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SPH modelling

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Abstract

Wave loading and wave overtopping of the sea dikes around the Blankenberge Marina were previously quantified and reported in Altomare et al. (2015), where the SPH-based DualSPHysics model was used for the calculations. Recently, a new solution was proposed for one of the cross sections that represents the sea dikes in Blankenberge area. The new design requires re-considering the wave loading on the storm wall via additional numerical model runs. The flood defence has been modified from a 1/2 sloping dike into a vertical structure. The storm wall itself, while keeping the same elevation, has been moved landwards. Numerical modelling has been performed to assess the wave loading on the storm wall and the pressures on the vertical face of the dike exposed to the wave attack. The results have been compared with the ones from the previous configuration.

Contents

List of figures

1 Introduction

1.1 Motivation of the work

Wave loading and wave overtopping of the sea dikes around Blankenberge Marina were previously quantified and reported in Altomare et al. (2015). However, a new solution was proposed for one of the cross sections that represent part of the sea dikes (Figure 1). The new design requires re-considering the wave loading on the storm wall though additional numerical model runs.

Figure 1. Cross sections representative of sea dikes in Blankenberge marina (map source: Google Earth)

The section to be re-considered is section 4C. Previously, the geometrical layout design was comprising a sloping dike with a storm return wall, see Figure 2. Wave forces on the storm wall were characterised by means of the SPH-based DualSPHysics model (Altomare et al. (2015)). In this study, the same numerical model has been used to assess the loading to the new layout of cross section 4C. The new layout of the cross section is shown in Figure 3. The structure consists of a vertical quay with a storm wall and horizontal slab located at +2.8 m TAW. The storm wall is marked in red in Figure 3. Its maximum elevation is +8.2 m TAW. Pressure distributions on the vertical walls where the horizontal slab is located have been also requested.

Figure 2. Previous design of cross section 4C (Altomare et al. 2015).

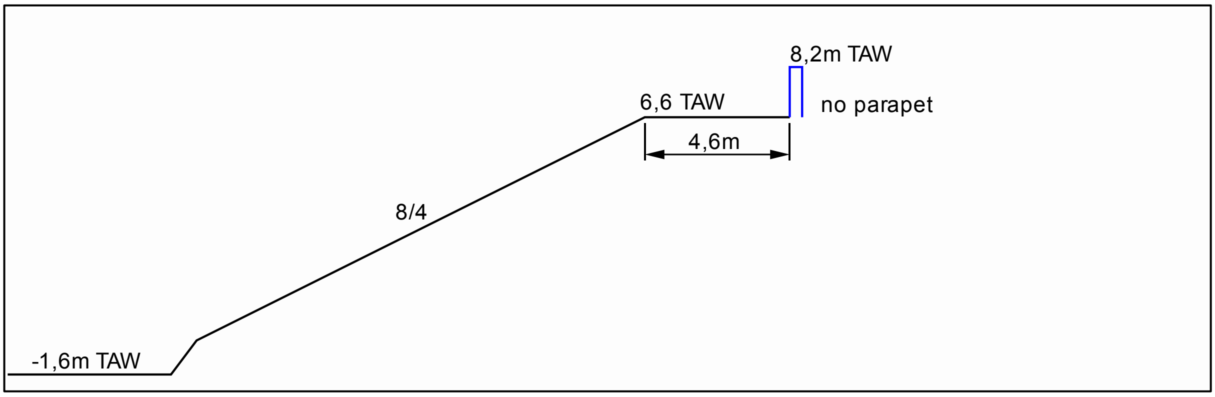
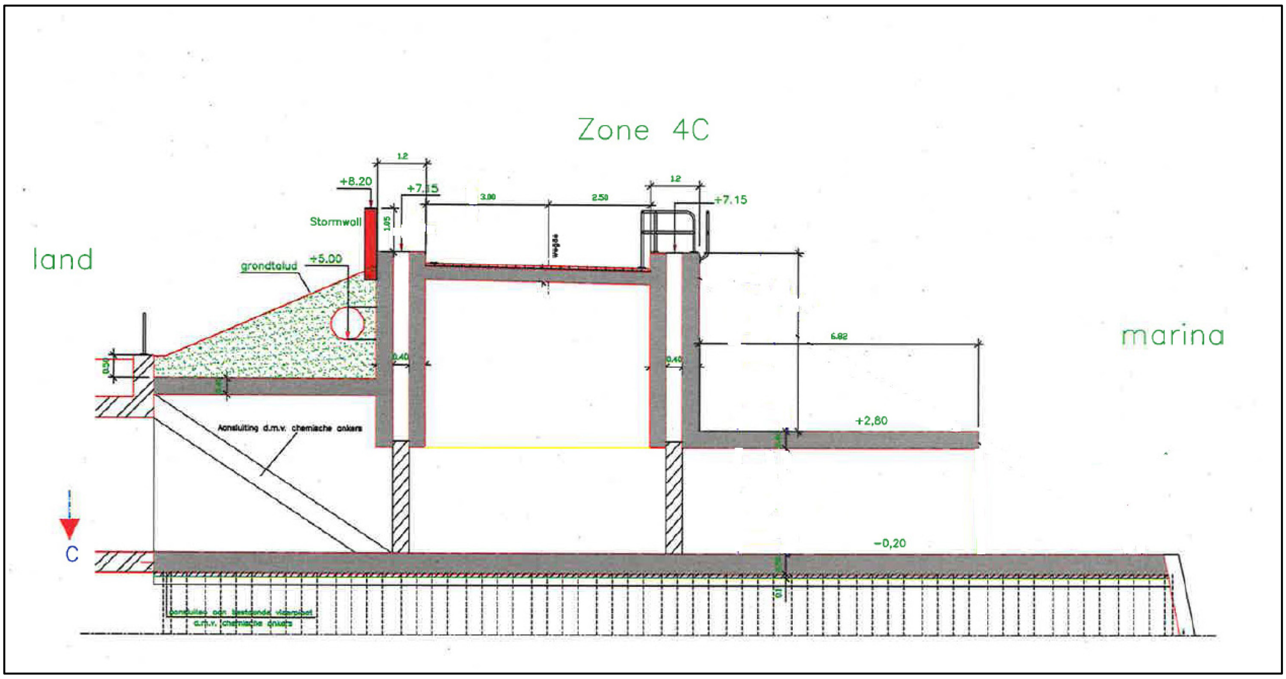


