IAHR CONGRESS PROCEEDINGS

4th International Short Conference/Course on APPLIED COASTAL RESEARCH

Honouring Professor Hans F. Burcharth

15th – 17th June, 2009
LIM - Universitat Politècnica de Catalunya (UPC)
Spain

PROCEEDINGS

Edited by

Agustin Sanchez Arcilla
and
Giuseppe Roberto Tomasicchio

Università del Salento

CIIRC International Centre for Coastal Resources Research

IAHR AIRH

International Association of Hydro-Environment Engineering & Research

4th International Short Conference/Course on *APPLIED COASTAL RESEARCH*

Honouring Professor Hans F. Burcharth

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SPONSORED BY
The International Association of Hydraulic Engineering & Research

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prof. Giuseppe Roberto Tomasicchio

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4th International Short Conference/Course on
APPLIED COASTAL RESEARCH

***Honouring Professor Hans F. Burcharth*, University of Aalborg, Denmark**

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FOREWORD

The International Short Conference and course on Applied Coastal Research (SCACR) is a technical conference series focusing on research issues related to harbour and coastal engineering and dynamics and bringing together students and top level researchers and practitioners. It is, therefore, of interest for PhD students, field and laboratory experimentalists, theoreticians and modellers, all of them with an interest on the physical processes driving water and sediment fluxes in the coastal zone and their interactions with the structures and ecosystems.

The 4th SCACR was hosted by LIM/UPC in Barcelona in June 2009. It covered state of the art formulations, numerical simulations (both classical and using advanced techniques such us SPH), physical experiments in the laboratory (both large and small scales) and field campaigns, etc.

We were fortunate to have invited speakers such us Prof. Marcel Stive, Dr. Jeff A. Melby and Professor Holger Schuettrumpf, together with a large group of invited specialists and engineers.

The conference was dedicated to honour the contributions of Prof. Hans. F. Burcharth from Aalborg University (DK) in the field of Coastal and Harbour Engineering, acknowledging his contributions to teaching, research and engineering applications in this field.

The conference had a participation of 95 people from Europe and overseas. Although every effort was made to keep the registration fee low, some researchers from 3rd world countries showed their interest but had difficulties in joining the conference for a variety of reasons (visas, travel costs, etc.).

The conference had the support of IAHR, Universitá del Salento, ETSE de Camins, Canals i Ports de Barcelona, UPC and CIIRC.

We want also to acknowledge the various sponsors and technical exhibitors. Due mention should also be made of the enthusiastic support of LIM and CIIRC staff who made possible and successful this event.

We want to thank as well the Sessions Chairmen and other support staff.

This successful 4th SCACR prepares the way for organizing future conference editions which will bring together in such a symbiotic manner young and experienced researchers and engineers, interested in coastal and harbour problems.

The 5th edition will be held in Germany at Lehrstuhl und Institut fuer Wasserbau und Wasserwirtschaft, RWTH, Aachen in June 2011.

Then, see You in Germany in June 2011.

Agustín Sánchez-Arcilla and Roberto Tomasicchio

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PROGRAMME

Monday 15th June 2009

15:30 – 16:00	Registration
16:00 – 16:15	**Opening and Welcome address** Agustín Sánchez Arcilla, UPC Roberto Tomasicchio, University of Salento
16:15 – 17:30	**Lecture 1:** "Changing Costal Environment" by Marcel Stive, T.U.Delft, The Netherlands
17:30 – 18:00	Coffee Break
18:00 – 20:00	**Technical Session 1** Chairman: Jose A. Jiménez (UPC)

