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Coastlab12

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Editors:

- Peter Troch
- Vasiliki Stratigaki
- Sieglien De Roo

UNIVERSITEIT GENT

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Peter Troch, Vasiliki Stratigaki and Sieglien De Roo

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DESIGN OF STORM RETURN WALLS FOR THE MASTERPLAN FOR COASTAL SAFETY: FROM CONCEPTUAL TO DETAILED DESIGN

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Abstract

This paper reflects on the design of measures for the Integrated Master Plan to protect the Flemish coastline against erosion and flooding on a short and long term basis, looking ahead at the year 2050. Different measures and alternatives to prevent present and future flooding are being worked out on the basis of safety checks and flood risk calculations along the entire Flemish coastline. This paper describes how initially the available knowledge about the effect of measures is used and how the physical experiments are used to refine this knowledge for detailed design.

1. Introduction

Although the Flemish coastline is merely 67 kilometers long, every meter is optimally used. Many stakeholders have specific interests in this varied area: housing, tourism, harbour activities, nature reserves, ... To balance the needs of all these interest groups, mutual cooperation is needed to guarantee their future. The Integrated Master Plan for coastal safety started in 2007 and brought all these factors together. This plan was approved in June 2011 by the Flemish Government and forms the basis for the development of the seafront along the Flemish coast in the nearby and distant future (looking at 2050) with safety against flooding as a primary aim. A combination of soft and hard coastal protection techniques will be used. To detail the design of these hard measures architectural, numerical and physical modelling is being executed. This abstract outlines the process of the conceptual and the detailed design.

2. Masterplan Coastal Safety

The Belgian coast is situated at the southern part of the North Sea. The coastline is 67 km long consisting mostly of sandy beaches with sea walls in front of the cities and dunes in between. There are 4 harbours at Nieuwpoort, Oostende, Blankenberge and Zeebrugge and the Zwin (tidal inlet) (Fig. 1). In the flood prone area live about 400.000 people.

Although Belgium has a small coast, every kilometer is intensively used. Residential neighbourhoods, ports, industries and important nature reserves are present. The pressure from tourism and recreation is immense. To balance the needs of all these interests, at present and in the future, an integrated approach is necessary. Nonetheless, special attention has to be given to coastal safety. Due to climate changes (e.g. sea level rise) and continuing development of the coastal zone, protection against coastal erosion and flooding will become increasingly difficult and costly to guarantee. To counter this problem, good spatial planning, cooperation between different governmental organisations and collaboration with neighbouring countries will be essential.

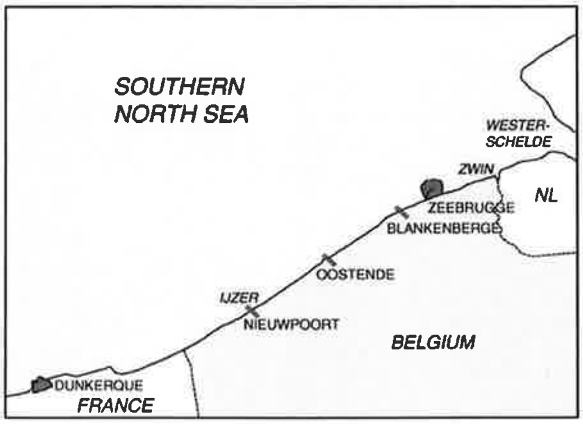

Figure 1. The Belgian coast is 67 km long and has 4 harbours at Nieuwpoort, Oostende, Blankenberge and Zeebrugge. At the borders two important nature reserves (Westhoek and Zwin) are located.

A lot of coastal communities however did not achieve the (intermediate) safety standard to be protected for the 1 in 1000 year storm. There was a need for long-term planning. Hence, for the first time, the Coastal Division of the Flemish region started up a study to work out an 'integrated master plan for Flanders future coastal safety'. The aim of this study was to protect the Flemish coast against erosion and flooding on a short and long term basis, looking ahead at the year 2050, based on the principles of Integrated Coastal Zone Management (ICZM). Therefore the time aspects of investments, sea level rise, beach erosion, ... are also taken into account. This integrated master plan was approved in June 2011 and defines the measures needed to develop and guarantee a safe coastline.