Figure 3. New layout design for cross section 4C.



2 Numerical modelling

2.1 DualSPHysics model

2.1.1 overview

The DualSPHysics model (http://dual.sphysics.org/) has been used for the present analysis. DualSPHysics is a numerical model based on the Smoothed Particle Hydrodynamics (SPH) method (Crespo et al., 2015). Smoothed Particle Hydrodynamics is a Lagrangian and meshless method where the fluid is discretised into a set of particles and each of these particles is a nodal point where physical quantities (such as position, velocity, density, pressure) are computed as an interpolation of the values of the neighbouring particles. In DualSPHysics, boundaries (walls, bottom, coastal structures, wave generators, vessels, wind turbines, etc.) are described using a discrete set of boundary particles that exert a repulsive force on the fluid particles when they approach. A dynamic boundary condition (Crespo et al., 2007) is used in DualSPHysics, where the boundary particles satisfy the same equations as the fluid particles, however they do not move according to the forces exerted on them. Instead, they remain fixed (fixed boundary) or move according to some externally imposed movement (gates, flaps, etc.).

DualSPHysics is capable of using the parallel processing power of either CPUs and/or GPUs making the study of real engineering problems possible. This new technology makes the study of real-life engineering problems possible at a reasonable computational cost on a personal computer (Altomare et al., 2014; Altomare et al., 2015).

2.1.2 New functionalities

Recently proper wave generation and absorption techniques have been implemented in DualSPHysics for long-crested regular and random waves. The waves are generated in DualSPHysics by means of moving boundaries that aim to mimic the movement of a wave-maker as in physical facilities. The wave generation using moving boundary in DualSPHysics consists of piston- and also flap-type wave-makers. All details about wave generation and wave active absorption system (AWAS) implemented in DualSPHysics can be found in Altomare et al. (2017).

2.2 Hydraulic Boundary conditions

The hydraulic boundary conditions (HBC) for the numerical modelling have been derived by the foreseen irregular waves in the harbour and they consist of locally generated waves by wind and waves from penetration, characterised by different values of wave period and wave height. The resulting HBC are:

- H_{m0}=0.51 m and T_p=6.5 s

The water level is equal to +7.11 m TAW.

