CHARACTERIZATION OF WAVE IMPACTS ON CURVE FACED STORM RETURN WALLS WITHIN A STILLING WAVE BASIN CONCEPT

Corrado Altomare[1,2], Toon Verwaest[1], Tomohiro Suzuki[1,3], Koen Trouw[1,4]

Low lying coastal areas are ones of the most vulnerable zones to the effects of sea level rise and storm surge. An example is the Belgian coastline. In order to protect it from erosion and flooding on the long-term, the Flemish Government approved in 2010 the Coastal Safety Master Plan, a driving plan that provides general solutions for coastal protections looking ahead to the year 2050. The coastal town of Wenduine is one of the weakest links along the Belgian coastal defense line due to the low freeboard of the existing dike and the high population density in this area. A solution could be to heighten the existing sea walls. However, a compromise needs to be found between social and technical requirements since the elevated touristic and recreational value of the area makes very high storm return walls not acceptable as solution for the upgrading of the existing dike. Therefore the construction of a new curve-faced wave return walls, coupled with beach nourishment has been adopted to meet required mean wave overtopping discharge standards foreseen in the Master Plan. The wave loading on such kind of walls have to be characterized for a proper design. The present work illustrates the final results of the experimental campaign conducted at Flanders Hydraulics Research (Antwerp) to assess the forces exerted by sea waves onto those curve-faced walls within a stilling wave basing concept.

Keywords: wave loading; stilling wave basin; shallow foreshore; physical model; coastal protection

INTRODUCTION

The Flemish Coastal Safety Masterplan (Afdeling Kust 2011) outlines the need for improved coastal defenses along the Belgian coastline that generally consists of a combination of soft and hard works: the beach nourishment and the upgrading of the existing sea dikes. The aim of the aforementioned actions is to provide adequate defense against extreme storm events. The coastal town of Wenduine is one of the weakest links along the Belgian coastal defense line due to the low freeboard of the existing dike and the high population density in this area. The presence of shallow foreshore governs the wave propagation and wave overtopping, triggering the generation of free long waves, main factor influencing high overtopping rates and huge flooding over the coastal defenses (Figure 1). The construction of a new curve faced wave return walls, coupled with beach nourishment has been finally adopted to meet required mean wave overtopping discharge standards foreseen in the Master Plan for the coastal town of Wenduine.

The curvature of the walls increases the efficiency against overtopping making possible to maintain the wall heights below architectural acceptable limits. The new walls are not only coastal defenses but also integrated part of the promenade (parapets, benches) so that the stakeholders perceived them tolerable and user-friendly. Even though curve-faced or parapet walls are effective solutions to prevent overtopping and overflows, they shows generally higher forces than vertical walls as also confirmed by Kortenhaus et al. (2004).

Physical hydraulic model tests have been carried out in the large wave flume at Flanders Hydraulics Research laboratory, in Antwerp, to verify and to optimize the design of the proposed wave return walls. The main objectives of the physical model experiments were to quantify the expected wave overtopping rates over the walls and to assess the wave loadings for the designed layout.

The experimental campaign helped the designers to find the best solution for the upgrading of the existing sea dike. This solution aims to represent a trade-off between technical and architectural issues with big attention focused on the social and environmental impact of the new construction. Several different modifications of the existing dike have been considered during the design process. The actual sea dike profile has been changed, widening the promenade and using a vertical quay or curve-faced quay instead of the actual sloping dike. The final design consists of two curve faced walls, with drainages on the seaward wall. The promenade is extended seawards and the sea dike is actually replaced by the curve seaward wall. Hence the walls form a so-defined “stilling wave basin” wherein the energy of the waves overtopping the seaward wall can be attenuated before reaching the second wall and overflow landwards. The wall layout is conceived to make them part of the coastal promenade

[1] Flanders Hydraulics Research, Berchemlei 115, B-2140 Antwerp, Belgium Email: corrado.altomare@mow.vlaanderen.be, toon.verwaest@mow.vlaanderen.be, tomohiro.suzuki@mow.vlaanderen.be
[2] Dept. of Civil Engineering, Ghent University, Technologiepark 904, B-9052 Ghent, Belgium
[3] Dept. of Hydraulic Eng., Delft University of Technology, Stevinweg 1, 2628 CN Delft, The Netherlands
[4] Flemish Agency for Maritime Services and Coast, Vrijhavenstraat 3, B-8400 Oostende, Belgium Email: koen.trouw@fidesengineering.be

and not just alien elements on top of that. The height of the walls and effectiveness of drainages in the seaward wall have been examined. A description of the first phase of the experimental campaign can be found in Veale et al. (2012).

The results of wave loadings on the curve-shaped storm return walls are reported in the present work.

Figure 1. Oblique aerial photograph of Wenduine sea dike and foreshore

EXPERIMENTAL CAMPAIGN

Physical model experiments have been carried out in the wave flume facility at Flanders Hydraulics Research Institute, in Antwerp, Belgium (Veale et al. 2012). Objective of the experimental campaign was helping the designers to find the most adequate solution to upgrade the existing sea dike. The experimental study was comprised of the following components:

- Design and construction of a scale model of the existing foreshore profile and sea-dike at Wenduine.
- Installation of proposed wave return walls on top of the dike.
- Measurement of landward overtopping discharge over the sea-dike/wave return walls.
- Measurement of horizontal impact forces and uplift forces that cause overturning moments on the wave return walls.
- Measurement of wave heights at the toe of the sea-dike and at other locations offshore.
- Visualization of experiments with digital video recording.

Several different alternatives have been tested where the wave overtopping discharge and wave loadings on new coastal elements have been measured for each configuration. We selected two representative the design conditions for the new structure. The standards are summarized as follows:

- Standard 1: Minimize risks for a storm with 1000 years return period ($q < 1$ l/s/m).
- Standard 2: Reduce risk of major economic damage and casualties for the so-called +8.0m TAW Superstorm (17000 years return period) ($q < 100$ l/s/m).

The prototype storm conditions calculated at – 5.0 m TAW water depth are indicated in Table 1. They have been estimated from a SWAN numerical model for the Flemish Coastal Safety Masterplan (Verwaest et al. 2008).