The different topics of the study are summarised in Fig. 2.

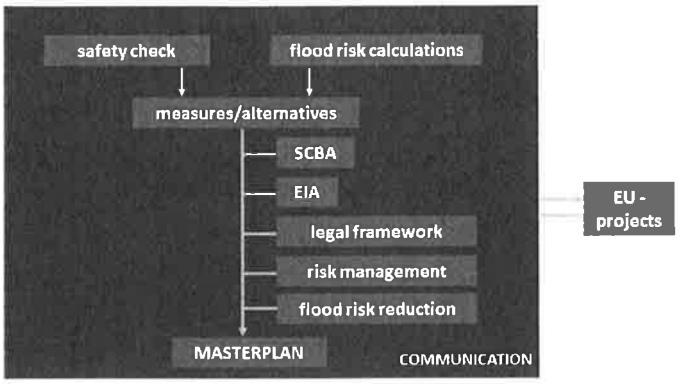

Figure 2. Different topics of the Integrated Master Plan.

3. Conceptual design

Both for the harbours and the coastal towns a social cost benefit analysis was executed. In this analysis different measures were compared. For the coastal towns (where overtopping is the main problem) the measures consist of a beach nourishment in combination with possible hard measures at the location of the current sea wall. Possible hard measures are storm return walls and stilling wave basins. The larger the hard measure, the less sand has to be nourished (or the higher the protection level). For the harbours (where also overflow can be a problem in the initial situation) measures consist of storm return walls all around the quays in the harbour, to prevent both overtopping and overflow, and a possibility to close the harbour with a storm surge barrier.

For all measures the cost was estimated based on experience of comparable projects and/or on a rough preliminary design of the structures. With flood risk calculations the (remaining) potential damage was evaluated. The higher the protection level of the measure the less damage.

All costs and benefits were compared, taking into account the social consequences. By doing so, a ranking of the different types of measures was obtained and the optimal height and /dimension of the measures was determined.

For this exercise no detailed calculations of wave forces and stability of the foundation were necessary. The cost benefit analysis only gives a ranking. If two measures are of comparable impact and cost, other considerations have to be taken into account for the decision, such as the social impacts. Therefore the outcome of the cost benefit analysis has to be robust for changes in costs or avoided damage, including the high uncertainty on extreme water levels, wave heights, ... It was not be realistic (and necessary) to investigate all possible measures with physical modelling or complicated modelling.

4. Used methodology for the conceptual design

4.1 Harbours

The wave penetration in harbours is calculated with the wave energy model SWAN. SWAN is not suitable to calculate diffraction, but for most locations the wave height is determined by local wind growth and/or direct wave penetration. For the detailed design a combination of a (semi-) time domain numerical wave model (e.g. Boussinesq modeling) (for the wave penetration) and a spectral model (SWAN, for the local wind growth) is used.

The possible flood protection measures in harbours consists of quay walls with a storm return wall at some distance from the quay wall, heightening of quays, storm surge barriers, ... To calculate overtopping over quay walls with a storm return wall, the method of Den Heijer is used (Den Heijer, 1998). However, it should be noted that in the physical modeling on which the method is based, only 1 test is done for a case with a water level lower than the quay level (h_B<0, cf. Figure 3.