2.3 Methodology and model setup

The latest implementation of wave generation and AWAS in DualSPHysics allows overcoming the limitations of the work of Altomare et al. (2015) where only a few monochromatic waves were generated to assess the wave loading on the storm return wall. At the present stage, long wave time series can be generated with the wave reflection compensated by the wavemaker position. Additionally, irregular wave trains from synthetic spectra (e.g. JONSWAP) can be generated. To be consistent with Altomare et al. (2015) two types of waves have been simulated: 1) regular waves with mean wave height equal to 2.2 times the significant wave height and mean period equal to the peak period; 2) irregular waves. For irregular waves, the series have been limited to few waves from the 1000-wave train, in order to reduce the computation costs. However, the time window has been selected in order to include the highest wave of the whole wave train. The numerical layout is depicted in Figure 4. The piston-type wavemaker (left side) is located 65 m from the structure, distance that is larger than the wavelength, *L*, which is about 52 m. The locations where forces/pressures have been measured are marked in red colour. On top the dike a sort of stilling wave basin is present and it has been filled with water prior to simulation in order to represent the effect of residual water on the largest waves that will overtop the structure and then hit the storm wall. The horizontal line emerging from the quay wall is actually the slab depicted in Figure 3 with elevation +2.8 mTAW.

A sensitivity analysis on the numerical model resolution has been conducted, resulting an initial inter-particle distance dp=0.05 m the best compromise between model accuracy and computational cost. All waves have been generated at second-order, which implies the generation of super-harmonics for regular waves and sub-harmonics (bound long waves) for irregular waves.

Prior to the numerical model to calculate forces and pressures, the wave generation and propagation in DualSPHysics has been checked by running a model where the structure was removed and the numerical tank was extended up to *3L* approximately. At the right-end side of the tank a relaxation zone was included in order to damp out the wave reflection. The water surface elevation has been therefore measured at 5 different locations (x=45.76, 48.36, 52.00, 56.68, 61.88 m) and the WaveLab software (v.3.742) from the Aalborg University (http://www.hydrosoft.civil.aau.dk/wavelab/) has been used to calculate the incident wave height at the location where the structure is foreseen.

Figure 4. Numerical layout of cross section 4C

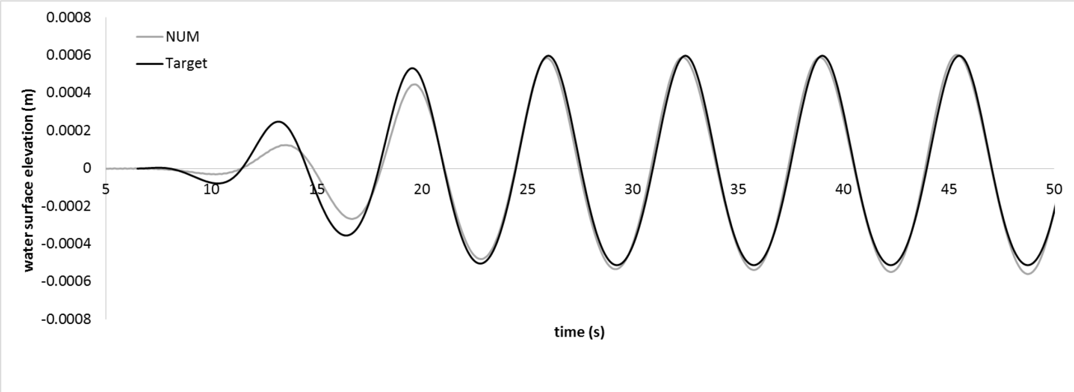
2.4 Regular wave modelling

2.4.1 Incident wave analysis

Following the methodology described in Altomare et al. (2015), a regular wave height of 2.2 times the significant one has been simulated (H=1.1 m) with a mean period equal to the spectral period (T=6.5 s).

Figure 5 shows the water surface elevation measured in the numerical model without the structure at x=52.00 m from the wave maker. The theoretical (target) water surface elevation is also depicted. It can be noticed that the numerical one matches the target one. The errors on H and T have been estimated and they are equal to 0.18 % and 0.25 %, respectively.

Figure 5. Comparison between the numerical and the theoretical water surface elevation for regular waves.