Table 1. Prototype storm conditions extracted at -5.0m contour offshore of Wenduine

Case	Prototype				1:25 Model Scale		
	SWL [m TAW]	H_{m0} [m]	$T_{m-1,0}$ [s]	T_p [s]	H_{m0} [m]	$T_{m-1,0}$ [s]	T_p [s]
+8.0 m Superstorm	7.94	4.97	9.00	12.75	0.20	1.80	2.55
1000 Year storm	6.84	4.75	8.60	11.70	0.19	1.72	2.34

A two-dimensional, 1:25 scale physical hydraulic model of the Wenduine sea dike and foreshore was constructed in the large wave flume at Flanders Hydraulics Research. The wave flume is 70 m long, 4.0 m wide and 1.45 m deep. The facility is equipped with a piston type wave generator with a stroke length of 0.6 m, which can generate monochromatic, bichromatic and random waves. The on-line computer facilities for wave board control, data-acquisition and data-processing allow for direct control and computation of relevant wave characteristics. Wave energy spectra can be prescribed by using standard or non-standard spectral shapes or by prescribing a specific time-series of wave trains.

The wave generator is not equipped with an Active Wave Absorption System (AWAS). Passive wave absorption was installed downstream of the sea-dike models to reduce the wave reflections. A sketch of the physical flume is depicted in Figure 2.

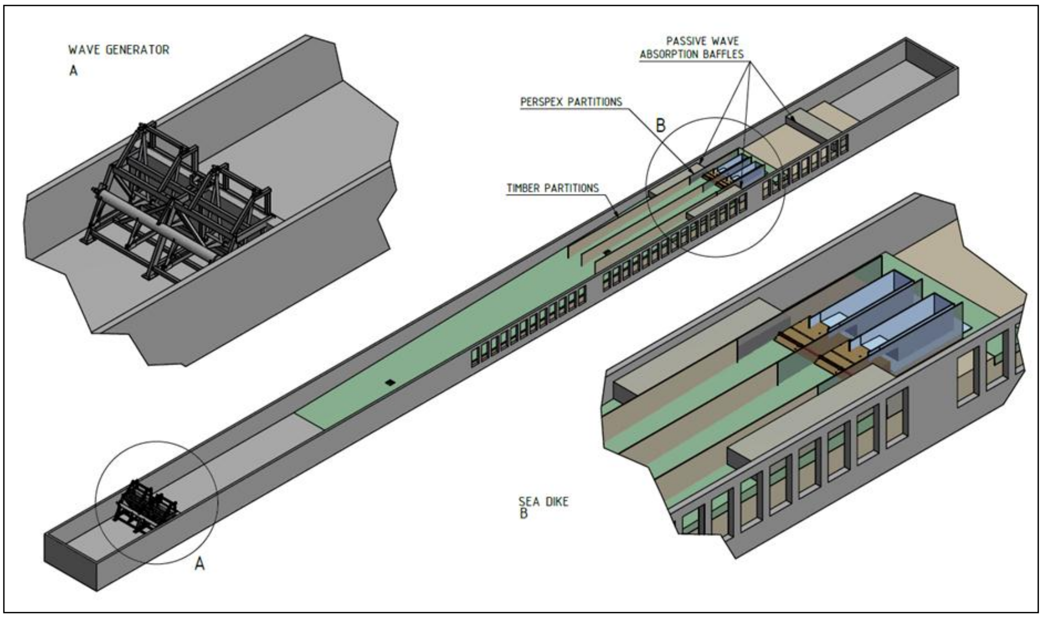

Figure 2 – 3D drawing of physical flume setup at FHR

The foreshore slope represented in the physical model was the conservative condition where the foreshore has eroded after the duration of an extreme storm event. It was important to include the foreshore slope in the model so that wave transformations occurring on the shallow foreshore were physically represented. The wave breaking leads in fact to the transformation of wave spectra with consequent shift of the wave energy towards the low frequencies and generation of long waves mostly responsible of the overtopping events. Finally the foreshore slope was represented with a consistent slope of 1:35 in the physical model in order to simplify the construction, once it was verified that this simplification had not significant effect on wave heights at the toe of the dike (H_{m0}, H_{max}) as well as corresponding mean wave overtopping discharges (Veale et al. 2011 and Suzuki et al. 2011).

Figure 3 provides an illustration of the final design for sea-dike and walls geometry. The dimensions are expressed in prototype scale. It is foreseen to widen the promenade to create a stilling wave basin between the two walls at distance of 10.75m (see Figure 4).

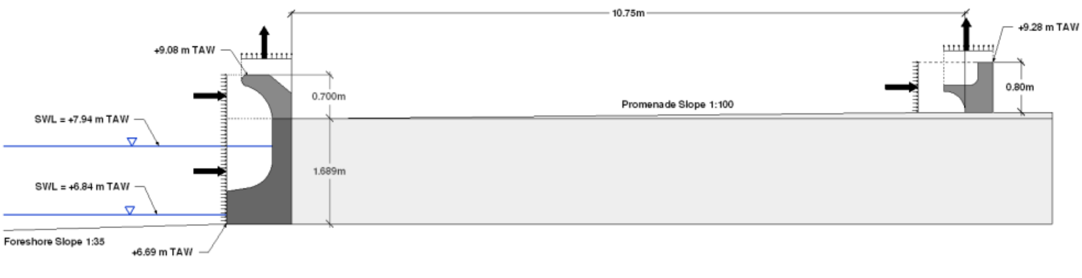

Figure 3. Schematicof sea-dike geometry

The walls are curve faced. The seaward wall is not just a topping element of the sea-dike but replaces the whole sea-dike: it can be considered as a vertical dike with a curved face exposed seaward. The crest freeboard of the seaward wall is at 9.08 m TAW. The second wall is only 0.80 m high, its final freeboard is at 9.28 m TAW. The second wall is designed to be an element integrated with the landscape. In fact its shape is conceived to represent a sort of bench along the entire promenade. The black arrows in Figure 3 indicate on which wall the horizontal and vertical forces have been measured. The seaward wall has been split into two parts, one above and one below the promenade level, for the measurements of the horizontal forces.