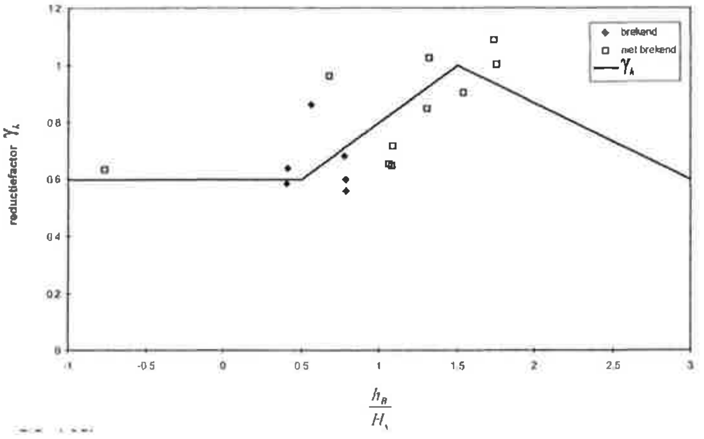

Figure 3 Reduction coefficient dependency on the relative water level

Also the obliqueness of the waves (for quays) is taken into account, using the formulaes of (Eurotop Manual) :

$$\gamma_\beta = 1 - 0.0062\beta \ voor \ 0° < \beta < 45°$$
$$\gamma_\beta = 0.72 \ voor \ \beta \geq 45°$$

It should be noted that in this approach, the reduction for wave directions of 80°(what is a common case in harbor (entrances)) is equal to 45° wave attack. This makes physical modeling necessary for the detailed design (optimisations). These experiments are scheduled for the near future.

4.2 Coastal towns

For coastal towns first the erosion of the beach during a storm is calculated with Durosta (Steetzel, 1993). The eroded beach profile is used as input for the wave modeling (SWAN) in order to calculate the wave parameters at the toe of the dike. In the detailed design SWASH is used for this calculation (Suzuki et al, 2012).

If a storm return wall at the dike crest is examined, again Den Heijer is used (outside the range of valid configurations).

5. Physical experiments for optimalisations

5.1 Wave overtopping on quay walls with a storm return wall

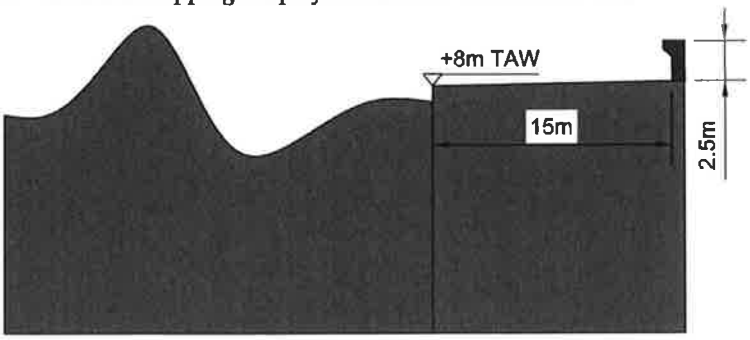

Figure 4 Model set-up

At UGent physical experiments are performed for different (relative) water levels, wave heights, crest widths and wall heights (set up as shown in Figure 4).
In Figure 5 the available data are clustered based on the ratio between the berm width (Lb) and the wall height (hm): Black: very small berm length Lb (LB/hm around 2) ; Red: broad berm (Lb/hm around 10) ; Orange (Lb/hm around 5) ; Blue (Lb/hm >>10) ;Green : no wall
The dimensionless overtopping (Q*) and freeboard (R*) are calculated as:

$$R_n = \frac{h_c}{H_s} \frac{1}{\gamma_{totaal}}$$

$$Q_n = \frac{q}{\sqrt{g H_s^3}}.$$

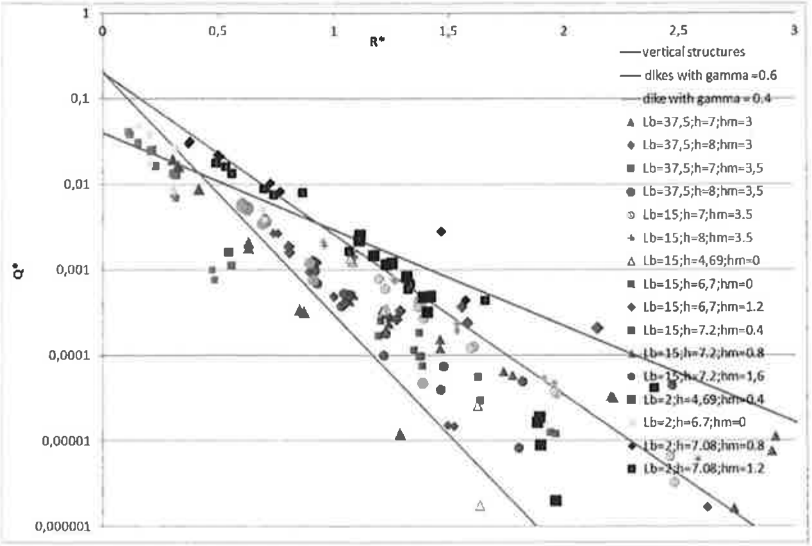

Figure 5 Dimensionless overtopping (Q*) related to the dimensionless freeboard (R*) for different quay layouts

Within the figure, the factor γ is left out to see where all data are located in the graph related to the each other. The points are clearly clustered. For each cluster the corresponding gamma is determined to fit the relation:

$$Q_n = 0,2 \exp\left(-2,6\ R_n \right)$$

This gives the dependency as shown in Figure 6:

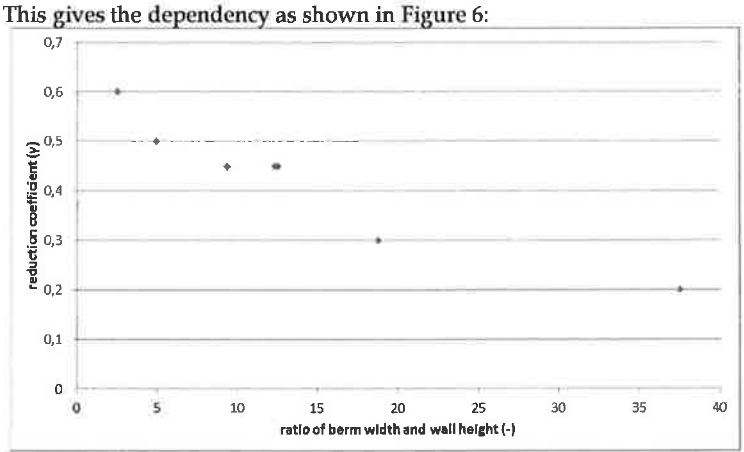

Figure 6 Reduction coefficient γ in relation to the ratio of berm width and wall height

Parapet walls result in a further reduction. For the used shape of parapet, a γ of 0.8 can be used (cf. Figure 7).