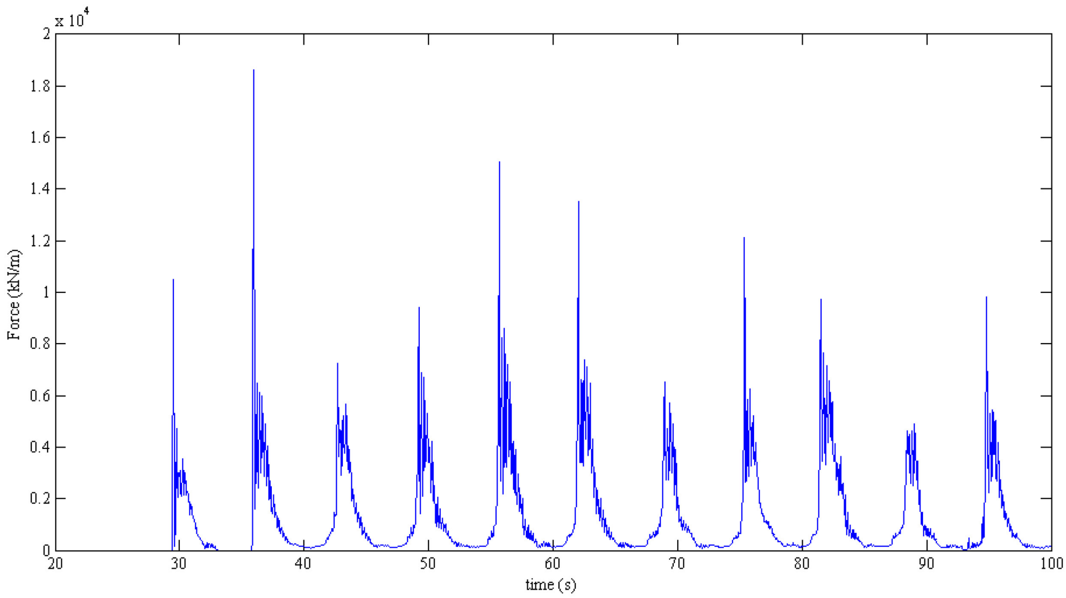


2.4.2 Wave forces on the storm wall

The structure has been reintroduced in the numerical model to calculate the forces exerted on the storm wall by the waves that overtop the dike crest. A total duration of 100 s (physical time) has been simulated. The number of fluid particles was equal to 226,696 using a dp=0.05 m. An Nvidia GeForce GTX 770 has been used for running the model, taking approximately 4.2 hours.

In the case of the regular waves, almost all waves overtop the dike and hit the wall, resulting in very high impacts (Figure 6). The maximum force is equal to 18.6 kN/m, in any case smaller than the force calculated in Altomare et al. (2015), which was equal to 28.5 kN/m. The new DualSPHysics functionalities, however, allowed simulating longer series than in Altomare et al. (2015), resulting in 11 impacts during 100 s simulation. Despite the fact that regular waves are generated, a certain variability of the wave force is present. This result is in agreement with Kisacik et al. (2012). The mean value has been calculated, the resulting value is equal to 10.5 kN/m, with a standard deviation equal to 3.9 kN/m.

Figure 6. Time series of forces on the storm wall for regular waves.



2.4.3 Wave pressures on the dike

Wave pressure and total forces on the vertical dike have been calculated. The seaward wall of the dike has been analysed by considering the parts below and above the horizontal slab separately (see Figure 4).

The total force time series and the pressure profiles on the bottom and top part of the vertical dike wall are depicted in Figure 7 and Figure 8, respectively. The hydrostatic solution is also represented. The pressure profile has been calculated at the moment when the wave crest reaches the dike. The pressure distribution resemble the hydrostatic one, with a relatively small increase due to the hydrodynamic component.

Figure 7. Horizontal force on the dike: below the horizontal slab (left) and above it (right)

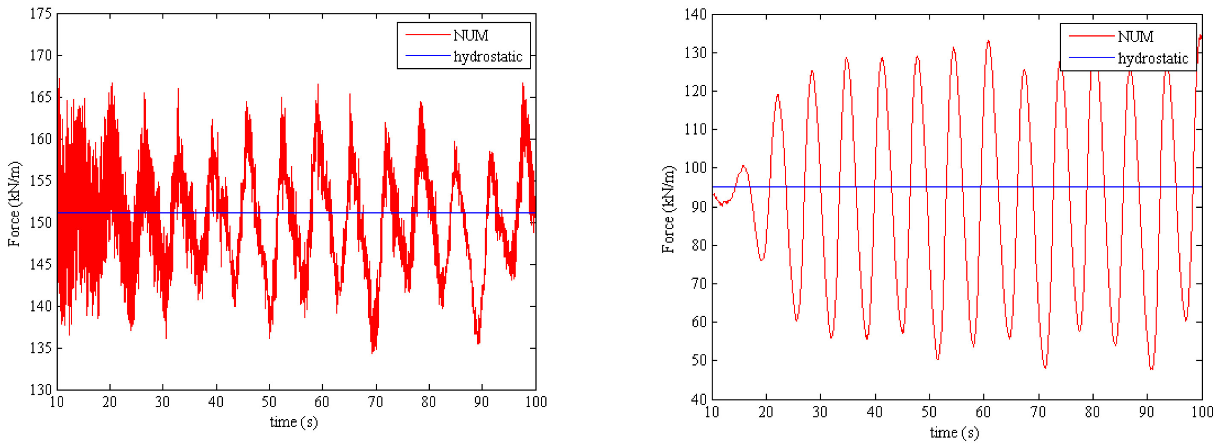
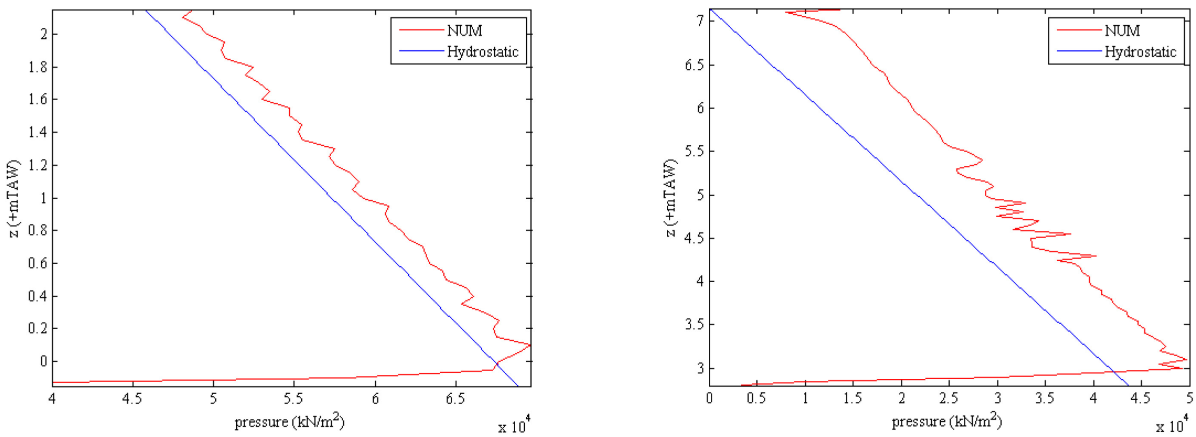


