INTEGRATED DESIGN OF COASTAL PROTECTION WORKS FOR WENDUINE, BELGIUM

William Veale^{1,2}, Tomohiro Suzuki^{1,2,3}, Toon Verwaest², Koen Trouw^{4,5} and Tina Mertens⁵

Wave overtopping tests were performed with a 1:25 physical scale model to determine the optimal geometry for design of new wave return walls at Wenduine, Belgium. Wave overtopping on the shallow foreshore at Wenduine was found to be dominated by low-frequency infragravity waves (f < 0.04 Hz at prototype scale). Mean wave overtopping discharge measured with the physical model compared well with the Van Gent (1999) empirical overtopping equations for shallow foreshores. Physical model tests confirmed that the stilling wave basin concept proposed by Geerearts, et al. (2006) and wave wall parapet concepts of Van Doorslaer & De Rouck (2010) were effective at reducing the wave wall height required to meet the tolerable discharge overtopping standards.

Keywords: wave overtopping, stilling wave basin, shallow foreshore, infragravity waves.

INTRODUCTION

A masterplan to strengthen weak links in the Belgian coastal defence line was established by the Flemish government in 2011 (see e.g. Mertens, et al. 2008 Afdeling Kust 2011). One of the measures outlined in this masterplan is to reduce the risk of wave overtopping at the Belgian coastal town of Wenduine, by a combination of beach nourishment and construction of a new wave return wall on the existing sea dike.

To optimize the geometry of the new wave return wall at Wenduine, wave overtopping tests using a 1:25 scale physical model were performed at Flanders Hydraulics Research laboratory. The objective of wave overtopping tests was to develop a design for a coastal defense that integrates aspects of engineering safety, architectural design, urban planning and community requirements. This paper describes the experimental setup and results from wave overtopping tests.

BACKGROUND

The existing sea dike and foreshore at Wenduine is depicted in Figure 1. The key features of this dike are its low level of existing freeboard; shallow foreshore; close proximity to an urban area; low lying hinterland behind the sea dike and the high recreational and touristic value of the beach and promenade.

Table 1 lists the standard of protection provided by the existing sea dike for the two design storm events, i.e. the 1000 year annual recurrence interval (ARI) storm and the "+8.0m Superstorm", which is an extreme storm with an estimated 17,000 year ARI (Van der Biest 2009). The existing mean wave overtopping discharges listed in have been estimated or extrapolated from wave overtopping tests performed with a physical model which is described in the following sections.





Figure 1. (a) Location of Wenduine on the Belgian coast (b) Existing sea dike at Wenduine (shaded in yellow)

¹ Flanders Hydraulics Research, Berchemlei 115, Antwerp B-2140, Belgium.

² Department of Civil Engineering, University of Ghent, Technologiepark 904, Ghent B-9052, Belgium.

³ Environmental Fluid Mechanics Section, Faculty of Civil Engineering and Geosciences, Delft University of Technology, P.O. Box, 2600 GA Delft, The Netherlands.

⁴ Fides Engineering, Sint Laureisstraat 69d, B-2018 Antwerp, Belgium.

⁵ Flemish Agency for Maritime and Coastal Services, Vrijhavenstraat 3, Oostende 8400, Belgium.

Table 1. Existing and tolerable mean wave overtopping discharge at Wenduine					
Storm Event	Existing Overtopping Discharge	Tolerable Discharge Criteria			
1000 Year ARI	q ~ 70 l/s/m	q < 1 l/s/m			
+8m Superstorm	q ~ 500 l/s/m *	q < 100 l/s/m			

* extrapolated value

STUDY OBJECTIVES

The objective of the present study is to perform wave overtopping tests with a physical hydraulic model to determine the optimal geometry of wave return walls to be constructed on the existing sea dike at Wenduine. The crest level of the proposed wave return walls must be as low as possible for community acceptance, but must also restrict the overtopping hazard by limiting mean overtopping discharge (q) to tolerable discharge criteria listed in Table 1 for the 1000 Year ARI and the "+8.0 m Superstorm" event, as prescribed by the Belgian coastal safety masterplan (Afdeling Kust 2011).

PHYSICAL MODEL

Model Geometry

Wave overtopping tests were performed at 1:25 scale in the large wave flume at Flanders Hydraulics Research laboratory. Wave flume dimensions are 70 m long, 1.4 m high and 4 m wide. A piston type wave generator with a stroke of 0.5 m was used for wave generation with a passive wave absorption system was located downstream of the sea-dike.

The absence of an active wave absorption in the flume means that wave energy could accumulate in the flume if reflections are high. However, analysis of wave energy density spectra at offshore locations in the flume, indicate that if there was energy accumulation in the flume it was limited.

