for Rav1E. The water levels are comparable as the wave periods. In the case of Rav1 test, due to the big wave height offshore (comparable with the water depth at the toe of the structure) the wave breaking occurs far from the toe: as a consequence, the wave height at the toe of the dike is smaller than the wave height offshore. In the Rav1B case the wave height at toe is smaller than the wave height offshore for the numerical model, bigger in the physical model. In the Rav1E case the wave breaking occurs very close to the toe of the dike, the wave shoaling is still the main responsible of the wave transformation: as consequence the wave height at the toe is bigger than the wave height at the wave generation.

The aforementioned results underline that a substantial difference exists between SWASH 1DH results and physical model results for cases where the water depth at the toe of the dike is not so shallow (i.e. H_{m0} at the toe is very similar or smaller than the water depth). From one hand, it is possible the process of wave breaking in SWASH does not represent properly the real wave breaking process as happens in the flume. But it is also possible that model effects exist related to the experimental facility.

The results in Figure 18 confirm that there are no significant differences up to the incipient wave breaking. After the wave breaking, numerical and physical results, tends to diverge. The reason of this divergence has not been clearly identified during the experimental campaign. The concern is whether or not model effects during the experiments prevail on some numerical model inaccuracy. For this stage of the project, due to the aforementioned considerations, the results from profile 099 have been excluded from the analysis that will follow. Further investigation has been conducted afterwards on profile 099 carrying out physical model experiment at UGent facilities (see Chapter 6).

Figure 12 – Rav7I: Very shallow foreshore case (Section 105.2),
comparison of SWASH 1DH results with physical model results. [WaveLab 3.66: Hm0=0.60 m, Tm-1,0=41.59 s]

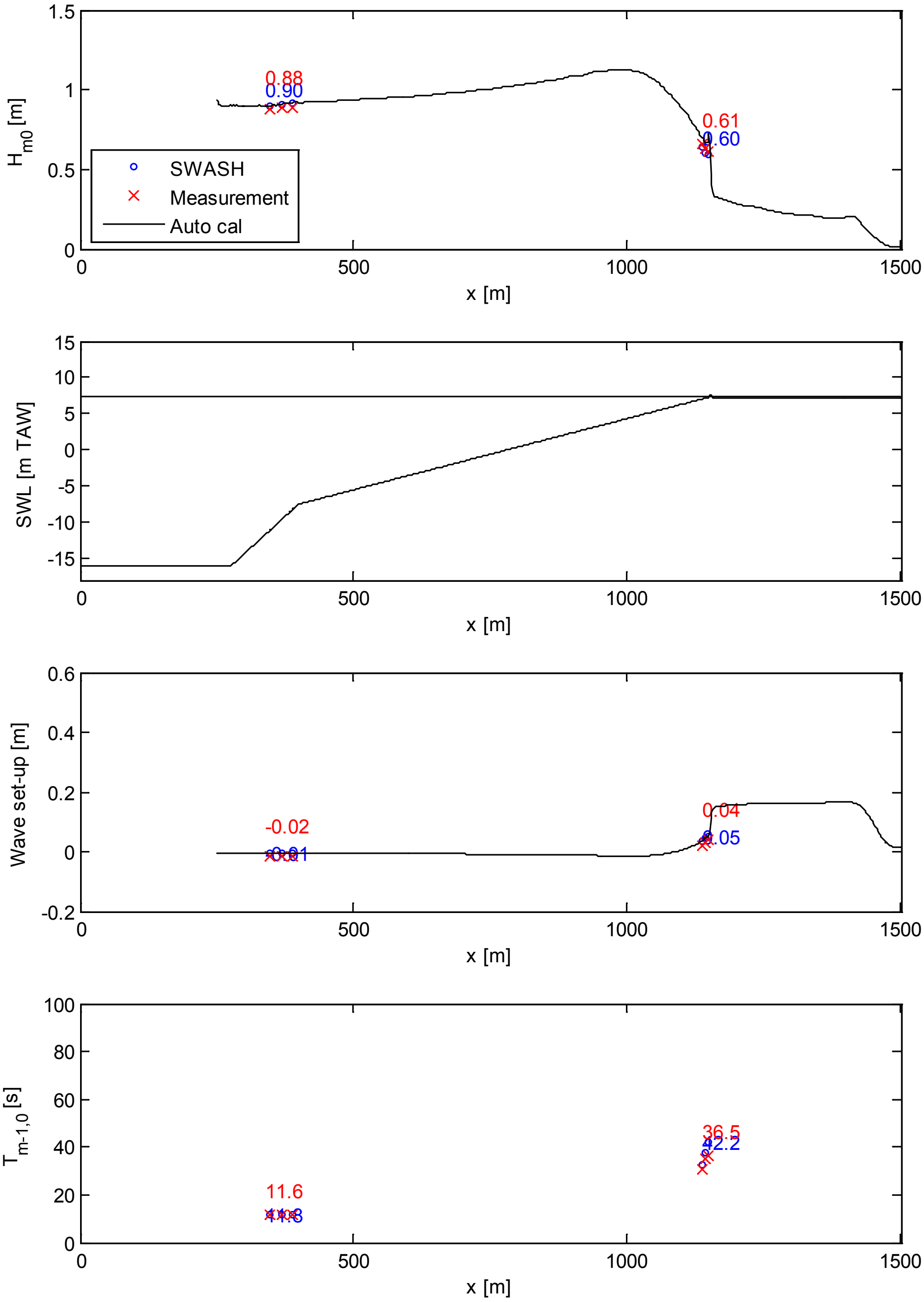


Figure 13 – Rav7I: Very shallow foreshore case (Section 105.2), comparison of SWASH 1DH wave spectra with experimental ones.

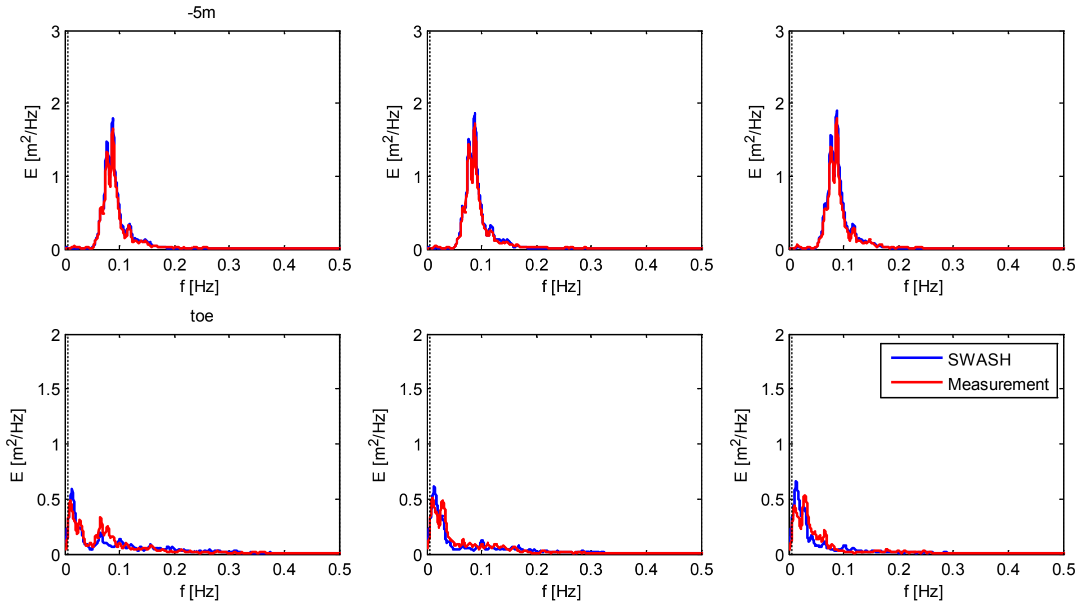


Figure 14 – Rav7I: Very shallow foreshore case (Section 105.2),
comparison of SWASH 1DH time series of water surface elevation with experimental ones.

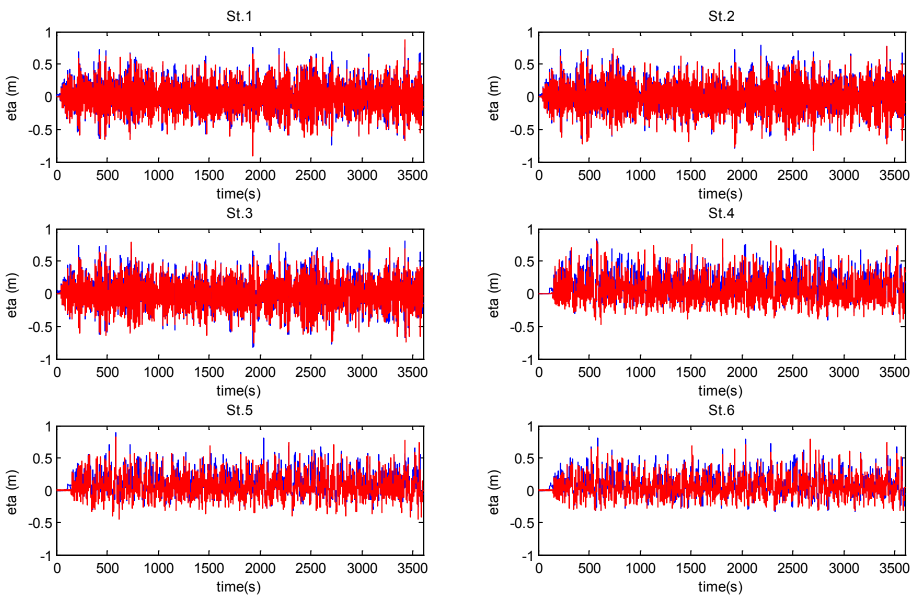


Figure 15 – Rav1, relatively deep water case 1 (Section 099) [Wavelab 3.66: H_{m0}=1.74 m $T_{m-1,0}$=15.76 s]

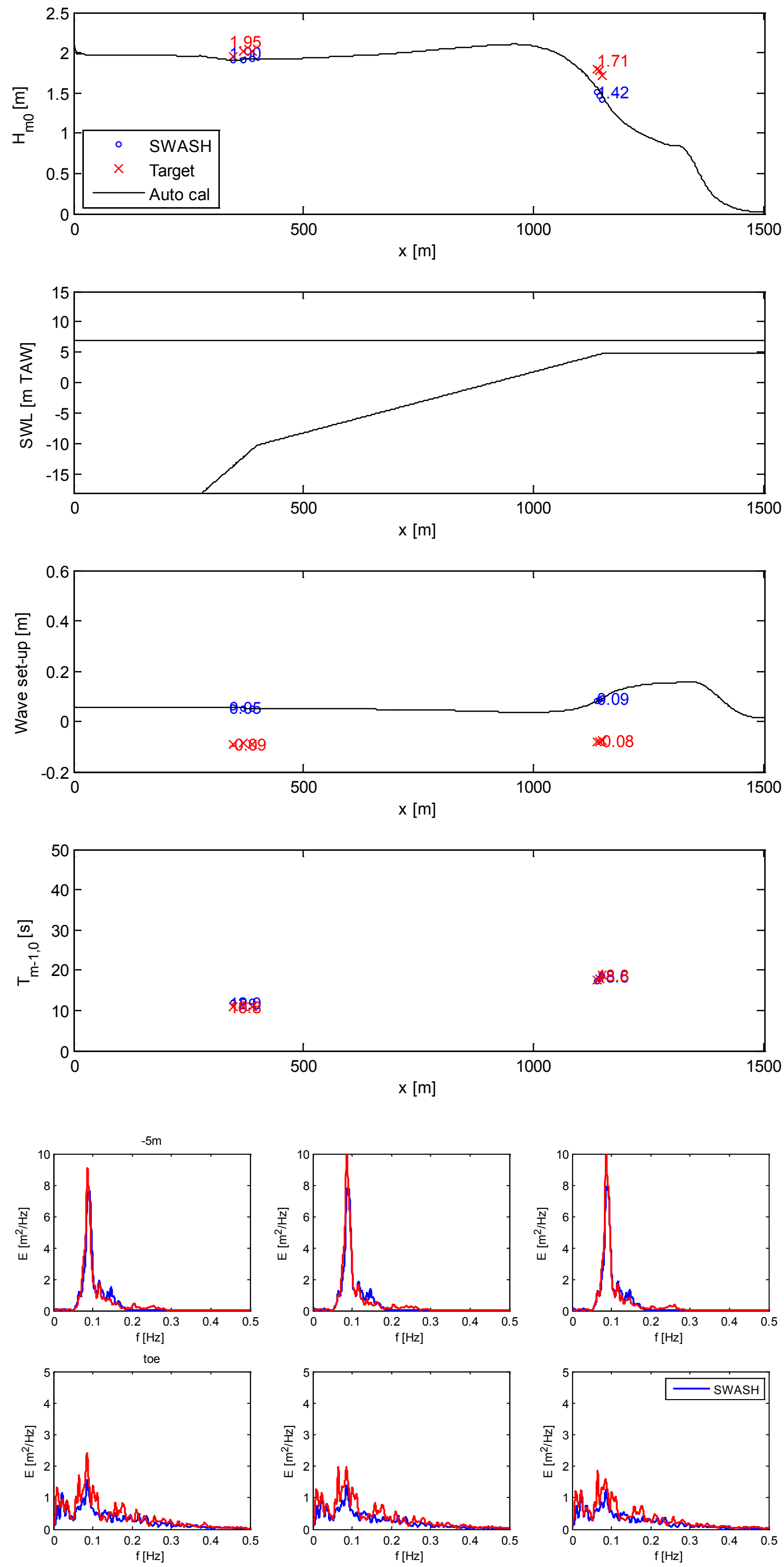


Figure 16 – Rav1B, relatively deep water case 2 (Section 099) [Wavelab 3.66: H_{m0}=1.47 m $T_{m-1,0}$=12.8 s]

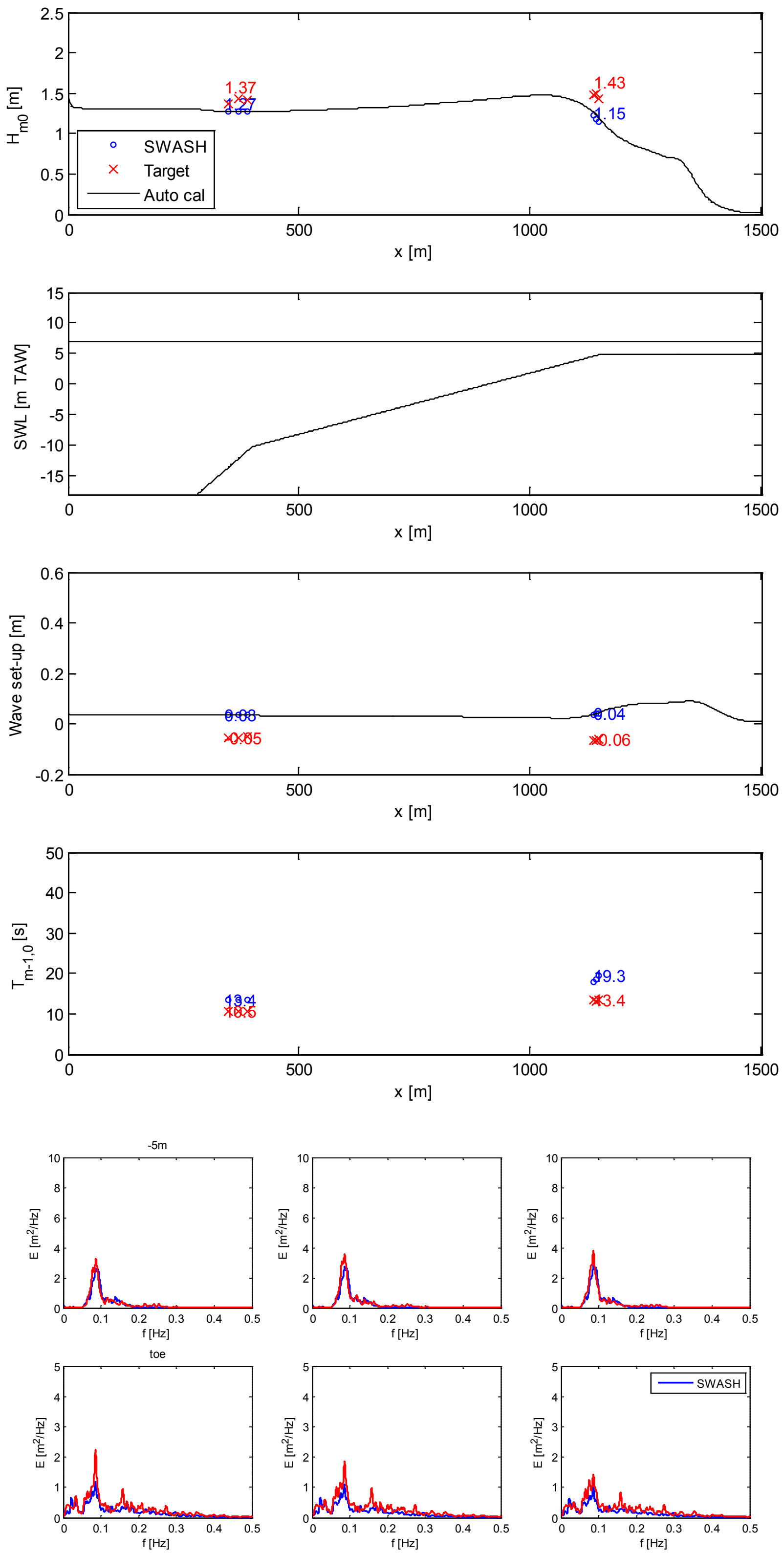


Figure 17 – Rav1E, relatively deep water case 3 (Section 099)

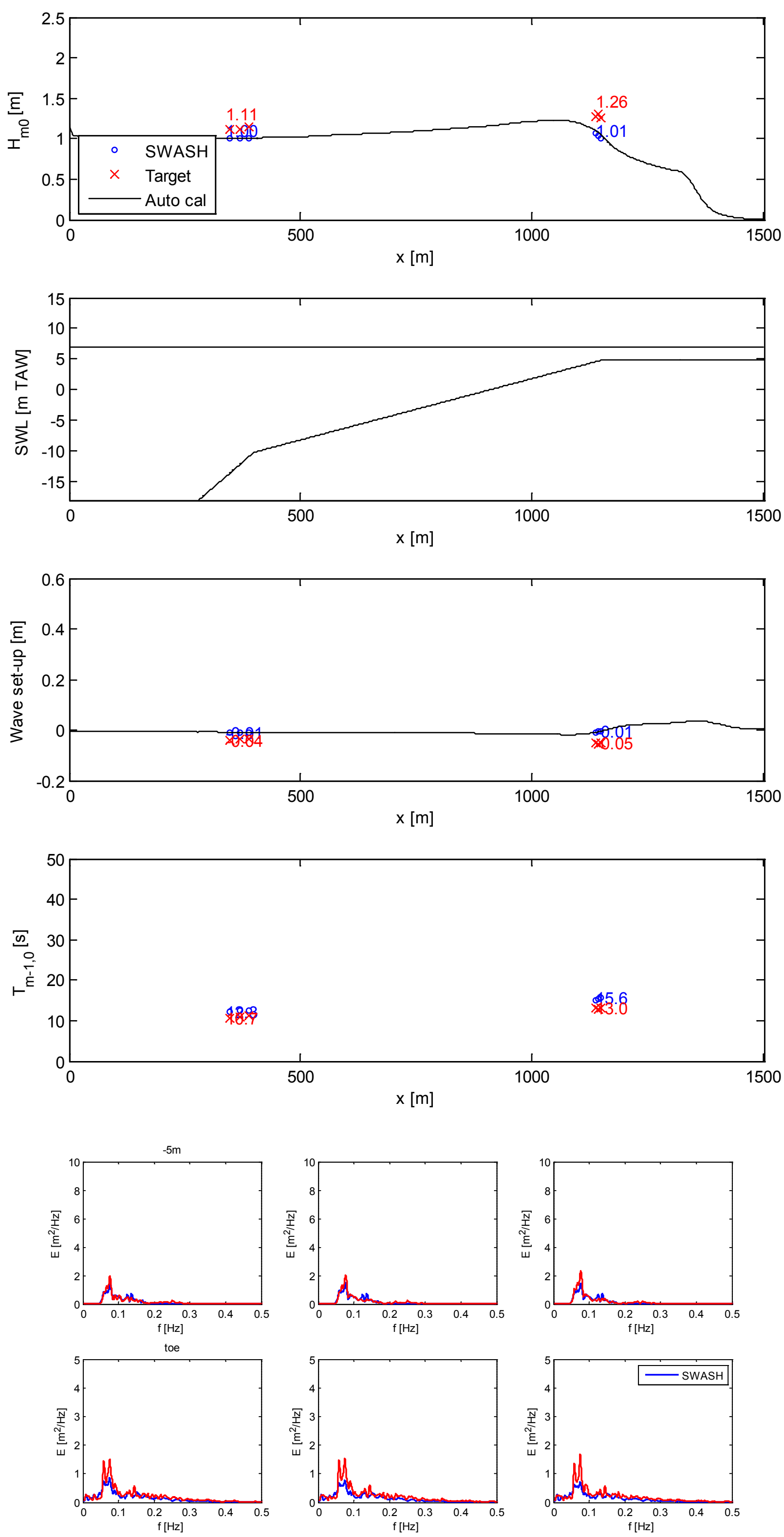
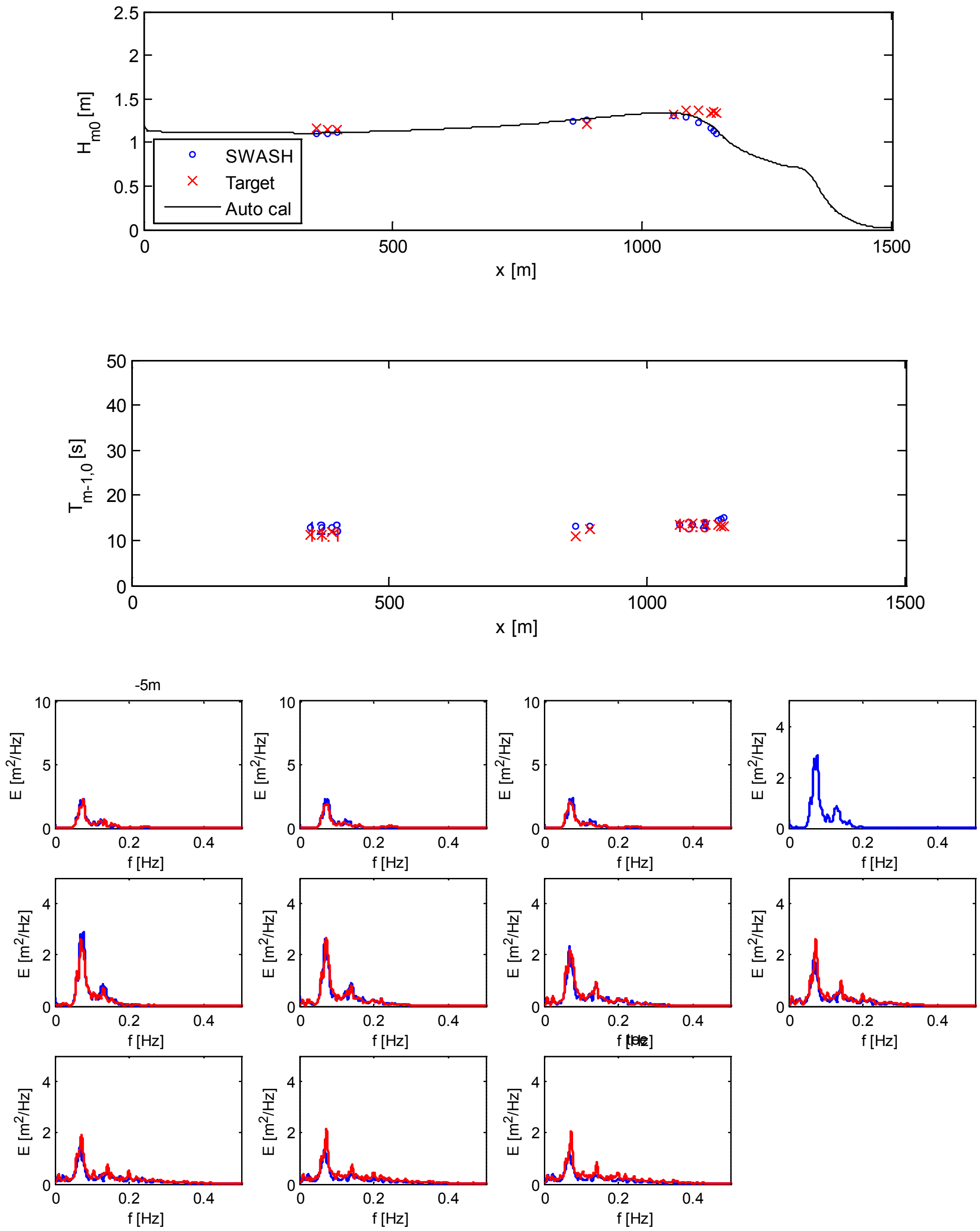


Figure 18 – Rav_extra4, relatively deep water case 4 (Section 099)



4.4 Results and discussions

Wave overtopping was measured for 271 tests: 111 tests have no overtopping. The following analysis refers to the 160 tests with overtopping over the dikes. Test cases whit wave boundary conditions that correspond to profile 099 have been excluded as previously discussed.