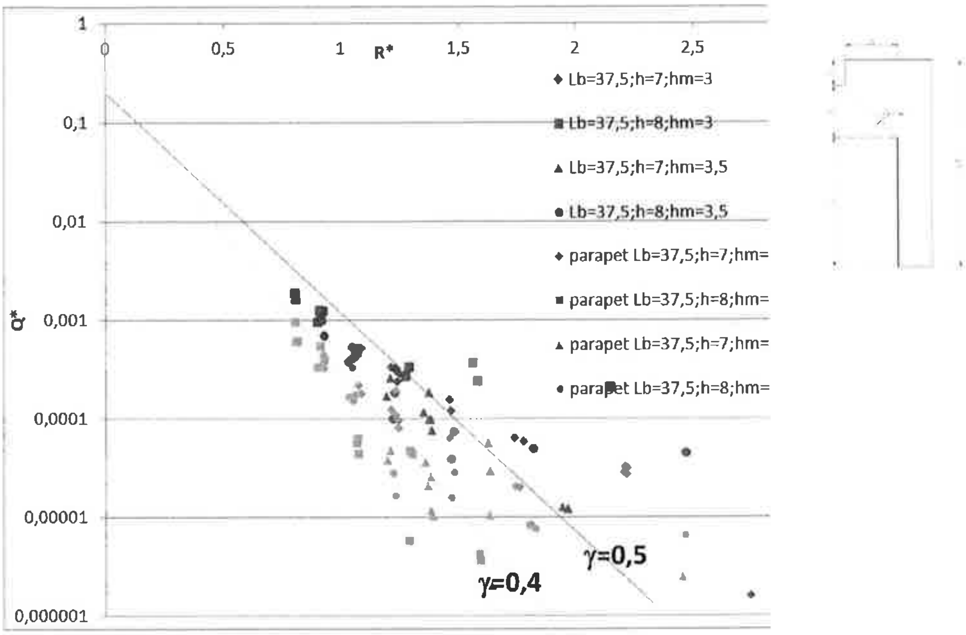

Figure 7 Comparison of overtopping discharges between walls with and without parapet.

5.2 Wave forces

For the determination of wave forces Den Heijer and Goda are used for the conceptual design. For oblique waves a reduction of the dynamic forces is applied (multiplying by $\cos^2(\beta)$) However, these formulae are used out of the range for which they were intented.

The idea is to find a relation between layer thickness, velocity and wave force, since already data exist about the relation between wave parameters and layer characteristics (cf. Figure 8) (e.g. Van der Meer et al, 2010).

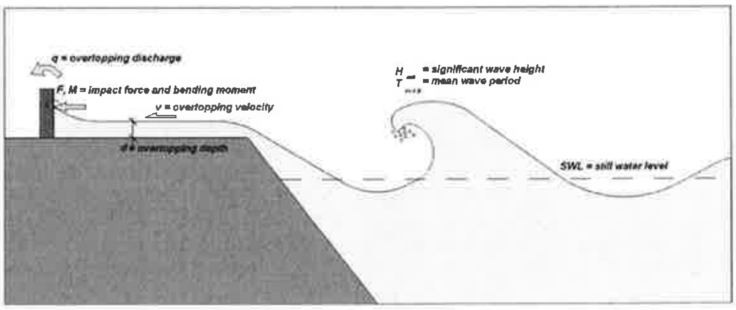

Figure 8 Definition of parameters to find relations

5.2.1 Experiments with the wave overtopping generator

A test campaign was carried out using the Wave Overtopping Simulator, (Van Doorslaer et al.,2012) In the wave overtopping simulator (see Figure 9) different volumes of water can be released in a short time, simulating individual overtopping events. At a distance of 10m from the simulator storm walls are placed, on which impact forces are measured.

Figure 9 Set up of the overtopping Simulator and storm return walls

The force record over time is shown for an overtopping wave of 3500l/m, measured on the horizontal plate (Figure 10) and one of the vertical plates (Figure 11). The 4 individual sensors (Bottom left (BL), bottom right (BR), top left (TL) and top right (TR)) are to be read from the left Y-axis, and are expressed per meter width (N/m). The sum of the four sensors, divided by the width of the measurement plate is to be read from the right Y-axis (also in N/m).

The force records of the horizontal plate (Figure 10) have a very steep rise over time: the maximum is reached in 0.1 to 0.2s for the bottom sensors (_BL and _BR) and about 0.3s for the top sensors (_TL and _TR). The bottom sensors measure the highest forces. The top sensors are located above the flow depth, and measure less high forces. The measurements of the left and right sensors on both top and bottom are very similar.