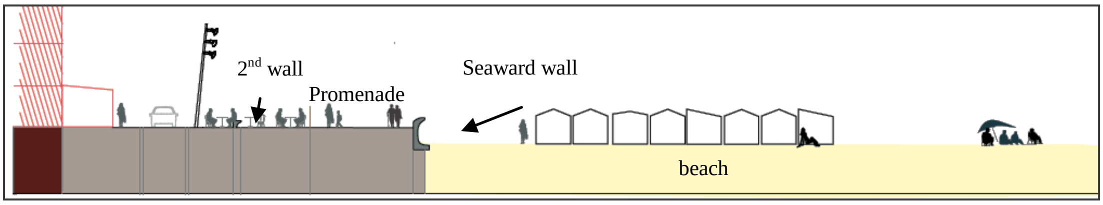

Figure 4. Schematic of the new promenade with curve-faced walls and stilling wave basin

Instrumentation

Wave height measurements were obtained with twelve resistance type wave gauges installed at the locations illustrated in Figure 5. To determine incident wave parameters reflection analysis was performed on data from the wave gauge arrays. Reflection analysis was performed with WaveLab 3.39, software for wave analysis developed at Aalborg University (Denmark), which utilises the Mansard and Funke (1980) method. As referred to in Van Gent and Giarusso (2003), the Mansard and Funke (1980) method assumes linear wave motion, which is not generally valid for the wave motion at Station 5, where severe wave breaking and surf-beat phenomena occur. As a result the Mansard and Funke (1980) method is expected to introduce inaccuracies when separating incident and reflected waves close to the toe of the dike.

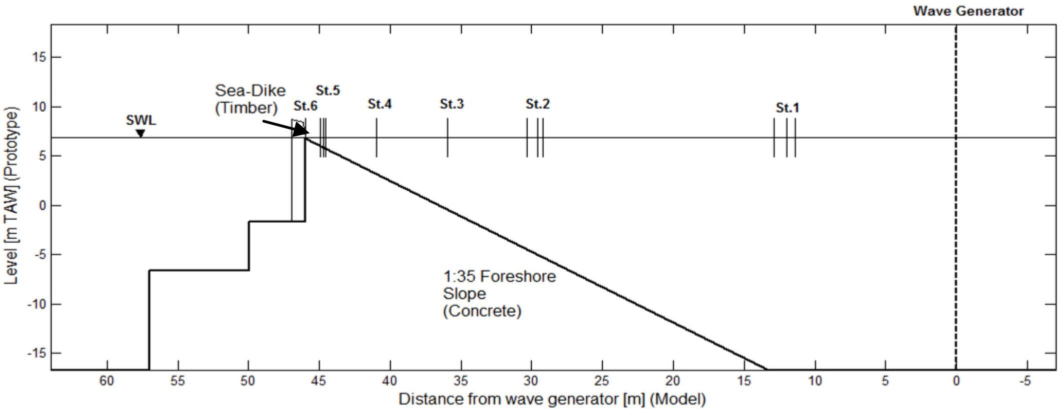

Figure 5. Wenduine sea-dike and foreshore as constructed in the large wave flume (NB: y-axis is in prototype scale, x-axis in model scale)

Wave forces on the storm return walls have been measured using five S-type load cells (Tedea Huntleigh Model No. 614, maximum capacity 50 kg). Three load cells have been used to measured horizontal forces (H1, H2 and H3 in Figure 6) and two load cells were measuring the vertical forces on the walls (V1 and V2 in Figure 6).

Hydraulic boundary conditions

For all model tests a JONSWAP wave spectrum with $\gamma = 3.3$ was generated with the wave paddle, and the total number of waves generated was at least 1000.

As remarked by Kisacik et al. (2012) although the generated waves in one test are nominally identical, their impact behavior can vary significantly in one run. The phenomena of wave propagation, transformation and impact on coastal structures can be thereby considered a stochastic process, so that no repeatability is possible and the impact has to be characterized following a statistical approach. Once the hydraulic boundary conditions have been defined as representative of extreme storm event on the Belgian coast, several tests (respectively 20 for the 1000 year storm and 19 for the +8.0m TAW Superstorm condition) have been carried out for the same target waves, but changing each time the wave train of the series.

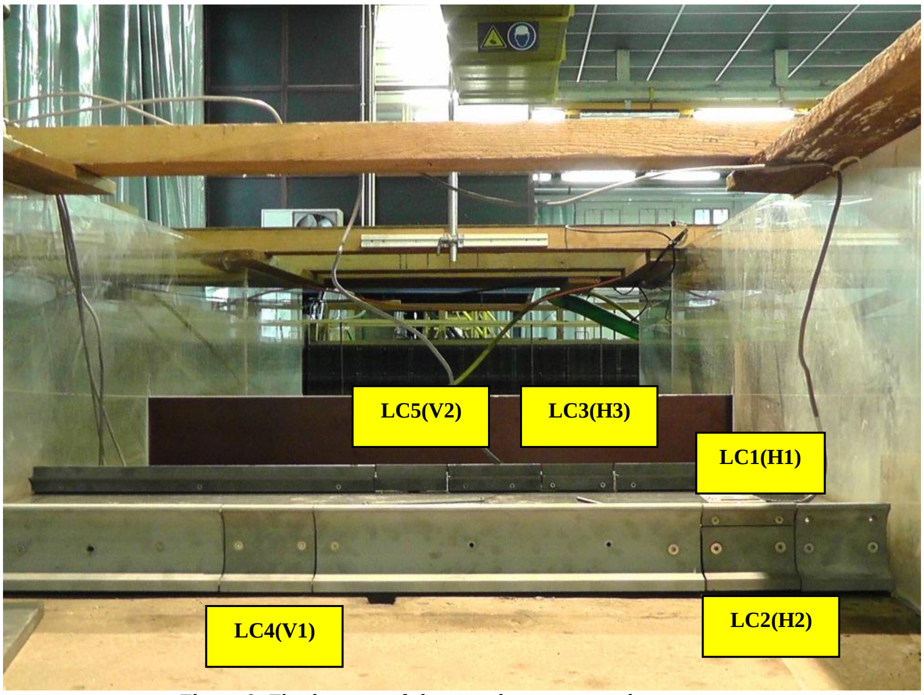

Figure6. Final setup of the new instrumentation system.

