# Numerical and physical modelling of wave penetration in Oostende harbour during severe storm conditions

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## Abstract

Hydrodynamic boundary conditions are needed for the design of new sea defence structures in the harbour of Oostende along all quays to defend the city centre and hinterland from flooding during a super storm. The wave climate in the harbour was decoupled to wave penetration and locally generated wind waves to allow separate modelling of both phenomena. The wave penetration was modelled with a physical wave model, a mild slope equations model MILDwave and a Boussinesq equations model Mike 21 BW. The locally wind generated waves were modelled with the spectral model SWAN. By using a thin sponge layer along the boundaries in the numerical wave penetration models to model partial reflection inside the harbour, very good correspondence is found between the numerical wave penetration models and the physical model. Finally, the wave penetration energy and locally generated wave energy were superposed to obtain the complete wave climate along all structures inside the harbour during severe storm conditions.

*Keywords: wave penetration in harbour, physical modelling, numerical modelling, mild slope equations, Boussinesq equations, local wind generated waves*

## 1        Introduction

### 1.1        Harbour of Oostende

The harbour of Oostende is located on the Belgian coast facing the North Sea. As a part of the master plan for coastal safety of the Belgian coast, safety against flooding and wave overtopping has to be assured during super storms. A minimum safety level is prescribed based on a super storm with a return period of 1000 years.

The still water level during such a storm at Oostende was determined to be +7.20m TAW, including a 0.30m sea level rise due to climate change expected until 2050. The crest level of the quay walls inside the harbour is approximately +6.90m TAW and the city centre is well below this level (+4.50m TAW). It is clear that during a 1000 year storm the quay walls are too low to prevent flooding of the city centre and hinterland. The flooding and overtopping danger is illustrated in Figure 1: even for a storm with return period of 5 years, water can already reach the quay crest level.

Therefore, new sea defence structures are needed to achieve this minimum safety level prescribed by the master plan (e.g. storm walls on the quays). The hydrodynamic boundary conditions along all the tide and wave afflicted constructions in the inner harbour are required for the design of these structures.

At the moment the configuration of the outer harbour is being changed dramatically as the old harbour dams are replaced by two new rubble-mound breakwaters (cf. Figure 2) and the access channel is broadened and deepened to improve maritime access. The wave modelling efforts in

this paper are only focussed on the future situation for which the new sea defence structures need to be designed.



Figure 1: Water reaches quay crest level during a 5-year storm in the harbour of Oostende.



Figure 2: Harbour of Oostende, original harbour dam layout (left), future breakwater layout (right)

Physical and numerical wave modelling was done to obtain the hydrodynamic boundary conditions. Boundary conditions such as the significant wave height $H_s$, the average wave period $T_{m-1,0}$ and wave direction are needed for the functional (wave overtopping) and structural (wave forces) design of the new sea defence structures. This paper will focus on the accurate determination of the significant wave height all around the harbour during severe storm conditions.

## 2    Physical wave modelling

The physical model of the future configuration of the harbour was built in the wave basin of Flanders Hydraulics Research (Hassan et al., 2011) on a scale of 1:100 (cf. Figure 3). A piston type wave paddle generates the waves in this basin and only long-crested wave generation is possible. All time series were based on a JONSWAP spectrum. Wave measurements were performed in the model at more than 40 locations with non-directional wave gauges. Some of these locations are shown in Figure 7.
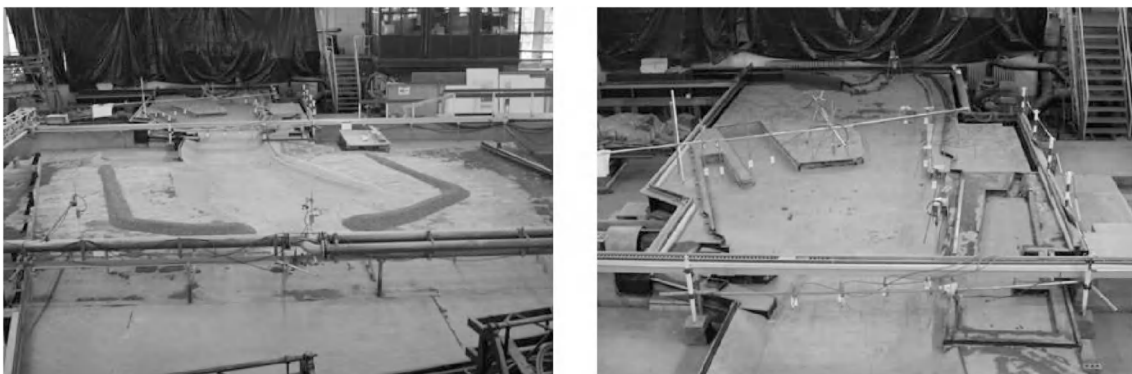


