8th INTERNATIONAL CONFERENCE ON COASTAL AND PORT ENGINEERING IN DEVELOPING COUNTRIES

COPEDEC 2012, IIT Madras, Chennai, INDIA.
20-24 Feb. 2012

PROTECTION AGAINST FLOODING OF THE HARBOUR OF OSTEND (BELGIUM) BY THE CONSTRUCTION OF STORM WALLS

P. DeWolf[1], K. Trouw[1], A. Bolle[2], K. Van Doorslaer[3], P. Troch[3], T. Verwaest[4], D. Ballieu[5]

Abstract: The city of Ostend is threatened by floodings due to wave overtopping over the quays of the harbour of Ostend. For this reason storm return walls have to protect the city. This paper describes the design of these storm return walls, starting from the determination of the hydrodynamic boundary conditions, the calculation of wave overtopping discharges and wave forces and the final geotechnical and structural design of the storm return walls.

Keywords*: wave penetration, harbour, wave overtopping, forces, structural design, storm wall.*

INTRODUCTION

The town of Ostend is the service centre of the Belgian coast. It is an attractive sea resort with a rather small harbour at present, although Ostend has been one of the important ports on the Southern North Sea for many centuries.

Fig. 1 : Location and aerial view of Ostend centre and its harbor

(simulation of the harbor extension; the new breakwaters are at present nearly completed)

1 Agency for Maritime and Coastal Services (MDK), Coastal Division, Vrijhavenstraat 3, 8400 Oostende, Belgium,
tel.: +32 59 55 42 11, fax: +32 59 55 42 66, peter.dewolf@mow.vlaanderen.be
2 IMDC, annelies.bolle@imdc.be
3 Ghent University, Dept. of Civil Engineering. Technologiepark 904, B-9052 Zwijnaarde.
Peter.Troch@UGent.be, koen.vandoorslaer@ugent.be
4 Flanders Hydraulics, toon.verwaest@mow.vlaanderen.be
5 Technum-Tractebel Engineering, dries.ballieu@technum-tractebel.be

The old town centre is situated at the west of the harbour and its level is situated at about mean high water level. The quay walls of the harbour have crest heights which are at many locations lower than the design water level. Waves penetrating into the harbour can cause damage to nearby buildings and severe flooding of the old city centre. But newer city areas situated around the harbour are also insufficiently protected against high storm surges.

The city of Oostende is vulnerable to coastal flooding because large areas are low-lying.

The paper describes the flooding problem in the harbour and the design of the protection measures.

THE MASTERPLAN FOR COASTAL SAFETY OF THE FLEMISH REGION (BELGIUM)

The region of Flanders (Belgium) has a coastline of 67 km, with 10 coastal cities which are also important bathing resorts, with the harbours of Zeebruges and Ostend and the yachting harbours of Nieuwpoort and Blankenberge, and a number of important nature reserves. The coastal area is eminently a domain where recreation, naturalness and economic prospects go together. The area is on the other hand vulnerable because of the risk of flooding by the sea. To reduce this risk to an acceptable level the Flemish government decided to lay down a Masterplan for Coastal Safety.

The Coastal Division of the Agency for Maritime and Coastal Services of the Flemish region was responsible for the content of this plan. The plan details the measures which are necessary to protect the entire Flemish coastline against flooding.

The Masterplan for Coastal Safety was drafted to guarantee a basic safety along the Flemish coastline against minimal a 1000-year storm surge and this for a medium long period till 2050. On the basis of a safety check of the coastal protection and flood risk calculations for different superstorm ranges, the zones which required attention were defined and the priorities established. The protection measures for the weak zones were defined on the basis of flood risk calculations, an environmental impact study, a societal cost benefit analysis, budgetary considerations and consultation of the stakeholders.

FLOODING PROBLEM IN THE HARBOUR OF OSTEND

Ostend has a high concentration of inhabitants in the city centre and an important economic value. From the studies it appeared that the city of Ostend constitutes one of the weakest links in the flood protection of the Flemish coastline and that measures have to be executed with high priority to protect the city centre from flooding via the seaside and via the harbour.