4.4.1 Wave overtopping of smooth dikes

Repeatability

The repeatability with respect to wave overtopping has been checked. The results for two different wave conditions are shown in Table 11. For profiles 106.1 and 105.1, the test has been repeated three times using the same time series as wave generation input. The scatter of the overtopping discharge, *q*, among tests within the same group seems reasonable and the tests can be considered repeatable.

Table 11 – Repeatability of wave overtopping results.

Profile	Test Id.	$H_{m0,}$ offshore [m]	$T_{p,}$ offshore [s]	H_{m0} [m]	$T_{m-1,0}$ [s]	R_c [m]	h_{toe} [m]	h_{wall} [m]	Promenade [m]	*Q* *[l/s/m]*
106.1	**Rav5F**	1.21	10.78	0.84	24.54	1.43	0.80	0.0	0.0	7.79
106.1	**Rav5G**	1.24	11.38	0.84	23.91	1.43	0.80	0.0	0.0	8.81
106.1	**Rav5H**	1.25	11.38	0.85	23.97	1.43	0.80	0.0	0.0	9.27
105.2	**Rav7G**	1.28	12.80	0.59	42.43	1.18	0.00	0.0	0.0	1.11
105.2	**Rav7H**	1.27	12.80	0.59	42.54	1.18	0.00	0.0	0.0	1.04
105.2	**Rav7I**	1.28	12.80	0.60	41.59	1.16	0.02	0.0	0.0	1.23

Equivalent slope

The wave overtopping results have been validated against the methodology proposed in Suzuki et al. (2015) to calculate the mean wave overtopping for sea dikes with shallow and very shallow foreshores.

The original formula of Van Gent (1999) for wave overtopping in shallow foreshore is used, but the breaker parameter is calculated using the so-called equivalent slope. The breaker parameter $\xi_{m-1,0}$ is a combination of slope angle and wave steepness. It is defined as:

$$\xi_{m-1,0} = \frac{\tan(\delta)}{\sqrt{2\pi H_{m0} / gT_{m-1,0}^2}} \tag{1}$$

where δ is the equivalent slope angle [°]. In the case of (very) shallow foreshore, large breaker parameters may be found for very low wave steepness. The formula proposed by Van Gent (1999) is expressed by the following two equations:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^c exp\left(-\frac{R_c}{H_{m0}\gamma_f\gamma_\beta(0.33+0.022\xi_{m-1,0})}\right) \tag{2}$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.21 exp\left(-\frac{R_c}{H_{m0}\gamma_f\gamma_\beta(0.33+0.022\xi_{m-1,0})}\right) \tag{3}$$

where:

- q is the overtopping discharge per meter width of the structure [m^3/s/m]
- R_c is the crest freeboard [m]
- γ_f is the reduction coefficient that considers the effects of the slope roughness [-]
- γ_β is the reduction coefficient that considers the effects of the obliqueness [-]

Equations (2) and (3) refer respectively to the probabilistic and deterministic approach as defined in EurOtop (2007). The *c* parameter in equation (2) is assumed as normally distributed parameter with mean value equal to -0.92 and a standard deviation σ equal to 0.24. The 5% upper and under exceedance limits can be calculated as $(-0.92)\pm1.64\sigma$. The value of $10^{-0.92}$ is replaced in equation (3) by 0.21 ($\approx 10^{-0.92+\sigma}$), that represents a safer evaluation of the mean overtopping discharge.

The equivalent slope in shallow foreshore is defined as the average slope, $tan\delta$, in the zone between $SWL-1.5H_{m0}$ and $SWL+Ru_{2\%}$, where SWL is the still water level, H_{m0} is the significant wave height at the toe of the structure and $Ru_{2\%}$ is the wave run-up exceeded by 2% of the incident waves. The average slope is calculated as follows:

$$tan\delta = \frac{(1.5H_{m0}+Ru_{2\%})}{L_{slope}} \tag{4}$$

where the quantity L_{slope} is the horizontal length of the zone between $SWL-1.5H_{m0}$ and $SWL+Ru_{2\%}$. To calculate the wave run-up the following formula is used (eq. 5.4, EurOtop, 2007):

$$Ru_{2\%} = 1.75\gamma_b H_{m0}\xi_{m-1,0} \quad \text{with maximum of}$$

$$Ru_{2\%} = \left(4.0 - \frac{1.6}{\sqrt{\xi_{m-1,0}}}\right)\gamma_b H_{m0} \tag{5}$$

Eq. (5) corresponds to the calculation of the wave run-up height following a probabilistic (i.e. average) approach meanwhile as defined in EurOtop (2007). The wave run-up is not known a priori because its calculation depends on the slope used to calculate the surf similarity parameter. Hence, the application of equation (5) requires an iterative procedure that is stopped when the result for wave run-up is starting to converge. The iterative procedure can be summarized as follows:

1) The first run-up value can be estimated as $1.5*H_{m0}$.
2) With this first run-up value, the new slope is calculated using equation (4).
3) This slope is reintroduced in equation (1) to calculate the new value of the breaker parameter.
4) Then equation (5) is used for the 2nd run-up calculation.
5) These steps have to be iterated until the wave run-up value starts to converge: the difference between 2 consecutive values is smaller than 1%.
6) The final calculated slope will be used as the equivalent slope $tan\delta$.

The dike slope only is used instead of the equivalent slope to calculate the breaker parameter, in case $h_{toe}>1.5*H_{m0}$ where h_{toe} is the water depth at the toe of the dike. For $h_{toe}\leq1.5*H_{m0}$, L_{slope} can be calculated as:

$$L_{slope} = \frac{(Ru_{2\%}+h_{toe})}{tan\alpha} + \frac{(1.5H_{m0}-h_{toe})}{tan\theta} \tag{6}$$

In this way, the average slope can be calculated also in cases with a water level below the toe level, (i.e. emergent toe, $h_{toe}<0$).

The results of overtopping measurements are plotted in Figure 19 and Figure 20. The measured overtopping rates are depicted versus the dimensionless freeboard $R_c/H_{m0}(0.33+0.022\xi_{m-1,0})$, and compared with predictions using Eq. (2) (black solid line) and Eq. (3) (blue dashed line). The overtopping is expressed dimensionless as $Q=q/(gH_{m0}^3)^{1/2}$. The dotted lines indicate the 5% under and upper exceedance limits.

The data plotted in Figure 19 correspond to all results from FHR (including cases with promenade and storm walls). The results from SWASH modelling (IMDC, 2015) and the data from Van Gent (1999) are also plotted for further comparisons. The quite large scatter is mainly caused by the fact that the proposed formula does not consider the influence of storm walls or different promenade widths. Therefore, the data of only smooth dikes without walls and without promenade have been analysed separately. The results are plotted in Figure 20. The equivalent slope concept is proved to be a reliable approach.

However, for a proper evaluation of the formula performance, it is necessary to quantify the error of the prediction. The method proposed in Goda (2009) is used here. The geometric mean (*Geo*) is introduced:

$$\bar{x}_G = exp\left[\frac{1}{N}\sum_{i=1}^{N} lnx_i\right] \text{; with} \quad x_i = \frac{q_{est,i}}{q_{meas,i}} \tag{7}$$

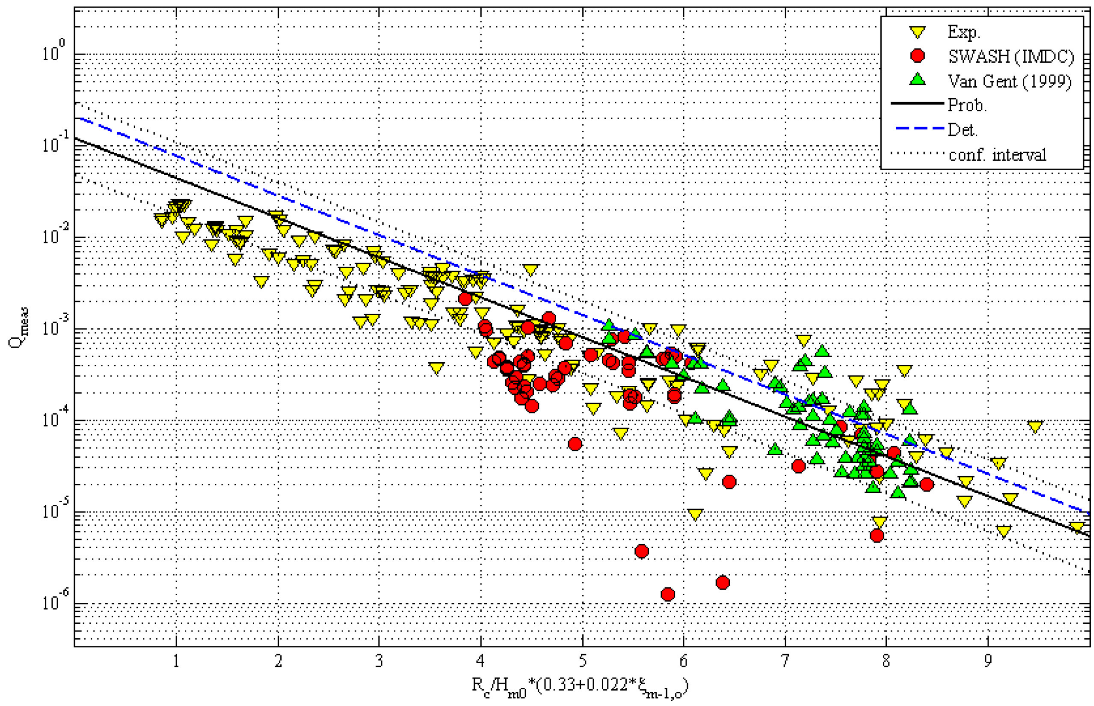
where N is the number of data, $q_{est,i}$ and $q_{meas,i}$ are respectively the *i-th* predicted and measured mean overtopping discharge. The scatter of the data is assessed through the geometric standard deviation (*GSD*) calculated as the exponential value of the standard deviation of the logarithm:

$$\sigma(x_G) = exp\left\{\left[\frac{1}{N}\sum_{i=1}^{N}((lnx_i)^2-(ln\bar{x}_G)^2)\right]^{0.5}\right\} \tag{8}$$

Considering a quantity normally distributed, 90% of the data will be contained in the range between the mean divided by 1.64 times GSD and the mean multiplied by 1.64 times GSD.

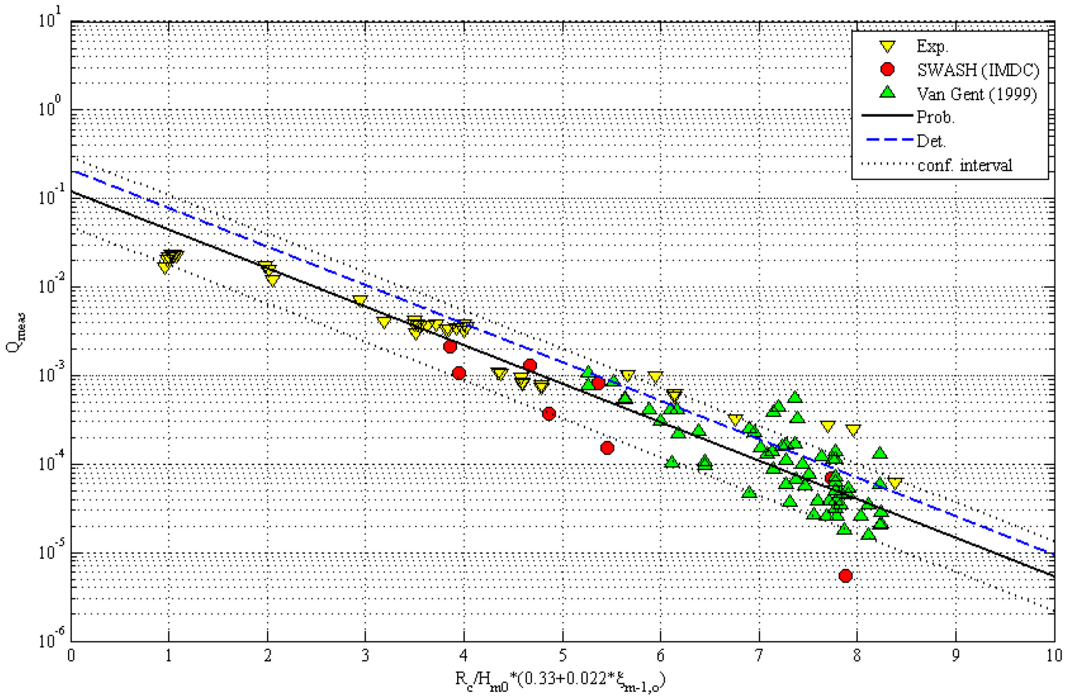
The geometric mean and the geometric standard deviation calculated on the 42 FHR results (no wall, no promenade) are equal to 0.92 and 1.87 respectively. This means that, if the overtopping rate is assumed to be normally distributed, 90% of the predicted overtopping rate is to be located in the range between 0.30 and 2.82 times the measured overtopping discharge. In general, there is a small underestimation of the mean wave overtopping discharge. However, the results are far better than using only Eq. (2) and only the dike slope as proposed in Van Gent (1999) and EurOtop (2007). In fact, the error, calculated using Van Gent (1999) led to the 90% of the predicted overtopping rate in the range between 0.27 and 25.78 times the measured overtopping discharge, with an enormous overestimation of the overtopping. Figure 21 shows the results when Van Gent (1999) formula is applied: both experimental an numerical results are clearly overestimated.

Figure 19 – Wave overtopping data and prediction using the equivalent slope concept



(the dotted lines indicate the 5% under and upper exceedance limits)

Figure 20 – Wave overtopping data and prediction using the equivalent slope concept



(the dotted lines indicate the 5% under and upper exceedance limits)
only tests without storm wall and without promenade.

Figure 21 – Wave overtopping data and prediction using Van Gent (1999).

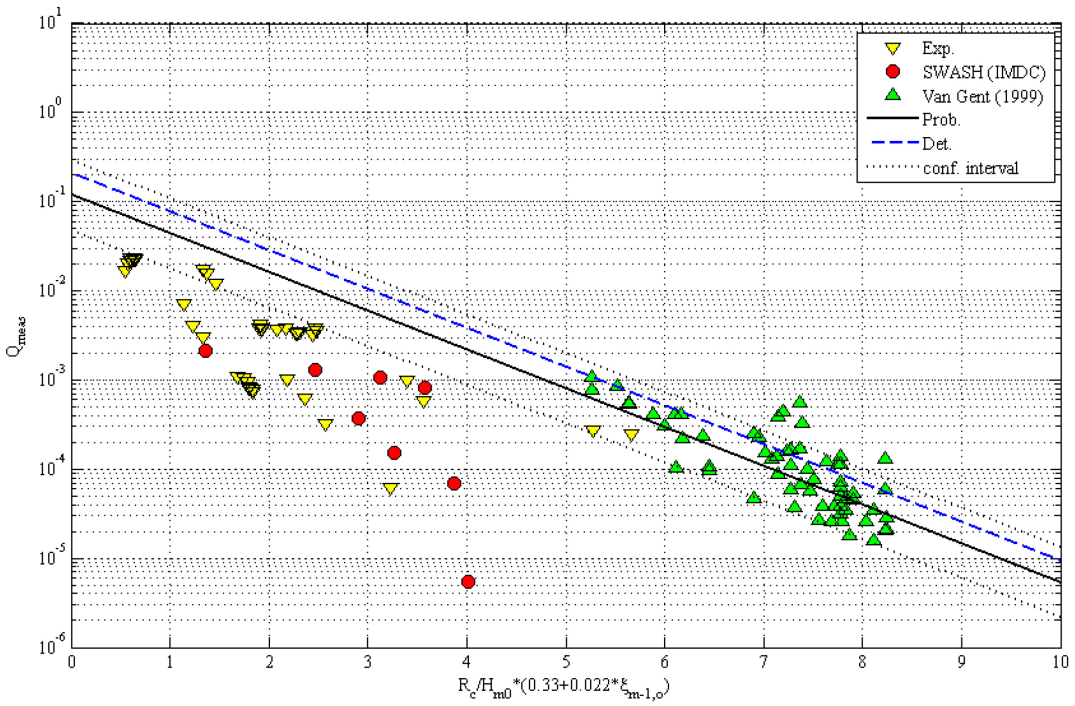
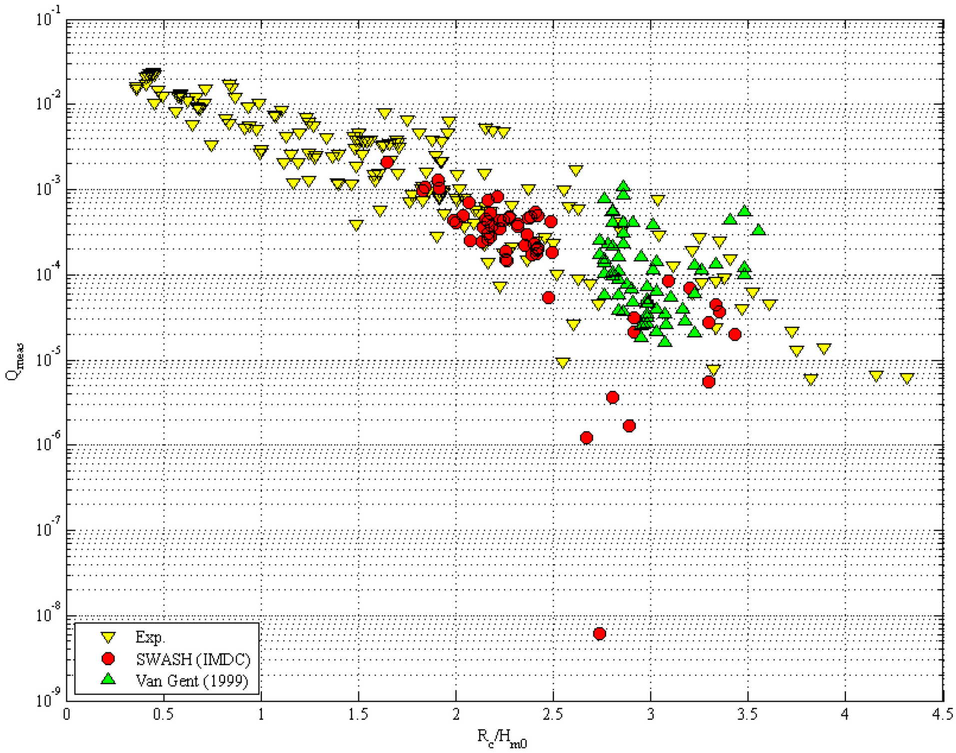


Figure 22 – Dependence of dimensionless overtopping on the non-dimensional freeboard.



Importance of the non-dimensional freeboard

The dimensionless wave overtopping is depicted in Figure 22 versus the non-dimensional freeboard, defined as the ratio between R_c and H_{m0}. It is well known that the non-dimensional freeboard is the most relevant parameter influencing the wave overtopping. The relationship between Q and R_c/H_{m0} is normally represented in a semi-logarithmic plot. The physical model results, plotted together with SWASH 1DH results from IMDC (2015) and Van Gent (1999), data confirm this trend. Few cases from SWASH 1DH seems to diverge due to possible numerical model inaccuracies for low overtopping values.