WAVE LOADING RESULTS

The wave forces on the storm return walls have been sampled at 1000 Hz. Both horizontal and vertical forces have been measured. The wave loading signal has been analyzed to identify and characterize the peak values of wave forces, the quasi-static forces and the rise and duration times of the biggest impact events. Peak forces from wave impact events have been extracted from measured force time series. As in McConnel and Kortenhaus (1997) the chosen threshold value for forces on the seaward wall is function of the incident wave height as $F_{threshold}=0.1H_{m0}^2\rho g$ [N/m]. A value of 50% of the $F_{threshold}$ has been considered as threshold value for the loadings on the second wall which is installed landward on the promenade. The same algorithm proposed by McConnel and Kortenhaus (1997) has been used to identify the quasi-static forces and rise and duration time of each impact event.

A huge spread in the result has been noticed from test to test. The shape of the impact, the peak forces and quasi-static forces, the rising time and duration time have been assessed. Both horizontal and vertical (uplift) forces have been measured on both the walls.

+8.0 m TAW Superstorm

Maximum peak wave impact forces (F_{max}) measured by each sensor for all model tests are listed in Table 2 for the +8.0 m Superstorm hydraulic boundary conditions, as well as the total value of the horizontal wave force on the seaward wall (H1+H2). The mean value and standard deviation are also indicated. The vertical component on the seaward and promenade wall are expressed as positive if directed upwards and negative if directed downwards.

The analysis of the signals of each load cell has shown that the horizontal components of the forces present the so-called "roof" values or quasi-static forces after the peaks. However for H1 (force on the upper part of the seaward wall) there are quite often no roofs after the peaks. It is evident that H1 is not representative of the entire wave loading on the wall and the definition of roof values is made in literature for the loading on the entire walls. For the upward vertical force on the seaward wall V1+ roof values are presented. For the second wall no roof values were determined because of very low values or not at all a roof. Since it has been observed that in many time series a certain noise is present, this can lead to wrong readings for the assessment of the quasi-static force. A filter has been applied to the data to extract the quasi-static forces, as also suggested in McConnel and Kortenahus (1997). It is worthy to notice that the application of this low pass filter (40 Hz) affects the peak values but doesn't affect the quasi-static forces. It only removes the noise present in the signal of the quasi-static forces. A detail view of such kind of effect can be seen in Figure 7.

Table 2. F_{max} [kN/m] peak wave impact forces for +8.0m Superstorm and incident wave characteristics (prototype scale)

TEST ID	H1	H2	H3	V1 +	V1 -	V2 +	V2 -	H1+H2	H_{m0} [m]	T_p [s]
WEN_276	167	392	43	317	-193	21	-46	524	5.04	12.80
WEN_278	204	350	32	389	-284	24	-41	526	4.96	12.80
WEN_279	200	368	44	388	-255	28	-23	568	4.94	12.80
WEN_282	96	291	45	266	-137	44	-32	374	4.97	13.47
WEN_283	116	345	59	325	-184	21	-28	456	5.04	12.80
WEN_284	115	284	53	199	-120	26	-25	399	5.03	12.80
WEN_285	135	357	48	245	-167	27	-39	381	4.92	12.80
WEN_286	131	232	34	318	-192	35	-27	348	5.11	12.80
WEN_287	148	345	46	353	-132	42	-36	493	5.05	12.80
WEN_289	150	372	43	231	-135	35	-42	503	5.04	12.80
WEN_290	146	388	74	268	-148	58	-33	482	4.81	13.48
WEN_292	265	466	48	249	-132	42	-44	731	5.08	13.48
WEN_293	105	234	38	256	-123	75	-53	312	4.95	12.80
WEN_295	128	273	45	466	-287	66	-59	372	4.87	12.80
WEN_296	223	306	116	279	-141	62	-41	525	4.90	12.80
WEN_297	297	358	50	369	-240	36	-33	655	5.02	12.80
WEN_298	123	264	71	467	-276	40	-41	359	5.02	12.19
WEN_299	159	327	39	311	-172	64	-46	407	5.05	12.80
WEN_300	136	302	59	286	-161	36	-36	437	5.08	12.80
WEN_321	145	386	42	407	-242	34	-24	516	5.10	12.80
Mean Value	159	332	51	319	-186	41	-37	468		
St. Dev.	52	57	18	74	56	16	9	103		

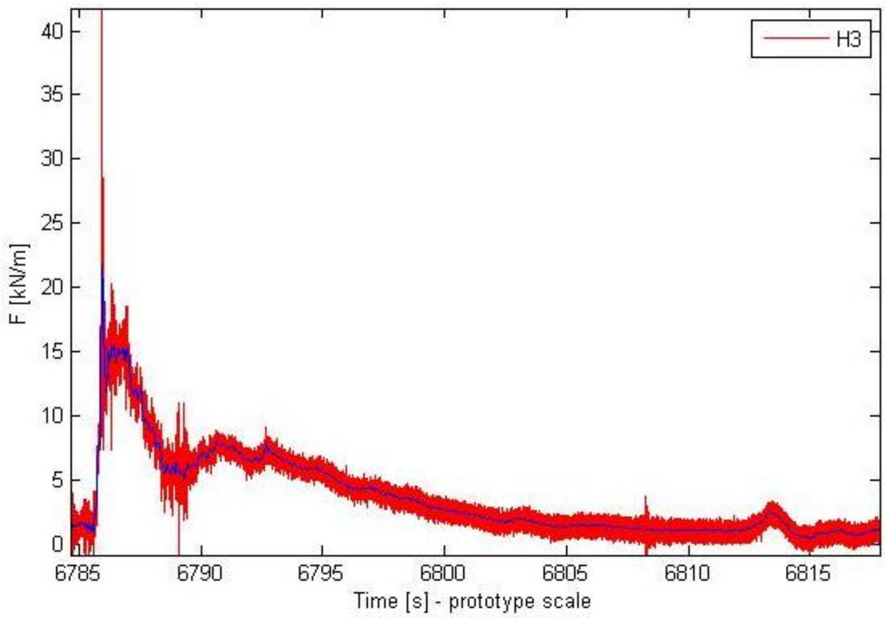

Figure 7. Example of the effect on the filtering for the determination of the quasi-static forces (blue line gives 40 Hz low pass filter smoothing).