The water levels corresponding to storm surges with different return periods can be estimated from tidal records in the harbour of Ostend. A disastrous flooding occurred already in 1953 when a storm surge with a 250 year return period struck the city. Since then measures were taken over a number of years to improve the protection of the city against flooding. Quay walls were heightened, storm walls were built and in 2004 a beach nourishment in front of the city centre was carried out. The city of Ostend is now protected against a 100 year storm surge, as well from the seaside as the harbour side. At present two new breakwaters are under construction to make the harbour accessible for ships with a length of 200 m.

In accordance with the Masterplan for Coastal safety, Ostend has however to be protected against storm surges with a minimum return period of a 1000 years. A cost benefit analysis was executed to decide if a further increase of the safety level was cost efficient. For this reason, also calculations were done for higher and lower water levels with their corresponding probability of occurrence. Integration of the damage and casualties for these water levels resulted in flood risk estimations. A chain of numerical models was used starting with wave propagation simulations towards the coastal defence line in the harbour area, overtopping and overflow calculations, breach modeling, flooding simulation of the low-lying coastal plain, and finally a GIS-based empirical model to estimate direct economic damages and probabilities of human casualties in relation to the maximum water levels and velocities occurring during the flooding.
At the seaside, it has already been decided to strengthen the coastal defence by a new beach nourishment and by building a new seawall with a stilling wave basin between the new western breakwater and the old western harbour jetty. The old western harbour jetty is a wooden construction that is protected as a heritage monument.

The new harbour breakwaters will allow larger ships to enter the harbour and prevent sedimentation of sand in the harbour entrance channel. Measures to prevent flooding of inhabited areas via the harbour are however necessary.

In the harbour mainly 3 options were possible to increase the safety level: 1) heightening the quay walls 2) construction of flood walls and 3) the construction of a flood barrier.
Many buildings are already present in the harbour area and consequently heightening the quays walls was not very feasible. Because of the large width of the harbour entrance (more than 100m) option 3 was not retained either. This last measure would be very effective to reduce the water level in the harbour, but is too expensive compared to other possible measures.

Finally option 2 was chosen and flood walls will be built around the harbour area.

DESIGN OF THE FLOOD WALLS

Design water level

The design water level is the summation of the astronomical water level, the storm surge, wave set up, harbour oscillations, seiches, long waves and sea level rise. The first 3 components are incorporated in the wave statistics (extrapolation of the observed water levels during a period of 75 years). Seiches are believed not to occur preferentially during storms and have not been taken into account. Sea level rise was estimated on the basis of the IPCC(2007)-recommandations (30 cm till 2050). Long waves are still under investigation and are not yet accounted for. The extreme water level (with a 1000 year return period) is about 7m above MLLWS.

Wave penetration

The diffraction coefficients were determined on the basis of a comparative study of the different approaches and expert judgment.

Three numerical models were used for the study of the wave penetration: SimWave, based on Nwogu's extended Boussinesq model equations, Mike21 BW (also a Boussinesq model) and MILDwave, a mild-

slope wave propagation model based on the equations of Radder and Dingemans. Since none of the models include wind growth of waves, which is important due to the length of the harbour, SWAN was used to estimate the effect of the wind.
For the verification of the numerical model, field measurements of wave heights with pressure sensors were executed at 7 locations in the harbour over a period of 2 years.
Because of the high investment cost for building the flood walls and the disastrous consequences if the flood protection measures would fail, it was decided to verify the results of the numerical models by comparing them to the results of physical model tests. The harbour was therefore reconstructed in a wave tank at scale 1:100.

In situ measurements of 22 events with rather high waves were available (measured over a period of 3 years). Pressure sensors were installed at the quays. The pressure was converted to water levels in the frequency domain, using linear wave theory. Because short waves give less response (smaller pressure variations), the pressure sensors can only be used to measure the longer swell waves and not the locally wind generated waves in the harbour with periods smaller than 4s.
Since the pressure sensors are measuring in the neighbourhood of reflective quay walls, the measured wave heights contain also the reflected wave. For the estimation of wave overtopping and wave forces, only the incoming wave is relevant, so the measurements give a slight overestimation. It is difficult to estimate the contribution of the reflected wave to the total wave height, since the waves attack the quays very obliquely.

