MODELLING OF BYPASS OF SEDIMENT AT HARBOURS

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Abstract

The conditions for bypass of sediment and the sedimentation is studied by composite modelling where physical and numerical modelling have been used to support and complement each other. A harbour placed on an exposed straight coastline with a slope of 1:25 is considered. The mathematical modelling is made by the area model system MIKE 21. The physical modelling is made in a 25 m by 35 m wave basin with directional wave generators along one side of the basin. The sediment transport is illustrated by fine sand acting as tracer over a hard concrete bed; this illustrates the bypass of sediment and the intrusion into the harbour. Further, experiments are made with a moveable bed over part of the model domain. The physical model provides details on the flow and the wave field, on the areas of sedimentation and the mechanisms for bypass of sediment. The numerical models run in parallel to the physical modelling simulating the detailed flow and wave patterns and are used to provide the basis for determining the extent of the model area and the scale of the physical model and defining the proper boundary conditions for the physical model.

1 Introduction

A harbour on a coast blocks the littoral drift causing accretion and erosion at the up- and downdrift side, respectively. Eventually a significant part of the littoral drift may bypass the harbour. In this case it is important to minimize the deposition of the passing sediment in the harbour basin and the harbour mouth. Different measures have been proposed to counter sedimentation problems associated with sediment bypass. A solution is to design the protective works of the harbour to promote the bypass of sediment by making the harbour mouth face the incoming waves and streamlining the breakwaters. This increases the flow velocity and transport capacity past the harbour mouth due to contraction.

This study has investigated the conditions for bypass of sediment and the associated sedimentation by application of composite modelling where physical and numerical modelling has been used to support and complement each other. A generic example was considered: a harbour placed on an exposed straight coastline exposed to oblique waves. The protective works of the harbour consist of two curved rubble mound breakwaters located inside the surf zone.

2 Physical modelling

2.1 Definition of the physical model layout and boundary conditions

The basin host of the physical model is DHI's shallow water basin which is 35m long, 25m wide and 0.8m deep. An 18m long 3D wavemaker is fixed along one side of the basin generating irregular, multidirectional waves. Two long-crested wavemakers were also used: these 2D movable wavemakers are 5.5m long and generate irregular, unidirectional waves. The shallow water basin is also equipped with a pump system able to supply a discharge of maximum 1.2m^3/s.

An alongshore uniform bathymetry defined by constant slope of 1:25 was chosen. This slope characterises a dissipative beach. In order to produce a significant longshore sediment transport, waves were generated obliquely to the coastline with an angle of 25° at the offshore boundary of the model. The significant wave height was set to 8m, and the peak wave period was 12s, corresponding to a storm event. Note that the unidirectional wavemaker was rotated 25° compared to the 3D wave generator system to create the desired offshore wave direction (Fig. 1).

The harbour had to be placed far enough from the inflow boundary in order to obtain a good cross-shore distribution of the fully developed current magnitude. To determine this location, a numerical model including the physical constraints and boundary conditions described above was run against a periodic model representing an idealised solution. It was established that the wave driven current fully develops after 2,500m. Due to the dimension of the basin, a model scale of 1:100 was defined and consequently, the harbour was located 25m from the inflow boundary of the basin. As the integrated longshore flux from the numerical model yields a discharge on the order of 10,000m^3/s, a pump supplied a discharge of 0.10m^3/s at the inflow boundary; at the outflow boundary, water flows out over a weir. In the following, all model results are given in prototype scale, hence, as shown in Fig. 1, the harbour is located at x=1,000m.

