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Development of a high-resolution 2D hydrodynamic model
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Sub report 3 – Development of a high-resolution 2D hydrodynamic model for the Lower Sea Scheldt

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Abstract

In the present study, a detailed TELEMAC2D hydrodynamic model is developed for the modelling of three scenarios of interventions at the area of the INEOS facilities, i.e. the deepening of the existing jetty and a new quay wall construction (two variations). The main objective is to provide insight into the future maintenance requirements by making use of the numerical modelling results. Apart from this, the present study focus on how selected numerical and physical parameters (grid formation, bed roughness, turbulence) can affect the performance of the numerical model when simulating the hydrodynamics of tidal estuaries, through a series of sensitivity tests. The effect of local grid adaptation on the numerical diffusion and consequently on the results is further discussed. For the model's calibration and validation, ADCP transect measurements, single-point velocity measurements and water level measurements at different stations in the Lower Sea Scheldt are utilized. Numerical results on the effect of the investigated interventions on the flood and ebb currents, and on the sediment transport at the area of interest, are presented.

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1 Introduction

INEOS Oxide has constructed a jetty for tankers at the left bank of Scheldt (close to Punt van Melsele Figure 1). To optimize the capacity of the jetty, INEOS wants to make the inner side of the jetty accessible for inland vessels (Figure 2). At the moment, this inner side of the jetty is barely accessible, if not at all, due to the natural topography of the shore. The possibility of constructing a new container terminal approximately 300 m downstream of the existing jetty is also investigated (Figure 4). To this end, two variations are being investigated even further: one with a completely closed sheet pile wall and another one in which the front part of quay deck will be supported by piles.

To improve the accessibility of the existing jetty, INEOS considers to realize a deepening of the inner side of the jetty. To prevent the deepening works from any possible instability of the river bank, the dike will be reinforced by a sheet pile wall, as shown in Figure 3. After the deepening works, it is expected that the excavated area will be subject to sedimentation and therefore maintenance dredging works will have to be carried out at regular intervals. The present study aims to provide insight into the future maintenance requirements and nautical accessibility by means of numerical modelling. Specifically a detailed TELEMAC-2D hydrodynamic model was developed, the so-called BZS model, for the modelling of the considered scenarios of interventions, i.e. the deepening of the existing jetty and the new quay wall construction. Apart from this, the present study gives an insight of how numerical and physical parameters (grid formation, bed roughness, turbulence) can affect the performance of the numerical model when simulating the hydrodynamics of tidal estuaries.

The morphological evolution of the area of interest has been investigated by use of the available surveys and dredging data (source: Maritime Access, MDK-Flemish Hydrography and Port of Anwerpen) and reported in a separate sub-report (De Maerschalck *et al.*, 2020). The nautical accessibility of the inner side of the jetty, in relation to a possible installation of a downstream container terminal, has also been investigated and reported in another sub-report (Verwilligen *et al.*, 2020). In both studies the BZS model results are used to interpret and analyse the effects on morphology and accessibility.

Figure 1 – Location of INEOS-South jetty.



Figure 2 – Focus on the inner side of INEOS-South jetty.