Figure 3: Physical model, outer harbour (left), inner harbour (right)

Storm conditions with a return period of 1000yrs are a water level of +7.20m TAW[5], significant wave height of 5.20m and peak wave period of 12.0s. In addition, several other wave conditions were modelled corresponding to lower and higher water levels in order to deliver input for the flood risk modelling of Verwaest et al. (2008). All wave conditions are shown in Table 1. Three wave directions were simulated in the wave basin: NW, NNW and -37° which is the direction for which the most wave energy penetrates the harbour. In total more than 40 wave conditions were modelled.

Table 1: Hydrodynamic boundary conditions used in the physical model. The prototype values are given.

| Still water level [m TAW] | Max. water depth [m] | Significant wave height $H_s$ [m] | Peak wave period $T_p$ [s] |
|---|---|---|---|
| 6.00 | 14.90 | 4.60 | 12.0; 10.0 |
| 6.50 | 15.40 | 4.80 | 12.0; 10.0 |
| 6.80 | 15.70 | 5.00 | 12.0; 10.0 |
| 7.00 | 15.90 | 5.10 | 12.0; 10.0 |
| 7.20 | 16.10 | 5.20 | 12.0; 10.0; 8.0; 6.0 |
| 7.50 | 16.40 | 5.40 | 12.0; 10.0 |
| 8.00 | 16.90 | 5.70 | 12.0; 10.0 |

The physical model wave data provide calibration and validation data for the numerical models.

## 3     Numerical wave modelling

### 3.1     The numerical models

The most important physical processes in a harbour are diffraction, depth refraction/shoaling, (partial) reflection, transmission and non-linear wave-wave interactions (Battjes, 1994). Phase-resolving numerical models are used for the modelling of wave penetration because they can simultaneously account for diffraction and reflection (and standing waves) as opposed to phase-averaged wave models. Two phase-resolving models were used: MILDwave (Troch, 1998) and Mike 21 BW (DHI, 2009). MILDwave is a linear time dependant wave model based on the mild-slope equations of Radder and Dingemans (1985) and Mike 21 BW is a more complex non-linear time dependant wave model based on the enhanced Boussinesq equations of Madsen et al. (1991, 1992). By comparing both numerical models and validation with the physical model data, the applicability of each model is identified.

A severe storm is accompanied by extreme wind speeds (20m/s – 30m/s and higher), which cause local generation of very short waves with relatively high wave heights even for the relatively short fetch lengths in a harbour (van der Meer et al., 2002). Especially at the more landward areas of the harbour, these waves are substantial and cannot be ignored because of the low wave penetration energy in these parts. The spectral model SWAN (TUDelft, 2010) is a phase-averaged wave model, based on the action balance equation (Booij, 1999) and can model wave generation by wind. However, because it is phase-averaged, SWAN is not able to simulate diffraction[6] and standing waves due to reflection and cannot therefore model the wave penetration into the harbour accurately. Due to the local nature of wave generation by wind, diffraction plays a minor role in the harbour for these waves. The use of the phase-averaged wave model SWAN in harbours is therefore expected to be acceptable for and is limited to the simulation of local wave generation by extreme wind speeds.