Tuesday 16th June 2009

9:30 – 10:45	**Lecture 2A:** "Design of costal structures" by Jeff A. Melby, USACE, USA
10:45 – 11:15	Coffee Break
11:15 – 12:45	**Lecture 2B:** "Design of coastal structures" by Jeff A. Melby, USACE, USA
13:00 – 14:30	Lunch Break
4:30 – 16:00	**Lectio Magistralis** "Breakwater engineering" by professor Hans F. Burcharth, Aalborg University, Denmark
16:00 – 16:30	Coffee Break
16:30 – 18:30	Invited speakers for Specialist Session
16:30- 6:50	Extreme wave events (Agustín Sánchez-Arcilla - UPC)
16:50-17:10	Deterministic formulations. Transmision, reflection and overtopping (Xavier Gironella - UPC)
17:10-17:30	Probabilistic breakwater design (César Vidal – Univ. Cantabria)
17:30-17:50	Instrumented breakwaters. The Barcelona harbour case (Javier Uzcanga – Puerto Barcelona)
17:50-18:10	The design and construction of large breakwater. The Coruña harbour case (Enrique Maciñeira – Puerto La Coruña)
20:45	Social Dinner

Wednesday 17th June 2009

9:30 – 10:45	**Lecture 3A:** "Overtopping modelling" Holger Schuettrumpf, Institute of Hydraulic Engineering and Water Resources of RWTH, Aachen, Germany
10:45 – 11:15	Coffee Break
11:15 – 12:15	**Lecture 3B:** "Overtopping modelling" Holger Schuettrumpf
13:00 - 14:30	Lunch Break
14:45 – 16:45	**Technical Session 2** Chairman: Eugenio Pugliese Carratelli, University of Salerno, Italy
17:00	Discussion and Closing

Dean, R. J., Chen, R. And Browder, A. E., 1997. 'Full Scale Monitoring Study of a Submerged Breakwater, Palm Beach, Florida, USA'. Coastal Engineering, vol. 29, pp. 291-315.

Tanaka, N., 1976. 'Effects of Submerged Rubble-mound Breakwater on Wave Attenuation and Shoreline stabilization' (in Japanese). Proc. of 23rd Japanese Coastal Engineering Conference. pp. 152-157.

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REDUCTION OF WAVE OVERTOPPING: FROM RESEARCH TO PRACTICE

Koen Van Doorslaer[1], Julien De Rouck[1] and Stefaan Gysens[2]

Abstract: The present paper shows the purpose, the test results and the practical implementation of two innovative crest designs, based on scale model test with mainly non-breaking waves. First, the parapet and its geometrical parameters are shown. Second, the reduction of a dike with stilling wave basin is presented. Both crest designs will be built along the Belgian coast in order to fulfill current and anticipate future safety norms. Alternatives and/or performance are discussed within the present paper.

INTRODUCTION

To ensure the safety in Belgian coastal regions now and in the future (up to 2050, including the sea level rise) a Coastal Safety Plan[3] is being worked out, in which is declared that cities near the coastline have to be protected against storms with a return period of 1000 year (which is a minimal acceptable protection level). If this storm would appear now, theoretical and physical models have shown that the safety of one third of the Belgian coastal zone is insufficient. Such a storm may create breaches causing flooding of the hinterland with an economical damage of billions of Euros and the risk of life of at least 4000 people.

A beach nourishment is an option to increase the safety against these severe storms, but is not always possible close to harbors, may be very expensive since constant monitoring and restoration will be necessary during its lifetime, and are also limited to a certain level. Another safety measurement could be the heightening of the dikes, but this is often restricted because of its inconveniences; the visual attraction of the open sea can not be lost, apartments and promenades near the coastline have to remain untouched, ... A sound practical solution is needed within the spatial restrictions and dealing with the architectural desire of a touristic area.

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[3] http://zeeweringenkustbeheer.afdelingkust.be/

Fig. 1. Overtopping in Oostende, Belgium

Therefore innovative crest designs, with a significant reduction of wave overtopping but without heightening the crest level, are studied at Ghent University. Two of those designs and their practical use are discussed in this paper: the Parapet and the Stilling Wave Basin (SWB). Model tests have been carried out and prototypes will be built along the dikes of some coastal cities such as Oostende, Wenduine, Middelkerke, Nieuwpoort. This paper shows how research can lead to sound practical solutions.