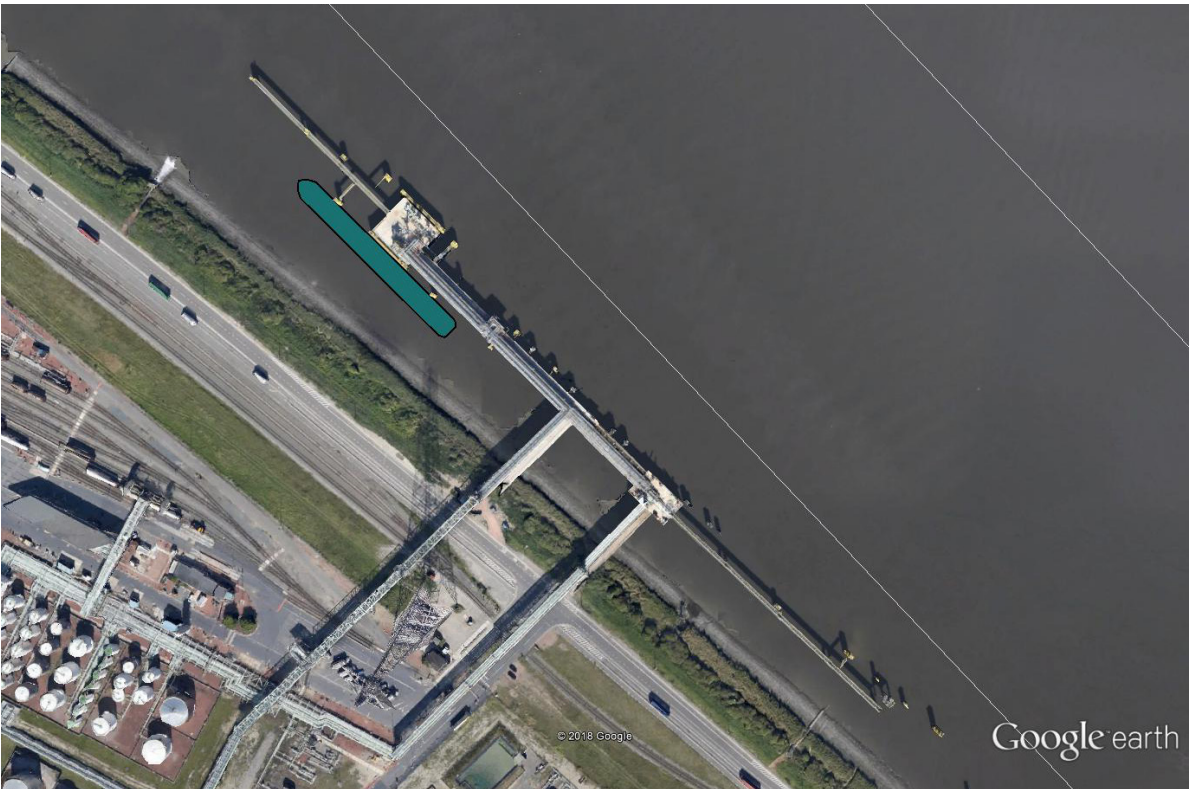


Figure 3 – Plan view of the dredging works (red shaded area) and the sheet wall (blue line) [top figure];
Cross-section (A-A) of the dredging works and the sheet wall [middle figure];
Cross-section (D-D) along the dredging works [bottom figure].