Mase et al (2002) studied the evolution of the total wave height along a quay wall under oblique wave attack. The figure below shows that the wave height increases along this quay wall. This makes it difficult to derive the incoming wave height. It was decided to use conservatively the total wave height.

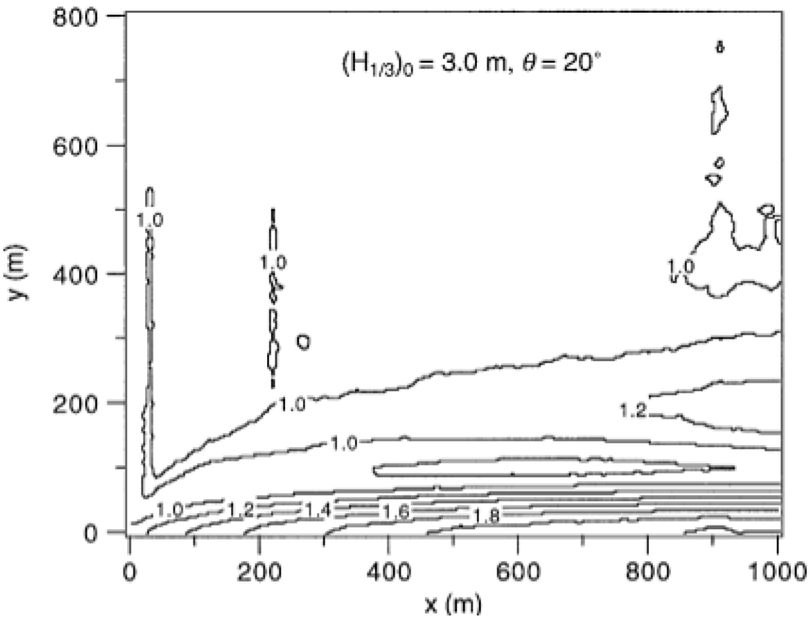


Fig. 2 Ratio of the total wave height over the incoming wave height (Mase et al, 2002)

The reflection and transmission coefficients depend on the water level. If the water level is higher than the quay wall, more energy is transmitted and thus less reflection occurs. This results apparently in a lower wave height.

Due to the reflection from quay walls,, the wave pattern in the harbour is very patchy. This is illustrated in Fig. 3. For each zone inside the harbour, the results were interpreted to permit to derive the appropriate wave height for that zone. Since the presented wave height is the total wave height, including reflected waves, peaks in the wave height pattern are not taken into account. Also the results of the different types of models were considered. However, these models were calibrated, so that the differences between the results were rather small.