New fitting

The physical model results have been reanalysed to find out a better fitting for the cases of smooth dikes without wall and without promenade. The present study employs an exponential expression for wave overtopping of smooth and impermeable sea dikes with very shallow foreshore, having the same form as Eq. (2):

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^{c\_new} exp\left(-\frac{R_c}{H_{m0}(0.33+0.022\xi_{m-1,0})}\right) \tag{9}$$

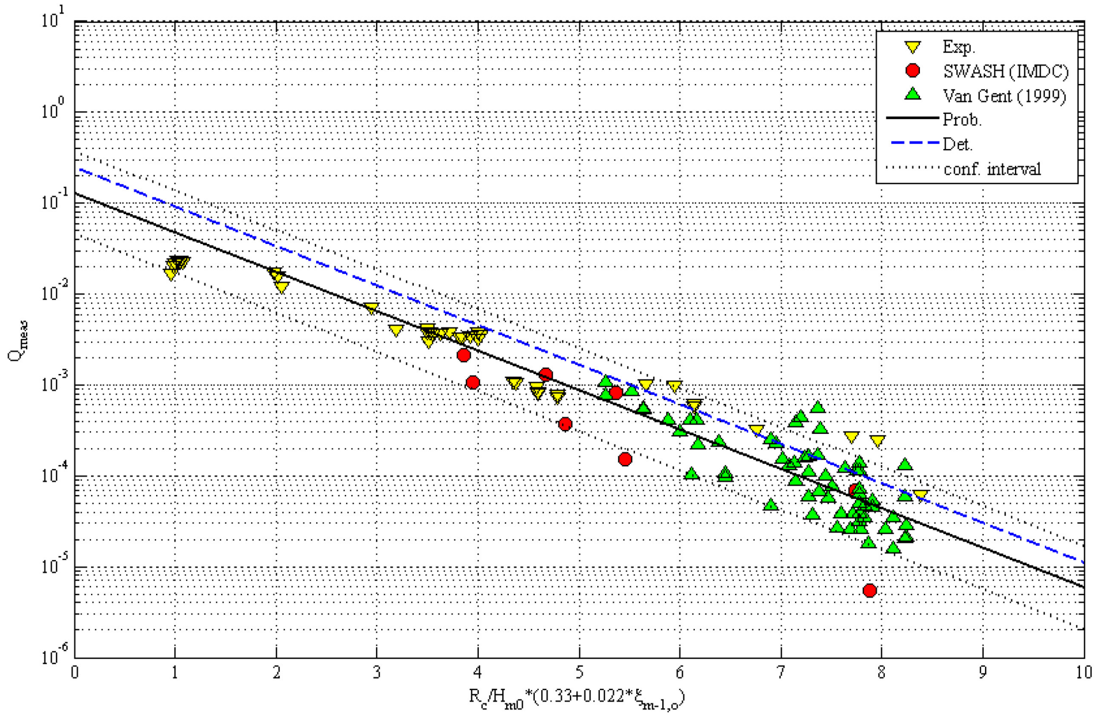
where the new *c_new* parameter calculated based on the analysis of the 42 data from FHR. The new fitting is based on the assumption to use the equivalent slope in shallow foreshore concept and uses the incident significant wave height and spectral period defined at the toe of the sea dike. With respect to Eq. (2) and Eq. (3), the influence factors γ_β and γ_f have been omitted since the effects of wave obliqueness and surface roughness are not analysed in the present study.

The *c_new* parameter in equation (9) is assumed normally distributed: under this hypothesis the mean value of *c_new* is -0.883 and the standard deviation σ is 0.276. The new coefficient of equation (9) is then $10^{-0.883} \approx 0.131$. The 5% upper and under exceedance limits can be calculated as *(-0.883)±1.64σ*. Assuming a safer approach, like in Eq. (3), the value of $10^{-0.883}$ can be replaced by 0.247 ($\approx 10^{-0.883+\sigma}$), that corresponds to the mean value of *c_new* plus one standard deviation.

The results of the new fitting are shown in Figure 23. The dashed blue line in Figure 23 represents the safer approach (still called deterministic, using definitions from EurOtop, 2007). The solid line represents the predictions using Eq. (9), meanwhile the two dotted lines are the upper and lower 5% exceedance probability.

The geometric mean (Geo) and geometric standard deviation (GSD) of each dataset using Eq. (9) have also been calculated. Considering the overtopping rate as normally distributed, 90% of the predicted overtopping rate is to be located in the range between 0.32 and 3.07 times the measured overtopping discharge.

Figure 23 – Wave overtopping data and prediction Eq. 9



(the dotted lines indicate the 5% under and upper exceedance limits)
only data from tests without storm wall and without promenade.

4.4.2 Influence of wall height

The final layout for several profiles along the Raversijde-Mariakerke-Wellington West zone foresees the construction of a storm return wall on top of the sea dike. The wall can be located at the edge of the dike (promenade width = 0 m) or far from it (usually at a distance between 4 m and 14 m, depending on the location). Therefore, model tests with storm walls and promenades have been conducted at FHR to investigate the influence of the wall height and location on wave overtopping discharge.

Tests with storm return wall located at the edge of the dike have been conducted. The results have been analysed to evaluate the influence of the wall height on the wave overtopping.

A distinction needs to be made once again between structural freeboard R_c and the dike freeboard A_c: the former one corresponds to the difference between the maximum structural elevation and the still water level, thereby it includes the wall height; the latter one is the difference between the elevation of the dike crest and the mean water level.

The influence of the wall height on the wave overtopping discharge is shown in Figure 24, where 5 examples from profile 106.1 are shown. For each example the wave conditions and the dike freeboard are indicated. Figure 25 shows the influence of the structural freeboard R_c on the mean wave overtopping for the same cases.

Analysing these graphs, the influence of the wall height appears less remarkable than the influence of the structural freeboard. In fact, in both figures, the overtopping is decreasing almost exponentially when the wall height or the structural freeboard increases. Higher walls mean higher freeboards, so an exponential decrease of the wave overtopping is expected anyway. The same behaviour has been noticed for similar cases with the wall located at 6.75 m or 12.5 m from the edge of the dike. Examples of these results are shown in Figure 26-Figure 29.

Considering two dikes with the same structural freeboard R_c, but one without wall ($A_c=R_c$) and one with wall ($A_c+h_{wall}=R_c$), it might be expected that the overtopping over the first dike is larger than the one over the second dike, due to the effects of the wall that "bends" the overtopping flows seawards.

Therefore, the data have been reanalysed in the following way:

- data from cases with the wall on the edge of the dike have been collected (in total 42);
- the data have been divided in 3 groups, each one characterized by a wall height (0.27 m, 0.52 m, 0.75 m);
- for each *i-th* test, Eq. (9) has been applied to calculate the predicted overtopping discharge $q_{pred,i}$.
- the reduction due to the wall height $\gamma_{wall,i}$ has been calculated as the ratio $q_{meas,i}/q_{pred,i}$ for each *i-th* test.
- for each of the three wall heights a mean reduction factor has been calculated as the average of the $\gamma_{wall,i}$.

This led to the calculation of three reduction factors, respectively for the 0.27 m, the 0.52 m and the 0.75 m wall. The results are plotted in Figure 30. A Reduction equal to 1 means no wall. Finally, the results have been interpolated and a relationship between wall height and wave overtopping reduction has been found. This relationship can be expressed as follows:

$$\gamma_{wall} = \frac{q_{with\ wall}}{q_{without\ wall}} = \exp(-0.672 h_{wall}) \tag{10}$$

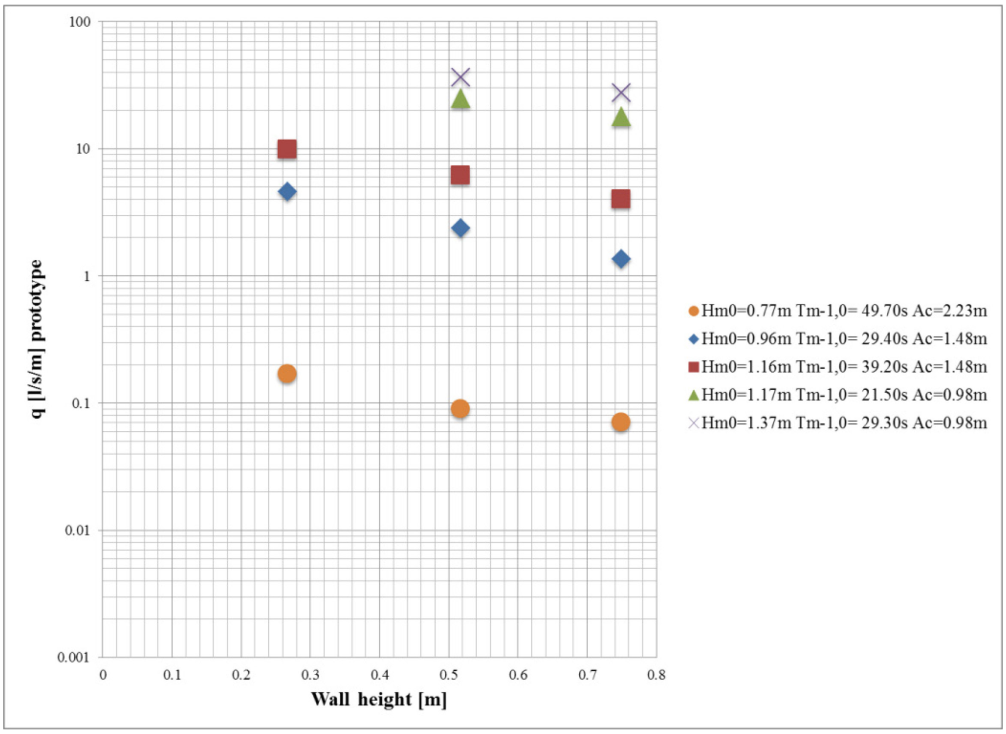
It must be noticed that this expression is function of the only wall height that is a dimensional quantity. Therefore Eq. (10) cannot be generalized and extended to other cases different from those tested for the Raversijde-Mariakerke-Wellington West zone. In other words, Eq. (10) can be used to evaluate the influence of the wall height on wave overtopping for all the profiles in the Raversijde-Mariakerke-Wellington West zone but not for different locations.

Table 12 reports an example where two cases from the experimental dataset have been extracted, with similar water depth at the toe, equal non-dimensional freeboard (R_c/H_{m0}) and similar wave period. The major difference is that for test Rav33 a wall of 0.27 m is installed, meanwhile in case of test Rav10D there is no wall. Despite the wave conditions for the test with wall are slightly larger than the case without wall, both in wave height and period, the mean overtopping discharge results smaller.

Table 12 – Influence of storm wall on wave overtopping: example case.

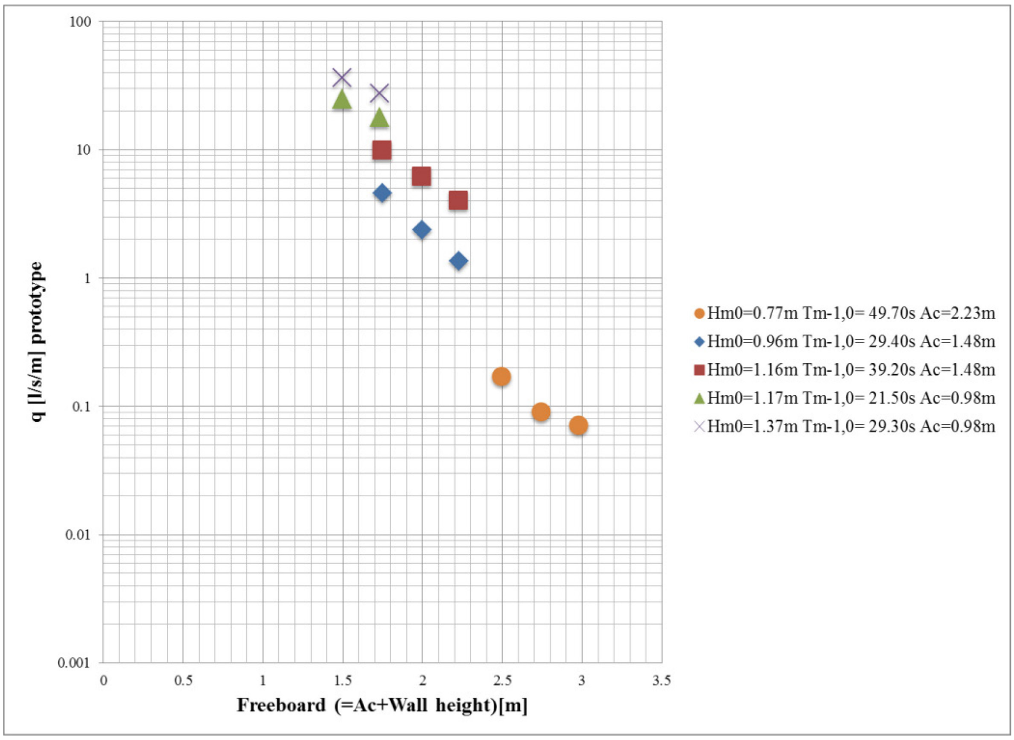
Test Id.	H_{m0} [m]	$T_{m-1,0}$ [s]	R_c [m]	R_c/H_{m0} [m]	h_{toe} [m]	h_{wall} [m]	Promenade [m]	*q* *[l/s/m]*
Rav33	1.14	39.54	1.73	1.52	0.75	0.27	0.0	9.90
Rav10D	0.97	33.19	1.48	1.52	0.75	0.00	0.0	11.01

Figure 24 – Influence of wall height on wave overtopping



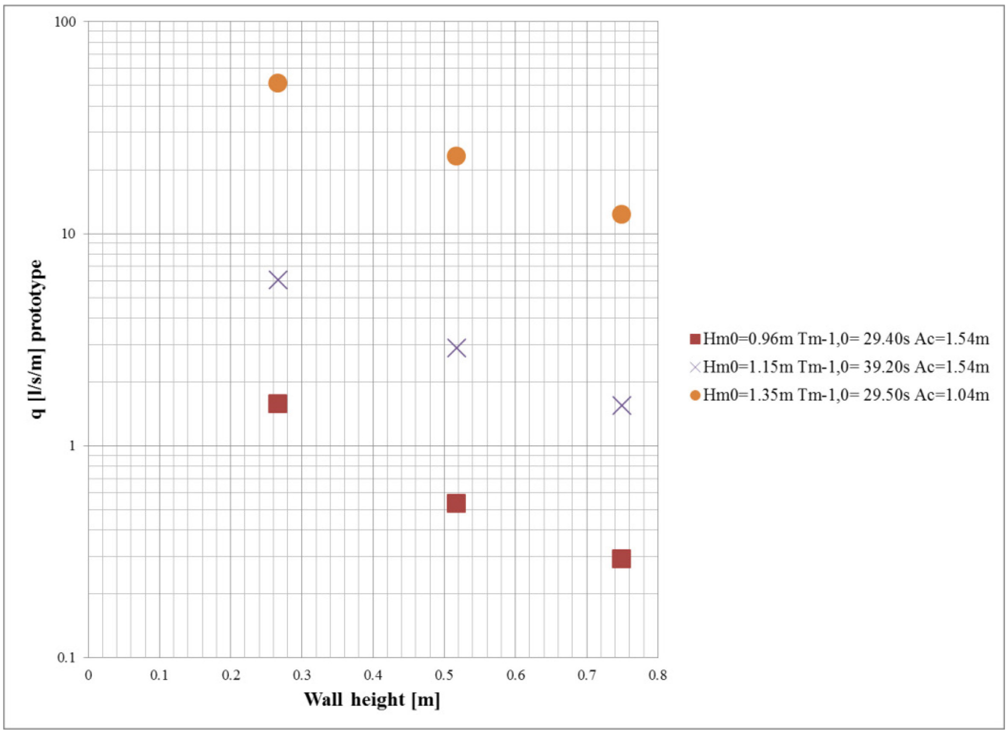
(dike model corresponding to profile 106.1)

Figure 25 – Influence of crest freeboard on wave overtopping



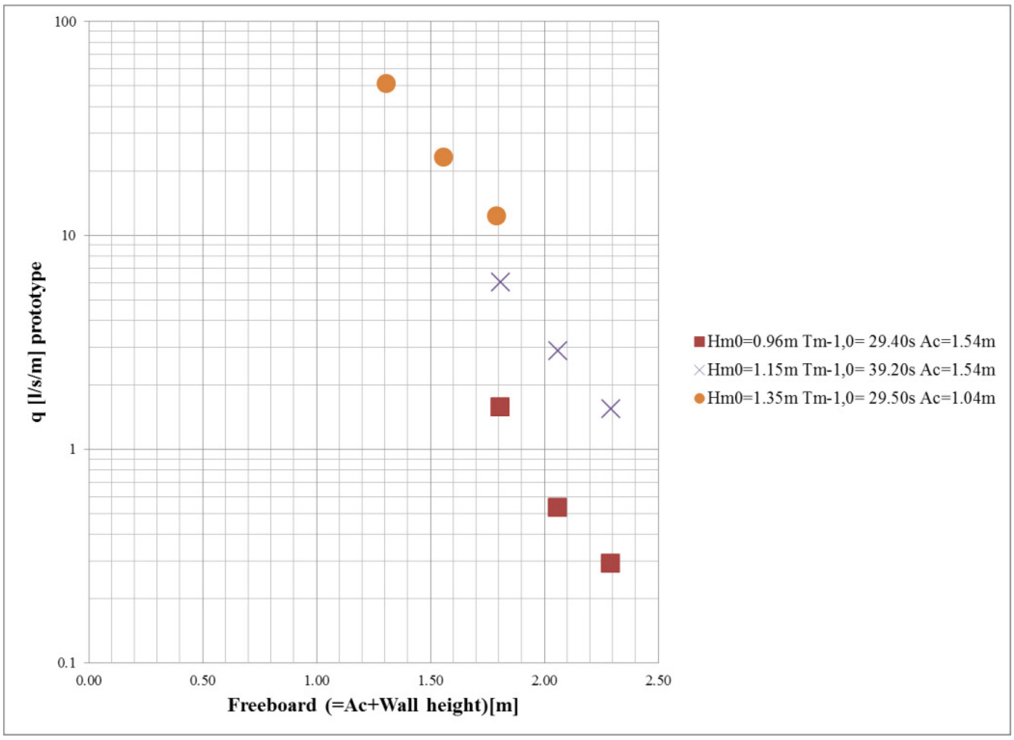
(dike model corresponding to profile 106.1)

Figure 26 – Influence of wall height on wave overtopping



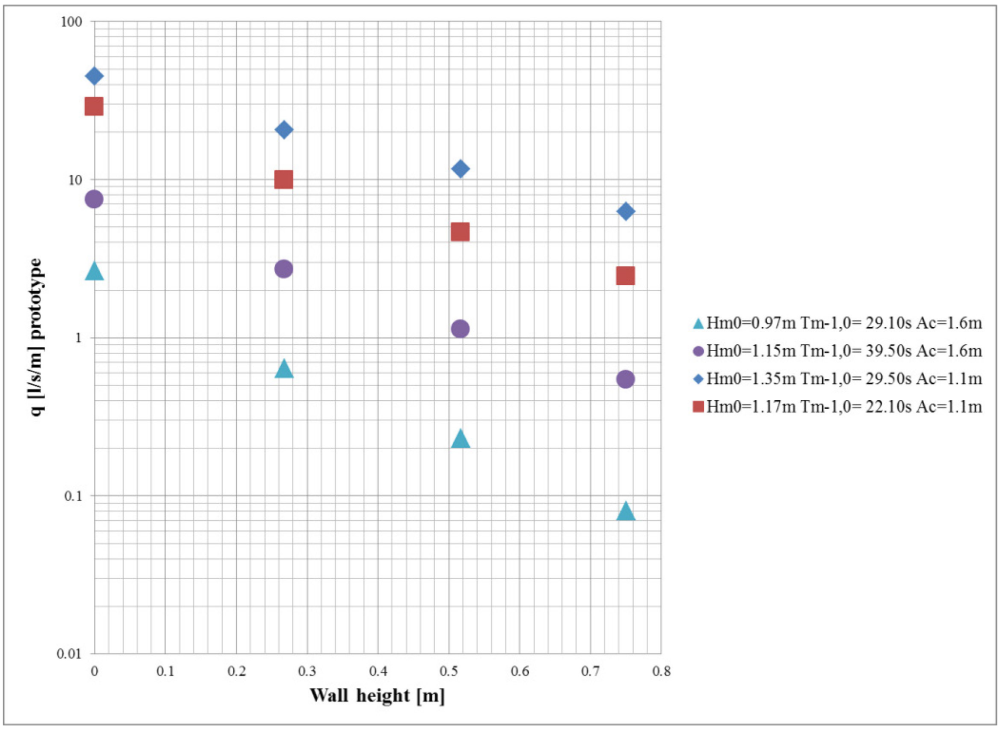
(dike model corresponding to profile 106.1 with 6.75m promenade)

Figure 27 – Influence of crest freeboard on wave overtopping



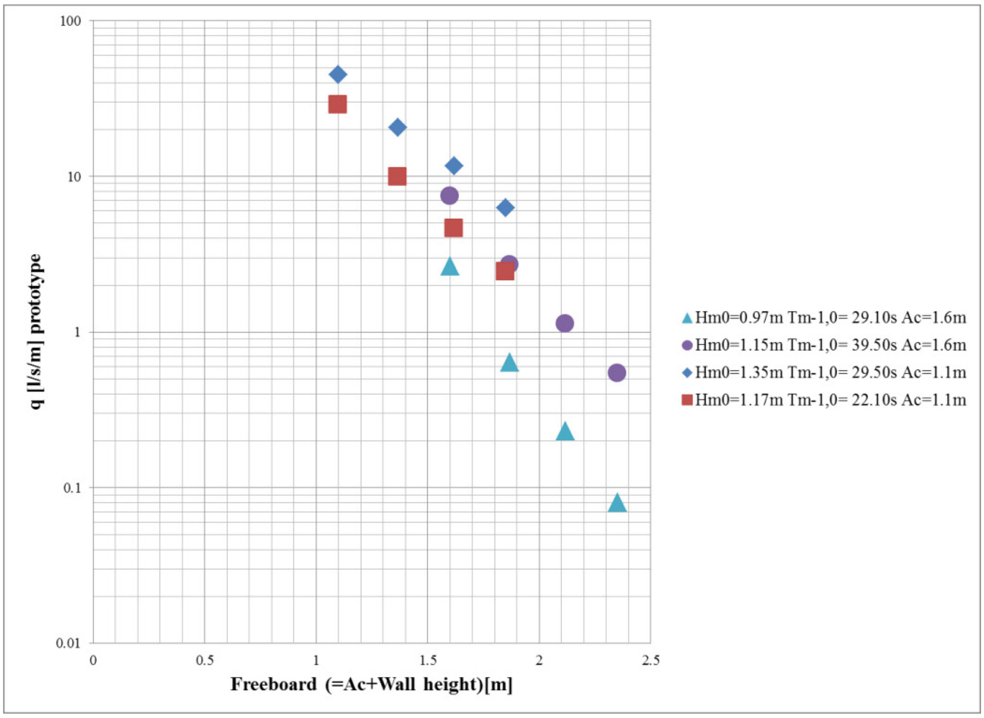
(dike model corresponding to profile 106.1 with 6.75m promenade)

Figure 28 – Influence of wall height on wave overtopping



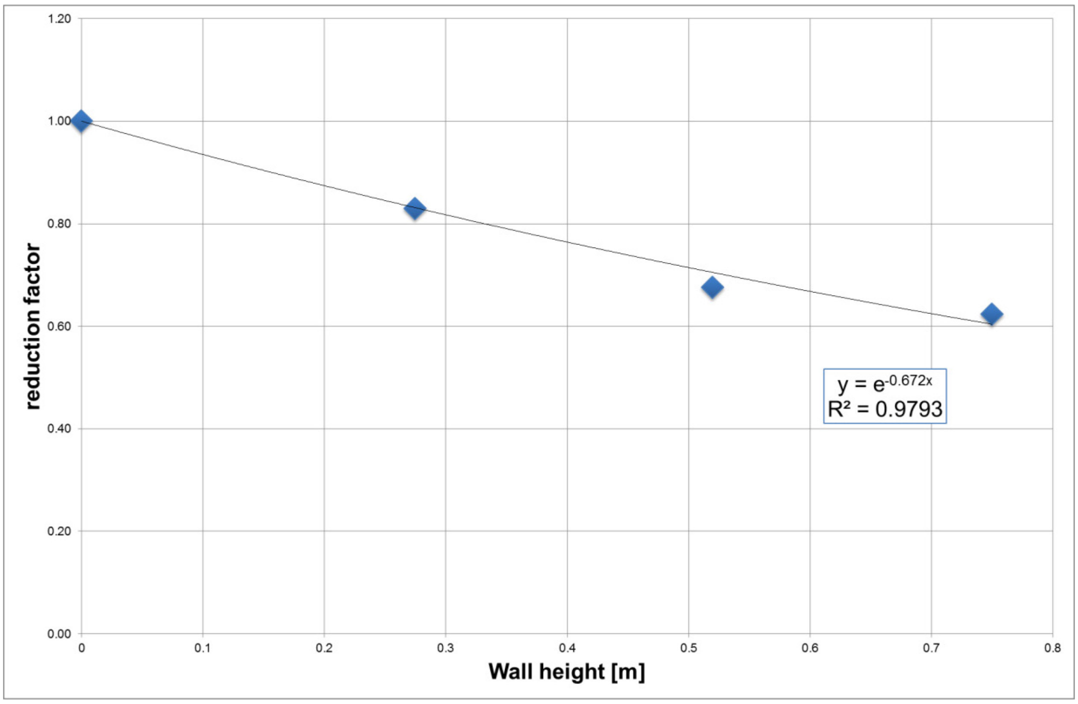
(dike model corresponding to profile 106.1 with 12.50m promenade)

Figure 29 – Influence of crest freeboard on wave overtopping



(dike model corresponding to profile 106.1 with 12.5m promenade)

Figure 30 – Mean reduction of wave overtopping discharge as function of the wall height



4.4.3 Influence of promenade width

The presence of a promenade can reduce the wave overtopping discharge. The reduction is expected to be dependent on the promenade width, calculated starting from the seaward edge of the dike crest. An additional storm return wall can be located at the landward end of the promenade to reduce further the overtopping discharge. To understand the influence of the promenade width on the mean overtopping discharge, several tests have been conducted at FHR where a promenade have been installed after the dikes. Two promenade widths have been tested, corresponding to 6.75 m and 12.5 m (prototype scale). Both tests with and without storm walls at the end of the promenade have been conducted. Examples of overtopping reduction due to different promenade widths are shown in Figure 31 and Figure 32, respectively for the dike models corresponding to the profile 106.1 and the profile 105.2. The overtopping decreases when the promenade width increases.