Figure 8. Pressure profile on the dike: below the horizontal slab (left) and above it (right)

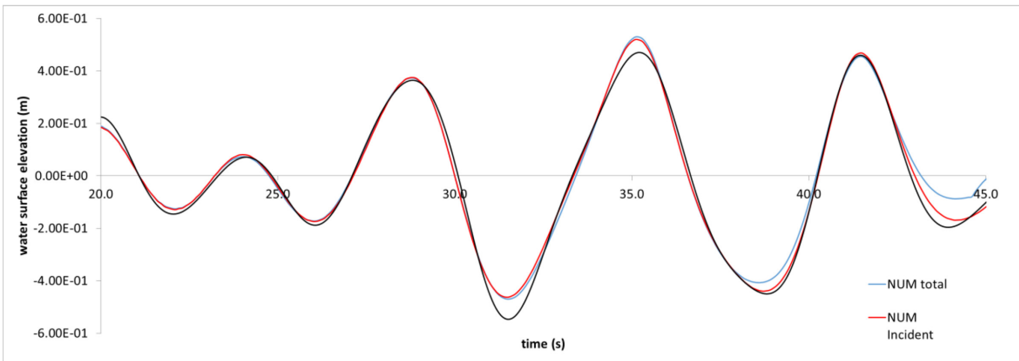


2.5 Irregular wave modelling

2.5.1 Incident wave analysis

Irregular waves have been generated in DualSPHysics using JONSWAP spectra (γ=3.3) and extracting a time series of 100 s out of the whole 1000-wave time series. The highest waves in the extracted time series are depicted in Figure 9. The numerical results is reported in terms of total and incident water surface elevation. The wave height of the target and generated maximum wave has been quantified. The wave height of the highest wave in the theoretical signal is equal to 1.01 m, meanwhile the numerical results give 0.99m (-1.98%). It must be noticed that the highest wave height is therefore equal to 1.94 H_{m0} while the simulated wave height for regular waves is 2.2 H_{m0}. The consequence of this difference will be discussed later.

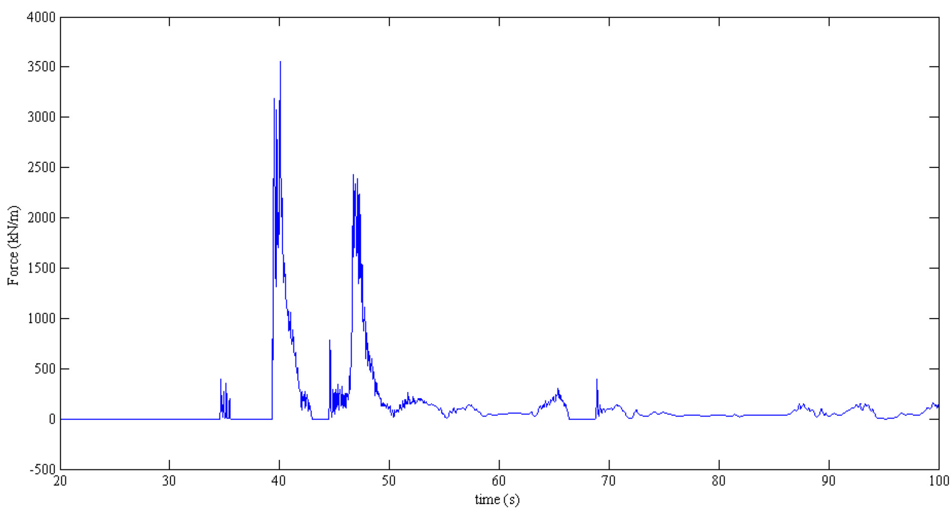
Figure 9. Comparison between the numerical and the theoretical water surface elevation for irregular waves.