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[5] TAW or „Tweede Algemene Waterpassing" is a reference level used in Belgium. A level of 0.00m TAW corresponds to the average low tide level at Oostende.

[6] SWAN does have a limited capability to model diffraction, but not in combination with reflection (TUDelft, 2010)

The physical model and both phase-resolving models cannot on their part account for locally wind generated waves[7]. This is why the modelling of wave penetration and locally wind generated waves was decoupled as proposed by van der Meer et al. (2002).

## 3.2     Wave penetration modelling

### 2.3.1     Set-up of models

The bathymetry used for the numerical models is shown in Figure 4 and is similar to the bathymetry used in the physical modelling. The bathymetry of the outer harbour was based on plans of the future harbour entrance channel. The bottom levels in the inner harbour were based on the prescribed dredging depths, resulting in a bathymetry with the highest possible water depths, allowing the most wave penetration.
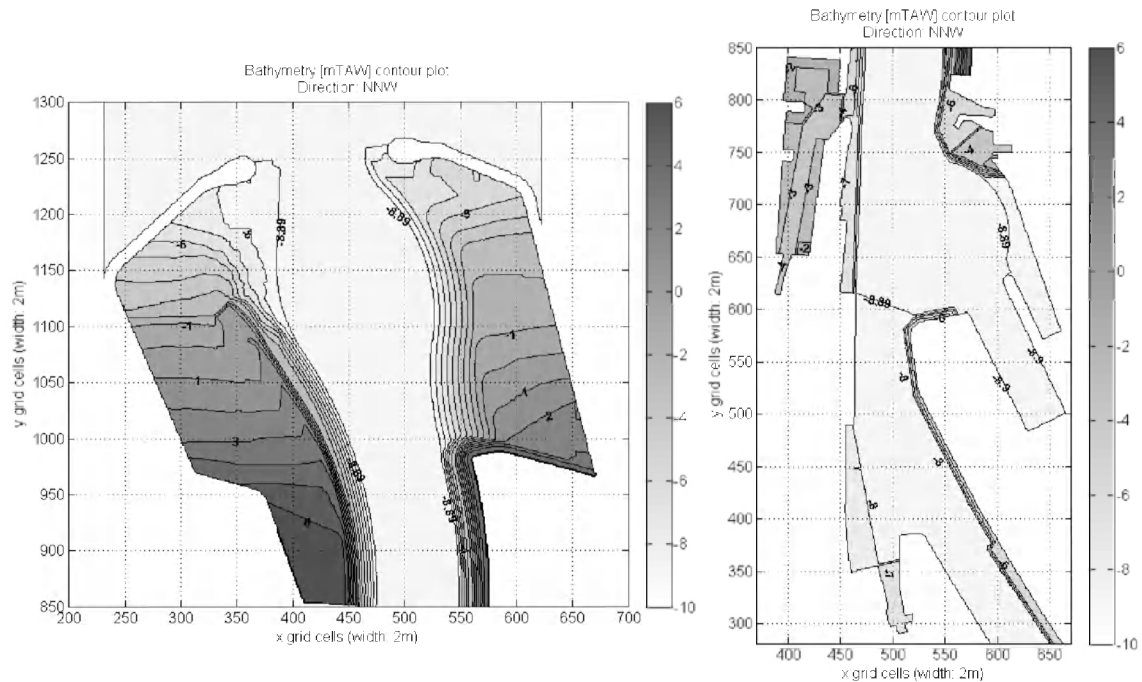


Figure 4:   Bathymetry of outer (left) and inner harbour (right). Bottom levels are given in [m TAW].