EXPERIMENTEL SET-UP

Test Facility and Wave Generation

Model tests on wave overtopping were performed in the wave flume (L x W x H = 30.0m x 1.0m x 1.2m) of the Department of Civil Engineering at Ghent University. In this flume waves are generated using a piston type paddle. The wave paddle steering features active wave absorption based on Frigaard and Brorsen (1995) as mentioned in Troch and De Rouck (1999). Waves were generated using the standard Pierson-Moskowitz spectra. The length of the tested time series was chosen to represent approximately 1000 waves in order to obtain reliable average overtopping discharges.

Measurements

Waves are measured using resistance type wave gauges. Incident wave conditions are determined using the method by Mansard and Funke (1981). Wave overtopping amounts were determined by weighing overtopping volumes.

THEORETICAL FORMULAS

Results are presented using the van der Meer approach as presented in TAW (2002). In this formulas, a distinction between breaking and non breaking waves is made using the Irribaren number ξ_0. The formula for breaking waves is

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0{,}067}{\sqrt{\tan\alpha}} \cdot \gamma_b \cdot \xi_0 \cdot \exp\left(-4{,}75 \cdot \frac{R_C}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \qquad (1)$$

with a maximum for non-breaking waves

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0{,}2 \cdot \exp\left(-2{,}6 \cdot \frac{R_C}{H_{m0}} \cdot \frac{1}{\gamma_f \cdot \gamma_\beta}\right) \qquad (2)$$

where q = overtopping discharge, g = gravity acceleration, H_{m0} = the wave height at the toe of the structure, α = the average slope of the structure, $\xi_0 = \tan\alpha / \sqrt{s_0}$ = wave breaker parameter, $s_0 = H_{m0}/L_0$ = wave steepness, $L_0 = g \cdot T_{m-1,0}^2 / (2\pi)$ = deep water wave length, $T_{m-1,0}$ = spectral wave period at the toe of the structure, γ = reduction factor (indices: b = berm, f = roughness, β = wave angle, v = vertical wall). Remark that □ stands for the average slope, meaning that any vertical wall or other structure on the slope is included in the definition of the average slope as presented in TAW (2002).

Both formulas represent the dimensionless overtopping rate on the left side as to be an exponential function of the dimensionless freeboard. In this research a smooth dike without berm ($\gamma_f = 1$, $\gamma_b = 1$) is constructed in a 2D wave flume ($\gamma_\beta = 1$). The goal is to find reduction factors $\gamma_{parapet}$ and γ_{SWB}. This will be discussed in the next chapters. Mainly non-breaking waves have been used, and only some breaking waves for a first comparison between both. But the focus of this research topic was to find the reduction factor s in the non-breaking formula (2).

DIKE WITH PARAPET

Basic Geometry

A parapet, or wave return wall, can be built on the slope of a dike. A simple return wall can be built (left on Fig. 2), but it is advisable to fill up the space and create a wider dike as shown right on Fig. 2. One of its main characteristics is the crest level, which remains as high (low) as the initial dike.

Fig. 2. Possible lay-outs of a parapet

Nevertheless, a major reduction of overtopping is expected. This is realized by projecting the incoming waves back towards the open sea, as shown in Fig. 3.

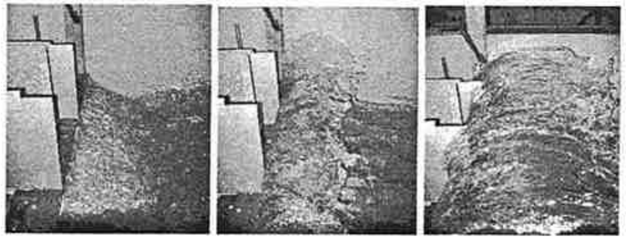

Fig. 3. Seaward projection of overtopping waves due to a parapet

During this research next geometrical variables haven been changed: the angle β, the total height of the parapet and the ratio of the heights $\lambda = l_k/l_v$.