The physical model geometry features a smooth, impermeable sea dike setup on a foreshore with a constant 1:35 slope; refer to Figure 2. The foreshore slope was constructed from smooth concrete, and is a simplified representation of the foreshore profile at Wenduine post a 1000 year storm event, estimated with the DUROSTA beach erosion model (Steetzel 1993). The sea dike and sea wall models were constructed from laminated timber, and a number of sea wall configurations were tested as outlined in the "Test Program" sub-section.



Figure 2. Physical model geometry (prototype scale)

Table 2. Incident wave boundary conditions at "target" wave gauge array								
Storm Event	Prototype Scale				Model Scale (1:25)			
	SWL (m TAW)	H _{m0} (m)	T _{m-1,0} (s)	T _p (s)	Model depth (m)	H _{m0} (m)	T _{m-1,0} (s)	T _p (s)
"+8.0 m Superstorm"	7.94	4.97	9.00	12.75	0.99	0.20	1.80	2.55
1000 Year Storm	6.84	4.75	8.60	11.70	0.94	0.19	1.72	2.34

Instrumentation

Wave height measurements were obtained with twelve resistance type wave gauges installed at the locations indicated in Figure 2 This allowed for analysis of incident and reflected waves at the wave gauge arrays labeled "Offshore" "Target" and "Toe" in Figure 2. Wave gauges were sampled at 20 Hz. Mean and instantaneous wave overtopping discharge was evaluated with a "Baluff Micropulse" water level sensor installed in the overtopping collection box.

Boundary Conditions

The wave parameters listed in Table 2 have been derived from SWAN model calculations of extreme storm events on the Belgian coastline (refer to Verwaest et al. 2008 for further details). Wave parameters were extracted from the SWAN numerical model at the -5.0 m TAW contour level offshore of Wenduine (TAW = Tweede Algemene Waterpassing; Belgian standard datum level).

The wave boundary conditions listed in Table 2 were reproduced at the -5.0 m TAW boundary in the scale model, which is located at the wave gauge array labeled "Target" in Figure 2. A JONSWAP wave spectrum with γ = 3.3 was generated at the wave paddle for all model tests, and the total number of waves generated for each test was at least 1000.

Test Program

The sea dike and sea wall variables listed in Table 3 were varied as part of the test program. Definitions of these parameters are indicated in Figure 2. Parapet designs were based on the geometry outlined in Van Doorslaer & De Rouck (2010), which consisted of a parapet nose angle (β) of 50 degrees as illustrated in Figure 3, and the ratio of λ varied with SWL. Stilling wave basin (SWB) concepts were based on work outlined in Geeraerts et al. (2006), and consisted of two walls offset by approximately 10 m, with an opening in the seaward wall to allow drainage of the stilling wave basin. For full details of the model test program refer to Veale et al. (2011). In total approximately 150 wave overtopping tests were performed.

Table 3. Matrix of test parameters					
Variable	Parameters Tested				
Dike slope	1:2 slope	Vertical	-	-	
Wall height	0 m	0.6 m	1.2 m	1.8 m	
Wall location	No walls	А	В	A & B (SWB)	
Wall geometry	Vertical wall	Parapet	-	-	



Figure 3. Wave wall parapet geometry (after Van Doorslaer & De Rouck, 2010)

RESULTS

Model Validation

The physical model was validated by comparing measured mean wave overtopping discharge from physical model tests setup with a smooth sloping dike with the EurOtop empirical equation for shallow foreshores (see Pullen et al. 2007). The EurOtop empirical equation for wave overtopping on shallow foreshores is reproduced as Eq. 1, where R_c^* and q^* are defined in Eq. 2 and Eq. 3. Freeboard, Rc, is measured from the crest of the dike to the still water level.

The overtopping reduction factor (γ) has been set equal to 1.0 as validation analysis is carried out for smooth sloping dikes only. For calculation of the breaker parameter, $\xi_{m-1,0}$ as defined in Eq.4, the respective incident wave height and period, H_{m0} and $T_{m-1,0}$ has been extracted from the physical model using reflection analysis of wave gauge array labeled "toe" in Figure 2. Furthermore, the foreshore slope of 1:35 has been used as the characteristic slope, α , for calculations with Eq. 4.