The computational domain was restricted outside the harbour as much as possible to limit the calculation time by using artificial land (cf. Figure 5). Waves are generated in the phase resolving models by an internal wave generation line at a distance of two wave lengths from the harbour entrance. Seaward of the wave generation line, a sponge layer was added to absorb all outgoing wave energy.
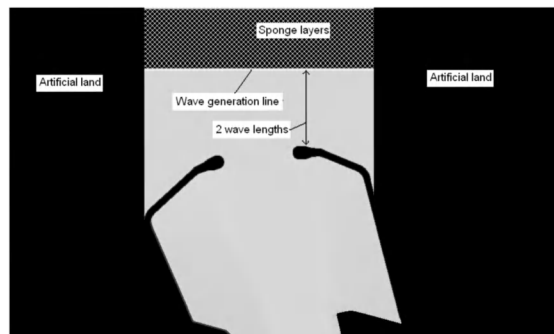


Figure 5:   Computational domain at the outer harbour with indication of artificial land, internal wave generation line and sponge layers

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[7] Wind generation of waves is currently under development for MILDwave by the University of Ghent.

The grid spacing is 2.00m in both x and y direction and a time step of 0.05s was used. A minimum water depth was set to 2.00m to avoid having to include wave run-up. Wave breaking was included in both models because wave breaking on the shallow areas in the outer harbour could have an important effect on the wave climate in the inner harbour. Transmission of wave energy over the harbour dams was neglected, since this is not important for the inner harbour.

To model the partial reflection in the harbour, a different approach than use of porosity layers – as suggested by DHI (2009) – was used. This is because the necessary width of a porosity layer is $L_p/4$ (with $L_p$ the wave length corresponding to the peak wave period $T_p$), which would take up too much space inside the harbour because of its small dimensions in relation to the wave length. Instead of using porosity layers, a thin sponge layer of one grid cell in front of each partially reflecting boundary was used. This was shown by Brorsen (2000) to be an effective and practical numerical partial reflection method for wave propagation models. The same approach has been used in the MILDwave model.

Appropriate reflection coefficients were determined for each structure in the harbour (e.g. (flooded) quay walls, dikes, rubble mounds,...). The corresponding sponge layer coefficients were calculated iteratively with a 1DH-model of a cross section of the considered structure and a reflection analysis, or where possible by calibration with the physical model results.

### 2.3.2   Results and comparative analysis

Most wave conditions from Table 1 were simulated with the wave penetration models. They were also based on a JONSWAP spectrum and only long-crested waves were generated. The results shown in this paper will only be limited to the 1000-year wave condition (cf. SWL = +7.20m TAW in Table 1) and for wave direction NNW, but the same conclusions count for all.
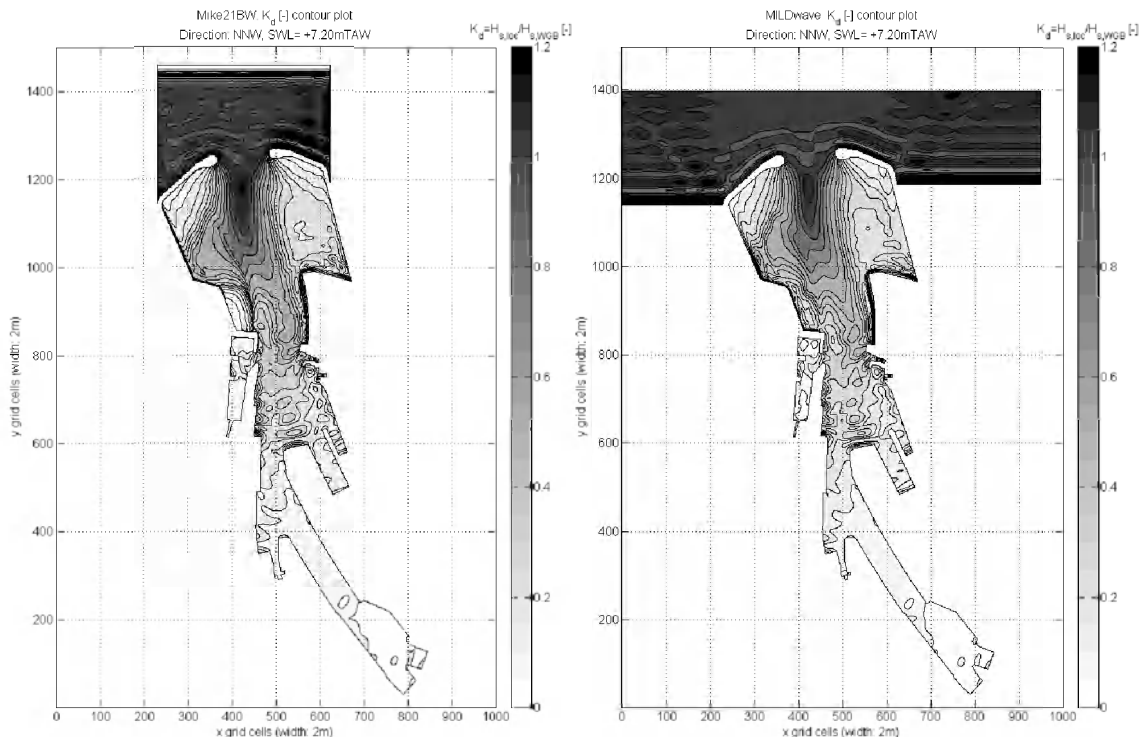