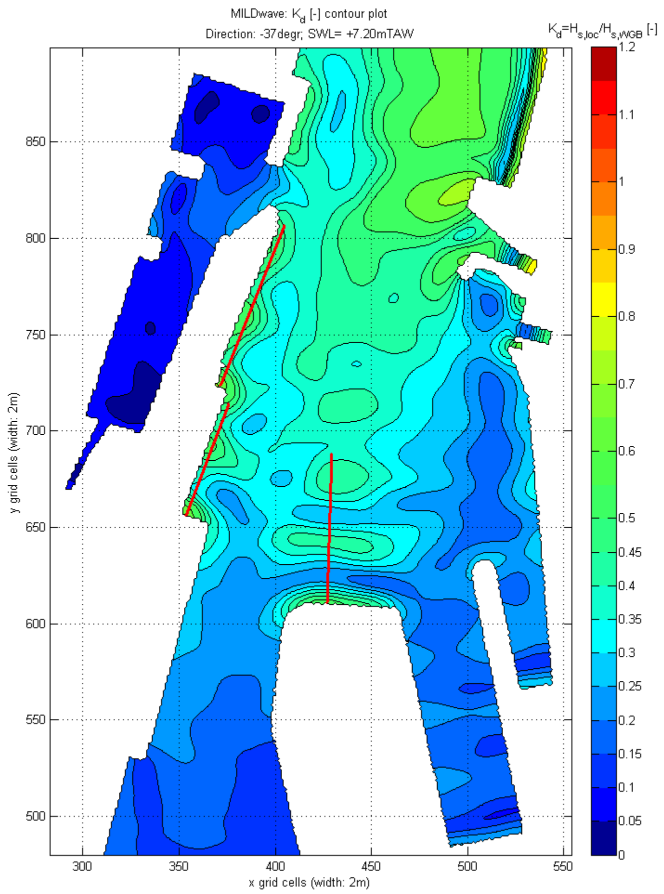
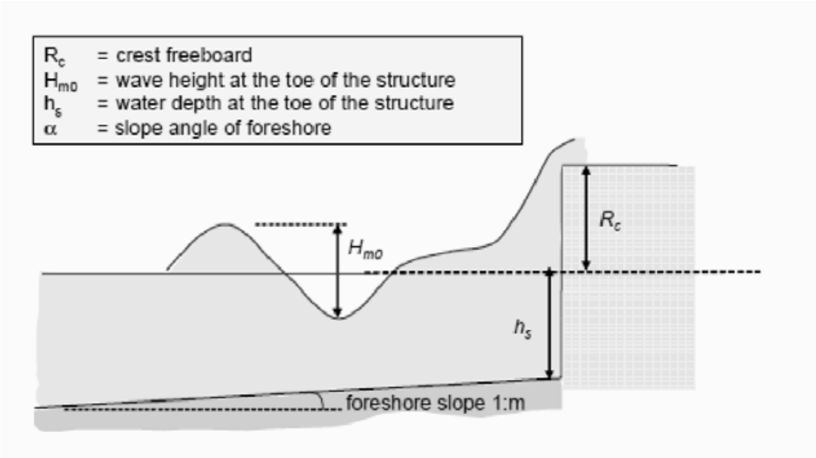


Fig. 3 Ratio of the local wave height over wave height outside the harbour (Gruwez et al, 2011)

Overtopping discharge

For most cases the overtopping formulae for vertical walls (Eurotop Manual, EA, ENW & KFKI, 2007) were used. Overtopping was limited to 1 l/s/m for the 1000 year storm. For flood risk calculations, where also higher water levels are considered (up to 8m above MLLWS), it is assumed that breaching (with a complete flooding of the hinterland) occurs at an overtopping discharge of 200l/s/m.
For non-impulsive conditions, the overtopping is calculated as:



$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04\exp\left(-1.8\frac{R_c}{H_{m0}}\right) \qquad \text{valid for } 0.1 < R_c/H_{m0} < 3.5$$

However, when the distance between the quay and the storm return wall (=berm) is significant (>10m) and when the quay is higher than the design water level, no suitable formulae are available. In these cases physical model tests were executed in a wave flume (scale 1:10 and 1:30).

Fig. 4 gives an example of the reduction of the overtopping discharge by using a berm width (distance between quay and storm return wall) of 11.7m instead of 2m for different heights of the wall, as obtained from the physical model tests.

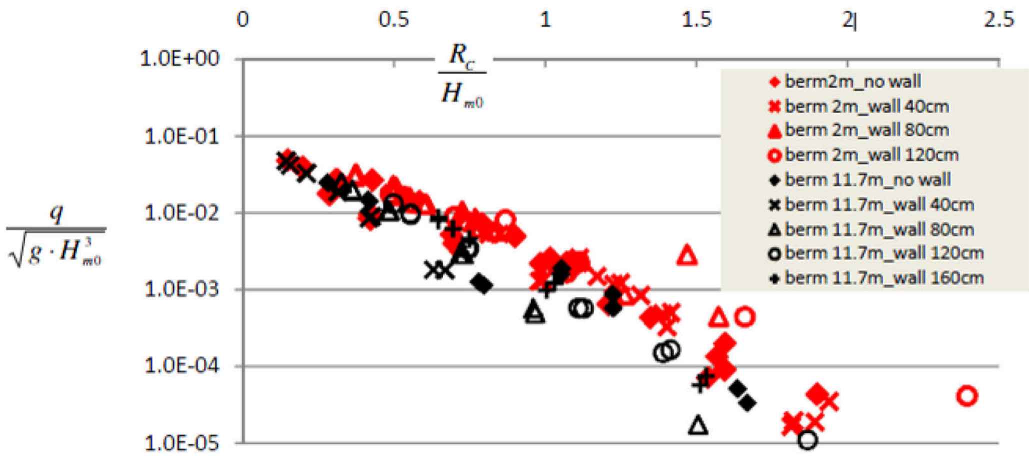


Fig. 4 Influence of the berm width on dimensionless overtopping discharge

The physical model was also used to estimate the reduction of the overtopping discharge by using a parapet.