The maximum measured values for the quasi-static forces are:

- 90 kN/m for the horizontal force on the seaward wall, H1+H2 (prototype scale)
- 27 kN/m for the horizontal force on the seaward wall, H3 (prototype scale)
- 21 kN/m for the vertical upwards force on the seaward wall, V1+ (prototype scale)

A possible correlation between the highest impact events (peak wave impact forces) for H1 and V1 have been investigated to check if the maximum events for the horizontal force on the parapet and the vertical force on the seaward wall occur at the same instant. The analysis has demonstrated that the correlation is low, with huge scatter of data as shown in Figure 8 where the 20 highest values of H1 and V1 are reported and for each of them the corresponding V1 and H1 value (occurring at the same time). Only one value shows the highest values of H1 and V1 occurring simultaneous (indicated with an red arrow in the graph). The vertical force corresponding to the instant when the maximum horizontal force occurs is called $V1(H1_{max})$; in the same way, $H1(V1_{max})$ has been defined for the horizontal force corresponding to the maximum vertical one.

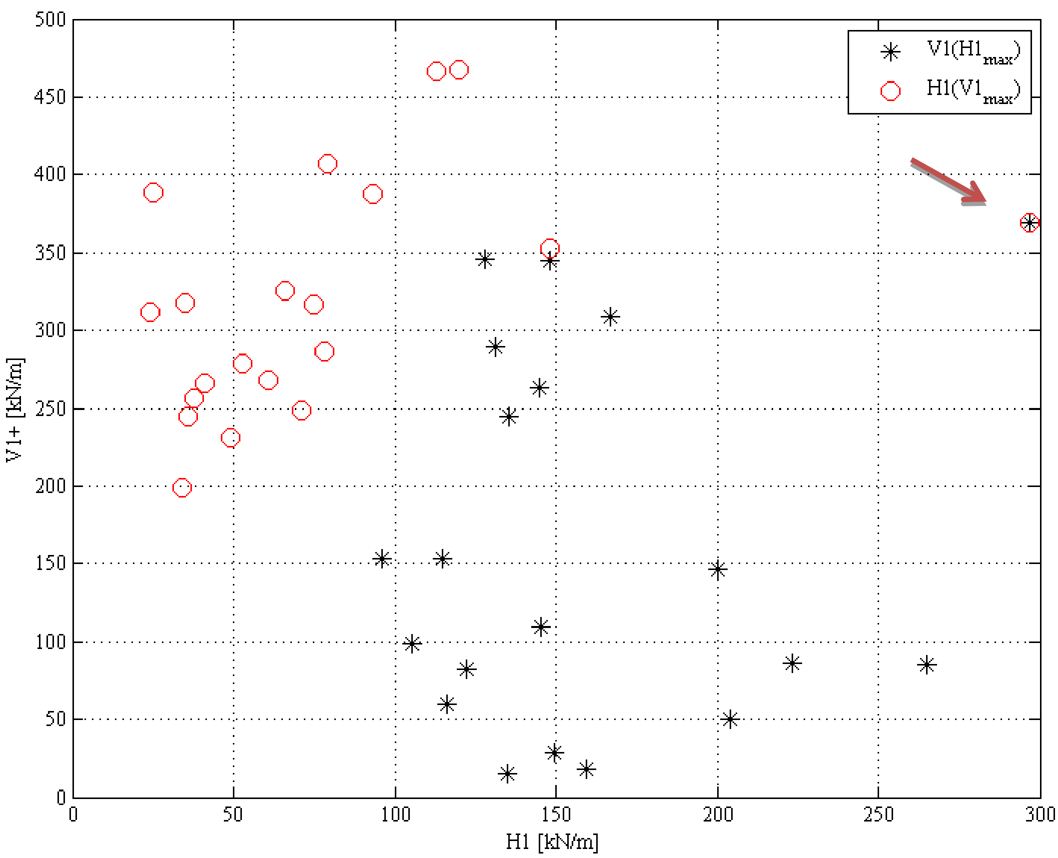

Figure 8. Correlation between H1 and V1 peak wave impact forces.

1000 years storm

Maximum peak wave impact forces (F_{max}) are listed in Table 3 (raw signal) for the 1000 year storm conditions. The total value of the horizontal wave force on the seaward wall (H1+H2) is shown as well. The mean value and standard deviation are also indicated. The vertical component on the seaward and promenade wall are expressed as positive if directed upwards and negative if directed downwards.

The previous results (average values) for "8m Superstorm" are shown in Table 4 and compared. The difference is assumed positive if there is a reduction of the 1000 year storm respect to the +8.0 m TAW Superstorm, negative if the mean values result higher than the previous ones.

The algorithm described in McConnel and Kortenhaus (1997) has been used to identify the quasi-static forces after each peak. The analysis of the signals for the quasi-static force is limited to the total horizontal force on the seaward wall. The maximum value for H1+H2 is 66.5 kN/m (prototype scale). For +8.0 m TAW Superstorm it was around 90 kN/m, thus 30% higher.

Table 3.Fmax [kN/m] peak wave impact forces for 1000 years storm storm and incident wave characteristics (prototype scale)

TEST	H1	H2	H3	V1 +	V1 -	V2 +	V2 -	H1+H2	H_{m0} [m]	T_p [s]
WEN_304	118	251	36	184	-98	47	-21	313	4.79	11.64
WEN_305	101	196	68	264	-144	54	-24	280	4.77	12.80
WEN_307	113	252	46	161	-78	54	-19	274	4.71	11.64
WEN_308	92	194	30	174	-93	55	-25	242	4.57	12.19
WEN_309	132	283	63	263	-148	54	-24	370	4.90	11.13
WEN_310	153	415	26	376	-154	55	-16	568	4.73	11.64
WEN_311	180	286	60	280	-133	39	-44	425	4.72	12.19
WEN_312	100	174	44	208	-99	73	-42	244	4.74	11.13
WEN_313	89	239	31	171	-93	35	-24	327	4.73	11.13
WEN_314	85	228	48	285	-141	35	-17	293	4.72	11.64
WEN_315	176	373	39	220	-143	81	-47	549	4.68	11.13
WEN_316	103	295	41	186	-110	47	-34	349	4.75	11.64
WEN_317	131	249	51	231	-100	53	-36	315	4.72	10.67
WEN_318	136	418	47	247	-107	57	-28	515	4.81	12.19
WEN_319	149	478	42	231	-145	162	-78	627	4.70	11.64
WEN_320	81	375	52	150	-88	43	-28	425	4.82	11.64
WEN_325	81	170	52	256	-148	85	-19	225	4.74	11.64
WEN_326	111	239	33	196	-114	45	-21	300	4.71	11.13
WEN_328	101	329	78	208	-103	90	-35	423	4.71	11.64
Mean Value	117	286	47	226	-118	61	-31	372		
St. Dev.	30	87	13	53	24	28	14	116		