2.5.2 Wave forces on the storm wall

When using irregular waves just two impacts occur during the selected time window (Figure 10). The maximum force is equal to 3.5 kN/m, 6 and 3 times smaller than the maximum and average force from the regular wave case. The reason of this difference has been further investigated (see later on).

Figure 10. Time series of forces on the storm wall for irregular waves.



2.5.3 Wave pressures on the dike

The total force time series and pressure profiles on the bottom and top part of the vertical dike wall are depicted in Figure 11 and Figure 8 Figure 12 respectively. The hydrostatic solution is also represented. The pressure profile has been calculated at the moment of where the wave crest reaches the dike. The pressure distribution resemble the hydrostatic one, with a relatively small increase due to the hydrodynamic component. The numerical solution is noisy due to the weakly compressible nature of the SPH method implemented in SPH, however the results are reasonably realistic.

Figure 11. Horizontal force on the dike for irregular waves: below the horizontal slab (left) and above it (right)

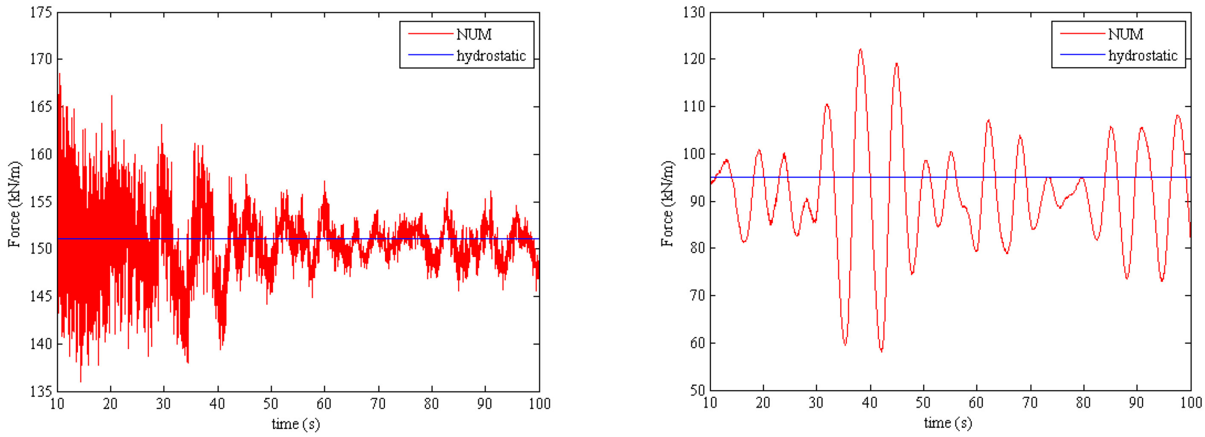
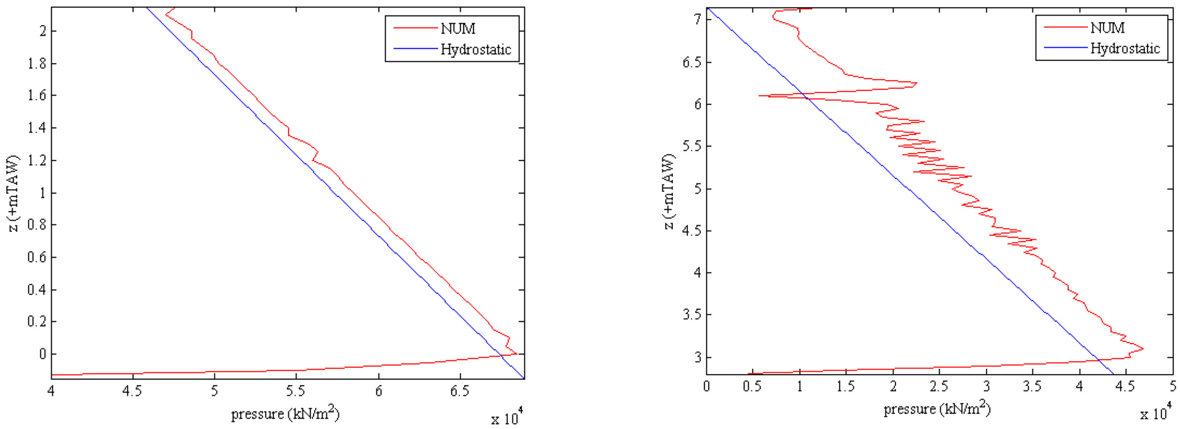