An extra reduction of the overtopping discharge is obtained by taking into account the direction of the waves. Waves that arrive oblique to a quay wall give less overtopping. The Eurotop-manual propose to use an extra reduction coefficient as:

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

$$\gamma_\beta = 0.72\ voor\ \beta \geq 45°$$

It is noted that in harbours the incoming wave direction relative to the quay wall direction, is often more oblique than 45 degrees. Probably for these cases a greater reduction should be used. However, model tests are not available to determine the extra reduction.

The height of the storm return wall is determined by the 2 requirements (safety assessment for a 1000 year storm and further risk reduction). Near the harbor entrance, where the waves are very high, the first criterium is dominant for the wall height. In this case, a parapet is useful. The final layout of the flood walls is shown in Fig. 5

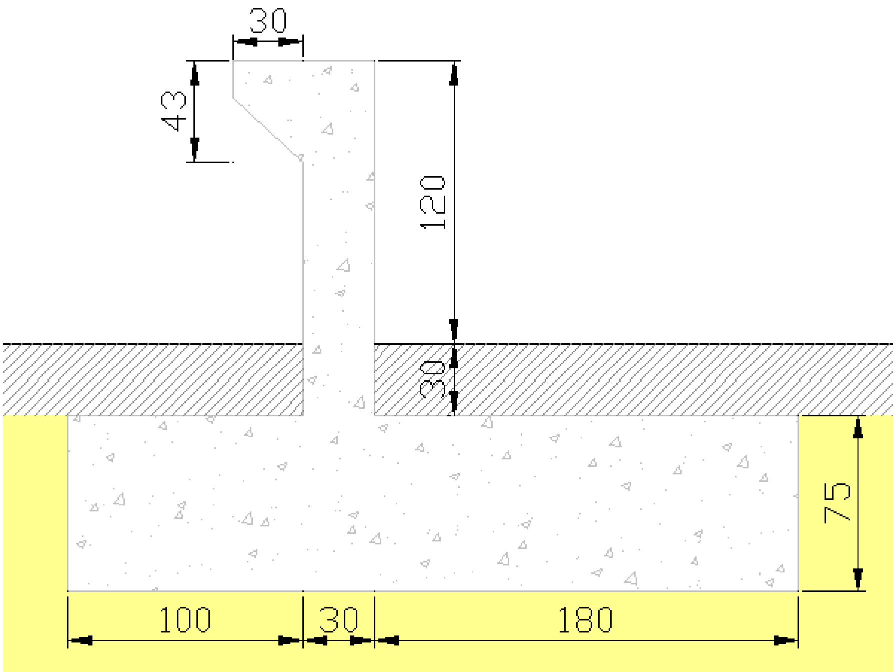


Fig. 5 Dimensions of the storm return wall near the harbor entrance (crest level 8.4m above LAT)

Deeper inside the harbor, where the waves are smaller, the risk reduction was dominant for the wall height.(cf. Fig. 6).