Table 4. Average values of the highest peak events for 1000 years storm and +8.0 m TAW Superstorm and differences [%] (force values in kN/m, prototype scale)

TEST	H1	H2	H3	V1 +	V1 -	V2 +	V2 -	H1+H2
1000 years storm	117	286	47	226	-118	61	-31	372
+8.0 m TAW Superstorm	159	332	51	319	-186	41	-37	468
Differences	26%	14%	8%	29%	37%	-49%	17%	21%

Statistical analysis

Figures 9÷11 plot the exceedance probability of the peak wave impact forces (red dots) measured by each sensor for all the model tests (no filter applied to the data signals). They are compared with the results of the +8.0 m TAW Superstorm (in blue in the figures). The y-axis is in logarithmic scale. The exceedance probability has been calculated on the total number of incoming waves for the whole dataset. These figures show that:

- All the data present an exponential tendency above a 0.05% exceedance probability with very low dispersion around the mean trend.
- The forces acting on the seaward wall (H1, H2, V1) present similar trends between the 2 storm events above 0.05% exceedance probability.
- The differences in H1 increase up to 120 kN/m below 0.02% exceedance probability.
- The values of H2 below 0.02% exceedance probality are quite similar to the +8.0 m TAW Superstorm results. For the biggest events in fact the bottom part of the wall is completely submerged; consider the small differences in the wave height and wave period of the 1000

years storm incoming waves respect to the +8.0 m TAW Superstorm, reason why the results are close.

- The H3 values for the highest events (below 0.1% exceedance probability) are quite similar to the +8.0 m TAW Superstorm ones. Notice that on one hand the overtopping rates responsible of this action are less than those in +8.0 m TAW Superstorm, but on the other hand the second wall is more exposed to their action than before. In fact, for the +8.0 m TAW Superstorm conditions, there is a sort of water pillow generated between the two walls that absorbs part of the energy of the waves that are hitting the promenade wall.
- V1 presents increasing differences between the two storm events below 0.02% exceedance probability.
- V2 presents values higher than the +8.0 m TAW Superstorm ones. Since the wall is not totally submerged as before (see what said for H3) it can be exposed directly to the wave action. This means high upwards vertical forces.

Figure 12 plots the exceedance probability of the quasi-static forces. Only the 10 highest roof values per each tests have been considered. As for the peaks, the exceedance probability has been calculated on the total number of incoming waves for the whole dataset and plotted together with the +8.0 m TAW Superstorm results.

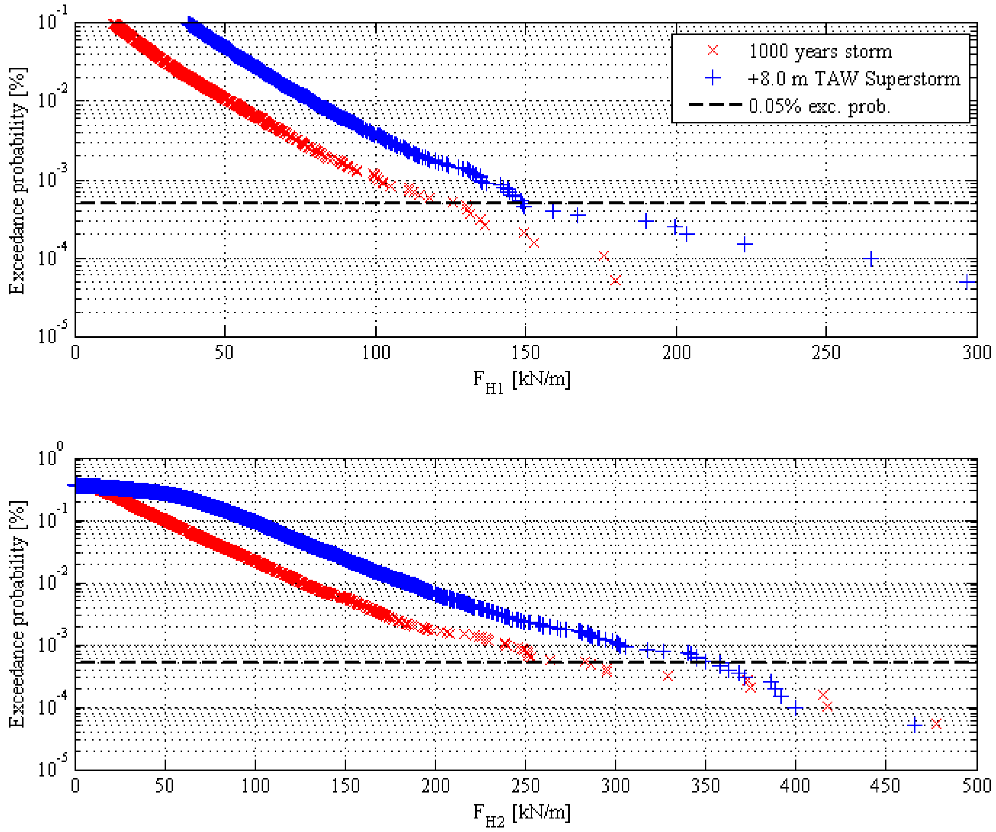

Figure 9. Wave impact exceedance probability of the horizontal forces on the seaward wall (peak values)

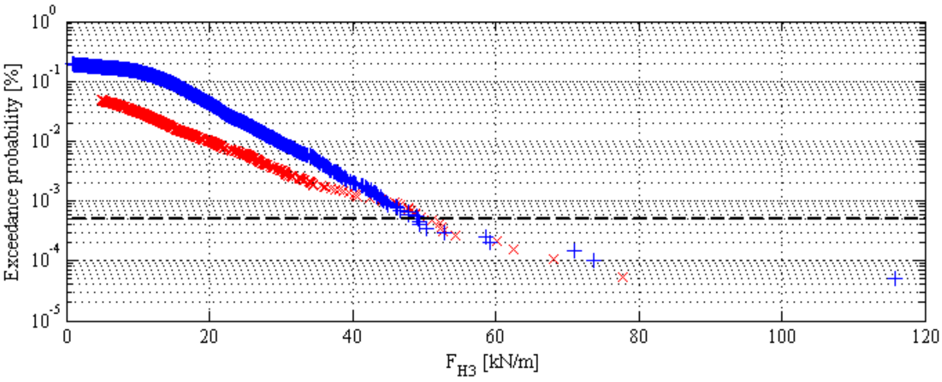

Figure 10. Wave impact exceedance probability of the horizontal force on the landward wall (peak values)