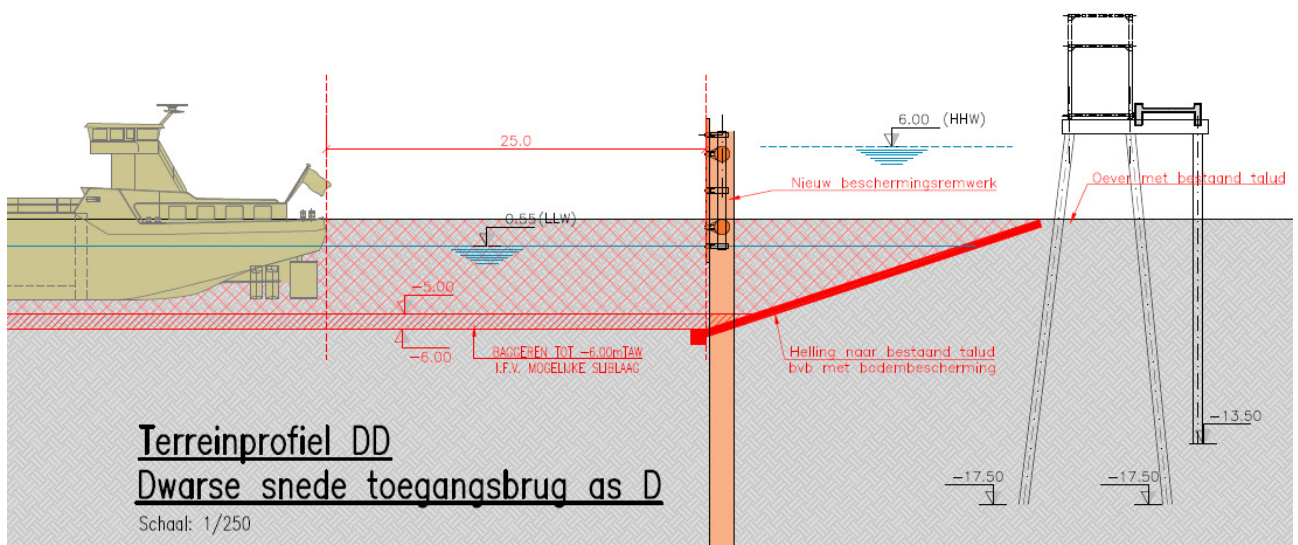
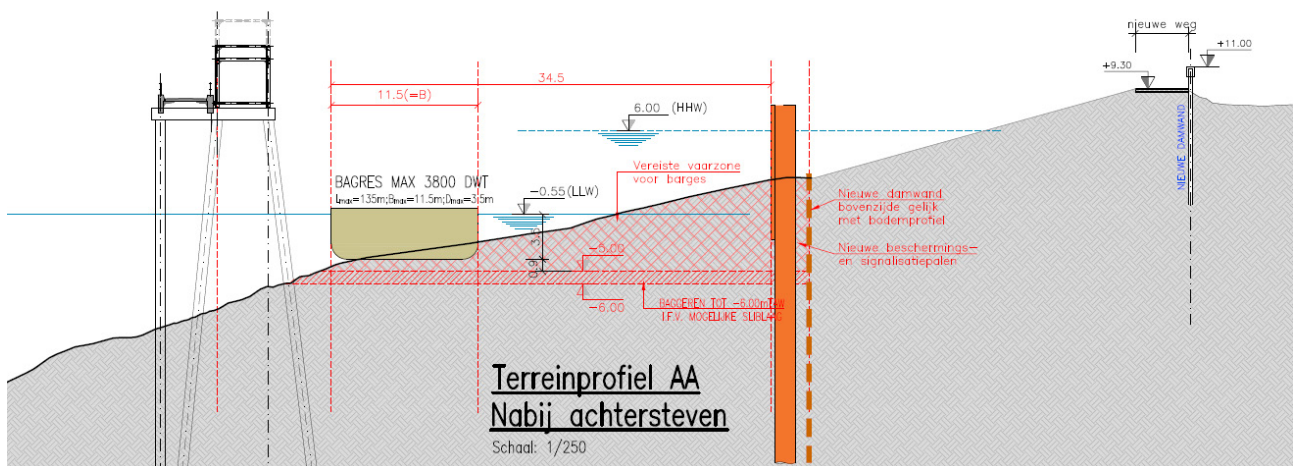