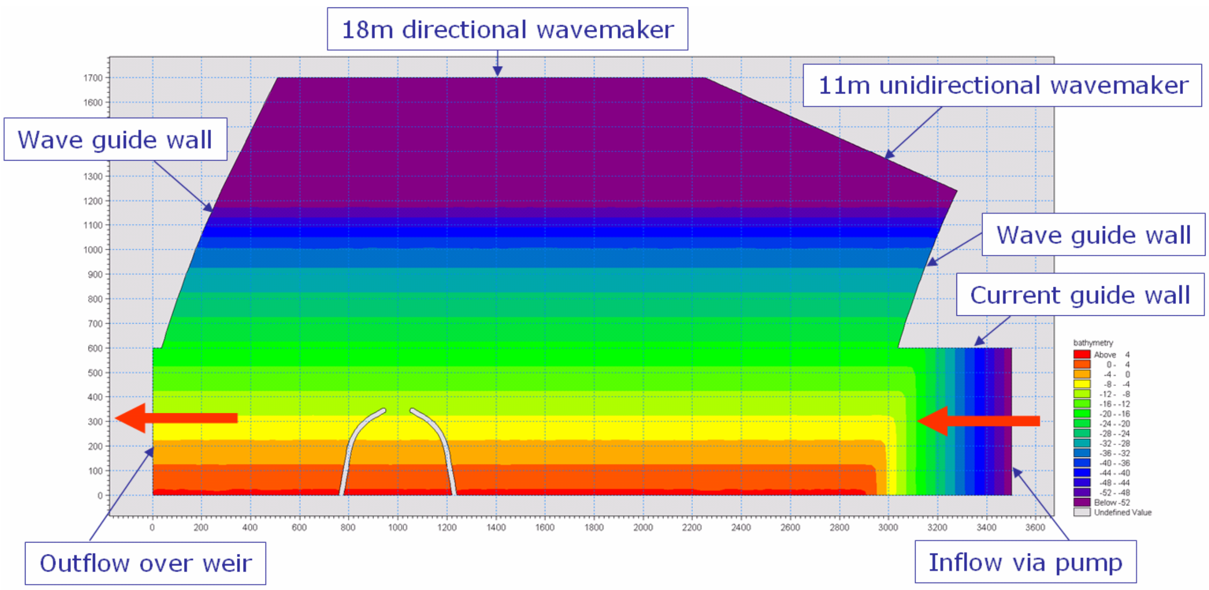

Fig. 1. Schematic layout of the physical model.

Different harbour layouts have been tested to study the influence of the harbour configuration on the sediment bypass. These are based on the angle between the two breakwaters at the harbour entrance as defined on Fig. 2: three angles have been tested corresponding to 20°, 40° and 60°.

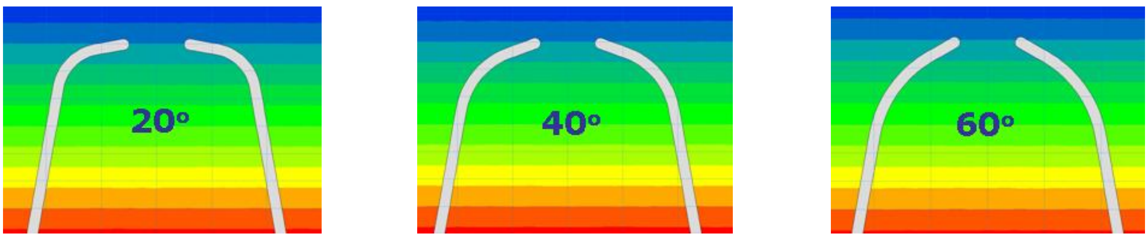

Fig. 2. Harbour configurations based on the angle between breakwaters.

Fig. 3. DHI's shallow water basin with wavemakers (left) and harbour layout (right).

2.2 Set-up of the morphological experiments

Two types of morphological experiments were carried out in the physical model: the first set of experiments was sand tracer tests on a concrete bed and the second experiment was a moveable bed test. The aim of the sand tracer tests was firstly, to check that the scaled model was able to bypass sediment with the applied boundary conditions and, secondly, to quantify this bypass for different harbour configurations. The aim of the moveable bed test was to study the morphological evolution of a sandy bed under constant hydrodynamic conditions. The applied sediment was fine sand with the lowest available mean diameter of 0.16mm.