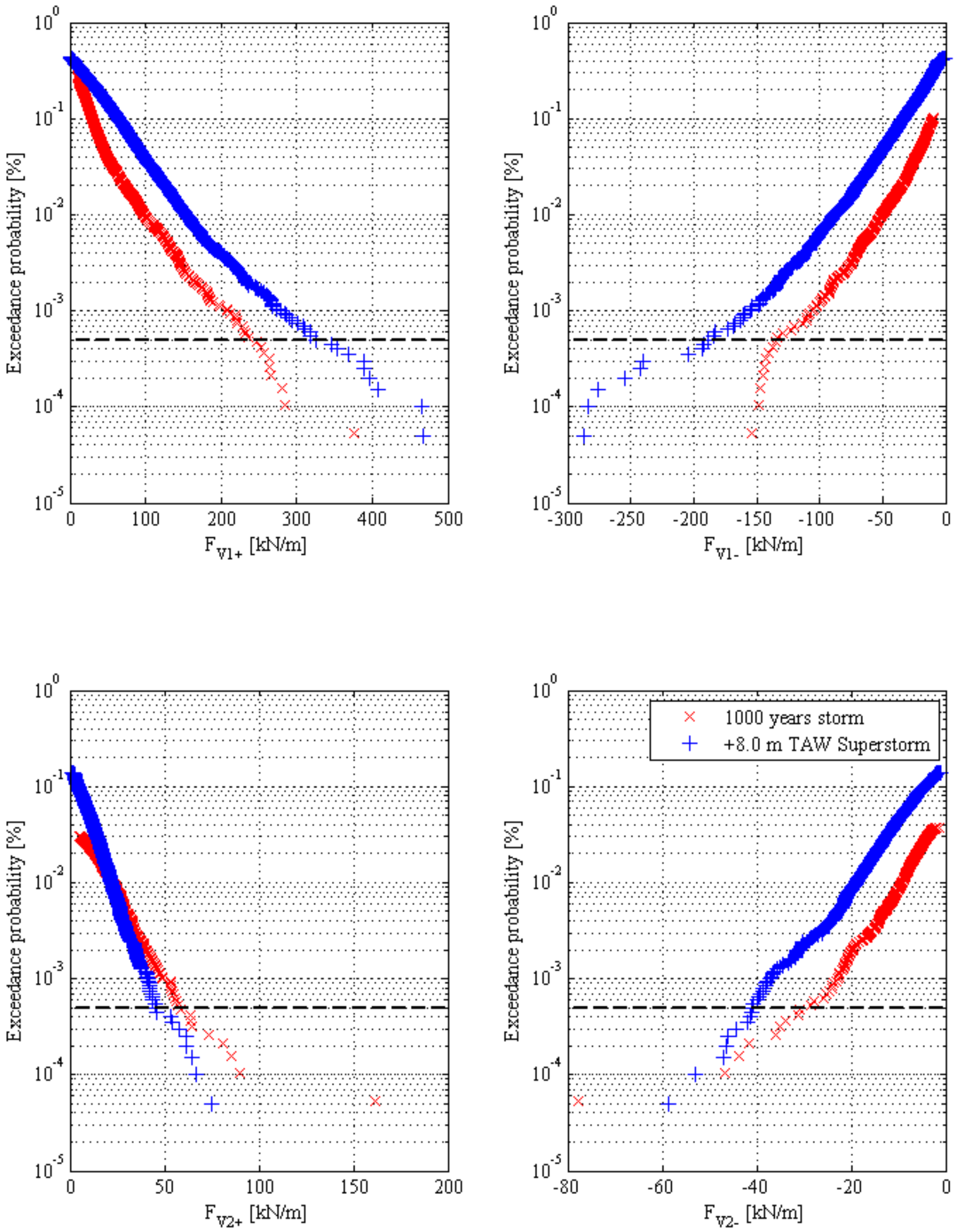

Figure 11. Wave impact exceedance probability of vertical forces (peak values)