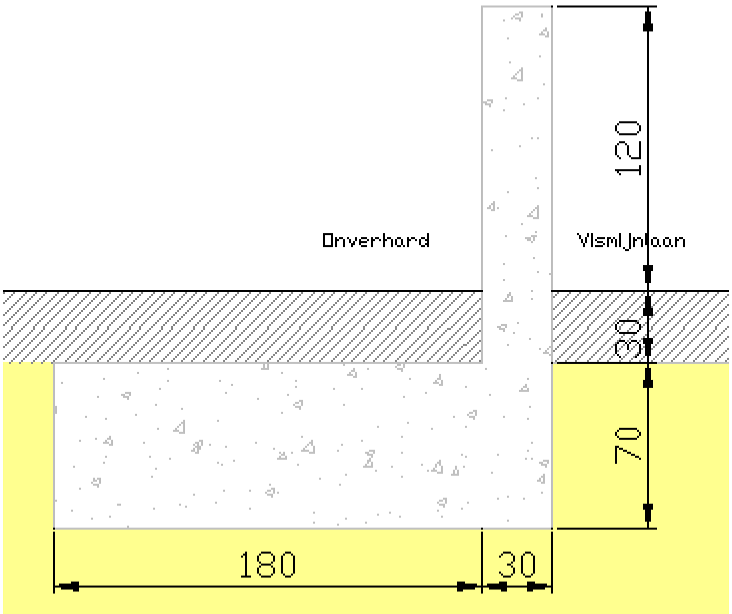


Fig. 6 Dimensions of the storm return wall deeper in the harbor (crest level +8m above LAT)

Wave forces

Wave forces on the flood walls were estimated by using the formulae of Goda. Where necessary, also the physical model was used, by placing wave force sensors on the constructions in the flume. Wave tests were also done for oblique waves.

The wall was attached to a static load cell, measuring the total force during each wave impact. All forces in a time series can be sorted from the highest to the lowest. This leads to the maximum force, but also the 1%, 2%, … greatest forces during a spectrum generated time series.
The disadvantage is that only the total force is measured, which requires an assumption on the point of application of this force. Numerical models may provide some extra information on this topic.
A typical signal of one wave impact is presented in Fig. 7.

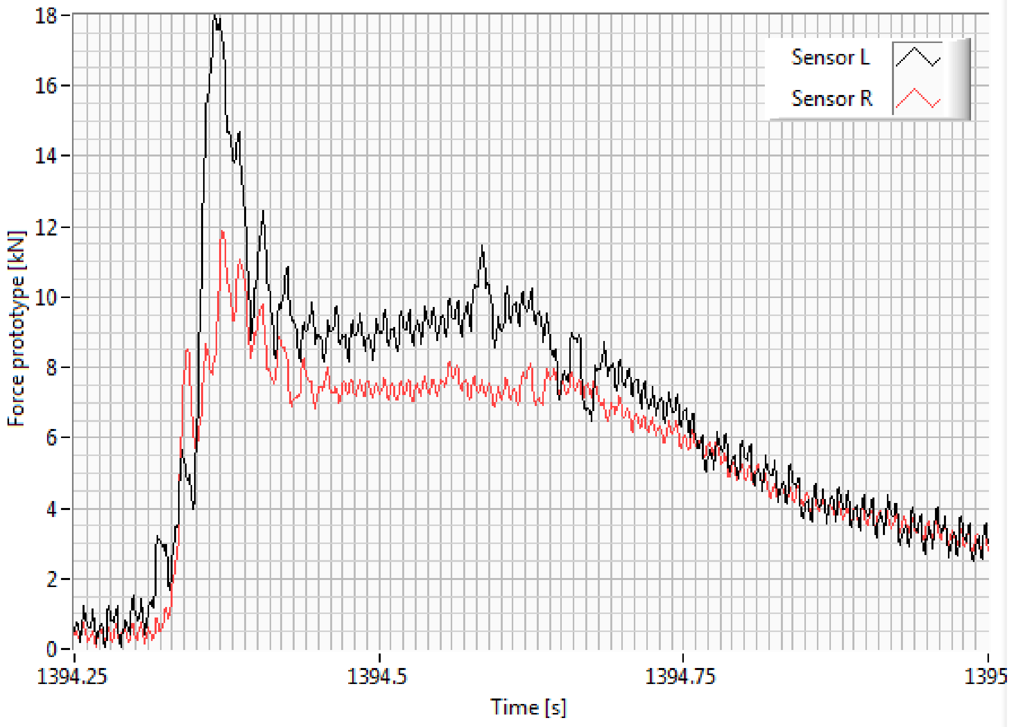


Fig. 7 Church roof signal of 2 load cells during the same impact

One of the major problems during analysis is the appearance of resonance, when the wave impact induces the structure to oscillate at its natural frequency, as can be seen on Fig. 8 (black signal). Detecting the natural frequency of the structure, and filtering at a frequency below this value leads to a reliable order of magnitude of forces during a wave impact. For structures sensible to resonance, the reduction of the wave forces can result in an unsafe design. Field experiments with scale 1/1 were carried out, using van der Meer's Overtopping Simulator (Van Doorslaer et al, 2011). Forces are measured on a wall built on a flat promenade at 10m distance away from the simulator. These tests provided more info on scale effects and resonance in the small scale tests.