Eq. 1 was developed from laboratory data by Van Gent (1999) for overtopping of smooth sloping dikes on shallow foreshores. Therefore measured and computed mean wave overtopping discharge only for tests with smooth sloping dikes have been compared against this formula in Figure 4. This figure indicates that the measured mean wave overtopping discharge from the physical model lies within the 95% confidence intervals given by Eq. 1.

$$q^* = 10^{-0.92} . \exp(-R_c^*) \tag{1}$$

$$q^* = \frac{q}{\sqrt{g \cdot H_{m0}^3}} \tag{2}$$

$$R_{c}^{*} = \left(\frac{R_{c}}{\gamma \cdot H_{m0} \cdot (0.33 + 0.022 \cdot \xi_{m-1.0})}\right)$$
(3)

$$\xi_{m-1,0} = \frac{\tan \alpha}{\sqrt{\left(2\pi / g\right) \cdot \left(H_{m0} / T_{m-1,0}^{2}\right)}}$$
(4)



Figure 4. Comparison of measured mean wave overtopping discharge (blue dots) with Van Gent (1999) empirical equation for shallow foreshores (black lines)

Wave Propogation

Figure 5 compares the water surface energy density spectra calculated from a single physical model test setup with a 1:2 dike slope, with no wave return walls on the dike (i.e. the existing Wenduine sea dike scenario). Spectra are plotted for one test case for the 1000 Year ARI storm at wave gauge No. 2, 5, 7, 8, 10 and 12 at prototype scale (refer to Figure 2 for wave gauge location). A similar trend in spectral shape observed in Figure 5 was found for all wave overtopping tests however these results are omitted for brevity. No reflection analysis has been performed on these data, and so spectra include both incident and reflected waves.

The influence of the shallow foreshore is clear from the shape of the energy density spectrum observed in Figure 5. The well-defined offshore peak period has been 'flattened' on the foreshore and energy shifted to low frequency infragravity waves (f < 0.04 Hz prototype scale). A similar trend in the evolution of wave energy spectra on shallow foreshores has been observed in previous physical and numerical model studies (see e.g. Alsina and Caceres 2011; Zijlema et al. 2011; Van Gent 1999).

The propagation of wave height (H_{m0}) , wave period $(T_{m-1,0}, T_p)$ and wave setup are compared in Figure 6 for the same physical model test. All physical model data has been converted to prototype scale in Figure 6. Wave setup has been calculated by subtracting the average of each wave gauge signal from the SWL before wave generation begins. These figures indicate the reduction in H_{m0} due to wave breaking on the shallow foreshore, and the increase in spectral period as energy is shifted to low frequencies. Wave setup in the order of 0.3 m is also observed at the toe of the dike.



Figure 5. Water surface energy density spectra measured for physical model test WEN_004



Figure 6. (a) Significant wave height, H_{m0} (b) wave period, $T_{m-1,0}$ (c) wave setup and (d) wave period, T_p measured for physical model for test WEN_004

Mean Wave Overtopping

Figure 7 and Figure 8 plots the effect of wall location on mean overtopping discharge as a function of wall height. Data is plotted for dikes with a vertical and 1:2 seaward slope, and a +6.92 m TAW foreshore level at the toe of the dike. Data points represent results from an individual physical model test, dashed lines represent an exponential line of best fit through the data. All data points are for the case of vertical walls with parapets, tested under the 1000 year storm wave boundary conditions.

Figure 7 indicates that, for the dike with a vertical seaward slope, the effect of setting back the wall 8 m from the edge of the dike acts to marginally reduce mean overtopping discharge relative to the case where the wall located at the edge of the dike. However, Figure 8 shows that this effect is not observed for the dike with a 1:2 seaward slope, where measured overtopping discharges are approximately equal for walls located at the edge of the dike or setback 8 m.

A more efficient solution at reducing mean overtopping discharge, given the same wall height, is by use of two walls offset to form a stilling wave basin, as proposed originally by Geererts et al. (2006). As illustrated in Figure 7 and Figure 8, this wall configuration shows a great efficiency at reducing mean overtopping discharge. If a mean overtopping discharge tolerance of 1 l/s/m is adopted, a single wall, over 1.8 m in height, located at the edge of the dike would be required to meet this criteria. However if a stilling wave basin is installed, this wall height can be reduced to approximately 1.0 m and 1.2 m, for dikes with a vertical and 1:2 seaward slope respectively

Figure 9 presents still images from three physical model tests, showing the largest overtopping wave impacting the sea wall. The same wave train was used for the three tests, and all tests were conducted with the 1000 year ARI boundary conditions. Figure 9a plots a 1.2 m vertical sea dike setup installed on the sea dike; Figure 9b a 1.2 m wall with a parapet; and Figure 9c a 1.2 m recurve wall. Mean wave overtopping discharge measured from the three tests is indicated below each figure. Figure 9 provides an indication of the effectiveness of a parapet or recurve wall at reducing mean wave overtopping discharge as first established in Van Doorslaer & De Rouck (2010). The physical process whereby the incoming wave is returned offshore is also evident in this figure.