The tracer sand was introduced upstream of the harbour at $0.025m^3$/hr and was added gradually in order to avoid any abrupt changes in the hydrodynamics as a result of sudden bathymetrical changes. A tracer experiment was considered finished when all the added sediment had passed the harbour entrance; the duration was 2 hours. At the end of each experiment, the sand deposited in the harbour was collected through a vacuum pump and dried so that it could be weighted.

A moveable bed experiment was then created by adding a 5cm layer of sand on top of the concrete bed over part of the model domain. In order to conserve the same depth at the harbour entrance, the breakwaters were extended accordingly. The above-defined forcing conditions were run for a total of 32 hours divided in 4-hour intervals. The thickness of the sand layer was measured in cross-shore profiles every 4 hours and the upstream boundary area was subsequently refilled at an average rate of $0.10m^3$/hr such as to cover the eroded area and maintain a regular input of sand.

2.3 Results of the physical modelling

The sand tracer results showed little sensitivity to the breakwater angle: the sedimentation inside the harbour was on the order of 4-5% for all the breakwater layouts. This is in good accordance with the value of 7% derived for the exchange of sand passing the harbour mouth (Dursthoff, 1970). Note that the deposition pattern inside the harbour resulted from the presence of two circulation cells (Fig. 4).

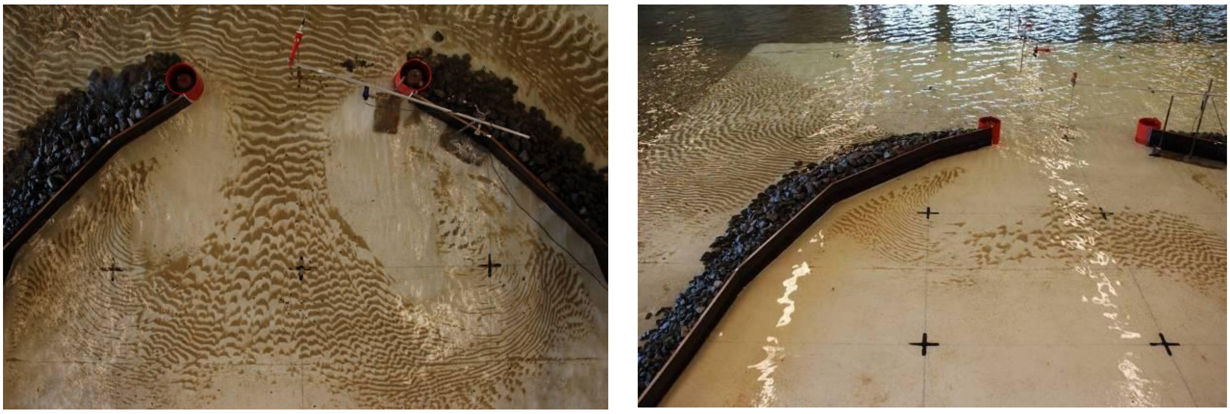

Fig. 4. Tracer evolution after 1 hour (left) and at end of experiment at 2 hours (right).

The main results of the morphological evolution of the moveable bed experiment (Fig. 5) are an upstream sedimentation, the creation of a bypass bar in front of the harbour entrance and downstream erosion. An equally important observation is the creation of an alongshore bar at all cross-shore locations. Out of the additional $4.3m^3$ of sand provided during the experiment at the upstream boundary, a majority was transported offshore: approximately 65% was transported in the offshore bar, 12% was impounded at the upstream breakwater and the remaining 23% bypassed the harbour.

Fig. 5. Evolution of the moveable bed experiment: upstream sedimentation (left) and bypass bar with downdrift erosion (right).

3 Numerical modelling

The numerical modelling was carried out with the MIKE 21 FM series developed by DHI using the SW, HD and ST modules for the simulation of wave propagation, hydrodynamics and sediment transport, respectively.