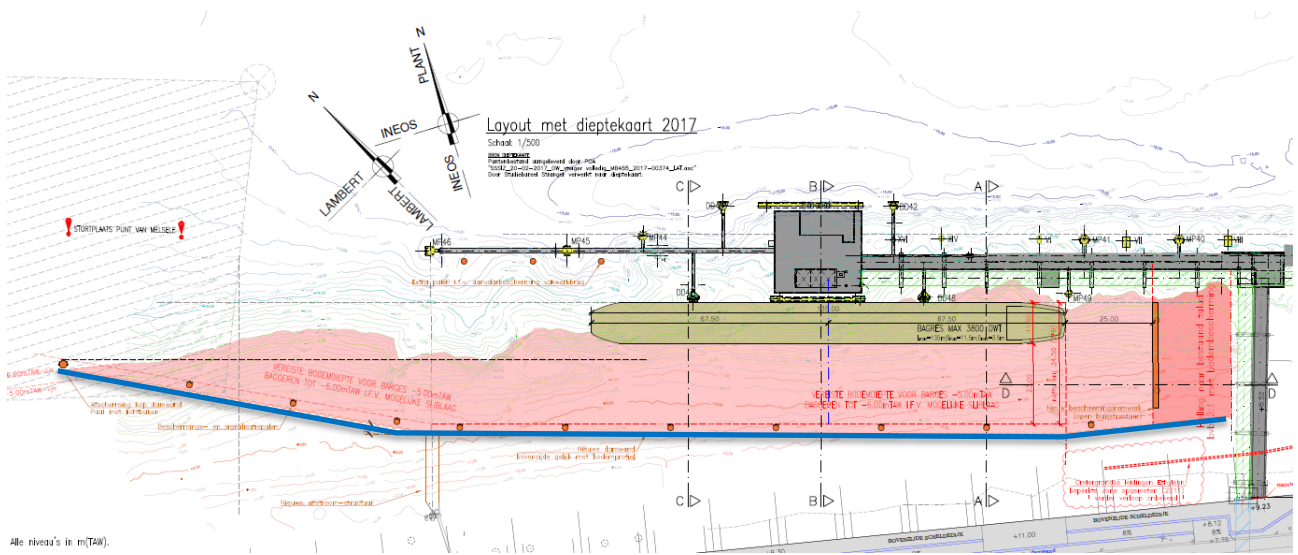
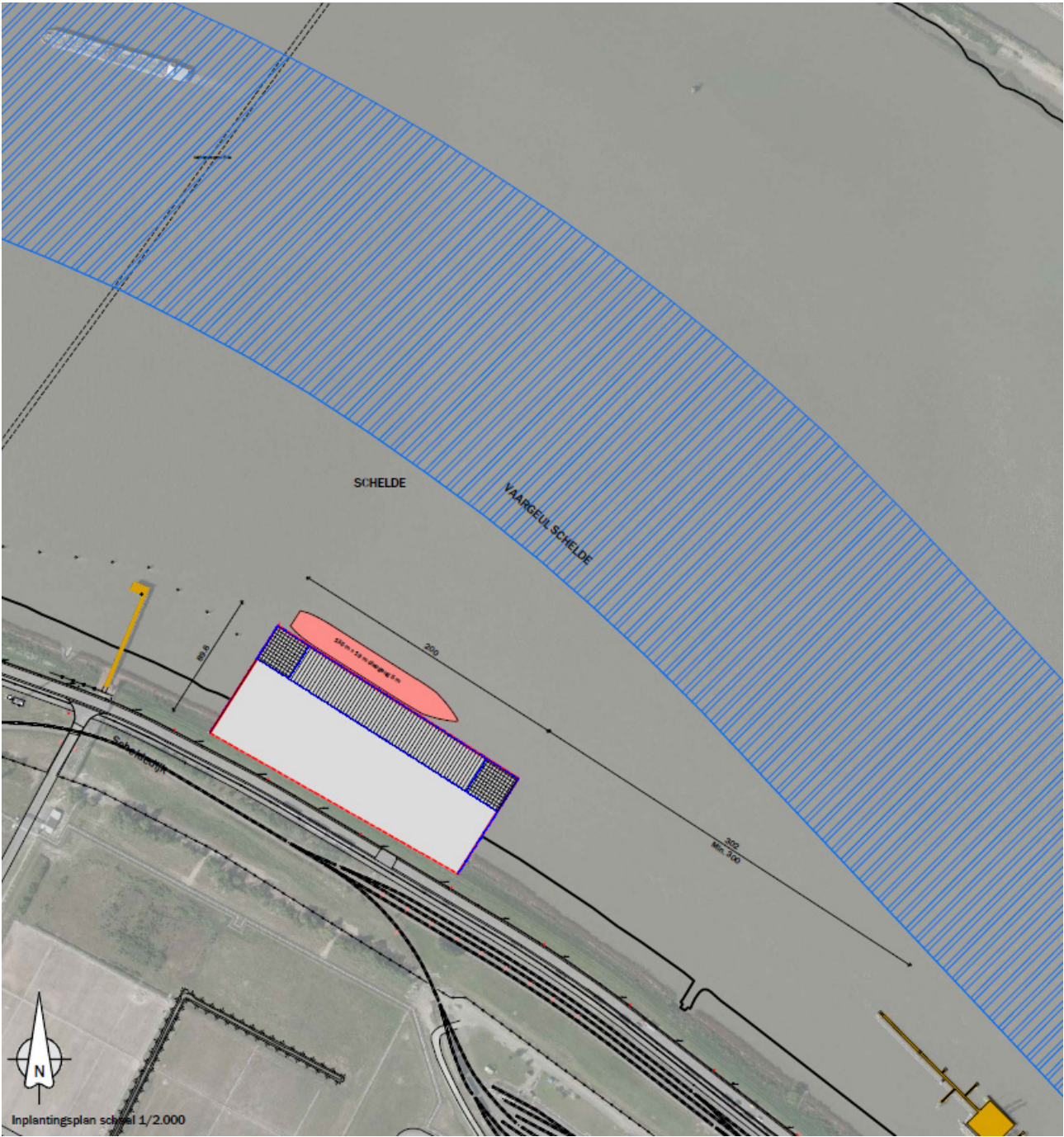