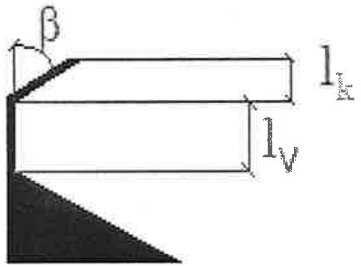

Fig. 4. Variable parameters β and λ

Test conditions

Table 1. Test Conditions For The Dike With Parapet, Scale Model Values

Wave Parameters		
Significant wave height at toe of structure	[m]	0.07 - 0.14
(H_{m0})	[s]	1.2 - 2.6
Peak wave period at toe of structure (T_P)	[-]	0.013 - 0.043
wave steepness (s_0)		
Structural parameters		
water depth at the toe of the structure (d)	[m]	0.44 - 0.57
Freeboard (R_C)	[m]	0.18 - 0.05
Dimensionless freeboard (R_C/H_{m0})	[-]	2.57 - 0.64
Slope of the structure (α)	[-]	1/2
Parapet parameters		
Angle (β)	[-]	0°; 15° ; 30° ; 45° ;
Inclined height (l_k)	[m]	60°
Total height of the parapet (l)	[m]	0.01; 0.02; 0.03
		0.02; 0.05; 0.08

It should be noticed that wave height is rather small compared to the water depth. This to avoid depth limited wave breaking in front of the structure.

Results

Test results have shown that the parapet has the biggest reduction in overtopping for $\beta \geq 45°$. The profit by increasing this angle beyond 45° is rather small (Fig. 5). A similar analysis is done for the height ratio λ. It is found that the reduction on wave overtopping is maximal for $\lambda \geq 0.33$ (Fig. 6). These conclusions are only valid for the tested parapet heights of 5cm and 8cm, where $h_{parapet}/H_{m0} > 0.3$. In case $h_{parapet}/H_{m0}$ is smaller than 0.3, more tests are needed to draw a conclusion.

Fig. 5 and Fig. 6 are the graphs of the reduction factor as a function of its geometrical variable, plotted for the parapet of 5cm high. The 8cm high parapet shows similar results.

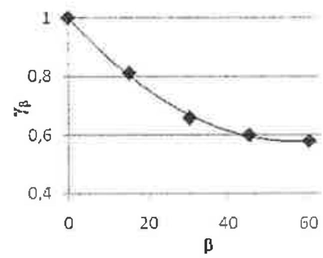

Fig. 5. Influence of β on the reduction of overtopping (graph: parapet of 5cm height)

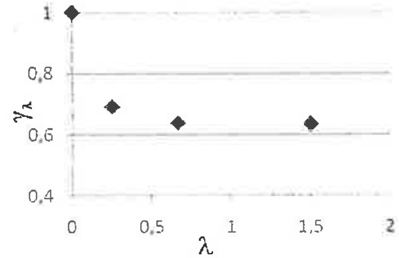

Fig. 6. Influence of λ on the reduction of overtopping (graph: parapet of 5cm height)

The reduction in wave overtopping over a dike with a vertical wall with a certain height h_{wall} is referred to the overtopping on a dike without vertical wall (but the same freeboard). This reference case is then defined as a wall with h_{wall} = 0cm. The reduction factor γ_v to be added to (2) is 0.93 in case $h_{wall}/H_{m0} < 0.6$ and 0.9 otherwise.

Similar, to compare the wave overtopping over a dike with parapet, one should refer to a dike with a vertical wall (with the same freeboard). This reference case is defined as a parapet with $\beta = \lambda = 0$. The reduction factor $\gamma_{parapet}$ to be added to (2) has a minimal value of 0.557 (in case $\beta \geq 45°$ and $\lambda \geq 0.33$).

The total reduction factor of a dike with parapet compared to a simple dike, then becomes the minimal value of 0.5.