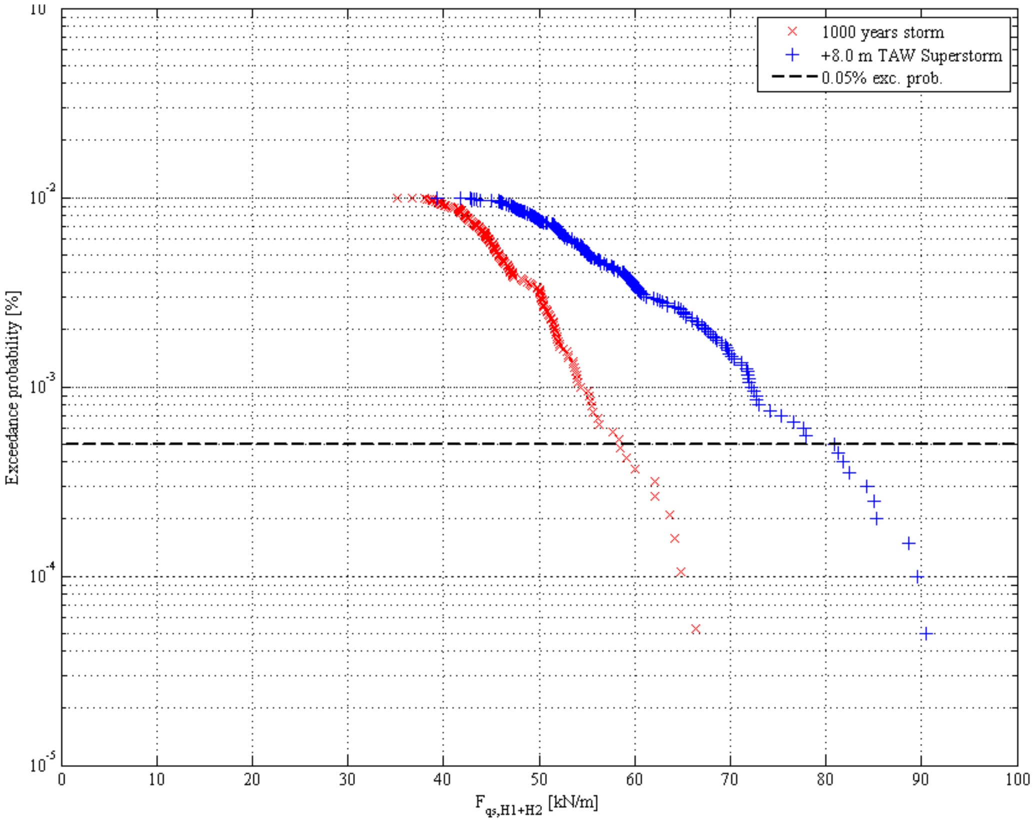

Figure 12. Wave impact exceedance probability of quasi-static horizontal forces exerted on the seaward wall

Time evolution - rise time and duration time

The time evolution of the loadings (the "shape" of the loading on a time scale plot) has been analyzed. Because of the nature of the impact and the hydrodynamic conditions, the impact shape can be compared with the so-called "church profile" where after a very steep and high peak a quite long and lower value can be identified (quasi-static force or "roof of the church").

Some essential characterizing parameters are the rise time and the duration time. The rise time t_r is the time between the start time of the event and the time of the maximum force (peak). The start time t_0 is the time when the force is starting to be higher than the threshold value. The duration time t_d is the time during which the signal is above the threshold value, so includes the rise time.

The 1000 years storm impacts are characterized by a rise time between 0.05 s and 1.2 s. The duration time is normally around 7-10 s. The +8.0 m TAW Superstorm impacts are characterized by a rise time between 0.06 s and 0.7 s. The duration time is normally around 6-16 s. Rise time and duration time vary case per case. There is then a quite wide range between the lowest and highest rise times that is strictly connected with the wave transformation occurring in the breaking zone and the wave by wave interaction.

CONCLUSIONS

The present work reports the analysis and characterization of wave loadings on the defined configuration for the new sea dike in coastal town of Wenduine which is identified as one of the weakest links along the Belgian coastline due to the low freeboard of the existing dike and its highly populated area.

The presence of shallow foreshore governs the wave propagation and wave overtopping, triggering the generation of the low-frequency waves, main factor influencing high overtopping rates over the coastal defense. To cope with it, a compromise has been found between social and technical requirements because of the highly touristic and recreational value of the area makes.

Physical model experiments have been carried out in the wave flume facility at Flanders Hydraulics Research Institute, in Antwerp, Belgium. Objective of the experimental campaign was helping the designers to find the most adequate solution to upgrade the existing sea dike. Several different alternatives have been tested where the wave overtopping discharge and wave loadings on new coastal elements have been measured for each configuration. The final design consisted in new two curve faced walls has been adopted to meet required mean wave overtopping discharge standards. A stilling wave basin is so formed between the walls.

Two target storm conditions have been modelled in the physical facility. For each one of them, several tests have been carried out where only the time series of the waves has been changed. In fact, as also remarked by Kisacik et al. (2012) although the generated waves in one test are nominally identical, their impact behavior can vary significantly in one run. The phenomena of wave propagation, transformation and impact on coastal structures can be thereby considered a stochastic process, so that no repeatability is possible and the impact has to be characterized following a statistical approach. For this reason, once the hydraulic boundary conditions have been defined as representative of extreme storm event on the Belgian coast, the tests have been carried out changing each time the wave train of the series.

A huge spread in the result has been noticed from test to test. The shape of the impact, the peak forces and quasi-static forces, the rising time and duration time have been assessed. Both horizontal and vertical (uplift) forces have been measured on both the walls.

REFERENCES

Afdeling Kust (2011). Masterplan Kustveiligheid [Online]. Available: http://www.afdelingkust.be/Userfiles/pdf/110628_RL_Rappor_%20KustveiligheidWEB4.pdf

Kisacik D., Troch P., Van Bogaert P. 2012. Description of loading conditions due to violent wave impacts on a vertical structure with an overhanging horizontal cantilever slab. *Coastal Engineering*, 60, 201-226.

Kortenhaus A., Pearson J., Bruce T., Allsop N. and van der Meer J. (2004) Influence of Parapets and Recurves on Wave Overtopping and Wave Loading of Complex Vertical Walls. *Coastal Structures 2003*: pp. 369-381.

Mansard E. P. D., Funke E. R. 1980. The measurement of incident and reflected spectra using a least squares method. *Coastal Engineering*1980, ASCE, 154–172.

McConnell K.J., Kortenhaus A. 1997. Analysis of pressure measurements from hydraulic model tests and prototype measurements, in: (1997). *Proceedings 1st overall project workshop MAST III / PROVERBS*, Las Palmas, Gran Canaria 18-22 February 1997: probabilistic design tools for vertical breakwaters. Report and papers task 1. pp. [1-14]

Suzuki T., Verwaest T., Hassan W., Veale W., Reyns J., Trouw K., Troch, P. and Zijlema M. 2011. The applicability of SWASH for modelling wave transformation and wave overtopping: A case study for the Flemish coast. *Proceedings in ACOMEN* 2011.

Van Gent M.R.A. Giarrusso C.C. 2003 Influence of low-frequency waves on wave overtopping; A study based on field measurements at the Petten Sea-defence. WL | Delft Hydrauilcs.

Veale W., Suzuki T., Verwaest T., Hassan W., Trouw K, Mertens T. 2012. Integrated Design of Coastal Protection Works for the town of Wenduine, Belgium. *Proceedings of 33rd International Conference on Coastal Engineering*, 2012, Santander, Spain.

Verwaest T., Van Poucke Ph., Vanderkimpen P., Van der Biest K., Reyns J., Peeters P., Kellens W., Vanneuville W., Mostaert F. 2008. Overstromingsrisico's aan de Vlaamse kust. Evaluatie van de zeeweing. Deel 1: Methodologie. *WL Rapporten, 718/2A. Waverboukundig Laboratorium & Universiteit Gent & Soresma-Haecon*.