Figure 4 – Sketch of the new container quay.



2 Model setup

The hydrodynamic simulations presented in this report are performed by means of TELEMAC-2D software (Hervouet, 2007), which solves the shallow water equations (Saint-Venant) on unstructured triangular meshes using the finite element method. TELEMAC software is capable of performing computations on adaptive meshes consisting of elements of various sizes making it suitable for complex geometries like in our study area. It also allows for efficient parallel computing of high scalability, carrying out ‘heavy’ computations in reasonable computational times.

2.1 Computational grid

2.1.1 Outline

The outline of the BZS model, shown in Figure 5, is based on that of the existing Scaldis model reported in Smolders *et al.* (2016). The drawing of the outline has been performed by taking the following steps:

- The existing Scaldis outline was cropped transversally at the locations of Liefkenshoek and Hemiksem measurement stations. These two locations were selected as the open boundaries of the BZS model so that the latter could be forced by the tidal flow measured at the corresponding stations.
- The outline was projected on the latest Google earth satellite image and updated where necessary, e.g. new constructions were included. Detailed tracking of the land boundaries for the more accurate representation of the most complex geometries also took place.
- The polders included in the existing Scaldis outline (see yellow lines in Figure 5) were excluded from the BZS model domain.

2.1.2 Grid construction method and main features

The computational grid is constructed by use of the finite element grid generator GMSH (Geuzaine & Remacle, 2009). An adaptive mesh refinement is necessary in order to achieve more accurate predictions at the area of interest and at complex geometries of the outline. The local adjustment of the mesh density locally is achieved by using the available options of GMSH (Size Fields) for the prescription of the characteristic size of the elements. The distribution of the element sizes over the BZS domain were based on the following basic considerations:

- A maximum size equal to 20 m was considered as the basic size of the elements of the inner computational domain.
- A medium size equal to 10 m was considered along the closed boundaries of the domain.
- A minimum size of 5 m was considered at an extensive area around the area of INEOS jetty and at complex geometries.

A typical grid structure considered in the calibration tests is shown in Figure 6, where part of the BZS domain is depicted. A grid configuration consisting of an unstructured mesh combined with a channel mesh (structured triangular grid) aligned with flow direction was also considered (Figure 7). The outline of the channel mesh in the BZS domain is shown in Figure 9. For the channel mesh the minimum element size of 5 m was more or less preserved in the transverse direction, while the size was increased to 10-15 m in the streamwise direction, reducing substantially the total node number.

The selection of the optimal grid configuration was based on the results of the sensitivity tests presented in Section 3.4.2.

Figure 5 – Outline of the BZS model (red line). Yellow lines denote the outline of the existing Scaldis model (Smolders *et al.*, 2016).