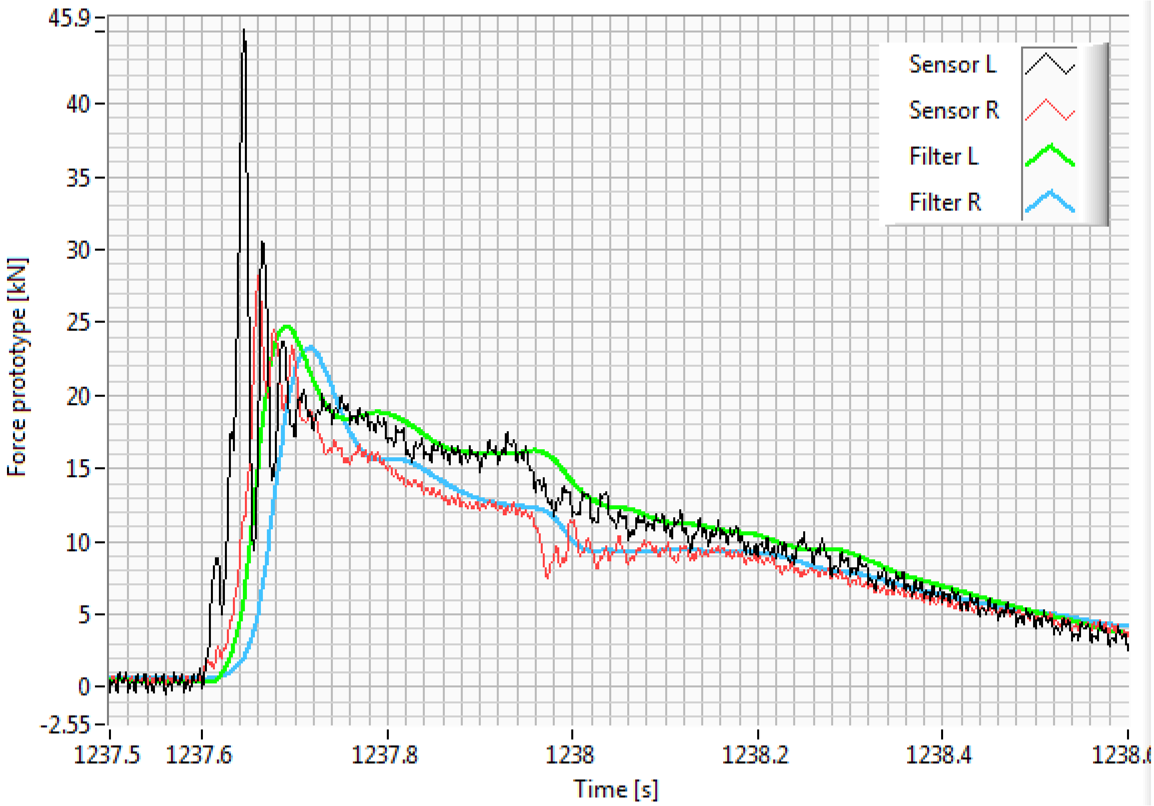


Fig. 8 Black signal is exposed to resonance. Filtering the signal leads to more reliable results (green). The red signal is also filtered, which leads to similar blue results

For the design, the average of the 0.4% highest wave force in the time series is used instead of the maximum wave force since F_{max}depends too much on the generated time series of waves (randomness). It is believed that design formulae do require this maximum wave height as input. However, the wave force is increased with the standard deviation on the measured forces to take uncertainty into account. Since the wave pressure distribution over the height of the wall is difficult to measure, numerical simulations are done with a CFD model. The results (cf. Fig. 9) indicated that the wave pressure distribution is roughly constant over the complete height of the wall for these geometrical and hydraulic conditions which induced a great overtopping discharge. This can be explained by the relative small height of the wall compared to the crest height of the waves. If the thickness of the water layer is small compared to the height of the storm wall, the pressure distribution gives a maximum at the bottom.