Figure 12. Pressure profile on the dike for irregular waves: below the horizontal slab (left) and above it (right)



3 Discussions and conclusions

Differences have been noticed between the results for regular and irregular wave in terms of wave forces on the storm wall (see Figure 6 and Figure 10). However, the global energy approaching the structure does not differ significantly. Proof of that is the force on the upper part of the vertical quay (left images in Figure 7 and in Figure 11). The regular wave case gives a force of around 130-140 kN/m, where the force corresponding to the highest wave from the irregular wave train is equal to 122 kN/m. This fact suggests that the differences on the storm wall are due to what is occurring on top of the quay, namely the wave overtopping, transformation and breaking mechanism.

Snapshots of the second and third impact for the regular wave case are depicted in Figure 14. Three instants of time for each impact have been represented. On the left side, the bottom picture represents the moment of the highest impact (F=18.6 kN/m), where the upper pictures represent the evolution of the wave running on the dike crest. Similar, the third impact (F=7.3 kN/m) is represented by the three pictures at the right side. The colours represent the velocity field, i.e. red stands for high velocities and dark blue for low velocities. The arrows are representing the pressure distribution. In general, the velocities that precede the second impact are higher than those preceding the third one. Besides, the residual water that is present at the toe of the wall just before the third impact interacts with the incoming flow reducing the water momentum. Each of the 11 impacts of the regular wave case presents differences such as those mentioned above. This explains the variability and non-repeatability of the forces. Alternatively, for the irregular waves case, only two waves are causing loading on the storm wall. The instant of the largest impact for the irregular wave train is shown in Figure 13.

Further analysis has been conducted, simulating a regular wave train that, instead of having a wave height H equal to $2.2H_{m0}$, is characterized by a wave height equal to the maximum wave from the irregular wave train, namely 0.99 m. Forces have been estimated, resulting in an average equal to 6.9 kN/m, therefore only twice as large than the irregular wave results. The standard deviation in this case is 3.3 kN/m.

Figure 13. Snapshot of the irregular wave case at the instant of the highest force on the storm wall.