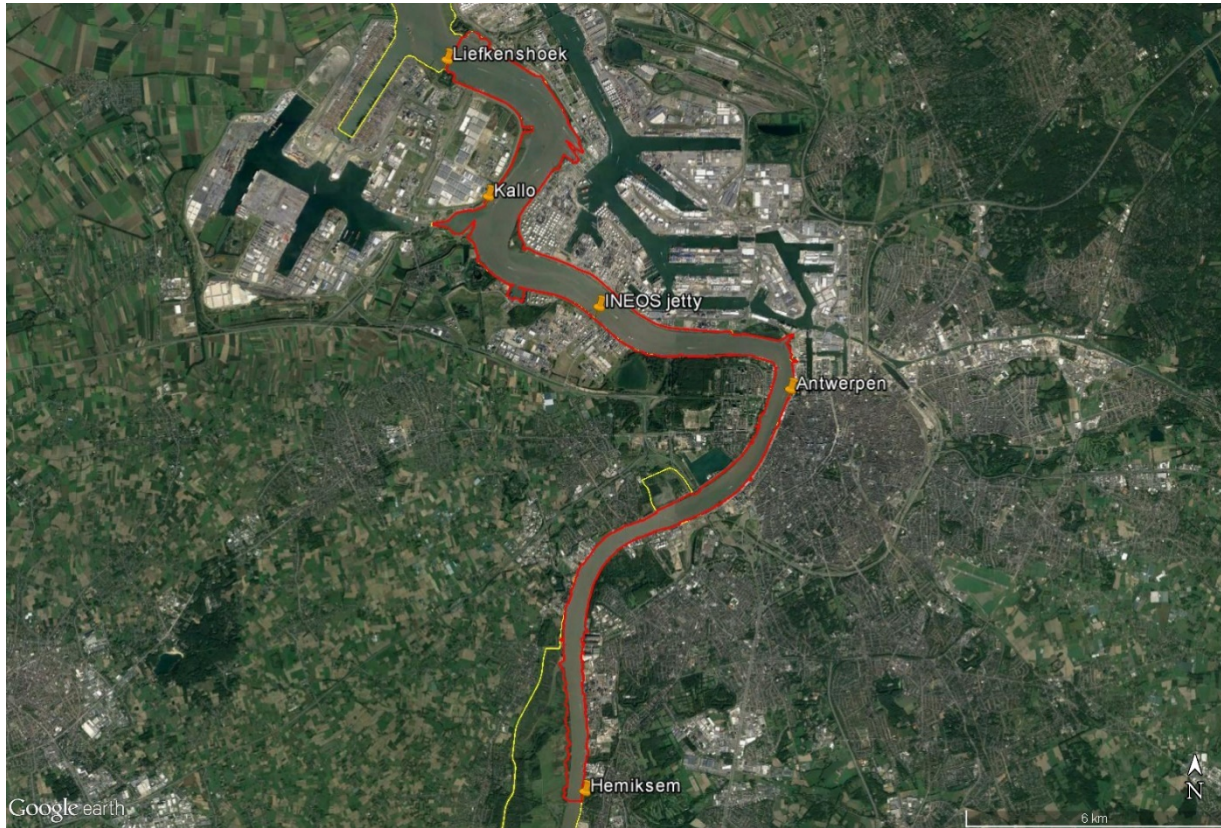


Figure 6 – Characteristic element sizes of the computational grid after adaptive mesh refinement using GMSH software.