Figure 31 – Effect of the promenade width on mean overtopping discharge (example case from 106.1 dike profile)

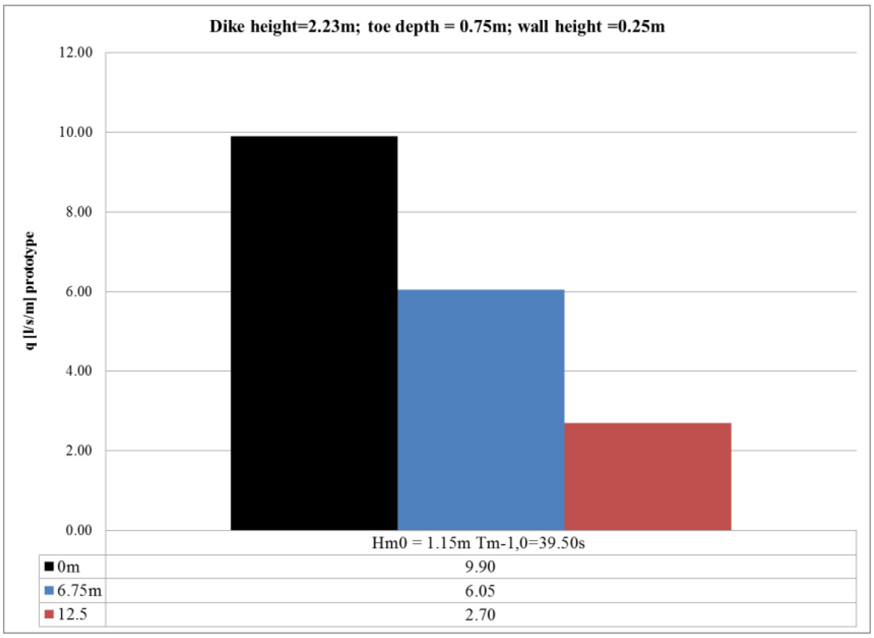
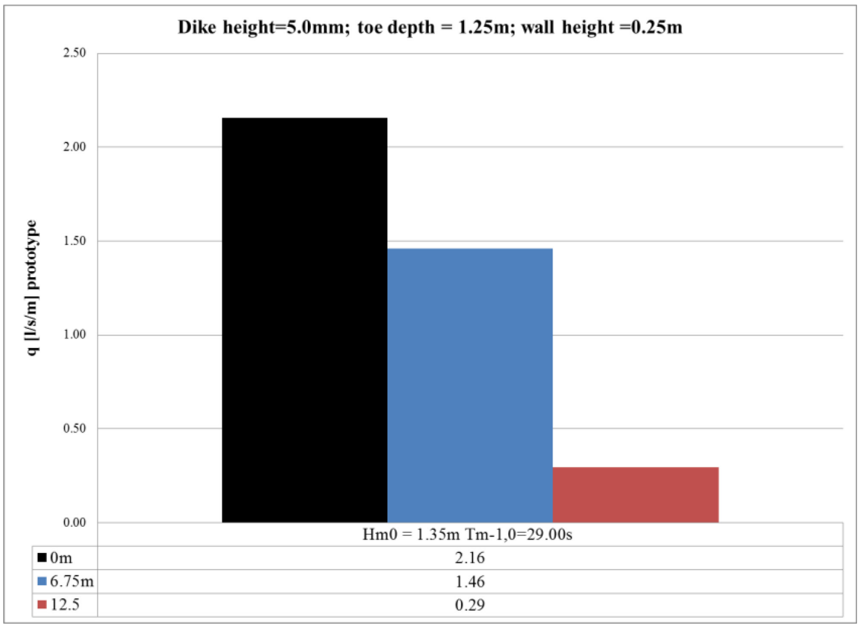


Figure 32 – Effect of the promenade width on mean overtopping discharge (example case from 105.2 dike profile)



To quantify the influence of the promenade width on the overtopping discharge, the data have been reanalysed in the following way:

- data from cases with promenade but without storm wall have been selected (in total 17);
- the data have been divided in 2 groups, each one characterized by a promenade width (6.75 m and 12.50 m);
- for each *j-th* tests, Eq. (9) has been applied to calculate the predicted overtopping discharge $q_{pred,j}$.

- the reduction due to the promenade width $\gamma_{prom,j}$ has been calculated as the ratio $q_{meas,j}/q_{pred,j}$ for each *j-th* test.
- for each of the two promenade widths a mean reduction factor has been calculated as the average of the $\gamma_{prom,j}$.

This led to the calculation of two reduction factors, respectively for the 6.75 m and the 12.50 m promenade. The results are plotted in Figure 33. Reduction equal to 1 refers to cases without promenade. Finally, the results have been interpolated and a relationship between promenade width and wave overtopping reduction has been found. This relationship can be expressed as follows:

$$\gamma_{prom} = \frac{q_{promenade}}{q_{without\ promenade}} = \exp(-0.059B) \tag{11}$$

where *B* is the promenade width.

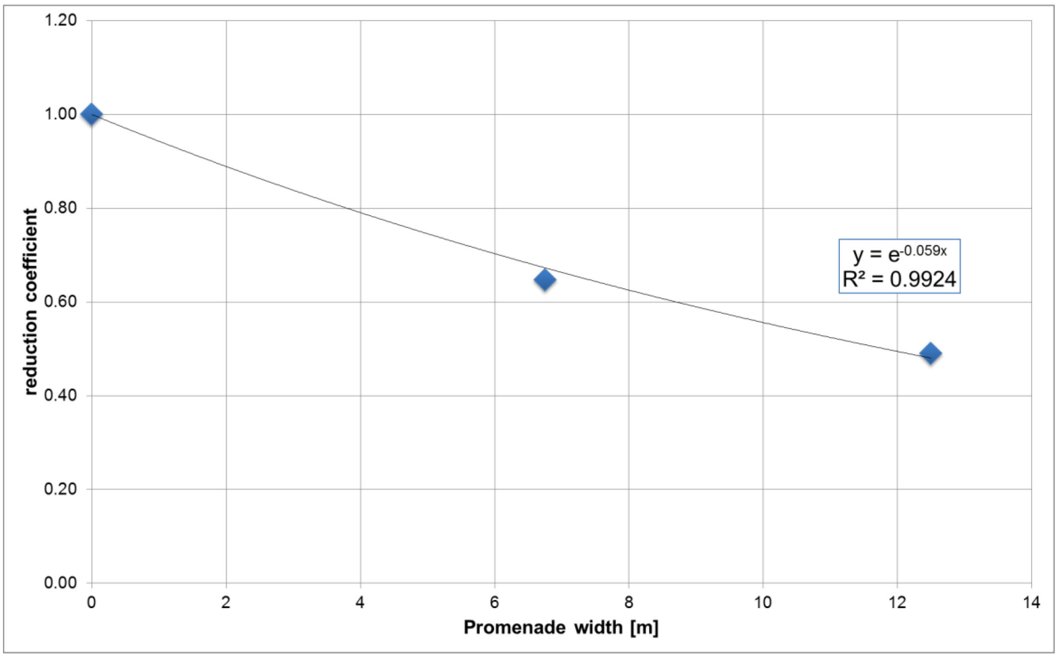
It must be noticed that Eq. (11) is function of the only promenade width that is a dimensional quantity. Therefore Eq. (11) cannot be generalized and extended to other cases different from those tested for the Raversijde-Mariakerke-Wellington West zone. In other words, Eq. (11) can be used to evaluate the influence of the promenade width on wave overtopping for all the profiles in the Raversijde-Mariakerke-Wellington West zone but not for different locations.

Table 13 reports an example where two cases from the experimental dataset have been extracted, with similar water depth at the toe, equal non-dimensional freeboard (R_c/H_{m0}) and similar wave period. The major difference here is that for test Rav107 a promenade of 12.5 m is present, but test Rav10C has no promenade. Despite the wave conditions for the test with promenade are slightly larger, both in wave height and period, than the case without wall, the mean overtopping discharge results smaller. This confirms the influence of the promenade for cases with same non dimensional freeboard, equal water depth and similar period.

Table 13 – Influence of storm wall on wave overtopping: example case.

Test Id.	H_{m0} [m]	$T_{m-1,0}$ [s]	R_c [m]	R_c/H_{m0} [m]	h_{toe} [m]	h_{wall} [m]	Promenade [m]	*q* *[l/s/m]*
Rav107	1.16	39.54	1.73	1.49	0.75	0.00	12.5	7.47
Rav10C	0.99	33.12	1.48	1.49	0.75	0.00	0.0	12.04

Figure 33 – Mean reduction of wave overtopping discharge as function of the promenade width



4.4.4 New equation for overtopping assessment of dikes with shallow foreshore and storm walls

A new equation for wave overtopping assessment is here introduced, based on results of §6.1, §6.2 and §6.3. The equation is expressed as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^{-0.883}\gamma_{wall}\gamma_{prom}exp\left(-\frac{R_c}{H_{m0}(0.33+0.022\xi_{m-1,0})}\right) \tag{12}$$

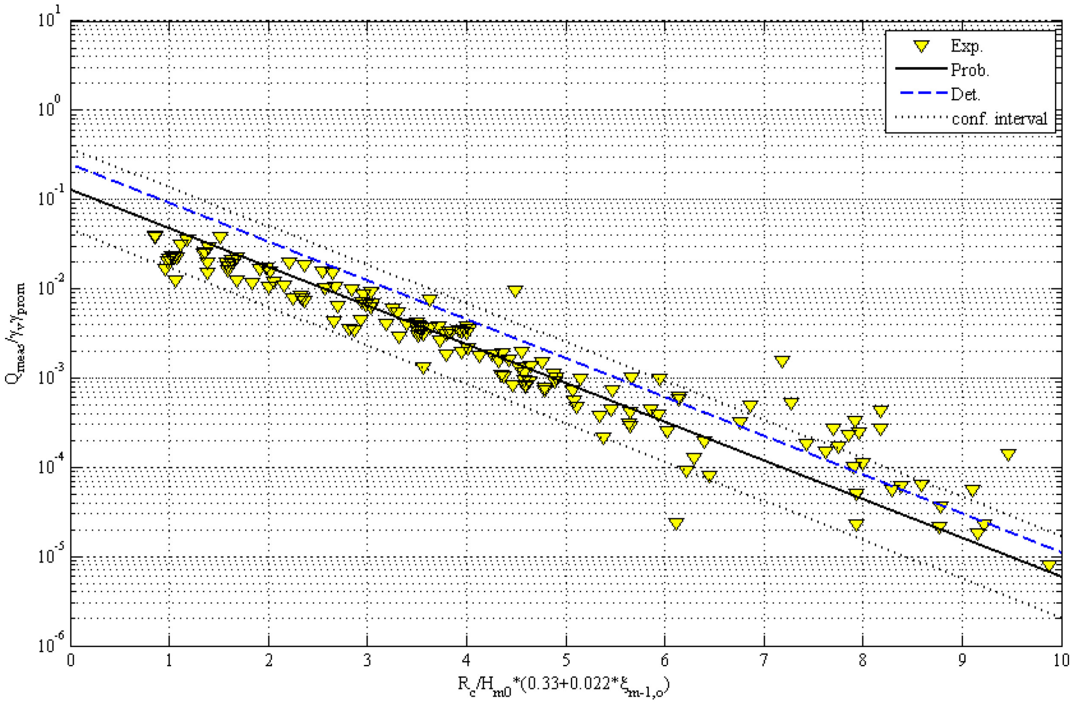
where the reduction coefficients γ_{wall} and γ_{prom} are calculated using Eqq. (10) and (11). The exponent -0.883 is assumed as a normally distributed parameter with standard deviation σ equal to 0.276. The 5% upper and under exceedance limits can be calculated as *(-0.883)±1.64σ*. Assuming a conservative approach, the value of $10^{-0.883}$ can be replaced by 0.247 ($\approx 10^{-0.883+\sigma}$).

<u>The proposed Eq. (12) must only be applied to cases from the Raversijde-Mariakerke-Wellington West zone, within the range of the wave height, wave period, freeboard, wall height, promenade width and toe depth as tested at FHR (see </u>Table 8<u>). Any use outside those ranges, can lead to a misjudgement of the wave overtopping discharge.</u>

The result of application of Eq. (12) is reported in Figure 34 and Figure 35. A scatter is noticeable for very low values of the overtopping discharge, however this was expectable. In fact, low overtopping usually means small wave height and high freeboard, few single overtopping events. The time series of the waves approaching the structure can play a significant role and different wave trains might lead to big differences in overtopping discharges. Further discussions on such source of uncertainties is reported in Williams et al. (2014).

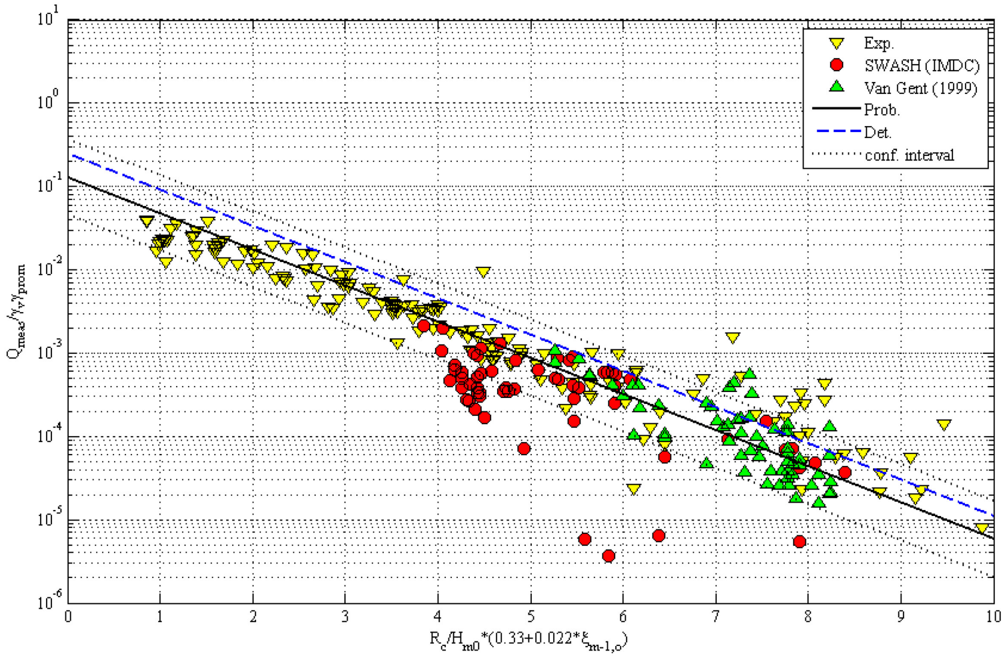
The geometric mean and the geometric standard deviation, calculated on the all FHR results, are equal to 0.95 and 2.09 respectively, in case Eq. 12 is used. This means that, if the overtopping rate is assumed to be normally distributed, 90% of the predicted overtopping rate is to be located in the range between 0.27 and 3.25.

Figure 34 – Wave overtopping data and prediction Eq. 12



(the dotted lines indicate the 5% under and upper exceedance limits)
only experimental data

Figure 35 – Wave overtopping data and prediction Eq. 12



(the dotted lines indicate the 5% under and upper exceedance limits)
all data.

Performance of reduction coefficients

Prior to apply the new formula to the results proposed by IMDC (2015), the performance of the reduction factors is here analysed. In order to do that, results of wave overtopping have been plotted in different colours: each colour corresponds to a different wall height or different promenade width. Firstly, only Eq. (9) has been applied, without any correction for the wall height and promenade width. The results are plotted in Figure 36 and Figure 37. The overtopping discharge is plotted using the non-dimensional quantity $Q_{meas}=q_{meas}/(gH_{m0}^{3})^{0.5}$, where q_{meas} is the mean overtopping discharge expressed in $m^3/s/m$ and g is the gravitational acceleration. All the cases, with and without storm wall and promenade are represented. The effect of the promenade width is immediately noticeable from Figure 37, where systematically the cases with the widest promenade give the minimum overtopping meanwhile the cases without promenade are characterized by the highest overtopping values. Similar conclusions can be drawn for the influence of the wall height, despite the differences are not so discernible as for the promenade effects. However, a trend can be identified.

The performance of Eq. (12), that includes the correction factor for wall height and promenade width, is demonstrated in Figure 38 and Figure 39, where there is not anymore distinction between cases with different wall height or promenade width. The accuracy of the prediction does not depend on the particular geometry and is uniformly spread over all the data.

Figure 36 – Influence of wall height on wave overtopping without the use of correction factors for wall height and promenade width.

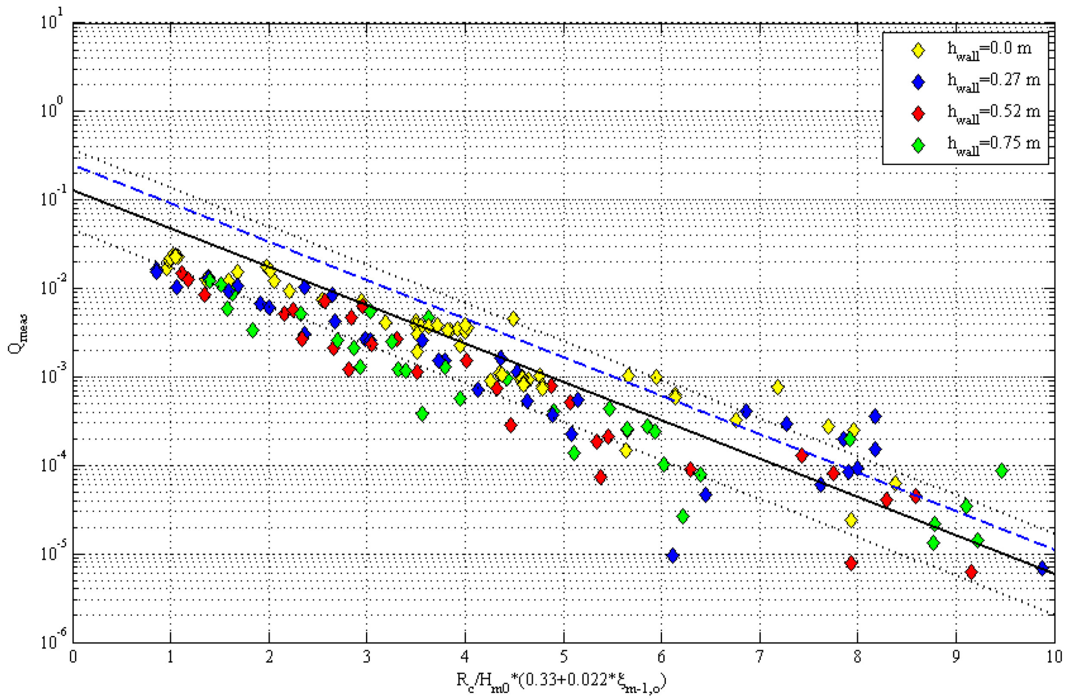


Figure 37 – Influence of promenade width on wave overtopping without the use of correction factors for wall height and promenade width.