3.1 Numerical verification of the hydrodynamics of the physical model

The cross-shore distribution of the significant wave height and the wave-induced longshore current were measured at several locations during the experiments; the measured versus the modelled results presented on Fig. 6 show a good accordance of the wave height decay in the surf zone and a satisfactory fit of the longshore current is also observed.

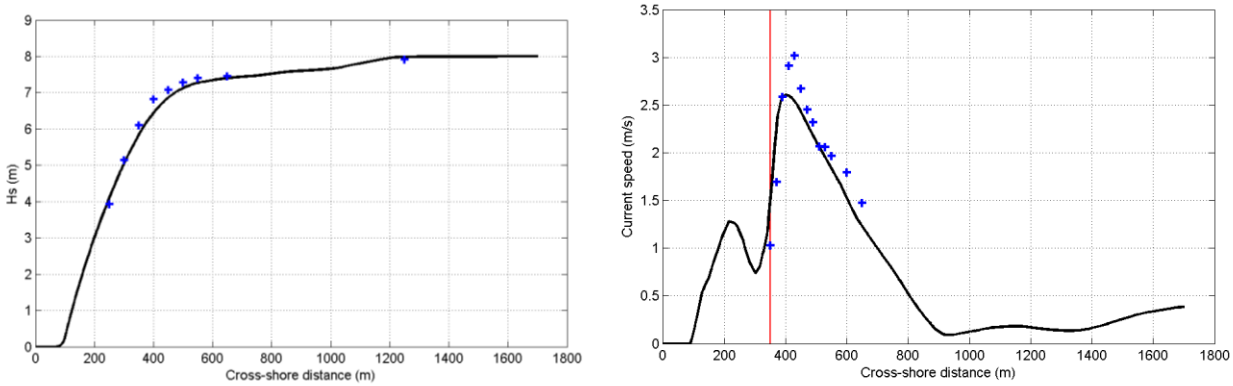

Fig. 6. Measured (blue crosses) versus modelled (black line) cross-shore distribution of wave height (left) and longshore current velocities (right) at x=1,700m and x=1,000m, respectively.

The increasing flow velocities past the harbour mouth due to the flow contraction is clearly illustrated in Fig. 7. Inside the harbour, the current field is characterized by two flow recirculation cells directed towards the breakwater arms which is in good agreement with the observed sediment transport pattern in the physical model (see previous Fig. 4).

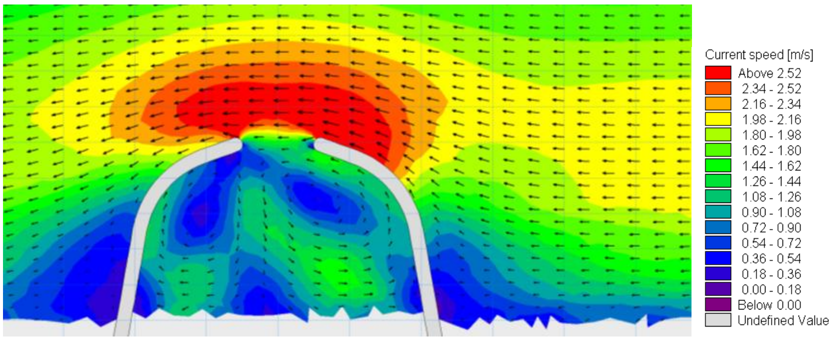

Fig. 7. Simulated flow field in the vicinity of the harbour (here shown with 40° between breakwaters).

3.2 Sediment transport scale effects

Recall that sediment characterised by a mean grain diameter of 0.16mm was used in the physical model. With the chosen 1:100 scale of the physical model, in order to obtain the right sediment transport scale, a grain diameter of 16mm should be used in the numerical model. With a corresponding fall velocity of 0.42m/s, the simulated morphological evolution with 16mm uniform grains was soon found to be to slow. Another possibility was to scale the fall velocity of the sediments. Under normal conditions, uniform sediments defined by a diameter of 0.16mm have a fall velocity of 0.017m/s. Thus, a 16mm grain size should have a fall velocity scaled by the Froude model law of 0.17m/s but this value is far beyond the input parameters defined in the sediment transport model. As a result, sediment characterised by the same properties as in the experiments has been used in the numerical modelling of sediment transport.