Figure 6:   Comparison of Mike 21 BW (left) and MILDwave (right) wave disturbance result.

A very good correspondence of the wave disturbance coefficient $K_d$ (=$H_{s,location}/H_{s,wave\ generation\ line}$) between both phase resolving models is found (cf. Figure 6). This is also shown in Figure 7 where the wave disturbance is compared at the exact locations of the wave gauges in the physical model. A very good correspondence is also found of both numerical models with the physical model wave disturbance results. Some exceptions (cf. $G06_{left}$ and $G14_{left}$ in Figure 7) are mainly due to the sensitive location of standing wave oscillations.
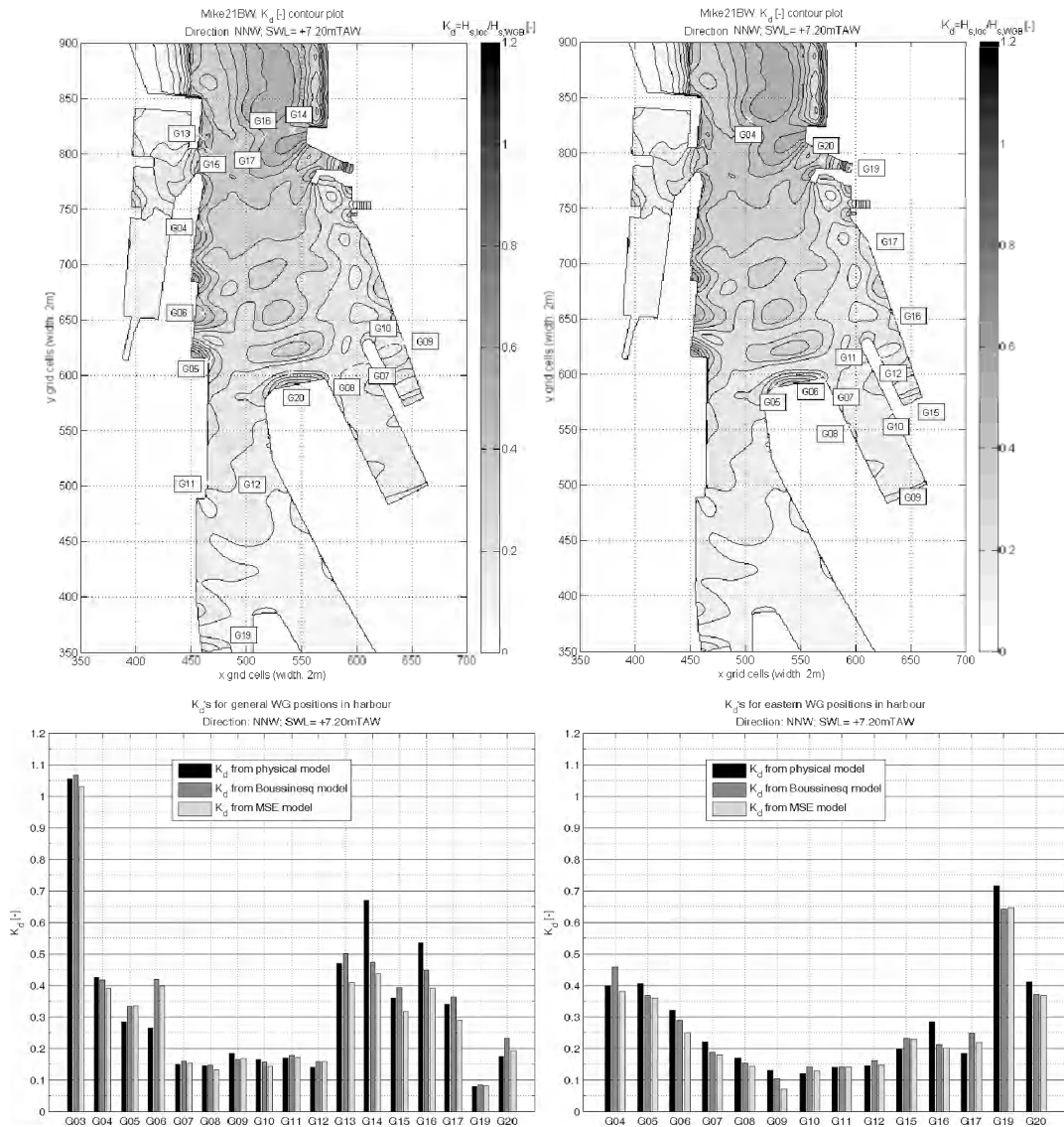
Figure 7: Comparison of the wave disturbance at the wave gauge locations of the physical model. Location of G03$_{left}$ is at the harbour entrance.