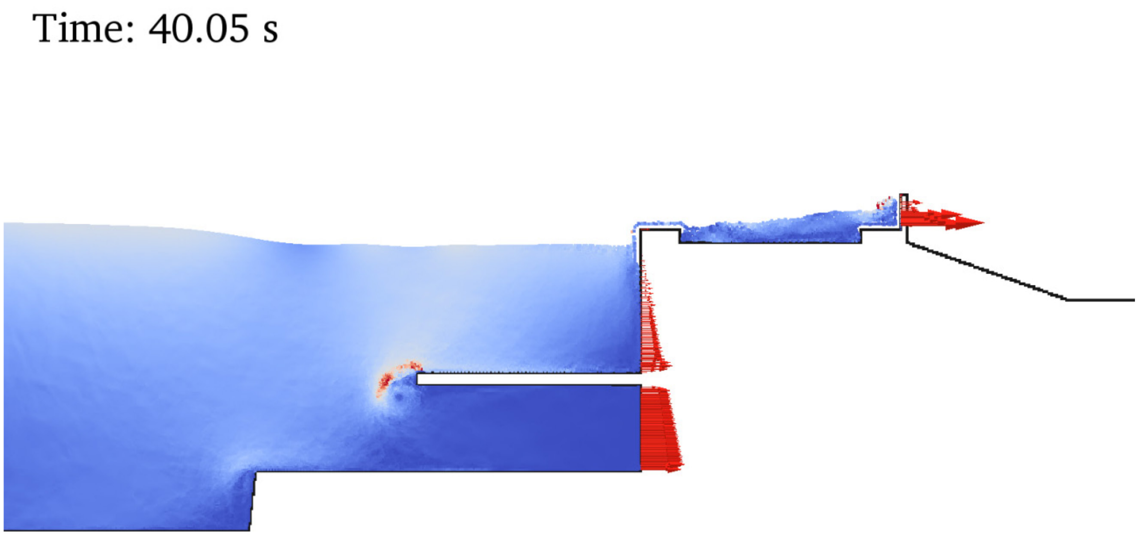
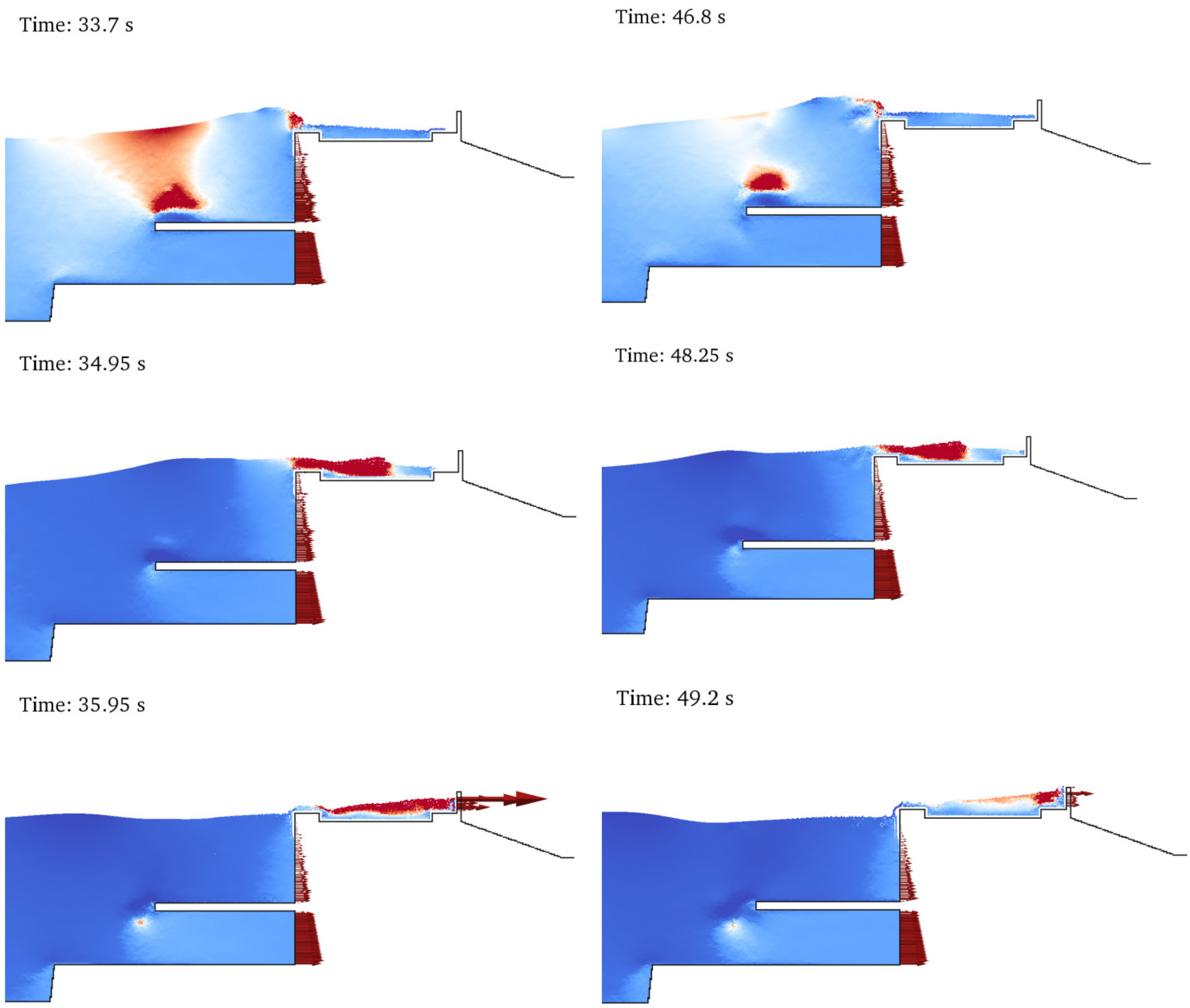


Figure 14. Snapshots of the regular wave case for the second and third impact on the storm wall.



Based on the aforementioned analysis, it is clear that adopting a methodology such as the one used in Altomare et al. (2015) is leading to very high values of the wave forces. Using these values will lead to a conservative design. Here, using irregular waves instead, results in smaller forces to be applied. For design purposes one should consider a safety factor on top of the irregular wave result, which has not been estimated for the present project. Differences might be noticed when the simulated time series are longer than 100 s as well. Lacking this information, the results from regular waves are recommended for design, also because they are consistent with the former methodology (Altomare et al., 2015).

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