Practical use

In Nieuwpoort, Belgium, the current borderline between the walking promenade and the beach is a 1/2 smooth dike. In stormy weather, the sea level will rise until it reaches the smooth dike. On Fig. 7 it is to be seen that an open railing is built as a fence between promenade and smooth dike. This does not prevent the water to flood over the promenade into the city. This open railing (height 0.5m above the promenade) can be changed into a parapet with β = 45° and λ = 1/3 as shown on Fig. 8.

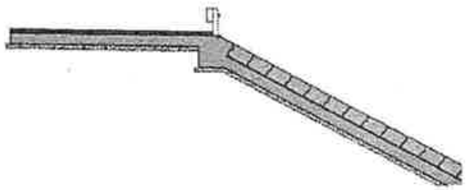

Fig. 7. Current dike with open railing

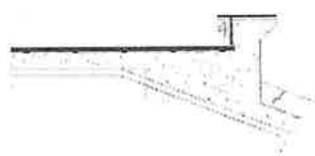

Fig. 8. Solution with parapet

In Table 2, the theoretical reduction factors are calculated for a storm with increasing return period. The promenade in Fig. 7 and Fig. 8 is situated at level +7.82m TAW, while the sea level is located between +6mTAW and +7.3mTAW.

Table 2. Reduction factors of a parapet in storm conditions

Return period (year)	sea level rise	water level (m TAW)	R_C/H_{m0} (-)	$H_{s,design}$ (m)	q (open railing) (l/s.m)	q (parapet) (l/s.m)	reduction factor (-)
1000	yes	7.3	0.51	2	877	129	6.8
	no	7	0.66	2	584	60	9.8
100	yes	6.8	0.84	1.8	326	20	16.5
	no	6.5	1.01	1.8	208	8	24.7
10	yes	6.3	1.26	1.6	97	1.6	62
	no	6	1.45	1.6	58	0.6	101

A major reduction, up to 100 times the overtopping discharge, can be obtained by a little adaptation in the existing construction in Nieuwpoort. According to the Belgian Coastal Safety Plan, the overtopping discharge should be reduced to an average of 1 l/s.m. A storm wall can be built behind the parapet, as shown in Fig. 9. Due to the parapet, this storm wall should only be setup once every10 or more years, but it is advisable to test the storm procedure and placements of the wall once a year during a lower storm flood warning, to minimize the risk of failure and as a training.

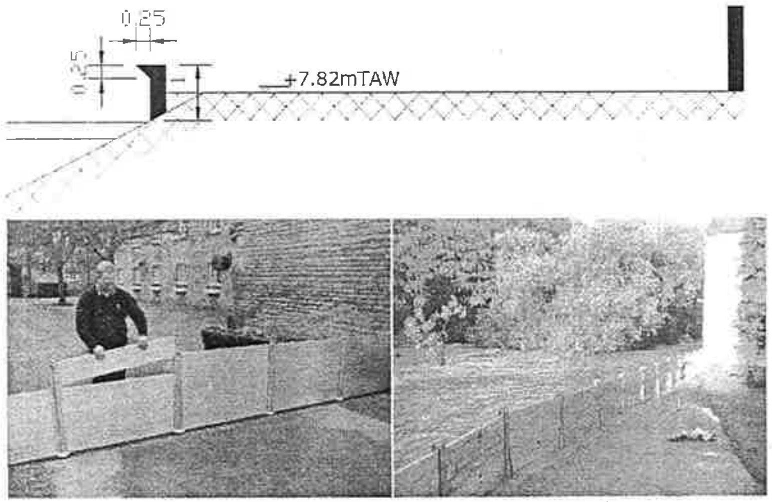

Fig. 9. Installation of a storm wall in case q > 1l/s.m

DIKE WITH STILLING WAVE BASIN

Basic Geometry

As presented on ICCE 2006 in San Diego by Geeraerts, De Rouck et al. (2006), a stilling wave basin is made up of a vertical seaward wall, a basin and a 2nd landward wall. Waves hitting the seaward wall are projected upward and "drop dead" in the basin and lose their energy. Since the seaward wall is actually a double row of shifted walls, the evacuation of the overtopped water is possible. In case of large overtopping, the water runs back and forth the basin between the seaward and the landward wall, meanwhile losing its energy. The remaining energy is insufficient to overtop the landward wall. An important parameter in this SWB is the blocking coefficient, which is the ratio between the open and the closed part of both rows of shifted walls. The optimum between inflow (as low as possible) and outflow (as high as possible) needs to be found.