Figure 7. Effect of wall location on overtopping discharge for 1000 Year Toetsing storm conditions for dike with vertical seaward slope. All tests carried out for wave return walls with parapets.



Figure 8. Effect of wall location on overtopping discharge for 1000 Year Toetsing storm conditions for (a) dike with 1:2 seaward slope. All tests carried out for wave return walls with parapets.



Figure 9. Still image of largest overtopping wave impacting sea dike from test of 1000 waves for (a) 1.2 m vertical sea wall (b) 1.2 m sea wall with parapet (c) 1.2 m wall with recurve.

Figure 10 plots the mean wave overtopping discharge as a function of wall height, for wave walls tested under both the +8.0 m Superstorm and 1000 year ARI storm wave boundary conditions for the dike with a vertical seaward slope. As outlined previously, the tolerable mean wave overtopping discharges at Wenduine are q < 1 l/s/m for the 1000 year ARI storm and q < 100 l/s/m for the +8.0m.

From interpolation of Figure 10, the wall crest levels summarized in Table 4 are required to achieve the above tolerable discharge standards. Wall heights, relative to a dike crest level of +8.38m TAW are listed in brackets below wall crest levels. Table 4 also lists the same data available for a sea dike with a 1:2 seaward slope.

This table indicates that in order to meet the tolerable discharge standards with a vertical seaward slope, either a single wall of with a crest level of +10.42 m TAW (i.e. a wall height of 2.05 m) is required, or a stilling wave basin with a crest level of approximately +9.40 m TAW (i.e. a wall height of 1.02 m) is necessary. For a dike with a 1:2 seaward slope, crest levels need to be increased to +9.55 m TAW for the stilling wave basin arrangement.

Note that for both the 1000 year ARI storm and +8.0m TAW Superstorm a foreshore level at the toe of the dike of +6.92 m TAW has been used in the physical model. Model tests have shown that mean wave overtopping discharge is sensitive to this level. It should be noted that the foreshore level of +6.92 m TAW has been used as it represents a conservative value for the eroded beach level post an extreme storm event.



Figure 10. Overtopping discharge for 1000 Year Toetsing and +8.0m Superstorm conditions (for dikes with vertical seaward slope and wave walls with parapets)

Table 4. Wall crest levels required to meet tolerable mean overtopping discharge standards								
Standard of Protection	Storm Event	Wall Crest Level [m TAW]						
		Dike with vertical seaward slope and wall(s) at location:			Dike with 1:2 seaward slope and wall(s) at location:			
		А	В	A & B	А	В	A & B	
q < 1 l/s/m	1000 Year ARI	+10.42 (2.05)	+10.29 (1.91)	+9.40 (1.02)	+10.40 (2.02)	+10.49 (2.11)	+9.55 (1.17)	
q < 100 l/s/m	+8.0 m Superstorm	+9.79 (1.41)	+9.53 (1.15)	+9.29 (0.91)	NA	+9.59 (1.21)	NA	

Note: Wall heights, relative to minimum dike crest level of +8.38 m TAW, are stated in brackets below crest levels

DISCUSSION

Stakeholder acceptance for a single wall constructed on top of the existing dike with a height of approximately 2.0 m would be very difficult to achieve. Therefore this option has not been considered further for design. For the stilling wave basin concept, wall heights in excess of 1.0 m are required, based on data presented in Table 4, to defend the town of Wenduine against the 1000 year ARI and +8.0 m TAW superstorms.

However discussion with stakeholders indicate that a wall level less than approximately 0.7 m is required for aesthetic reasons. Therefore further options are being considered to determine the final wall geometry for a design that integrates engineering safety, architecture/urban-planning, and community considerations.

CONCLUSIONS

Wave overtopping tests have been performed with a 1:25 physical scale model to determine the optimal geometry for design of new wave return walls at Wenduine, Belgium. The key conclusions from physical model results are as follows:

- Wave overtopping on the shallow foreshore at Wenduine was found to be dominated by lowfrequency infragravity waves (f < 0.04 Hz at prototype scale).
- Mean wave overtopping discharge measured with the physical model compared well with the van Gent (1999) empirical overtopping equations for shallow foreshores.
- Physical model tests confirmed that the stilling wave basin concept proposed by Geerearts et al. (2006) and wave wall parapet concepts of van Doorslaer & De Rouck (2010) were effective at reducing the wave wall height required to meet the tolerable discharge overtopping standards.
- Further investigations are required to determine the geometry of a wave return wall that meets tolerable wave overtopping discharge criteria as well as stakeholder requirements.

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