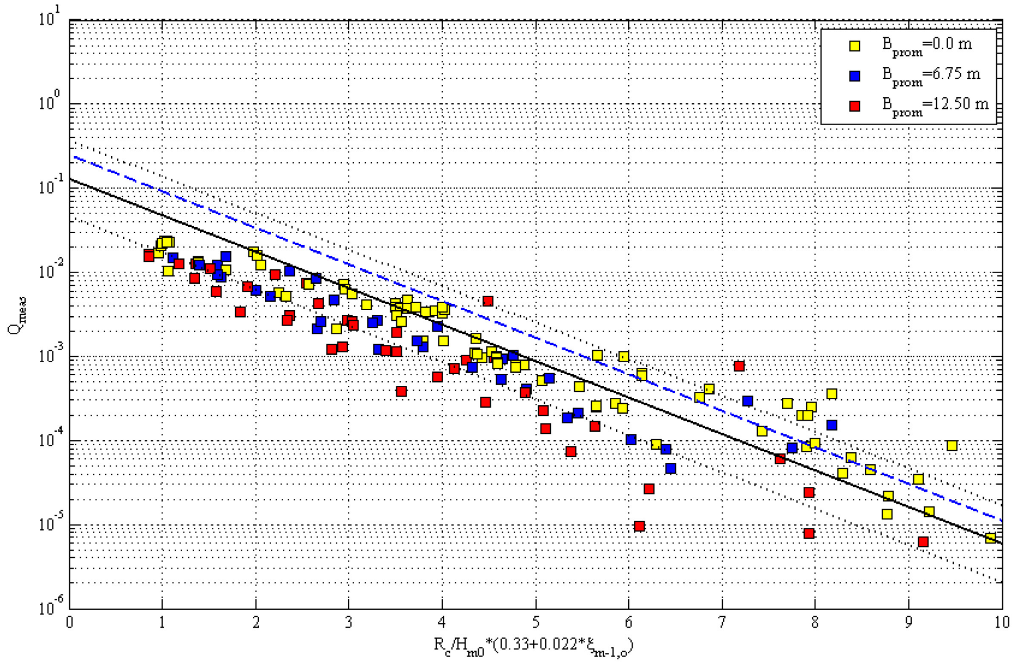


Figure 38 – Influence of wall height on wave overtopping with the use of correction factors for wall height and promenade width.

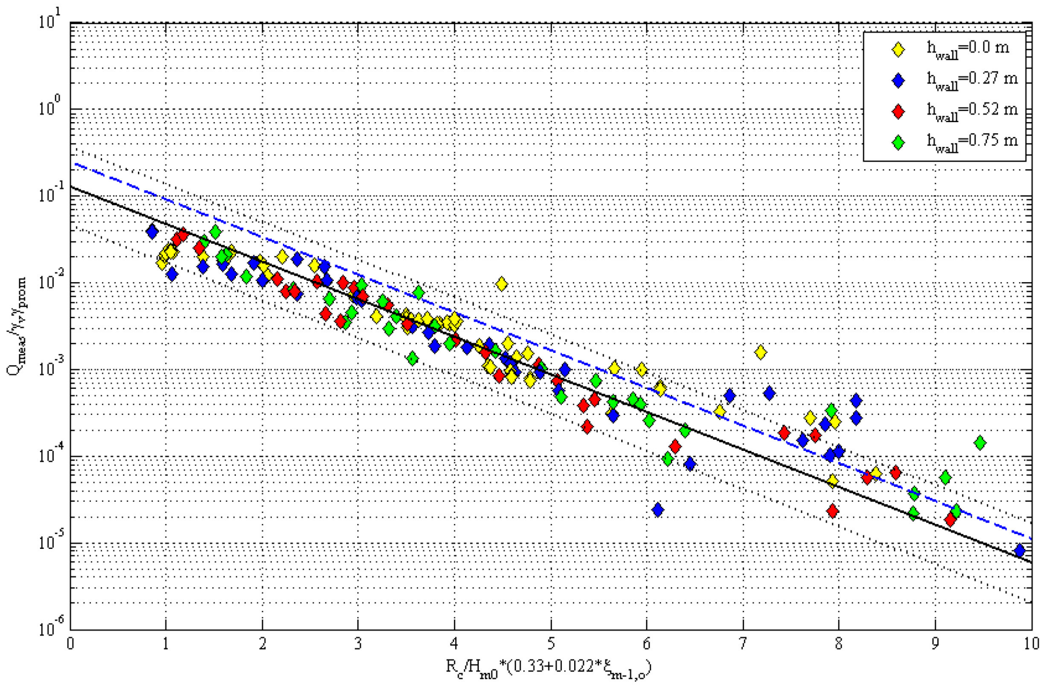
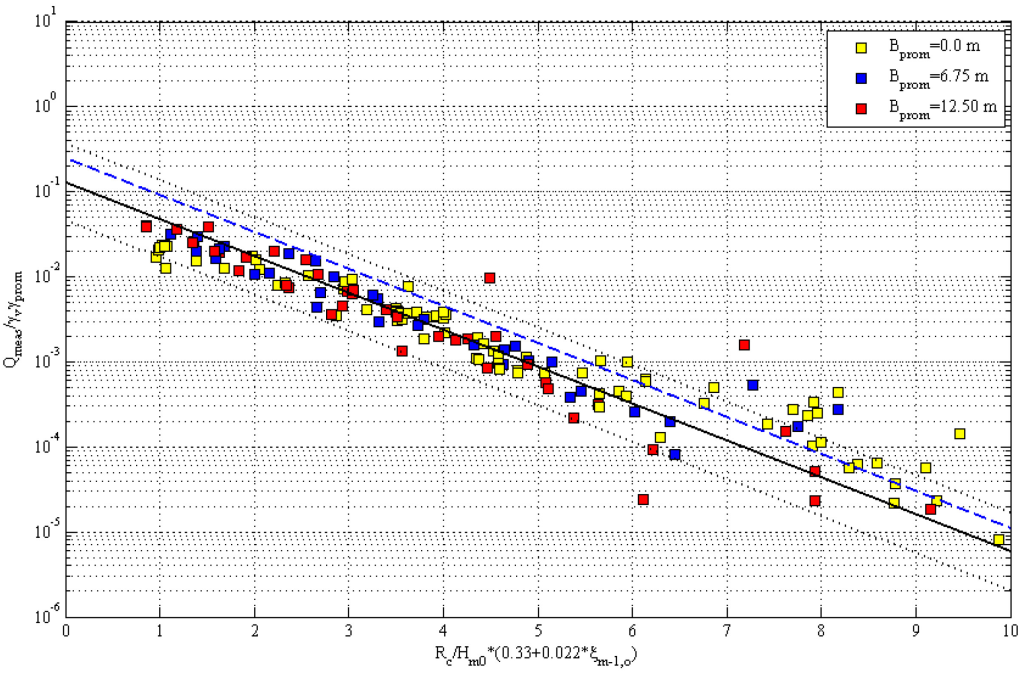


Figure 39 – Influence of promenade width on wave overtopping with the use of correction factors for wall height and promenade width.



Comparison with SWASH1D overtopping results

The new equation has been applied to all the profiles identified in the Raversijde-Mariakerke-Wellington West zone, in order to evaluate the difference between the SWASH results (IMDC 2015) and the predictions made using the semi-empirical formula above described (Eq. 12). The results are summarized in Table 14, where the predicted mean overtopping discharge and the ratio between numerical results and results from Eq. (12) are reported. In general, the application of Eq. (12) gives slightly higher overtopping than SWASH. Removing cases corresponding to profiles 103.21, 108.2 and 108.21 (for which there are concerns on the numerical model reliability on low overtopping discharge cases where uncertainty of the estimation is generally high), Eq. (12) leads to overtopping prediction in average 2.06 times larger than SWASH, excluding cases 103.21, 108.2 and 108.21 characterized by probable numerical model inaccuracies. This difference is not significant, considering all the uncertainties present in wave overtopping phenomena. However, the layouts for each profile will be adjusted to guarantee that the mean overtopping discharge, calculated using Eq. (12), is equal to or less than 1 l/s/m.

Application of the new formula to define the minimum wall height and position

Equation (12) has been applied to calculate the minimum wall height and promenade width (= wall location) to satisfy the overtopping criteria (q≤1 l/s/m). The results are summarized in Table 15. Based on architectural issues, the wall height and position can be varied. In any case, the expected overtopping discharge can be calculated based on Eq. (12). For the calculation the slope of the promenade has been assumed equal to 2% for all the profiles. Note that values shown in Table 15 is not the final layout for the design.

Table 14 – Comparison of predicted overtopping with overtopping results from IMDC (2015).

Profile	H_{m0} [m]	$T_{m-1,0}$ [s]	R_c [m]	Crest elevation at the seaward edge [mTAW]	Elevation of the wall toe [mTAW]	Crest elevation including wall and promenade [mTAW]	h_{toe} [m]	h_{wall} [m]	B [m]	q_{SWASH} [l/s/m]	$q_{(Eq.12)}$ [l/s/m]	*ratio* $q_{(Eq.12)}/q_{SWASH}$
099	1.24	17.28	3.02	**9.81**	**9.94**	**10.10**	2.28	0.16	6.7	1	**3.9**	*3.9*
0991 (variant)	1.24	17.28	3.02	**9.81**	**9.89**	**10.11**	2.28	0.22	4	1	**4.4**	*4.4*
0992 (variant)	1.24	17.28	2.96	**9.81**	**9.81**	**10.06**	2.28	0.25	0	0.97	**5.9**	*6.1*
103.1	0.75	34.34	2.47	**9.73**	**9.73**	**9.73**	0.32	0	0	0.14	**0.1**	*0.7*
103.2	0.7	33.01	2.38	**9.66**	**9.81**	**9.81**	0.35	0	7.3	0.05	**0.1**	*1.0*
103.21 (variant)	0.7	33.01	2.38	**9.66**	**9.66**	**9.66**	0.35	0	0	0.01	**0.1**	*8.2**
104.1	0.89	26.13	2.11	**9.23**	**9.38**	**9.38**	0.72	0	7.4	0.49	**0.8**	*1.5*
104.11 (variant)	0.89	26.13	2.11	**9.23**	**9.23**	**9.23**	0.72	0	0	0.4	**1.2**	*2.9*
104.2	0.92	22.53	2.05	**9.07**	**9.07**	**9.16**	1.26	0.09	0	1	**4.1**	*4.1*
105.1	0.74	23.66	1.9	**8.84**	**8.84**	**8.99**	0.6	0.15	0	1	**0.5**	*0.5*
105.2	0.57	41.49	1.32	**8.43**	**8.43**	**8.67**	0.09	0.24	0	0.94	**0.6**	*0.7*
105.3	0.78	27.09	1.98	**8.7**	**8.70**	**9.06**	0.51	0.36	0	1	**0.5**	*0.5*
106.1	0.83	26.16	1.76	**8.47**	**8.72**	**8.86**	0.86	0.14	12.5	0.97	**1.3**	*1.3*
106.11 (variant)	0.83	26.16	1.76	**8.47**	**8.47**	**8.86**	0.86	0.39	0	0.95	**2.2**	*2.4*
107.1	0.8	29.46	1.88	**8.8**	**9.05**	**9.05**	0.45	0	12.46	0.96	**0.5**	*0.5*
107.11 (variant)	0.8	29.46	1.82	**8.8**	**8.80**	**8.99**	0.45	0.19	0	1	**1.0**	*1.0*
108.1	0.83	25.11	2.18	**9.01**	**9.26**	**9.26**	0.88	0	12.67	0.43	**0.5**	*1.1*
108.11 (variant)	0.83	25.11	1.92	**9.01**	**9.01**	**9.01**	0.88	0	0	0.87	**1.9**	*2.2*
108.2	1.03	23	2.84	**9.07**	**9.27**	**10.00**	1.28	0.73	10.14	0.004	**0.4**	*92.9**
108.21 (variant)	1.03	23	2.91	**9.08**	**9.38**	**10.11**	1.28	0.73	14.8	0.00002	**0.2**	*12355.6**
108.22 (variant)	1.03	23	2.32	**9.07**	**9.07**	**9.40**	1.28	0.33	0	1	**2.5**	*2.5*

*There are concerns on the reliability of the numerical model results for profiles 103.21, 108.2 and 108.21

Table 15 – Minimum wall height and position that satisfy the overtopping criteria (q≤1 l/s/m) and overtopping results .

Profile	H_{m0} [m]	$T_{m-1,0}$ [s]	A_c [m]	h_{toe} [m]	h_{wall} [m]	new R_c [m]	B [m]	Crest elevation at the seaward edge [mTAW]	Elevation of the wall toe [mTAW]	Crest elevation including wall and promenade [mTAW]	$q_{(Eq.12)}$ [l/s/m]
099	1.24	17.28	2.86	2.28	**0.70**	3.69	**6.70**	**9.81**	**9.94**	**10.64**	***0.99***
103.1	0.75	34.34	2.47	0.32	**0.00**	2.47	**0.00**	**9.73**	**9.73**	**9.73**	***0.09***
103.21 (variant)	0.7	33.01	2.38	0.35	**0.00**	2.38	**0.00**	**9.66**	**9.66**	**9.66**	***0.07***
104.1	0.89	26.13	2.11	0.72	**0.00**	2.26	**7.40**	**9.23**	**9.38**	**9.38**	***0.48***
104.11 (variant)	0.89	26.13	2.11	0.72	**0.05**	2.16	**0.00**	**9.23**	**9.23**	**9.28**	***0.93***
104.2	0.92	22.53	1.96	1.26	**0.58**	2.54	**0.00**	**9.07**	**9.07**	**9.65**	***0.99***
105.1	0.74	23.66	1.75	0.6	**0.00**	1.75	**0.00**	**8.84**	**8.84**	**8.84**	***0.80***
105.2	0.57	41.49	1.08	0.09	**0.16**	1.24	**0.00**	**8.43**	**8.43**	**8.59**	***0.98***
105.3	0.78	27.09	1.62	0.51	**0.15**	1.77	**0.00**	**8.7**	**8.70**	**8.85**	***1.00***
106.1	0.83	26.16	1.62	0.86	**0.05**	1.92	**12.5**	**8.47**	**8.72**	**8.77**	***0.86***
106.11 (variant)	0.83	26.16	1.37	0.86	**0.63**	2.00	**0.00**	**8.47**	**8.47**	**9.10**	***0.98***
107.1	0.8	29.46	1.88	0.45	**0.00**	2.13	**12.46**	**8.8**	**9.05**	**9.05**	***0.21***
107.11 (variant)	0.8	29.46	1.63	0.45	**0.19**	1.82	**0.00**	**8.8**	**8.80**	**8.99**	***0.98***
108.1	0.83	25.11	2.18	0.88	**0.00**	2.43	**12.67**	**9.01**	**9.26**	**9.26**	***0.22***
108.11 (variant)	0.83	25.11	1.92	0.88	**0.19**	2.11	**0.00**	**9.01**	**9.01**	**9.20**	***0.99***
108.2	1.03	23	2.11	1.28	**0.20**	2.51	**10.14**	**9.07**	**9.27**	**9.47**	***0.98***
108.21 (variant)	1.03	23	2.18	1.28	**0.00**	2.48	**14.8**	**9.08**	**9.38**	**9.38**	***0.92***
108.22 (variant)	1.03	23	1.99	1.28	**0.66**	2.65	**0.00**	**9.07**	**9.07**	**9.73**	***0.98***

5 Experimental campaign at Ghent University (1st stage)

Physical model tests have been carried out at Ghent University (UGent) to quantify the wave forces exerted on the storm return walls foreseen along the coastal area of Raversijde-Mariakerke-Wellington west. Based on the overtopping results described in Chapter 3, IMDC has defined the minimum layout of the each cross section to guarantee mean overtopping discharge lower than or equal to 1 l/s/m for a design storm condition, namely a storm with 1000-years return period.

In a first stage, only one cross section has been selected to perform physical model tests at UGent. In particular, profile 104.2 has been chosen as one of the most representative profiles along the studied coastal area. The layout of profile 104.2 has been modelled in the big wave flume (Grote Golfgoot) at Ugent. The main aim of the experimental campaign was to quantify the forces exerted by overtopping sea waves on the 38 cm storm wall located on top of the sea dike. SWASH model (from 2D to 1D) has been used to provide the target wave conditions at the toe of the dike. Thanks to the capabilities of the selected facilities, the mean overtopping discharge has been also measured and compared with the predictions based on the formula described in Chapter 3.

The model scale has been chosen to be 1:25, based on the flume dimensions and capabilities. The foreshore length has been one of the constrains that led to this choice. The model scale is finally the same as the tests previously conducted at Flanders Hydraulics Research (FHR). The model consists of a 1:2.7 dike with a 38 cm storm wall located at 10.2 m from the seaward edge of the dike (dimensions expressed in prototype scale). The toe of the dike is located at +6.4 mTAW (prototype scale).

Excluding calibration of the incident wave boundary conditions, 41 tests in total have been conducted, respectively:

- 27 tests for the 1000-years storm conditions as initially provided by SWASH 1D calculations (H_{m0}=0.94 m, $T_{m-1,0}$=24.50 s, SWL=7.22 mTAW), of which 20 tests have different phase seed (i.e. time series).
- 7+7 with higher water levels (higher water levels means higher water depth at the toe, less breaking, shorter wave periods but higher wave height and bigger wave overtopping and wave forces).

5.1 Flume setup and instrumentation

The big wave flume at UGent has been divided into 2 partitions close to the toe of the dike. It was intended to use one partition, 0.30 m wide, to measure incident wave boundary conditions (IWBC) at the location of the dike toe and the other partition, 0.70 m wide, to measure wave overtopping and forces at the same time. In a preliminary phase, no dike was present in both partition and the IWBC have been measured in both. The results showed a quite perfect agreement between the two measurements, so we can say that the division of the flume does not create model effects that might bias the results. A sketch of the flume setup is depicted in Figure 40. A 1:10 transitional slope was needed between the horizontal bottom and the 1:50 foreshore slope, due to the limitation in length of the flume.

Two acoustic wave gauges and nine resistive wave gauges have been used for the measurement of the water surface elevation: the former ones are implemented for the active wave absorption system of which the flume is equipped; the latter ones are used to measure the transformation of wave height an period along the flume. Two of the resistive wave gauges are located at the dike toe to measure the IWBC. For wave forces, two load cells have been installed on the storm wall. Each load cell measured the loads exerted on a 10 cm wide plate (Figure 41,a). The mean wave overtopping discharge has been measured on 20 cm width: the overtopping volumes have been collected by means of a chute located just after the storm walls and weighted (Figure 41,b). The final cumulated mass has been divided by the duration of each test and the chute width (20 cm) to obtain the mean overtopping discharge (expressed in $m^3/s/m$ or l/s/m).

Figure 40 – Flume setup

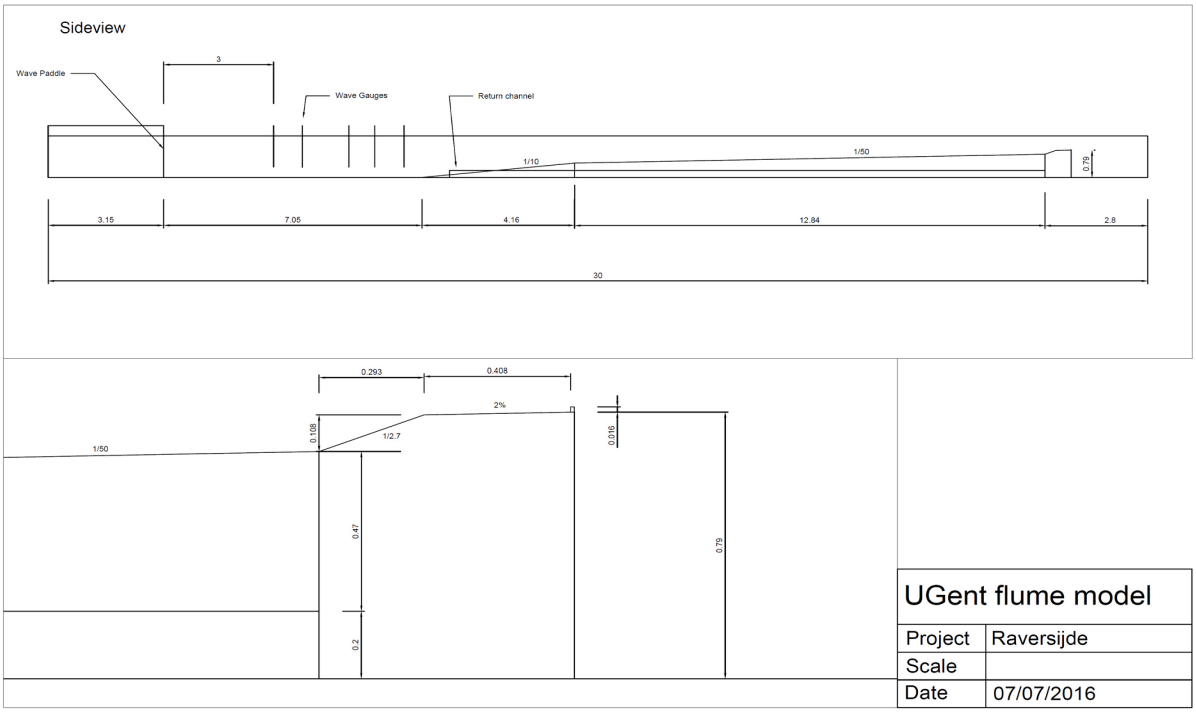
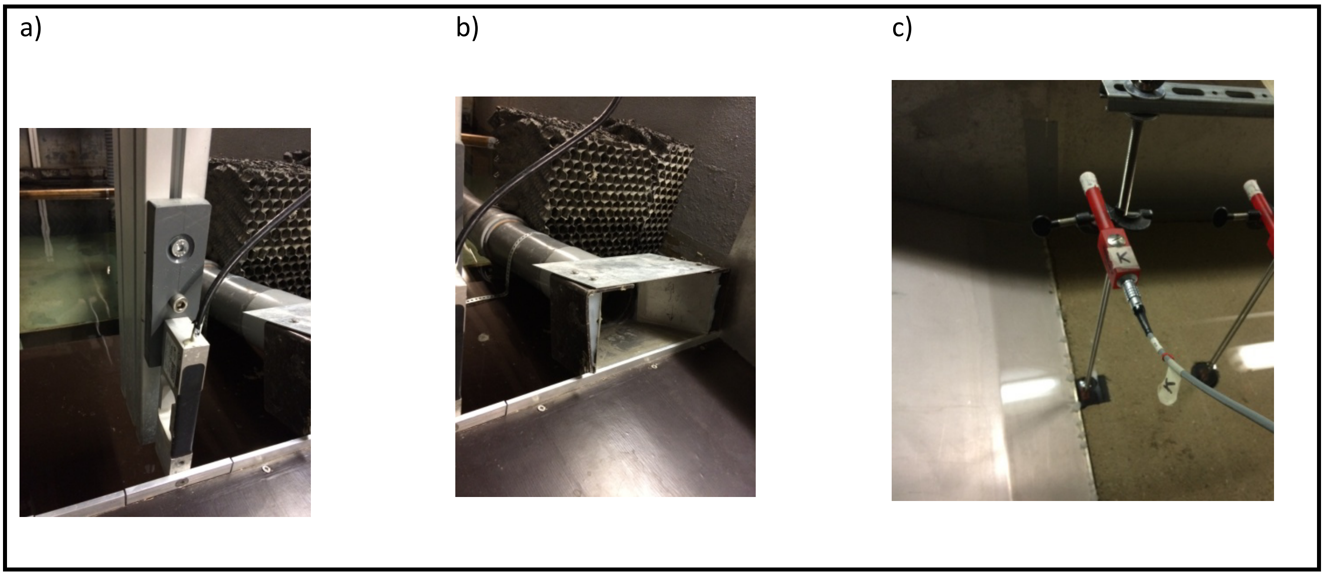


Figure 41 – Instrumentation setup: a) load cell, b) overtopping chute; c) resistive wave gauge at the dike toe



5.2 Incident WAVE boundary conditions (IWBC)

A preliminary calibration phase has been carried out to achieve the target wave height and period at the toe of the structure (H_{m0}=0.94 m, $T_{m-1,0}$=24.50 s). If we express the target waves in model scale, they result equal to:

- H_{m0}=0.94/25= 0.0376 m;
- $T_{m-1,0}$=24.50/sqrt(25)=4.90 s;

The results of the calibration phase are summarized in Table 13, where $H_{m0,WP}$, $T_{p,WP}$ and d_{WP} are respectively the input significant wave height and period and the water depth at the wave paddle, d_{toe} is the toe depth, H_{m0} and $T_{m-1,0}$ are the significant wave height and period measured at the toe of the structure, and $\varepsilon(H_{m0})$ and $\varepsilon(T_{m-1,0})$ are the deviations from the target wave conditions, expressed in percentage. Each test is characterized by a code, such as 010A_007A: in this case 010 indicated the test number 10, A means the layout (in this case with no dike, in case of the presence of the dike it will be B), 007 refers to the target wave conditions and the letter after that identifies a specific time series. That means that in Table 13 the time series has been changed 6 times, keeping the same spectra characteristics and the same configuration. It is worthy to remark as the difference between the measured and the target wave characteristics varies with the time series, in some case this difference is, in absolute value, bigger than 3% for the wave height and 5% for the wave period. Overall the target wave conditions have been achieved.