Similar to the parapet, the crest freeboard does not become higher.

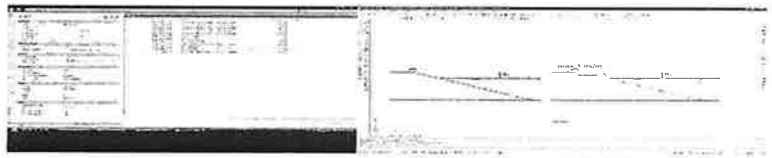

Fig. 10. Principle sketch of a SWB (right) versus the normal dike (left)

Depending on the existing situation, the promenade can be seen as the basin, but a stilling wave basin can also be built in front of the existing promenade. In this last case, the dike is extended towards the sea.

Test Conditions

Table 3. Test conditions for the dike with SWB

Wave Parameters		
Significant wave height at toe of structure (H_{m0})	[m]	0.08 - 0.18
Peak wave period at toe of structure (T_P)	[s]	1.28 - 2.56
wave steepness (s_0)	[-]	0.012 - 0.054
Structural parameters		
water depth at the toe of the structure (d)	[m]	0.30 - 0.52
Freeboard (R_C)	[m]	0.10 - 0.27
Dimensionless freeboard (R_C/H_{m0})	[-]	0.555 - 3.375
Slope of the structure (α)	[-]	1/2; 1/2.5; 1/3; 1/4; 1/6
SWB parameters		
height	[m]	0.096
length	[m]	0.480
blocking coefficient	[-]	0.5 (row 1) - 0.62 (row 2)

In the in 2006 presented SWB Geeraerts, De Rouck et al. (2006), only 1 basic geometry is tested with the above mentioned parameters. In the present paper, we discuss both the brief results of that research and the implementation of a SWB in Oostende based on an engineering, architectural and urban design process.

Results

The overtopping is measured on top of the seaward wall, as shown in Fig. 10. Compared to the reduction of a vertical wall $\gamma_v = 0.93$ or 0.9, as mentioned in the results section of the dike with parapet, a higher reduction with SWB can be expected due to the horizontal distance between the seaward vertical wall and the landward wall where the overtopping is measured. It is found that for non-breaking waves a total reduction factor γ_{SWB} of 0.48 should be included in formula (2). This reduction factor is deducted by dividing 1.97 by 4.08, the exponential coefficients in Fig. 11..

The actual reduction of overtopping discharge also depends on the value of the dimensionless crest freeboard. For $R_C/H_{m0} = 1$, a reduction factor of 8 is obtained. If R_C/H_{m0} increases to 1.5, a dike with SWB is overtopped by 24 times less water compared to a simple dike. Those reduction factors are only valid for the in Table 3 proposed geometry.

The research has also proven that increasing the height of the seaward wall is more effective than increasing the length of the energy dissipating basin.

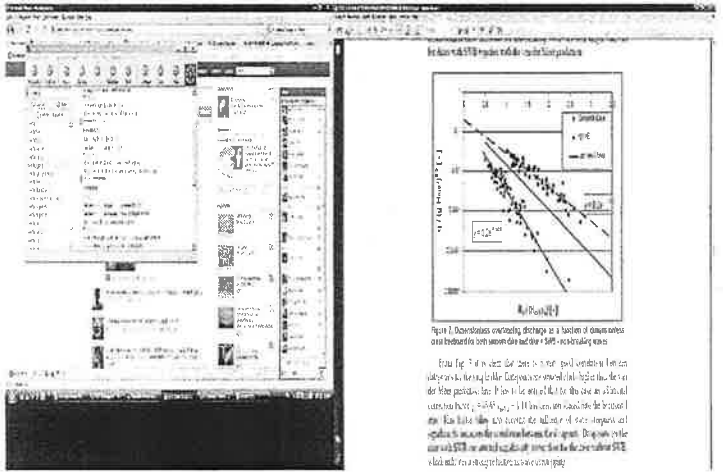

Fig. 11. Dimensionless overtopping discharge as a function of dimensionless crest freeboard for both smooth dike and dike + SWB (non-breaking waves)