The mild-slope model MILDwave seems to be just as capable as the Boussinesq model Mike 21 BW to model wave disturbance in harbours, and this with a lower calculation time.

## 3.3    Local wave generation by extreme wind speed

### 3.3.1    Set-up of model

The bathymetry of the SWAN model includes the most landward parts of the harbour as shown in Figure 8, which is a continuation of the inner harbour bathymetry from Figure 4. This part was not included in the wave penetration models because the wave penetration energy is negligible here as opposed to the locally generated wind waves.

The harbour entrance was closed and no waves were imposed at the boundaries. A uniform wind field was defined covering the complete calculation domain. All onshore wind directions and corresponding extreme wind speeds (~24 m/s at 10m height for RP = 1000yrs) were modelled to obtain the most disadvantageous conditions for each area in the harbour.

During a 1000-year storm, locally generated wind waves with significant wave height $H_{m0}$ of up to 0.80m and an average wave period $T_{m-1,0}$ of 2.1s are generated for the available fetch length inside the harbour. These waves are not negligible, certainly not at the most landward areas of the harbour (cf. right in Figure 9) where almost no wave penetration energy is left (cf. Figure 6).
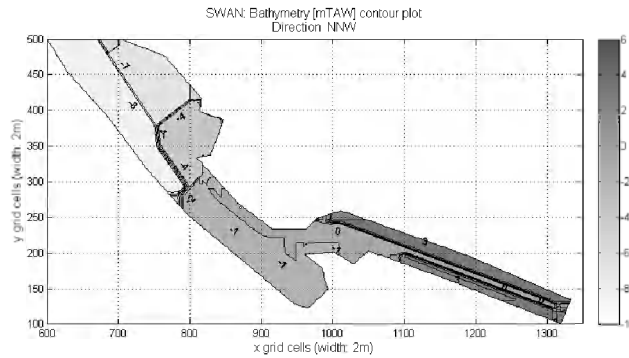
Figure 8:  Bathymetry of the most landward part of the harbour. Bottom levels are given in [m TAW] (attention: different origin of axis than in Figure 4).
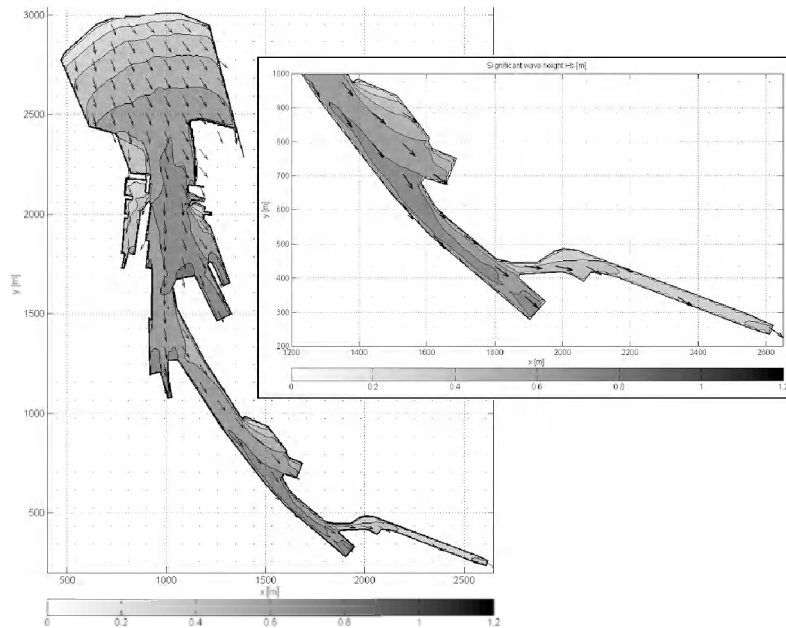


Figure 9:  Significant wave height $H_{m0}$ [m] and direction (arrows). Wind speed is 24m/s (from NW). Left: complete model domain, right: zoom to most landward areas of harbour.

## 3.4    Resulting boundary conditions in the inner harbour

The total wave climate on each location in the harbour is obtained by superposition of the wave penetration energy and the locally wind-generated wave energy according to equation (1) as proposed by van der Meer et al. (2002).

$$H_{m0} = \sqrt{H_{m0,1}^2 + H_{m0,2}^2} \tag{1}$$

with    $H_{m0,1}$    significant wave height of the wave penetration   [m]

        $H_{m0,2}$    significant wave height of the locally wind generated waves   [m]