Table 16 – Calibration phase

Test case	$H_{m0,WP}$ [m]	$T_{p,WP}$ [s]	d_{WP} [m]	d_{toe} [m]	H_{m0} [m]	$\varepsilon(H_{m0})$ [%]	$T_{m-1,0}$ [s]	$\varepsilon(T_{m-1,0})$ [%]
010A_007A	0.09	2.26	0.709	0.036	0.03691	-1.84	4.58	-6.53
011A_007B	0.09	2.26	0.709	0.036	0.03683	-2.05	4.662	-4.86
012A_007C	0.09	2.26	0.709	0.036	0.03777	0.45	4.797	-2.10
013A_007D	0.09	2.26	0.709	0.036	0.03664	-2.55	4.697	-4.14
014A_007E	0.09	2.26	0.709	0.036	0.03567	-5.13	4.733	-3.41
015A_007F	0.09	2.26	0.709	0.036	0.03617	-3.80	4.781	-2.43

5.3 Test matrix

In total 41 tests have been conducted to measure wave forces and wave overtopping. 27 tests correspond to the target wave conditions (1000-years storm), of which 20 have different time series (from A to T). The measured wave characteristics at the toe and the deviation from the target conditions are reported in Table 14.

Table 15 summarizes the measured wave conditions for two different initial water levels, respectively 1 cm and 2 mc higher than the previous one. That means to consider a 0.25 m and 0.50 m increase of the water level, in prototype. The mean wave overtopping discharge and the maximum forces for the two load cells are also reported in Tables 14 and 15 (these quantities are expressed in prototype scale).

For each water level, the mean value, μ, and the relative error, equal to the ratio between the standard deviation and the mean value, μ/σ, have been reported. It can be seen as the target conditions are achieved (Table 14) but still the time series plays a role (see for example the differences between tests

024B_007A and 043B_007R). The tests 030B and 041B are marked in red in Table 14, since the wave force results measured by one of the two load cells have been considered unreliable. Further details in §5.5.1 and 5.5.3.

Table 17 – Test matrix for target wave conditions.

Test case	H_{m0} [m]	$\varepsilon(H_{m0})$ [%]	$T_{m-1,0}$ [s]	$\varepsilon(T_{m-1,0})$ [%]	q [l/s/m] (prototype)	F_{max-A} [kN/m] (prototype)	F_{max-B} [kN/m] (prototype)
024B_007A	0.03768	0.21	4.764	-2.78	0.3775	2.79	2.77
025B_007A	0.03762	0.05	4.767	-2.71	0.3830	2.79	2.59
026B_007A	0.03749	-0.29	4.785	-2.35	0.3485	3.07	3.53
027B_007B	0.03736	-0.64	4.856	-0.90	0.2116	2.30	3.49
028B_007C	0.03867	2.85	4.996	1.96	0.3858	3.28	3.16
029B_007D	0.03690	-1.86	4.7	-4.08	0.0761	1.69	1.55
030B_007E	0.03660	-2.66	4.767	-2.71	0.1922	6.21	2.15
031B_007F	0.03778	0.48	4.862	-0.78	0.2254	1.77	2.35
032B_007G	0.03757	-0.08	5.033	2.71	0.3692	2.77	3.10
033B_007H	0.03816	1.49	4.928	0.57	0.2904	2.40	2.71
034B_007I	0.03751	-0.24	4.975	1.53	0.2696	3.23	3.54
035B_007J	0.03633	-3.38	4.743	-3.20	0.1742	2.28	2.45
036B_007K	0.03740	-0.53	4.94	0.82	0.1341	1.47	1.74
037B_007L	0.03713	-1.25	4.76	-2.86	0.2033	2.75	3.39
038B_007M	0.03967	5.51	5.131	4.71	0.2268	2.37	2.21
039B_007N	0.03992	6.17	5.125	4.59	0.2240	2.23	2.45
040B_007O	0.03940	4.79	4.875	-0.51	0.2489	2.46	2.40
041B_007P	0.03861	2.69	4.811	-1.82	0.2461	6.77	4.64
042B_007Q	0.04036	7.34	5.141	4.92	0.4895	2.80	3.43
043B_007R	0.03992	6.17	5.063	3.33	0.1659	2.80	3.19
044B_007S	0.03895	3.59	4.997	1.98	0.2406	3.17	1.81
045B_007T	0.03784	0.64	4.934	0.69	0.1327	1.57	2.29
061B_007D	0.03924	4.36	4.769	-2.67	0.1590	1.76	1.81
062B_007K	0.03991	6.14	4.924	0.49	0.2835	2.70	1.95
063B_007T	0.03920	4.26	4.806	-1.92	0.1742	2.20	2.54
064B_007R	0.04035	7.31	5.007	2.18	0.2212	4.04	4.05
065B_007R	0.04029	7.15	5.011	2.27	0.2696	3.57	3.30
μ	**0.0384**	**2.23%**	**4.906**	**0.13%**	**0.249**	**2.57***	**2.71***
σ/ μ	**3.2%**	**-**	**2.7%**	**-**	**37.9%**	**24.8%***	**25.0%***

*cases 030B and 041B are not considered.

Table 18 – Test matrix for higher water levels.

Test case	d_{WP} [m]	H_{m0} [m]	$T_{m-1,0}$ [s]	q [l/s/m] (prototype)	F_{max-A} [kN/m] (prototype)	F_{max-B} [kN/m] (prototype)
054B_012A	0.719	0.04285	4.097	2.1004	4.36	5.15
055B_012A	0.719	0.04283	4.100	2.0561	4.43	4.93
056B_012A	0.719	0.04279	4.094	2.2815	3.97	4.26
057B_012B	0.719	0.04351	4.201	2.6272	5.11	4.36
058B_012C	0.719	0.04335	4.141	2.2718	4.11	5.25
059B_012D	0.719	0.04328	4.162	1.7575	4.01	4.53
060B_012E	0.719	0.04344	4.137	2.3465	6.35	6.30
μ		**0.0432**	**4.133**	**2.206**	**4.62**	**4.97**
σ/ μ		**0.7%**	**1.0%**	**12.3%**	**18.5%**	**14.1%**
Test case	**d_{WP} [m]**	**H_{m0} [m]**	**$T_{m-1,0}$ [s]**	**q [l/s/m] (prototype)**	**F_{max-A} [kN/m] (prototype)**	**F_{max-B} [kN/m] (prototype)**
047B_011A	0.729	0.04745	3.534	8.1554	11.07	9.48
048B_011A	0.729	0.04741	3.540	7.7973	9.56	8.45
049B_011A	0.729	0.04740	3.543	7.9770	10.88	10.38
050B_011B	0.729	0.04642	3.487	7.3921	8.91	8.37
051B_011C	0.729	0.04685	3.652	8.6283	7.96	6.89
052B_011D	0.729	0.04700	3.575	7.5650	7.33	6.11
053B_011E	0.729	0.04626	3.529	7.5055	10.77	11.18
μ		**0.0470**	**3.551**	**7.860**	**9.50**	**8.69**
σ/ μ		**1.0%**	**1.4%**	**5.5%**	**15.7%**	**20.0%**

5.4 Wave overtopping results

Even though the overtopping measurement was not foreseen at the beginning as one of the measurements to carry out at UGent, it was possible to optimize the model setup in order to measure mean wave overtopping discharge and wave forces at the same time. The scope of the overtopping measurements was twofold:

1. To confirm the wave overtopping prediction based on Equation (12).
2. To evaluate the scatter in overtopping due to the different time series.

The results are depicted in Figure 42 and Figure 43. The following conclusions can be drawn:

1. There is a big scatter in the results from water level 0.709 m (design conditions, Table 14): the average overtopping discharge is equal to 0.249 l/s/m (in prototype) with a variation of 37.9% around the mean value.
2. The scatter reduces for higher water levels: for 0.719 m, the average overtopping is 2.206 l/s/m with a variation of 12.3%; for 0.729 m, the average overtopping is 7.860 l/s/m with a variation of 5.5%.
3. Less overtopping discharge (around the expected 1 l/s/m) leads to larger scatter.

4. There is a substantial agreement between formula prediction and measurements, however the formula slightly overestimates the smaller discharges (this is due to the calibration of the reductions coefficients for the storm wall and promenade that are based on the specific campaign conducted at FHR and do not consider the influence of the IWBC).

Overall the results in terms of overtopping can be considered acceptable and in agreement with the predictions. A predicted overtopping discharge twice larger than the measured one is considered within the overtopping uncertainties (see, for example, scatters in EurOtop, 2007 or Van Gent, 1999). The measured mean overtopping discharge for each test case is also reported in Tables 14 and Table 15.

Figure 42 – Wave overtopping results and formula prediction

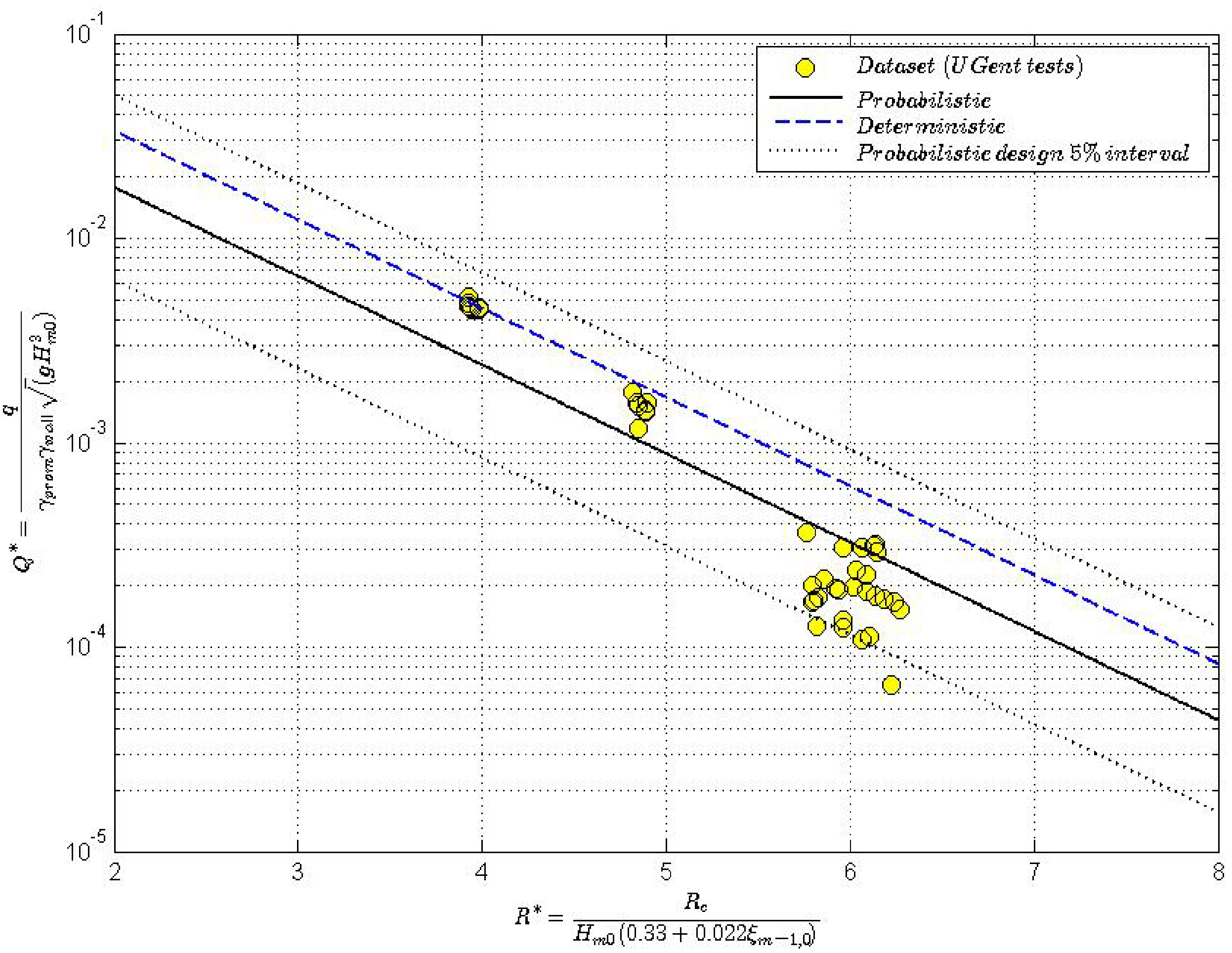
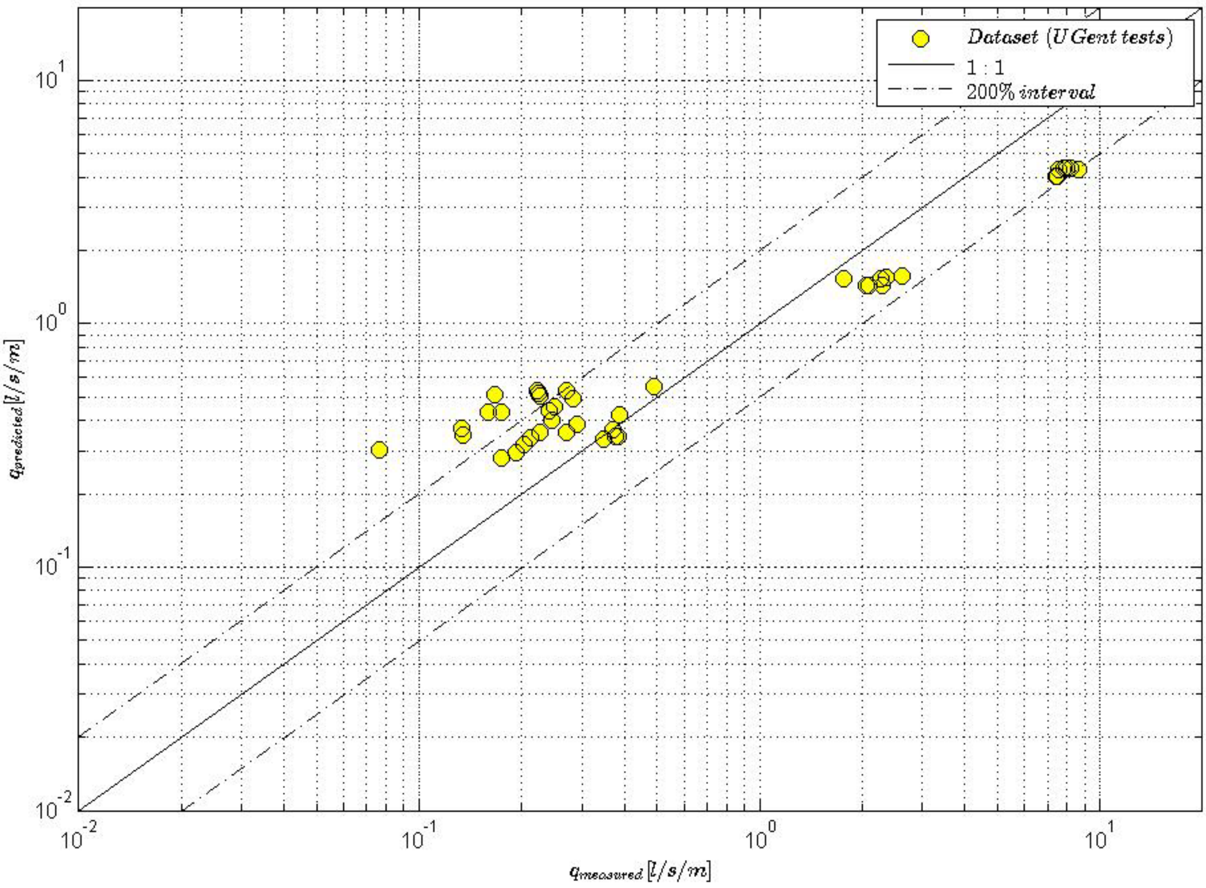


Figure 43 – Predicted versus measured overtopping discharge

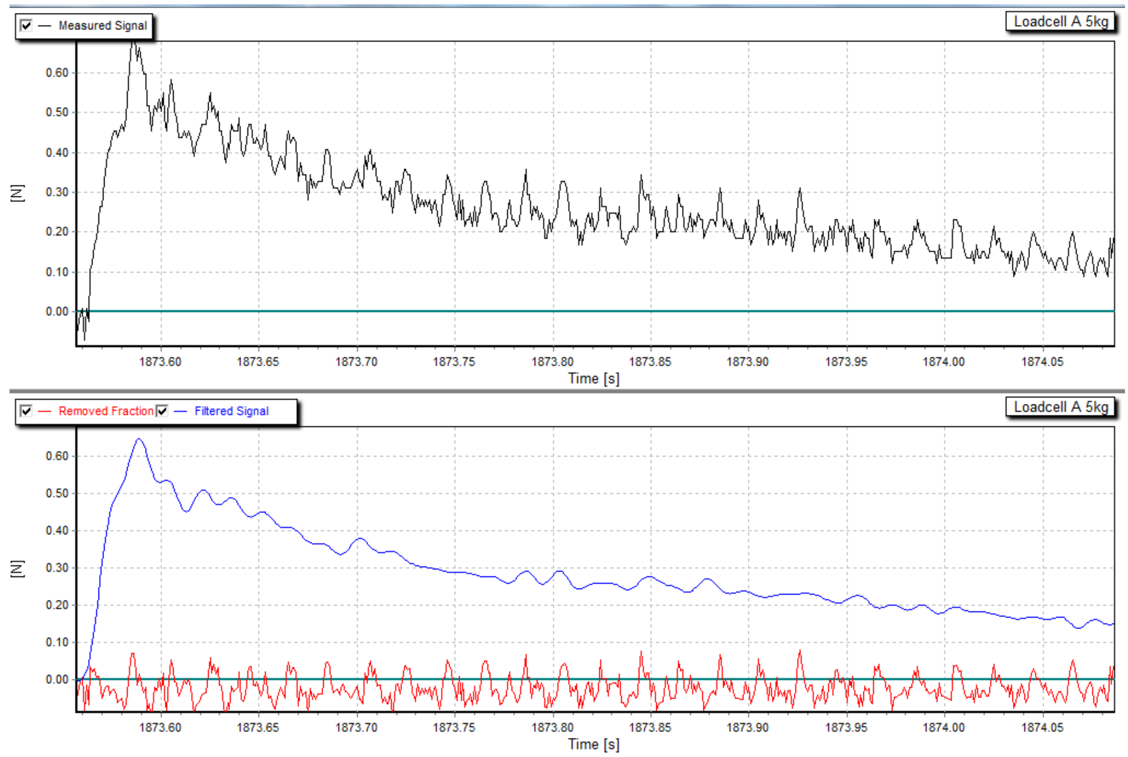


5.5 Wave force results

Main scope of the experimental campaign was to assess the force exerted by sea waves on the storm return wall located on top of the 104.2 sea dike. The forces have been analysed by means of two load cells (load cell A and load cell B) and the results have been compared showing some difference perhaps due to the effect of the flume side wall. A threshold has been set equal to the hydrostatic force in case of still water level covering the storm wall for its entire height. All the events greater than this threshold have been considered as impact events.

The raw signal from the load cells has been filtered. A band-pass filter (49-50 Hz) has been used to remove the electrical noise. A low-pass filter of 70 Hz has been applied to remove possible spikes mainly due to the resonance frequency of the load cells (checked by hammer-tests). The results of the filtering have shown that the filtering does not cut-off the peaks of the impacts but help only to filter out noise and spurious oscillations from the signal (see Figure 44).

Figure 44 – Example of raw and filtered signal for load cell A (model scale)



5.5.1 Design storm conditions

The 27 tests carried out for the design wave conditions (1000-years storm) have been analysed and all the impact events have been gathered to extrapolate from them the exceedance probability distribution of the wave forces. In average only 19-20 impact events per test have been measured. The maximum force for each test has been extracted. The 54 values (27 per load cell) have been gathered and they are plotted in Figure 45 in terms of their exceedance probability.

The results from the two load cells are substantially the same, except for the two highest events (load cell A, from cases 030B and 041B), that, moreover, do not seem to follow the same trend as all the other impacts. From Figure 45 it can be seen that a force of 4.5 kN/m corresponds to a 5% exceedance probability. The maximum force for each load cell is also reported in Table 14. It is worthy to see that a difference exists between the two load cells: the maximum value is really stochastic and some transversal effect can play a big role so that the maximum force measured on load cell A is sometimes very different from load cell B.