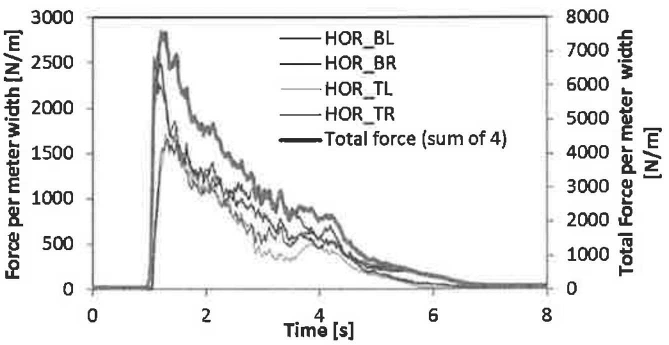

Figure 10. Force record of a horizontal plate, under an overtopping wave volume of 3500l/m

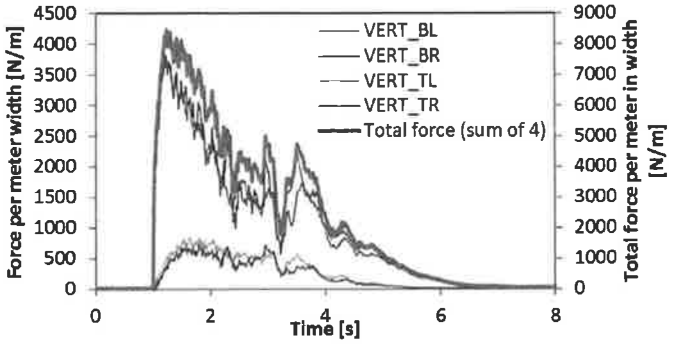

Figure 11. Force record of a vertical plate, under an overtopping wave volume of 3500l/m

5.2.2 Experiments at GWK

Another test campaign was carried out at the Large Wave Flume (Grosser Wellen Kanal, GWK) in Hannover. A dike with slope 1/3 was built in the flume with a crest level of +6m. on top of the dike, a 10m long promenade was built and storm walls were installed at the end of this promenade (see Figure 12). Both forces and pressures were measured in this test campaign and linked to the parameters of the incoming bore of the wave which overtopped the dike's crest.

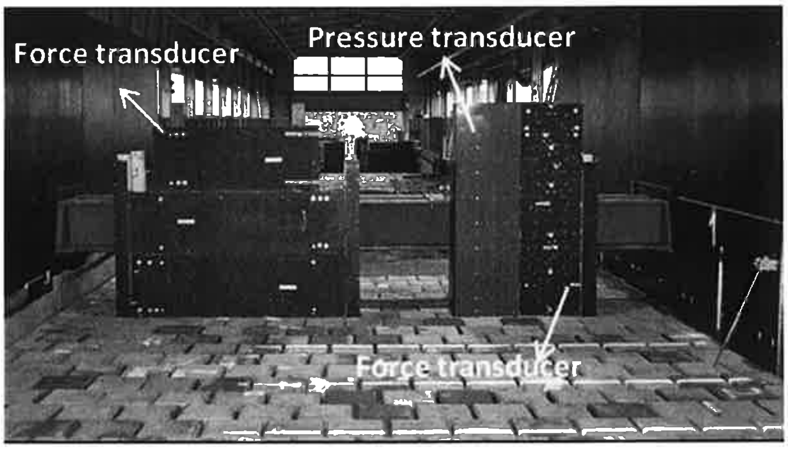

Figure 12 Model set-up experiments at GWK

More information about the test results can be found in Ramachandran et at, 2013.

5.3 Wave overtopping and wave forces over dikes with a shallow foreshore.

The foreshore at most of the coastal towns is very shallow and therefore wave breaking plays an important role for wave loading on the proposed wave return wall. Physical and numerical model tests have shown that the swash zone during extreme storm events is characterized by the generation of low-frequency infragravity waves (f < 0.04 Hz prototype scale), and the formation of solitary bores which collapse at the shoreline, propagate up the beach face and impact the sea dike. The physical experimental set up includes the foreshore. More information can be found at Veale et al, 2012.

6 Conclusions

- A master plan for coastal safety was set up using as much as possible available knowledge
- For the design phase many experiments are done/planned
- A reduction coefficient for overtopping over quay walls with a storm return wall on top is obtained
- In general the results of these experiments do not result in a need to reconsider the social cost benefit analysis
- However, communication to the public with preliminary results can be confusing if the results change after the detailed design.

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