The moveable bed experiments yielded a transport rate on the order of 0.10m^3/hr resulting in a complete erosion of the movable bed over a length of approximately 3m in 4 hours – with a model scale of 1:100, the prototype erosion would then be 400,000m^3. It was chosen to calibrate the numerical model against this result. It has to be pointed out that as sediment properties are the same in the physical and the numerical model, morphological changes from the numerical model will evidently develop faster than in the physical model. Consequently, the simulation time defined in the numerical model has not been scaled by a factor of 10. Numerical results were in fact extracted at a simulated time where the erosion of the movable bed at the upstream boundary is on the order 400,000m^3. This corresponded to simulated time of approximately 12 hours instead of the scaled 40 hours.

3.3 Simulated morphological results

The modelled spatial distribution of the tracer sand is illustrated in Fig. 8. The results from the physical and numerical models both indicate that nearly all the sediment added upstream (red area in Fig. 8) is transported downstream of the harbour; recall that a significant bypass of sediment was found to be on the order of 95% in the physical model. In the numerical model, an even smaller quantity of sand settles in the harbour: the bypass is found to be approximately 98% for all three considered breakwater angles. It has to be noted that the description of vortices close the harbour mouth, wave groups pumping sand inside the harbour and wave orbital motion are not taken into account in the numerical model. These phenomena would have the effect of bringing more sediment into the harbour which can explain such low percentages of sand trapped inside the harbour compared the results from experiments. Also note that for both models (compare Fig. 8 with Fig. 4), the current downstream of the harbour does not bend significantly in the onshore direction which induces that the tracer deposits mainly offshore.

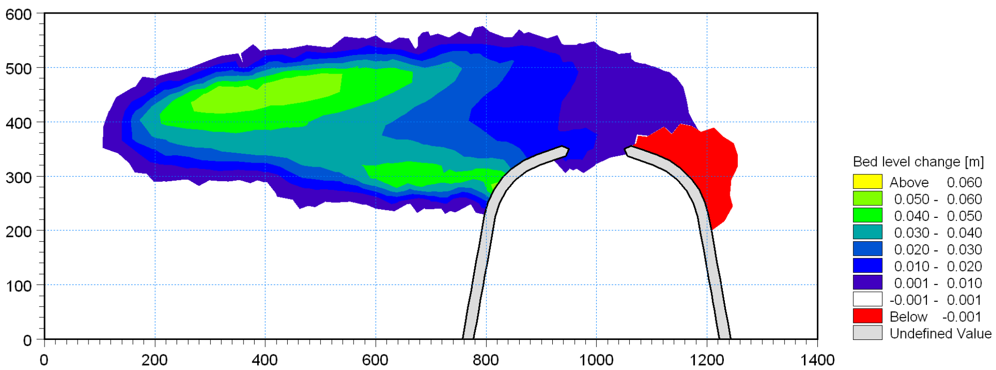

Fig. 8. Modelled spatial distribution of the tracer sand (40° between breakwaters).

For each of the eight tests representing the evolution of the bathymetry every 4 hours until 32 hours (in physical model time), the bed level changes were extracted at time steps where approximately 300m of erodable bed at the upstream boundary had eroded about 400,000m^3. In the following, the modelled versus the measured morphological evolution is synthesised. Results taken arbitrarily from t=0 to 4 hours are presented hereafter: the initial sediment transport field (Fig. 9), the modelled bed level evolution (Fig. 10) and a close-up of the modelled and measured bed level evolution around the harbour (Fig. 11).