5.5.2 Higher water levels

Seven tests per each water level, higher than the one used for the 1000-years storm, have been carried out. Among each group, 1 test has been repeated 3 times. Therefore 5 different time series have been carried out for each water level. The results in terms of exceedance probability are reported in Figure 46 and Figure 47.

Higher water levels led to more impact events, respectively 100 events for each test with d_{WP}=0.719 m and approximately 230 events for each test with d_{WP}=0.729 m. These values correspond to increase of the SWL in prototype with 25 cm and 50 cm respectively. The forces, as expected, increase as the water level increase. The maximum force for each load cell is also reported in Table 15.

Figure 45 – Exceedance probability of maximum wave forces on storm wall for load cell A and load cell B (1000-year storm)

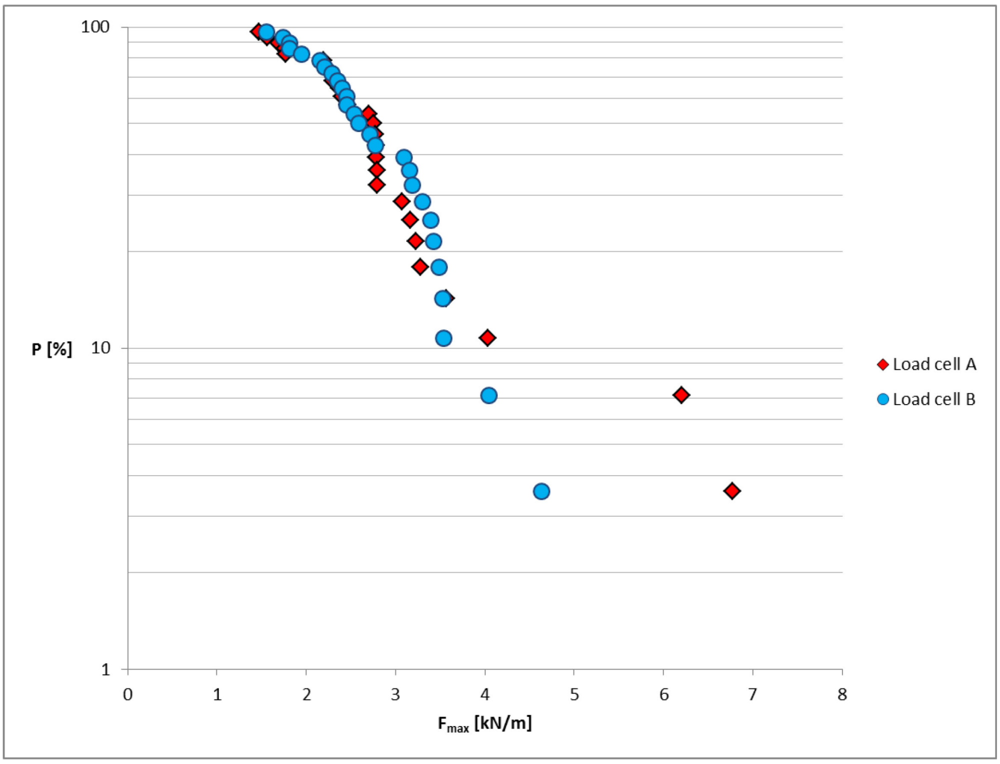


Figure 46 – Exceedance probability of maximum wave forces on storm wall for load cell A and load cell B (SWL increased 25 cm)

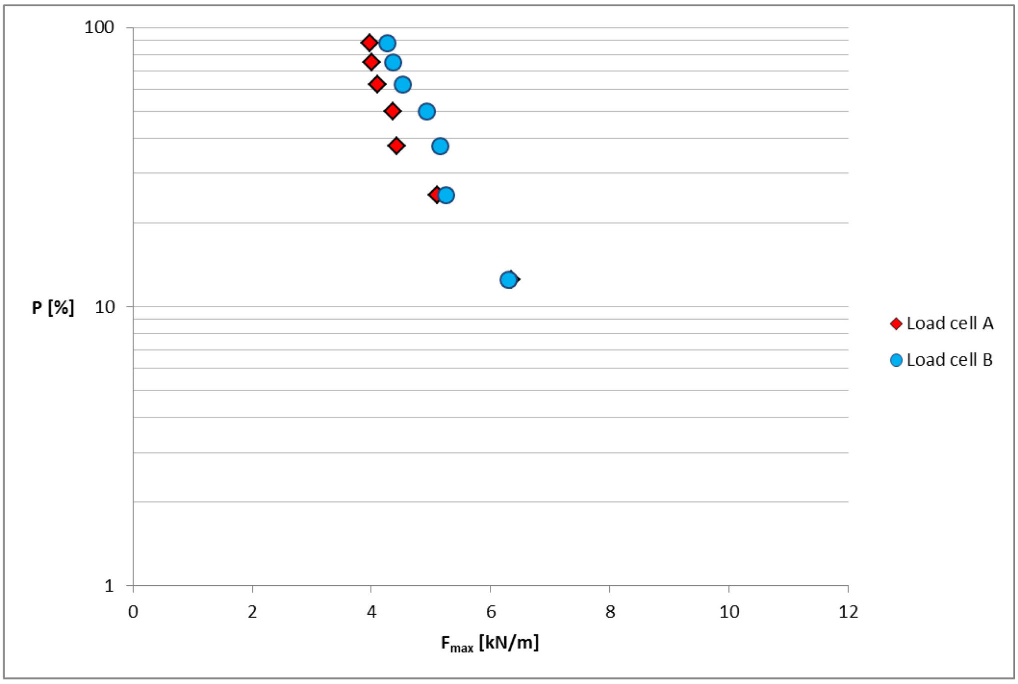
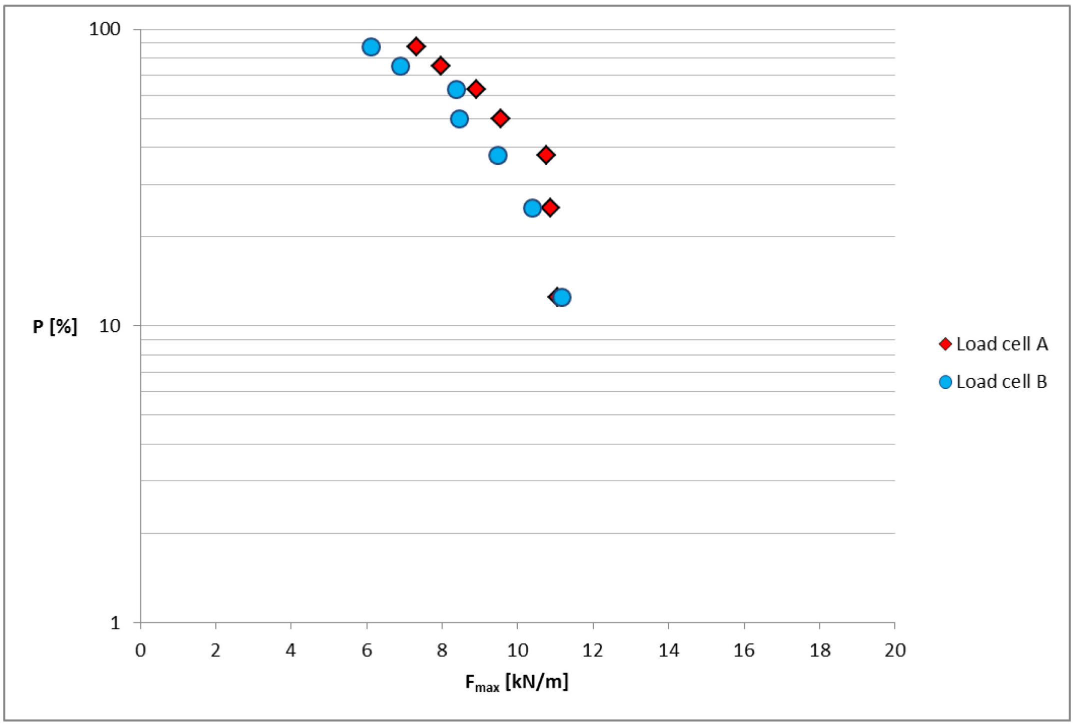


Figure 47 – Exceedance probability of maximum wave forces on storm wall for load cell A and load cell B (SWL increased 50 cm)



5.5.3 Relationship between forces and wave height for cross section 104.2

Van Doorslaer et al. (2015) analysed the wave impacts on storm walls located on sea dikes or quay walls in case of breaking and non-breaking waves, based on physical model experiments carried out in the big flume at Technical University of Catalonia in Barcelona. The authors show as the maximum or design forces are proportional to the non-dimensional freeboard, R_c/H_{m0}. They demonstrate as a good regression can be achieved if the non-dimensional quantity $F/\rho g R_c^2$ is considered. In the present case, we consider the maximum forces only, therefore $F_{max}/\rho g R_c^2$ is plotted in Figure 48 and Figure 49 as function of H_{m0} and R_c/H_{m0} respectively.

The two figures can offer an easy way to calculate, for cross section 104.2 or similar ones, the expected wave forces in case of increasing wave height or increasing water level (decreasing freeboard). This results do correspond to a storm wall of 38 cm in prototype. For different wall heights, they must be used carefully.

All the data are plotted together except those corresponding to tests 030B and 041B. Figure 48 and Figure 49 show in fact that those results are outliers with respect to the rest of the cases.

Figure 48 – Relationship between the non-dimensional wave force and the wave height at the toe of the dike.

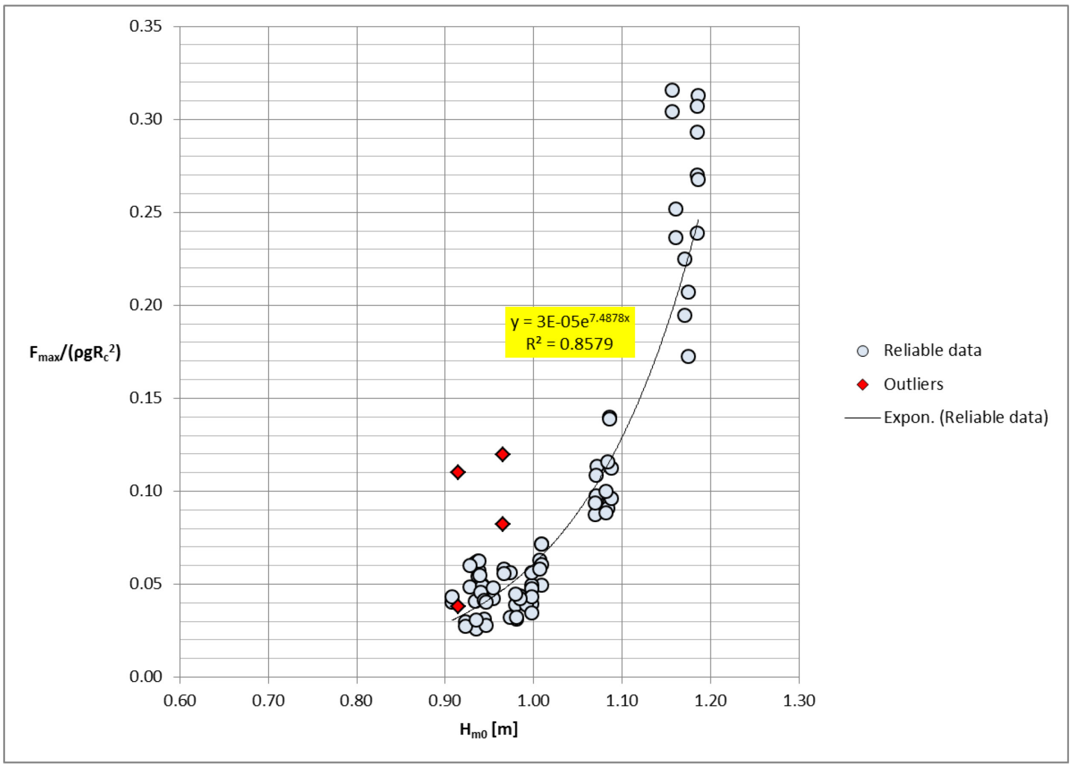
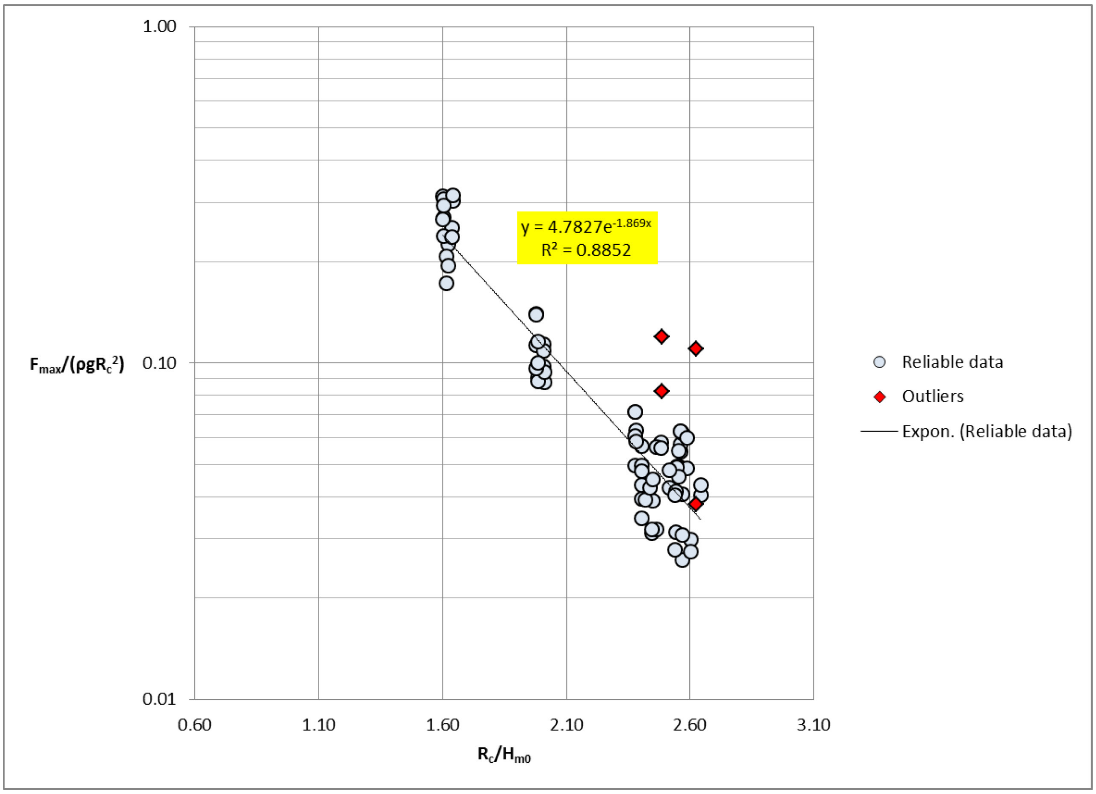


Figure 49 – Relationship between the non-dimensional wave force and the relative freeboard.



5.5.4 Characteristic values for design

For design, a characteristic value of the maximum wave force must be calculated as $F_k=\mu(F_{max})+1.64*\sigma(F_{max})$. The values for μ and σ/μ are reported in Table 14 and Table 15. The resulting characteristic values for the maximum wave forces are summarized in Table 16 for both load cells.

Table 19 – Characteristic values for maximum wave forces

Water level	**F_{k-A} [kN/m] (prototype)**	**F_{k-B} [kN/m] (prototype)**
Design SWL	3.6	3.8
SWL + 0.25m	6.0	6.1
SWL + 0.50m	12.0	11.7

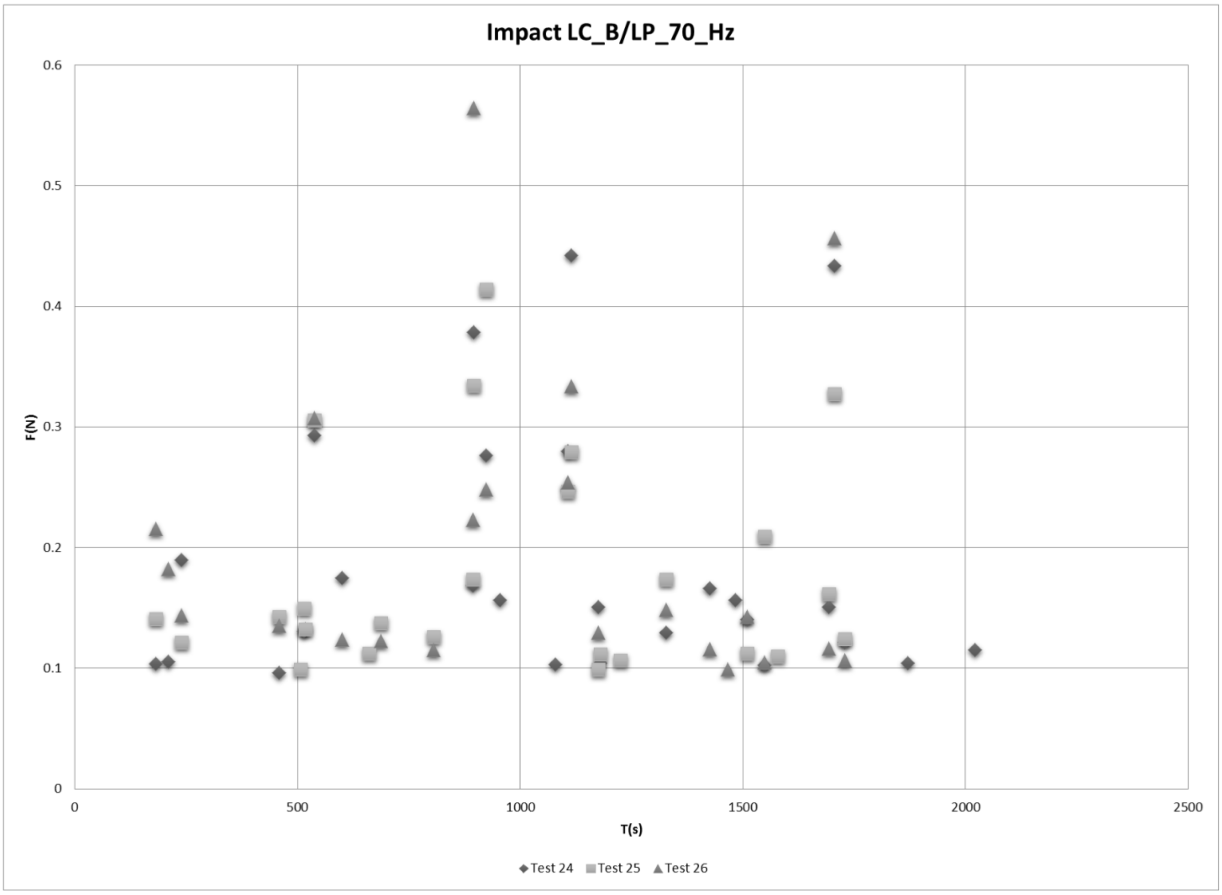
5.5.5 Repeatability

As for wave boundary conditions and wave overtopping, the repeatability of each test has been verified also in terms of wave forces. For that, three tests have been used. The results are summarized in Table 15. The maximum forces for load cell A (LCA) and load cell B (LCB) present some difference, meanwhile the IWBC and the mean overtopping discharge are almost the same. More clear view of the NO-repeatability of the test in terms of wave forces is given by Figure 50, where with different colours are represented the impacts from different test cases. The results are plotted in model scale. It can be noticed that the time at which each event occurs is pretty similar among the three tests. However, the magnitude of the impact changes and this change is completely random.

Table 20 – Tests for repeatability analysis

Test case	**H_{m0} [m]**	**$T_{m-1,0}$ [s]**	**Q [l/s/m] (prototype)**	**F_{max-A} [kN/m] (prototype)**	**F_{max-B} [kN/m] (prototype)**
024B_007A	0.03768	4.764	0.3775	2.79	2.76
025B_007A	0.03762	4.767	0.3830	2.79	2.59
026B_007A	0.03749	4.785	0.3485	3.07	3.53

Figure 50 – Impact event (in model scale) for repeatability analysis



6 Experimental campaign at Ghent University (2nd stage)

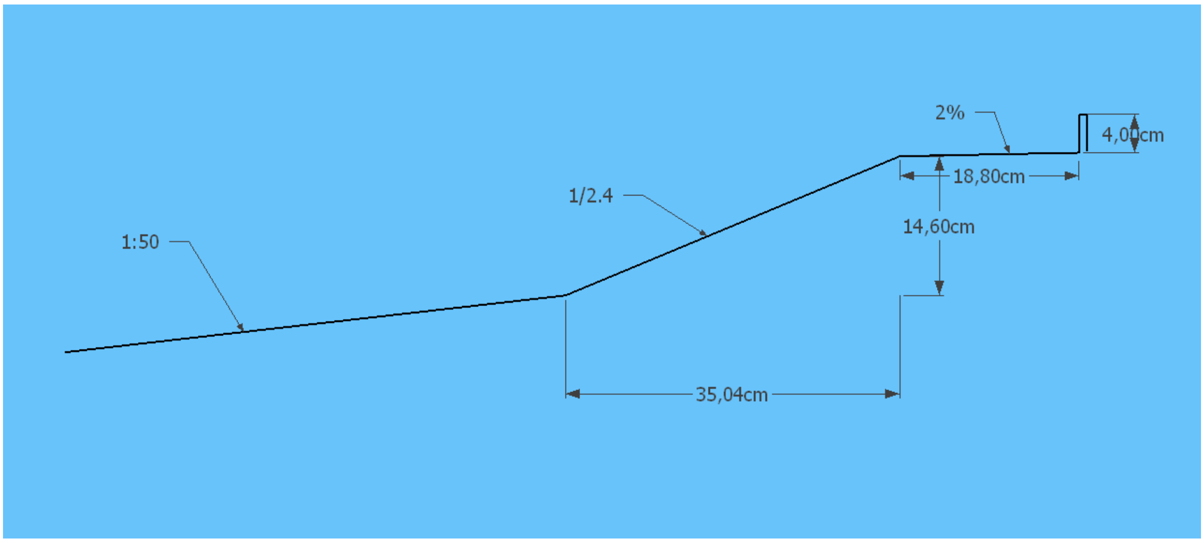
It was unclear if the results obtained for section 104.2 could be extended to another cross section, namely cross section 99. At that location, a wall of 1 m height and located at 4.7 m from the seaward edge of the dike, is employed as sand screen. The expected overtopping discharge over this wall is negligible, however the magnitude of the forces exerted by the overtopping waves is of difficult judgment. Chen et al. (2016) formula has been applied for a preliminary force estimation, however big uncertainties exist on the application of the formula to this specific case. Chen (personal communication) warned about the use of the formula in case of section 99, due to the fact that it is applied out of range.