Practical use

In 2007, the works started to ameliorate the access to the Oostende harbor and increase the safety towards flooding of the city centre of Oostende, Belgium. The city centre is at about +4.50mTAW (= mean high water level), i.e. about 2.5m lower than the design water level of the 1000 year storm.

Computer simulations have shown that a critical point of the shore protection is the promenade and the area between the planned western breakwater and the existing western jetty (which is a protected monument) This area is directly subjected to waves penetrating through the harbor entrance. The beach profile is rather low, since it has to stay 1m below the underwater dam on the left of the jetty to prevent sediment flow of this sand into the access channel. The construction of a SWB dike at this location perfectly fits into the works at the harbor of Oostende.

Two of the studied variants are to be discussed here: Fig. 12 shows the variant where the SWB follows the historical dike contour. In Fig. 13, the area is extended towards the sea. In this latter variant, the beach in front of the dike is smaller and thus the slope of the beach is steeper, so less wave breaking will occur. The main advantage on the other hand is the creation of a bigger area, which can be used for popular events in this touristic coastal city.

The critical section of both variants (blue line on Fig. 12 and Fig. 13) have been tested in the wave flume of Flanders Hydraulics Laboratory, and the main results of their behavior is given in Table 4.

If, by application of a stilling wave basin, the overtopping over the landward wall is still above 1l/s.m, a removable storm wall needs to be set up. In the experimental testing, the height of this wall and timing the set up has been tested.

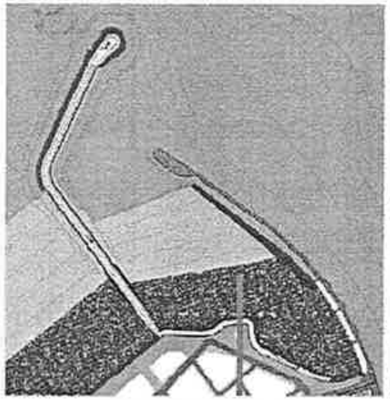

Fig. 12. Variant 1: existing coastline + SWB

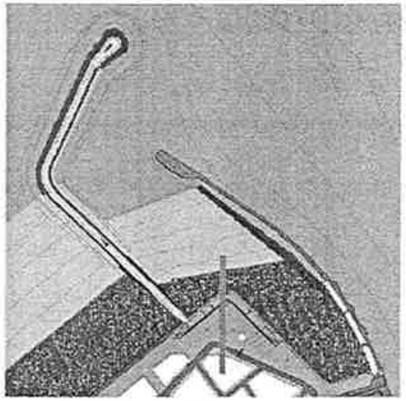

Fig. 13. Variant 2: extended to a square + SWB

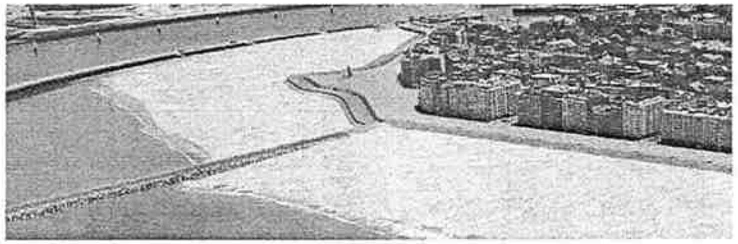

Fig. 14. Artist impression of variant 1: SWB around the existing dike

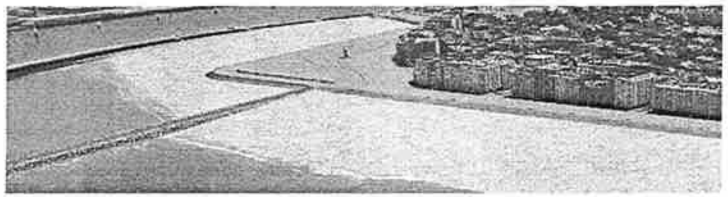

Fig. 15. Artist impression of variant 2: extension of the dike to a square with SWB in front