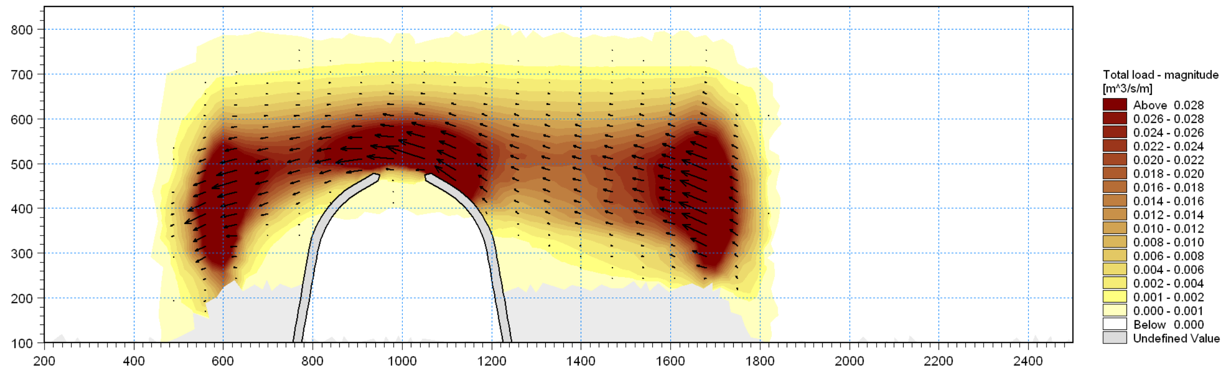

Fig. 9. Modelled sediment transport field at t=0 hr.

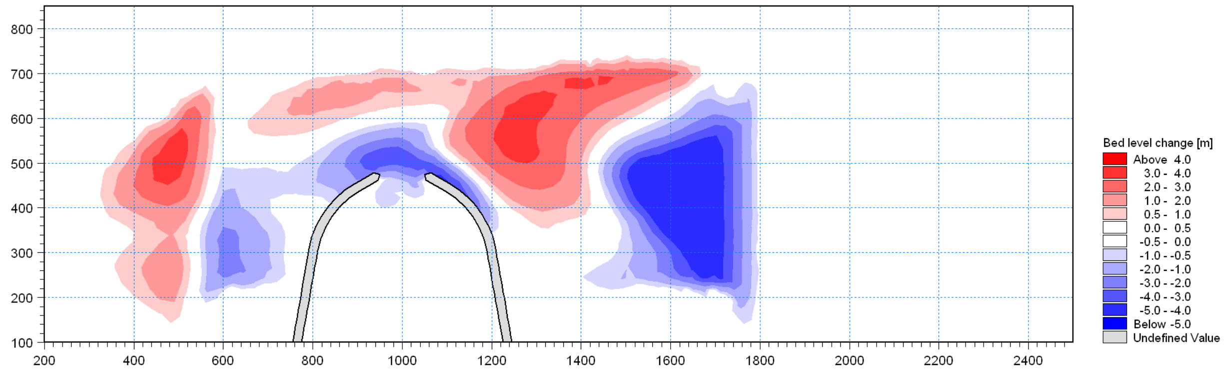

Fig. 10. Modelled bed level changes between t=0 and 4 hrs.

Due to the sudden decrease in water depth at the transition between the non-erodable and erodable bed (initially located at x=1,800m), the longshore current is strongly accelerated resulting in a large erosion at the upstream boundary. This phenomenon is immediately followed by a decrease of current magnitude inducing a positive gradient in sediment transport and thus the accretion of the area further downstream. This accretion rate is approximately 3.5m/40hrs at the beginning of the tests and it tends to decrease to 2.5m/40hrs towards the end at 32 hours; this is valid for both models.