Therefore, it has been decided to carry out extra tests at Ghent University to calculate the wave forces on such wall for cross section 99. The big wave flume (Grote Golfgoot) at Ghent University (UGent) has been used for the scope. The target wave conditions have been provided by IMDC as results of SWASH 2D and SWASH 1D.

6.1 Experimental campaign

The same model scale (1:25), foreshore layout and measurement setup of previous tests for cross section 104.2 have been used. Only the dike and the storm wall have been modified to be adjusted to cross section 99 (Profile A). The model finally consisted of a 1:2.4 dike with a 1 m storm wall located at 4.7 m from the seaward edge of the dike (dimensions expressed in prototype scale). The toe of the dike is located at +6.22 mTAW. The crest of the dike (excluding the wall) is located at +9.88 mTAW. A sketch of the sea dike is depicted in Figure 51.

Figure 51 – Flume setup: detail of sea dike (cross section 99)



6.1.1 Test matrix

In total 26 tests (excluding calibration of the incident wave boundary conditions and repeatability) have been conducted for the 1000-years storm conditions as initially provided by SWASH 1D calculations (H_{m0}=1.01 m, $T_{m-1,0}$=24.35 s, SWL=7.15 mTAW). Each test is characterized by a different time series. The difference between the measured and the target wave characteristics varies with the time series. In 17 cases the difference is bigger than 3% for the wave height and/or 5% for the wave period (in absolute value). Therefore, the wave height and period match with the target wave conditions within the accepted tolerance only for 9 tests. The results in terms of wave height, period and wave forces are reported in Table 18. The 9 tests with matching wave conditions are marked in red colour. The mean wave overtopping discharge for all cases was zero or negligible (less than 0.02 l/s/m), as expected.

Table 21 – Test matrix with measured wave boundary conditions and wave forces.

Test case	**H_{m0} [m]**	**$\varepsilon(H_{m0})$ [%]**	**$T_{m-1,0}$ [s]**	**$\varepsilon(T_{m-1,0})$ [%]**	**F_{max-A} [kN/m]**	**F_{max-B} [kN/m]**
079B_017A	1.03	2.22	24.07	-1.16	5.41	5.69
080B_017B	1.03	2.12	23.56	-3.24	5.71	4.27
081B_017C	1.04	3.66	24.35	0.01	6.29	5.18
082B_017D	1.04	3.34	23.92	-1.76	5.74	4.25
083B_017E	1.03	2.87	23.61	-3.05	4.01	5.39
084B_017F	1.02	1.87	23.08	-5.21	5.04	4.17
085B_017G	1.05	4.46	24.80	1.83	7.69	6.19
086B_017H	1.02	1.70	24.05	-1.23	5.27	2.82
087B_017I	1.06	5.33	24.18	-0.69	6.05	5.53
088B_017J	1.05	4.24	24.92	2.35	4.28	3.26
089B_017K	1.04	3.69	22.83	-6.26	6.11	5.51
090B_017L	1.05	4.26	23.79	-2.29	6.64	6.68
091B_017M	1.05	3.96	24.13	-0.90	6.91	4.87
092B_017N	1.06	5.85	24.48	0.52	9.29	3.98
093B_017O	1.05	4.38	24.14	-0.88	3.98	4.12
095B_017P	1.04	2.89	24.23	-0.51	4.42	3.38
096B_017Q	1.04	3.86	24.05	-1.23	6.43	5.33
097B_017R	1.05	4.48	24.02	-1.35	4.77	1.84
098B_018A	1.06	5.75	25.46	4.57	10.25	8.70

Test case	**H_{m0} [m]**	**$\varepsilon(H_{m0})$ [%]**	**$T_{m-1,0}$ [s]**	**$\varepsilon(T_{m-1,0})$ [%]**	**F_{max-A} [kN/m]**	**F_{max-B} [kN/m]**
099B_018B	1.04	3.84	24.18	-0.71	4.92	3.73
100B_018C	1.05	4.24	24.29	-0.24	3.23	3.33
102B_019B	1.03	2.67	24.66	1.26	3.71	3.94
103B_019C	1.01	0.48	24.48	0.52	4.46	3.51
104B_020A	1.04	3.34	24.80	1.83	3.71	2.38
105B_020B	1.02	1.75	25.52	4.81	5.26	3.75
106B_020C	1.02	1.10	25.21	3.54	1.68	3.06
μ					**5.43 (4.44)**	**4.42 (3.98)**
σ/ μ					**33.5% (27.95%)**	**33.4% (24.9%)**

6.2 Wave force results

A threshold value for forces equal to 0.13 N (in model scale) has been selected. All the events greater than this threshold have been considered as impact events. The raw signal from the load cells has been filtered. A band-pass filter (49-50 Hz) has been used to remove the electrical noise. A low-pass filter of 70 Hz has been applied to remove possible spikes mainly due to the resonance frequency of the load cells (checked by hammer-tests). The results of the filtering have shown that the filtering does not cut-off the peaks of the impacts but help only to filter out noise and spurious oscillations from the signal.

The 26 tests have been analysed and all the impact events have been gathered to extrapolate from them the exceedance probability distribution of the wave forces. In average only 15-20 impact events per test have been measured. The maximum force for each test has been extracted. The 52 values (26 per load cell) have been gathered and they are plotted in Figure 52 in terms of their exceedance probability. From Figure 52 it can be seen that a force of about 8-9 kN/m corresponds to a 5% exceedance probability.

Due to the fact that in only 9 cases the difference between measured and target wave boundary conditions lies within the acceptable limits (+-3% for H_{m0} and +-5% fot $T_{m-1,0}$), and that extreme wave forces measured on the storm wall might depends on higher wave height or longer wave period than the target ones, it has been decided to analyse the 9 tests separately. The results are plotted in Figure 53 in terms of exceedance probability. From Figure 3 it can be seen that a force of about 6-7 kN/m corresponds to a 5% exceedance probability, lower than what calculated for all 26 tests.

Figure 52 – Exceedance probability of maximum wave forces on storm wall for load cell A and load cell B (1000-year storm) – all tests included

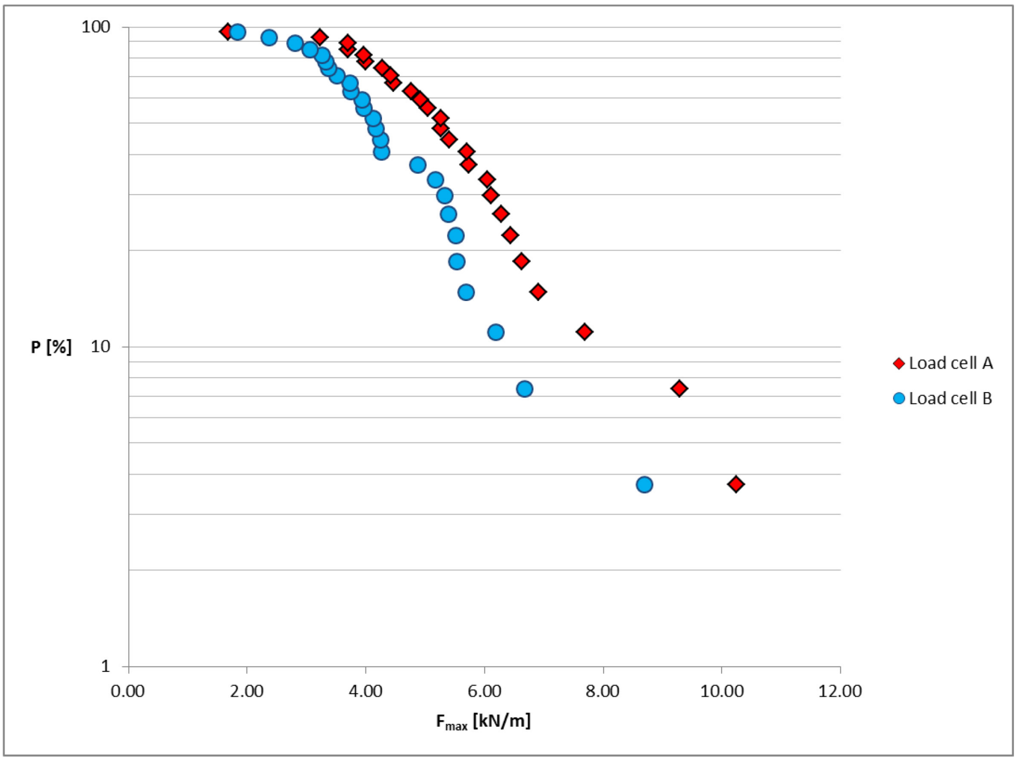
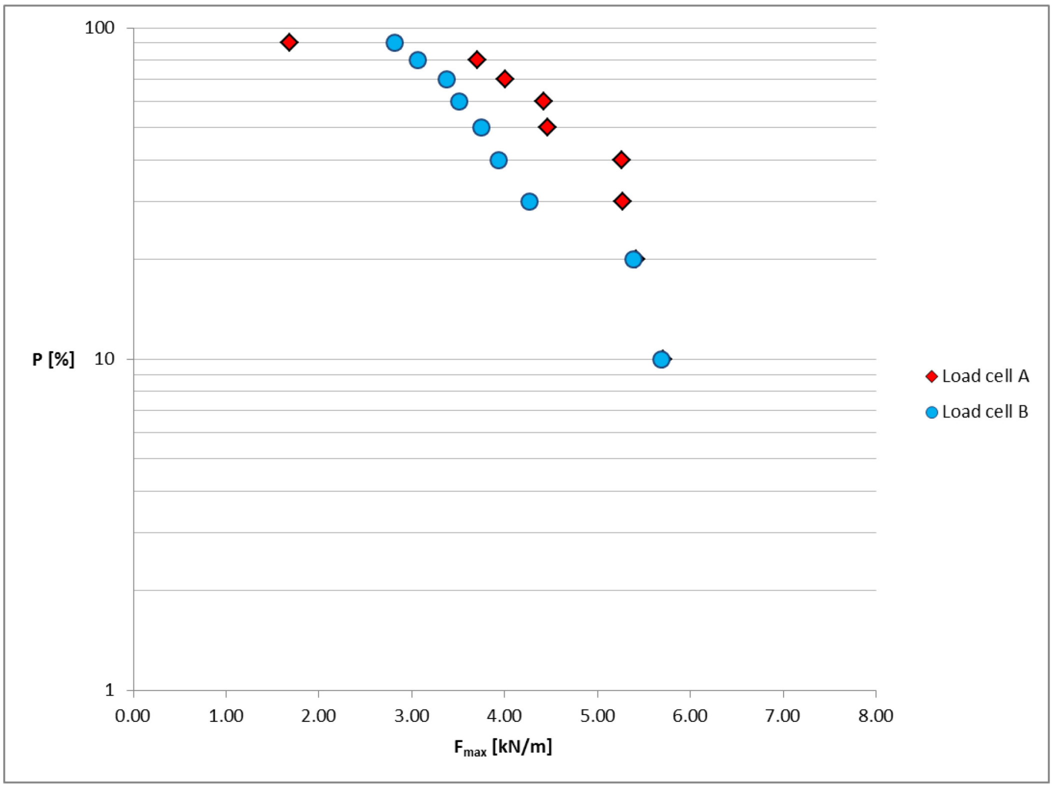


Figure 53 – Exceedance probability of maximum wave forces on storm wall for load cell A and load cell B (1000-year storm) – only tests with wave conditions matching with the target ones are included



7 Conclusions

The present report describes the main results of the physical model tests carried out at Flanders Hydraulics Research (FHR) and Ghent University (UGent) for the analysis of wave overtopping and wave forces on the storm return walls designed as upgrading of sea dikes located in the Raversijde-Mariakerke-Wellington West zone. These sea dikes are characterised by a shallow or very shallow foreshores reaching the toe of the dike with an average foreshore slope equal to 1:50. The dike slope varies between 1:1.7 and 1:2.6.

The purpose of the physical model tests was twofold: on one hand, to validate previous overtopping results obtained by numerical modelling (namely SWASH) and to provide information of new storm wall height and position which meet the wave overtopping criteria defined in the coastal safety master plan; on the other hand, to quantify the wave loading exerted on storm walls from few selected cases.

A new formula (Eq. (12)) is proposed for wave overtopping assessment, including the effect of wall height and position (= promenade width). Eq. 12 is here restated:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^{-0.883}\gamma_{wall}\gamma_{prom}exp\left(-\frac{R_c}{H_{m0}\left(0.33+0.022\xi_{m-1,0}\right)}\right)$$

This new formula must only be applied to cases from the Raversijde-Mariakerke-Wellington West zone, within the range of the wave height, wave period, freeboard, wall height, promenade width and toe depth as described in this report (see Table 8). The aforementioned ranges are summarized as follows:

- Significant wave height at the toe between 0.30 m and 1.81 m.
- Spectral wave period at the toe between 11.38 s and 61.50 s.
- Freeboard (including the wall height) between 0.38 m and 6.00 m.
- Water depth at the toe between -1.00 m and 2.20 m.
- Promenade width equal to 0.00 m, 6.75 m and 12.50 m.

Once the experimental campaign at FHR has been concluded, the main results have been used by IMDC to define the final layout for each profile in the Raversijde-Mariakerke-Wellington West zone. Later on, the profile 104.2 has been identified as representative profile of most of the cross sections in that area and, therefore, physical model tests have been carried out to measure wave forces exerted on the storm wall, necessary for a proper design of the same wall and its foundations. following a temporarily unavailability of the flume at FHR, a first phase of experimental force tests has been conducted in the big wave flume at UGent to measure wave forces on storm wall located on top of a 1:27 dike slope. A 1:50 foreshore slope is present before the sea dike. For 1000-years storm, a characteristic value for the maximum wave force of 3.8 kN/m has been assessed, corresponding to 7% exceedance probability as shown. As a conservative approach one can consider that the results for higher water levels do correspond to cases where the height of the beach is reduced (deeper waters at the toe). In particular, for a beach erosion of 50 cm in height, the results from cases with 50 cm increased water level should be considered: the characteristic value for the maximum wave force in case of 50 cm increasing of water level is equal to 12 kN/m.

In a second phase, experimental tests have been conducted at UGent to measure wave forces on storm wall located on top of a 1:2.4 dike slope, corresponding to cross section 99 (Profile A). Main characteristic of this profile is the presence of a storm wall, 1 m high, very close to the seaward edge of the dike. The height of the wall guarantees that no water will overtop it. However, the exerted wave forces could become quite large and must be taken into consideration for a proper design of the wall foundations.

The target wave conditions were not perfectly achieved in all tests. The differences between measured and target conditions was within the acceptable limits only in 9 test cases. Therefore, the exceedance probability has been assessed twice: first for all tests in phase 1, then for the 9 aforementioned tests. In the first case (all tests) a force of about 8-9 kN/m corresponds to a 5% exceedance probability; in the second case (9 tests) this value is equal to 6-7 kN/m.

All wave force results refer to specific cross sections that are representative of the studied coastal area. One should carefully judge whether or not to use the presented results for cases outside of the range of hydraulic and geometrical parameters for which the tests have been conducted.

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Appendix A: Cross wave investigation

In the physical model test for the design of the storm return wall in Raversijde, 4 m width large wave flume has been used. To have an efficient execution of the tests, the flume has been divided into 4 sections. Due to this separation, wave field can be distrupted by cross waves. In this memo, the influence of the cross waves are investigated.

In this investigation wave gauges are re-arranged as can be seen in **Fout! Verwijzingsbron niet gevonden.**. WG5, 6, 7, 8 are placed in line at 1.05 m from the edge of the 1/50 slope (Figure A-2). The distance of the each wave gauge is fixed as 80 cm. Those are used to see how cross waves influence to the result. WG10, 11, 12 and WG 21, 24, 27, 30 are also used to see the evolution of the waves in each section.

As a boundary condition at the wave paddle, 20 cm significant wave height and 2.2 s peak period (JONSWAP gamma=3.3) with 0.98 m depth are used. Those values are highest values used in the large wave flume in the past last 3 years.

Figure 54 – Plan view of the 4 m width large wave flume and allocation of wave gauges.

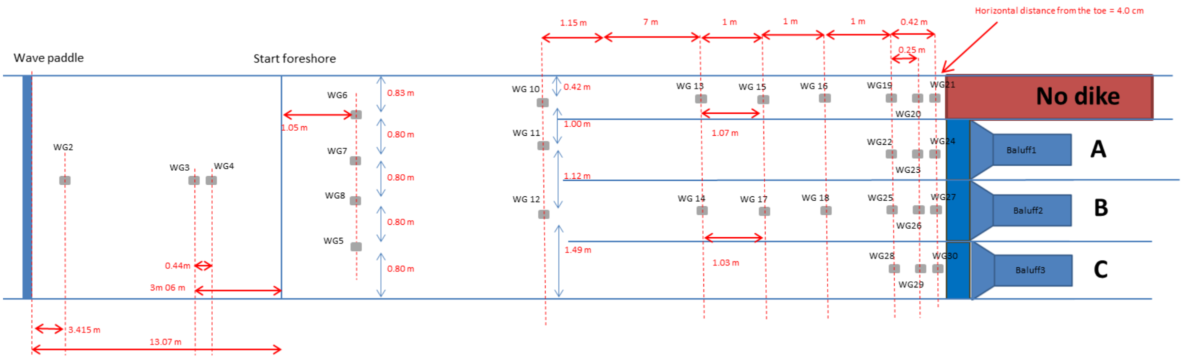


Figure 55 – The four wave gauges, WG5, 6, 7, 8.

Table 22 shows total significant wave height for WG5, 6, 7, 8 (in yellow, offshore), WG10, 11, 12 (in green, in front of the separation walls) and WG 21, 24, 27, 30 (in red, at 4 cm offshore side from the toe). Error % is a value calculated based on incident wave height (INC) obtained at the section 'No dike'.

Total significant wave height difference between sections in offshore (yellow) are up to 5% while one in front of the dike (red) is more than 50%: full reflection. Note that the values in front of the dike (red) is not so reliable since the calibration has not been conducted well. Thus this value is not an official value but the order of magnitude of those values can be judged.

Looking at the green, in front of separation walls, the value is not so much different compared to yellow, offshore ones. wave reflection become smaller because of the wave decay from wave-wave interaction etc and already reflected waves are small in front of separation walls.

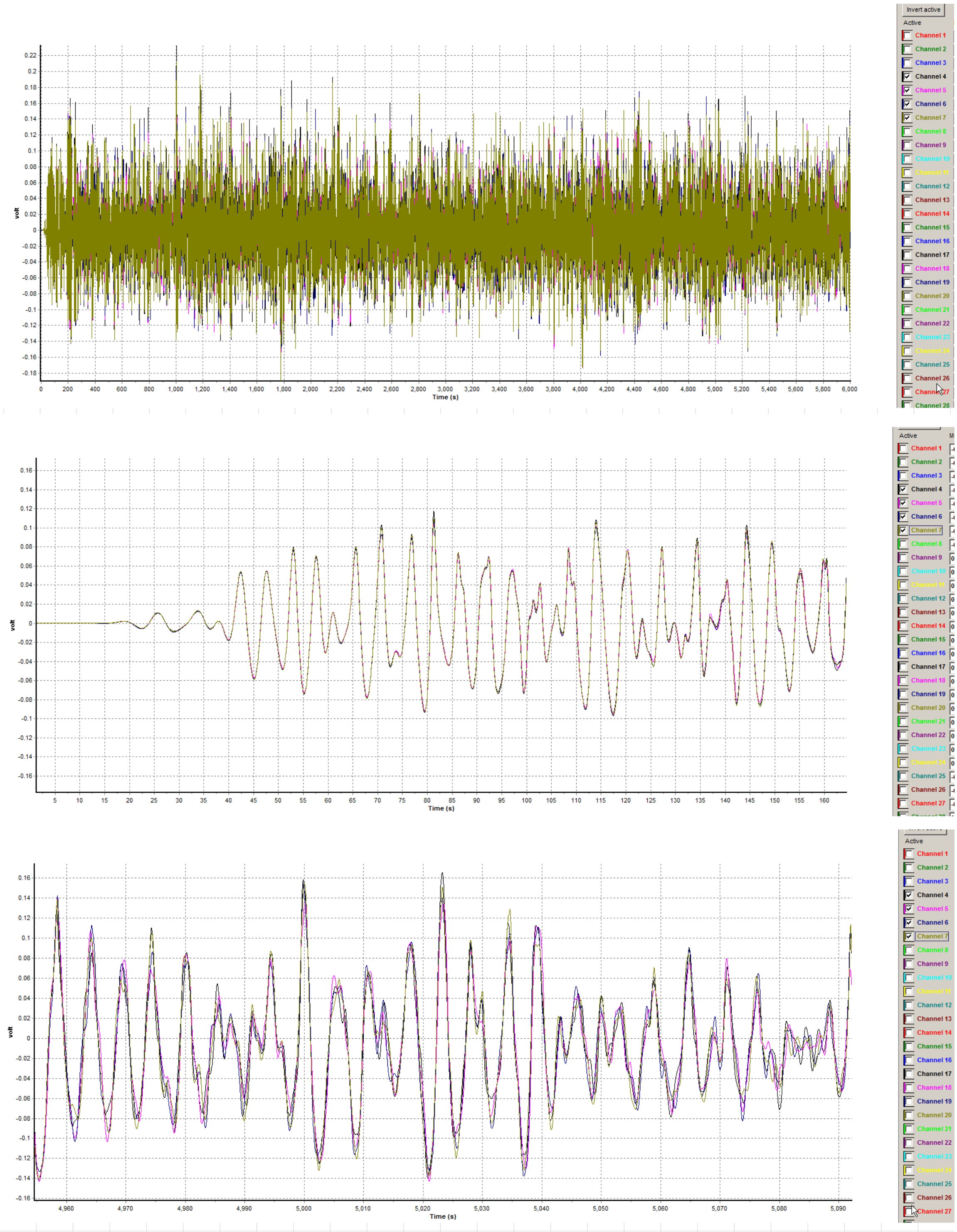
Fout! Verwijzingsbron niet gevonden. shows the wave time series for qualitative judgment (only offshore wave gauges).

Offshore wave height difference is 5%: it does not have so much impact for the results. Note that we accepts 3% error in our applications e.g. incident wave height for dike design testing.

Table 22 – Total significant wave height at each location

WG	Hm0	error %
5	0.187	3.4
6	0.181	INC
7	0.186	2.6
8	0.190	4.5
27	0.111	53.5
30	0.068	-6.7
21	0.073	INC
24	0.119	63.9
10	0.161	INC
11	0.165	2.8
12	0.170	5.9

Figure 56 – Time series of WG5, 6, 7, 8.



Upper figure shows entire time series,
middle figure shows the time series at the beginning,
lower figure shows the time series close to the end of the test (t = 5000 s)

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