Table 4. Test results of the SWB in the Oostende project

Variant	waveheight at the dike	overtopping discharge when SWL = +7mTAW	height of storm wall	waterlevel when stormwall needs to be set up	occurrence frequency
1	1.75m	1l/m/s	none	/	/
2	2.75m	45l/m/s	1m	+5.90mTAW	9y

In the environmental impact assessment (EIA), a 3rd and final variant has been proposed with a less seaward extended square. This variant has an overtopping of 1l/m/s (5l/m/s with the expected 30cm sea level rise) and is more flexible to anticipate a future increase of safety level.

CONCLUSIONS

In this paper 2 innovative crest designs with constant crest freeboard have been discussed: the parapet and a stilling wave basin.

It has been shown that a parapet is a very efficient way to reduce wave overtopping, which does not take much place and is easy to construct. A reduction factor $\gamma_{parapet}$ of 0.5 can be added to the formula for non-breaking waves of van der Meer. This maximum values can be obtained by a parapet with an angle β of 45° and the height ratio of the inclined part λ of 0.33. A parapet with this geometrical conditions will be built along the coast in Nieuwpoort, Belgium.

The principle of the 2nd crest design, dike with SWB, has already been proposed at ICCE 2006. A reduction factor γ_{SWB} of 0.48 has been extracted from the experimental testing for one specific layout. Two possible designs for the use of a SWB at Oostende and its overtopping reduction have been presented in this paper.

In the near future, further research will be done to formulate $\gamma_{parapet}$ and γ_{SWB} as a function of its dimensionless parameters.

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CONSEQUENCES OF A DETACHED BREAKWATER ON THE MORPHOLOGICAL RESPONSE TO STORMS OF LA BARCELONETA BEACH (BARCELONA, SPAIN).

Amanda Sancho, Jorge Guillén and Elena Ojeda[1]

Abstract: This study analyzes the morphodynamic behaviour under storm conditions of La Barceloneta, an artificial embayed beach, previous and after the construction of a detached breakwater using video image measurement. In general, the decreases in the emerged beach area due to storm events were higher after the construction of the detached breakwater. So the detached breakwater built in La Barceloneta beach do not seem to be completely effective in preventing beach erosion during the most energetic storms events.

INTRODUCTION

Many coasts are in erosion as a consequence of a variety of causes (e.g., wave action, maritime constructions interrupting sediment transport, decrease of river inputs, development of coastal areas and massive housing constructions, etc.). Nowadays the most common approach to mitigate beach erosion is the protection of the beach using hard engineering solution (e.g., seawalls, detached breakwaters, groins) soft engineering techniques (beach nourishment), or some combination between them. For instance the detached breakwater in conjunction with beach nourishment can provide an attractive engineering solution to solve erosion problems (Ilic et *al*, 2005).

The addition of sand to the emerged beach is used to protect the beach or the region behind it, and also to attain a certain beach width for recreational uses (Hanson et *al*, 2002).

On the other hand, detached or shore-parallel breakwaters are coast-parallel structures built at a certain distance from the shore and totally unconnected to it, protecting a certain shoreline area from wave action (Bricio et *al*., 2008). Their principal role is the protection of the coast from flooding or erosion although they have also been used to create artificial beaches (Ilic et *al*., 2005). However, the results obtained after building a detached breakwater have not always been as desired, because it depends on the effect that the detached breakwater has on littoral sediment transport (Bricio et *al*., 2008).

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