The presence of the harbour induces the acceleration of the flow in its vicinity, generating a gradient in sediment transport and thus the erosion of the bed. This erosion evolves with time in both numerical and physical model. Between 0 and 4 hours, it is located upstream of the harbour and at its mouth with a rate of 3-4 m. It then moves slightly downstream and decrease in the flow direction until 16hours. Rates of erosion for the two models are approximately the same during this period although some variations are experienced at later stage towards the 32 hours.

From the first hours of simulation, a bar is created in front of the harbour mouth at y=650m for the numerical model while it is located further onshore for the physical model. Its width is about 75 m and its growth rate is 1m/40hr and 2m/40hr for the numerical and physical model, respectively. Due to the presence of the bar, undertow generated by waves breaking on the bar, results in the offshore migration and width increase of the bar. The development rate of the bar tends to decrease in the experiments case while it is constant for the numerical model.

Downstream of the harbour, the physical model seems to generate a current which is more bent toward the onshore resulting in the erosion of the downstream part of the harbour, whereas with the numerical model, this erosion is much less.

These results indicate that the tendency predicted by the numerical and physical models are quite similar, the patterns seems however more pronounced with the numerical model. Both models show a morphological development whose intensity decreases slightly with time and tends to equilibrium.

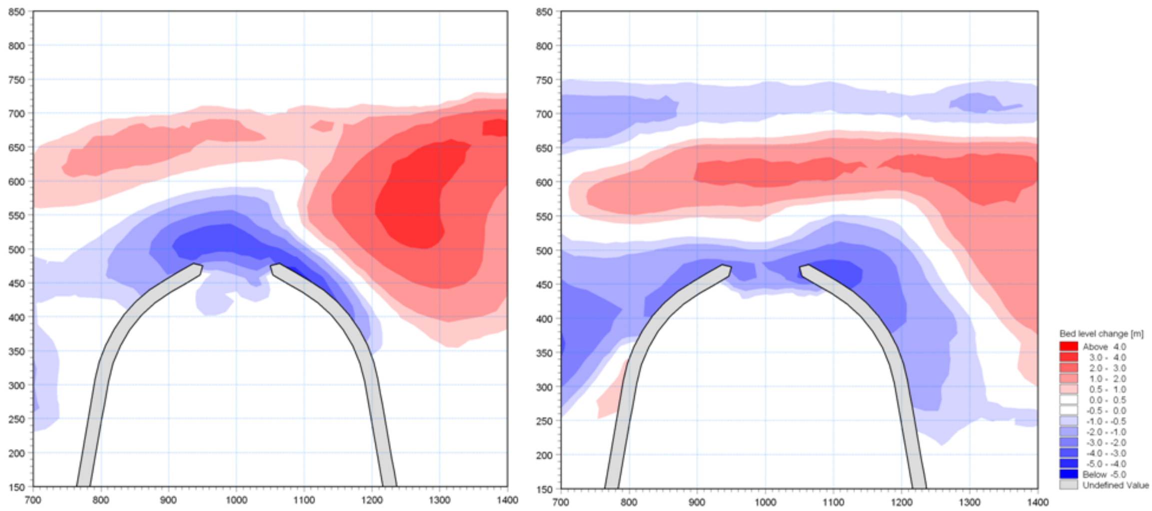

Fig. 11. Close-up of modelled (left) and measured (right) bed level change between t=0 and 4 hrs.

Conclusions

Taking into account restraints related to available facilities, the definition of the physical model layout was successfully optimized by the initial use of numerical modelling. The physical model predicted a sedimentation rate in the harbour of about 5% corresponding to the value of 7% derived on the basis of exchange of passing sediment. While inducing a significant bypass of sediment in the test basin, the flow acceleration in front of the curved breakwaters also maintained a depth along the bypass bar.

Two critical aspects in the design of harbour breakwaters are the navigational depth in front of the harbour and the amount of sediment entering the harbour basin. The good reproduction of the simulated morphological evolution supports that bypass mechanisms are well included in the mathematical model. Numerical modelling alone is thus well-suited to design the configuration of harbour breakwaters.

Acknowledgments

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