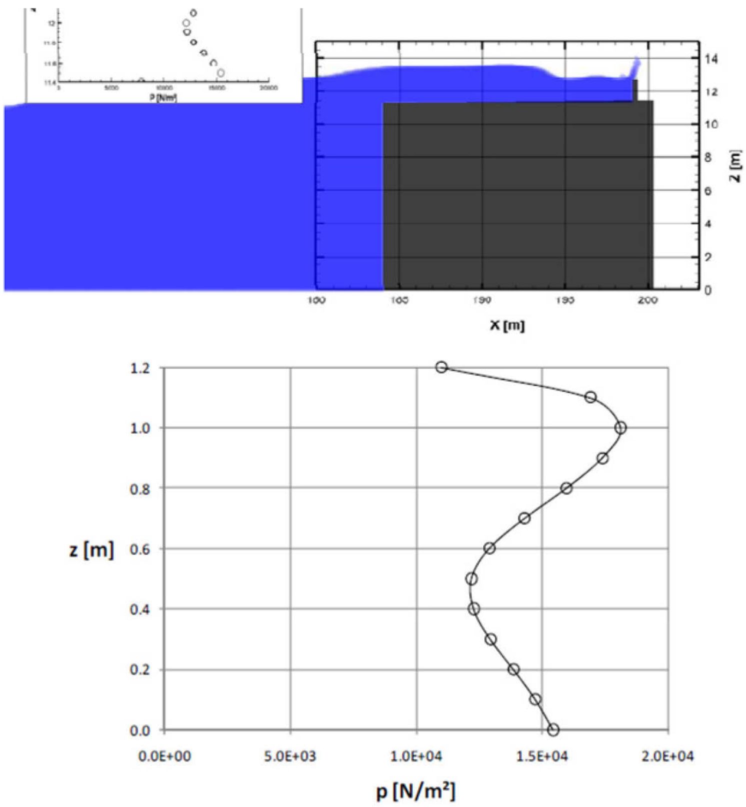


Fig. 9 Vertical distribution of the pressure

Detailed design

Once the wave forces are known, the flood wall is designed by checking the failure mechanisms (turning, sliding, ...). For the design Eurocode is used. The wave forces are considered as an accidental impact and as a consequence class 2 is used to determine the partial safety factor (1.3)

5. CONCLUSION

The paper gives an overview of the flooding problem in the harbour of Ostend due to storm surge water levels and waves, and details the design of the flood walls. Results of numerical and physical model tests are presented.

REFERENCES

EA, ENW & KFKI, (2007). *EurOtop: Wave Overtopping of Sea Defences and Related Structures:Assessment Manual.*

IPCC AR4 WG1 (2007), Solomon, S.; Qin, D.; Manning, M.; Chen, Z.; Marquis, M.; Averyt, K.B.; Tignor, M.; and Miller, H.L., ed., Climate Change 2007: The Physical Science Basis, Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change, Cambridge University Press, ISBN 978-0-521-88009-1, http://www.ipcc.ch/publications_and_data/ar4/wg1/en/contents.html (pb: 978-0-521-70596-7).

,

Gruwez, V.; Bolle, A.; Verwaest, T.; Hassan, W. (2011). Numerical and physical modelling of wave penetration in Oostende harbour during severe storm conditions, in: (2011). 5th International short conference on applied coastal research: book of abstracts - SCACR 2011, 6th to 9th June 2011 in Aachen, Germany. pp. 37-38

Mase, H., T. Memita, Y. Masatoshi, en T. Kitano, 2002. Stem Waves Along Vetical Wall Due to Random Wave Incidence. *Coastal Engineering 44*.

Van Doorslaer,K., De Rouck,J., van der Meer, J., Trouw,K., Full Scale Wave Impact Tests on a Vertical Wall, using the Wave Overtopping Simulator, Coasts & Ports 2011 Conference Perth

.