Functional design of the new sea defence structures is based on a limitation of the overtopping discharge and associated flooding risks for people and properties. Formulas to calculate overtopping require the incoming significant wave height. All wave models deliver the total significant wave height which includes both the incoming and reflected wave energy. Because of the small dimensions of the harbour and the complexity of the reflected wave field, conventional reflection analysis in the harbour is very difficult if not impossible. An expert judgement is recommended to take reflection effects into account when interpreting the results of the wave penetration models. Otherwise, it is possible that too conservative significant wave heights are used for the functional design.

## 4        Conclusions

New sea defence structures are needed in the harbour of Oostende to assure safety against flooding and wave overtopping during severe storm conditions. Therefore accurate hydrodynamic boundary conditions are needed for the design of these structures. This paper provided an overview of how the significant wave height in the harbour of Oostende during these severe storm conditions was obtained.

Modelling of the wave climate was decoupled to wave penetration and locally wind generated waves by the extreme wind speed. The wave penetration was simulated with the phase-resolving models MILDwave and Mike 21 BW, the local wind generated waves by the phase-averaged model SWAN. Using a sponge layer of one grid cell to model partial reflection in the phase-resolving models proved to be a very good method as the validation with the physical model data showed. The linear wave model MILDwave and non-linear wave model Mike 21 BW showed comparable results for the wave disturbance inside the harbour. The results of the SWAN model showed that severe storm wind can generate substantial waves for the relatively short fetch lengths inside the harbour. These waves are particularly important for the most landward parts of the harbour where almost no wave penetration energy is left.

The wave penetration energy and local wind generated wave energy were finally superposed, providing the complete wave climate inside the harbour during severe storm conditions.

## 5        Acknowledgements

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