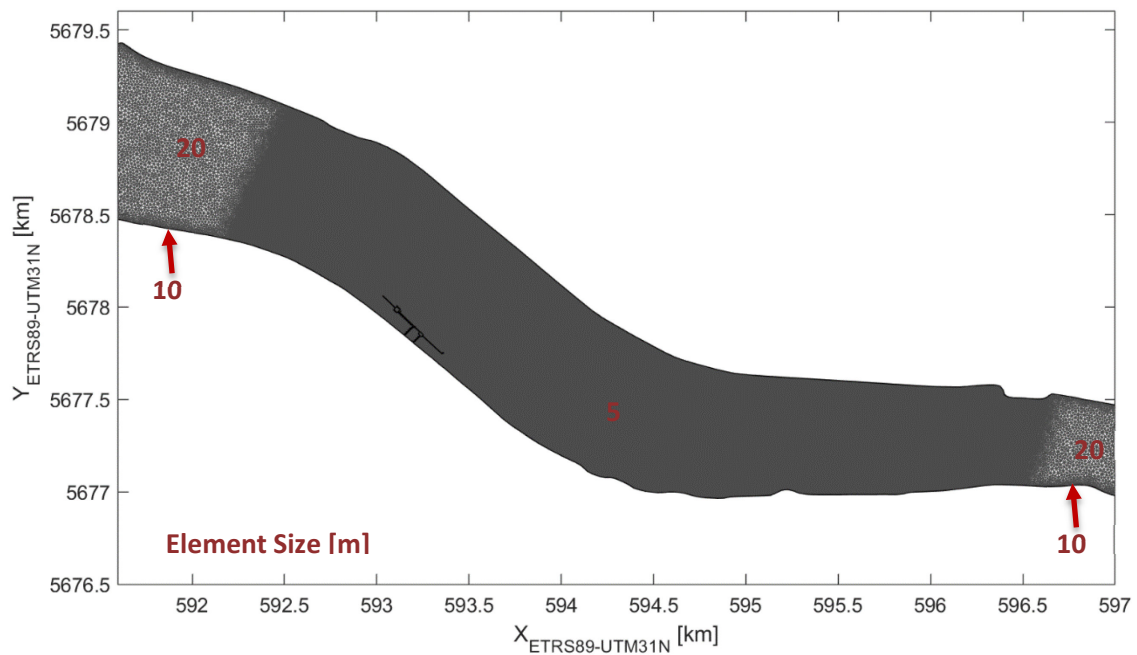
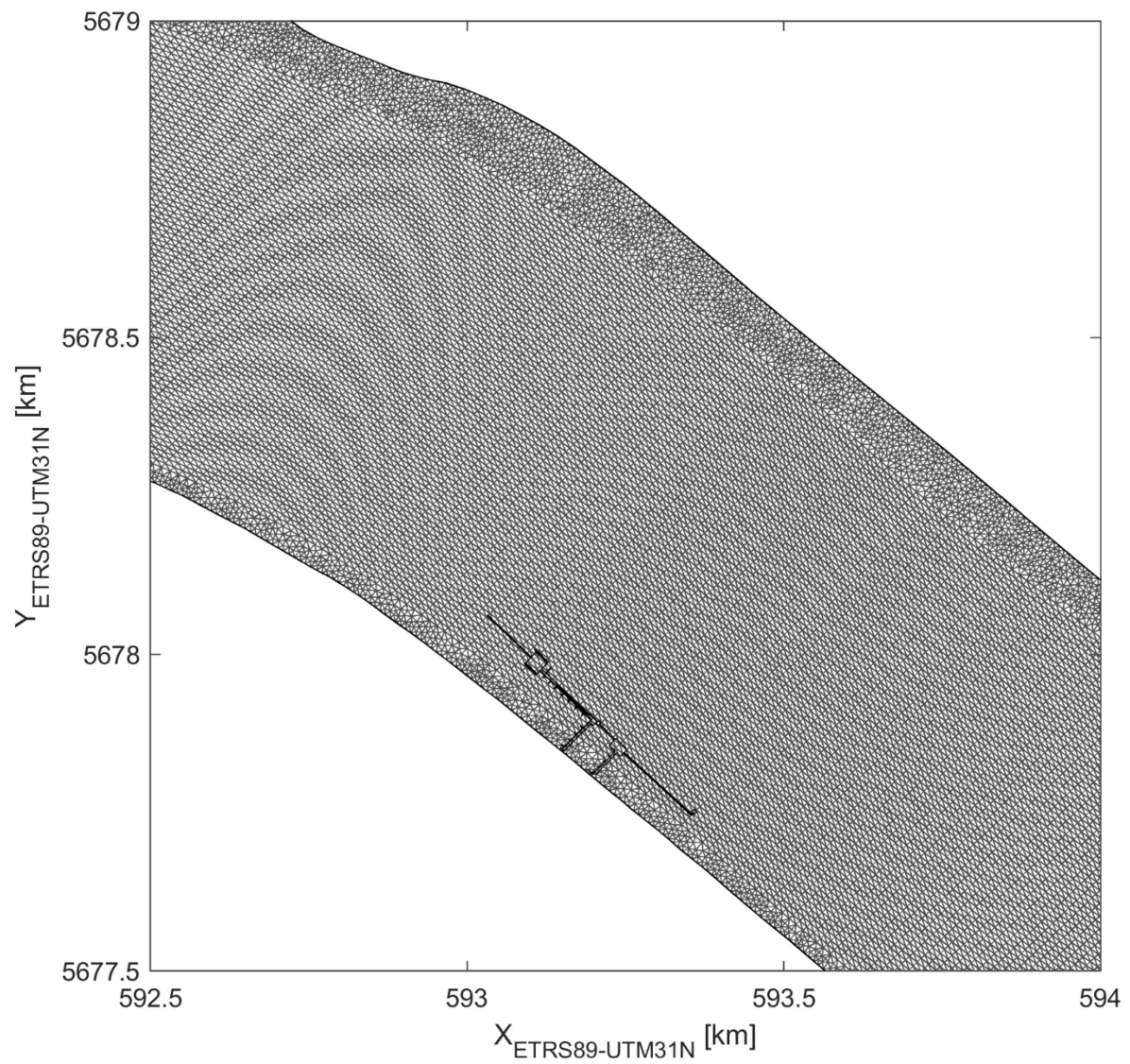


Figure 7 –Combining unstructured mesh with channel mesh.



2.2 Bathymetry

The bathymetric data for the BZS model were retrieved from the aMT (afdeling Maritieme Toegang) Master Archive. The horizontal reference system is the ETRS89 UTM31N and the vertical reference system is the TAW. The data come from the year 2018 and their original resolution is equal to 1 m.

In order to efficiently interpolate the bathymetric data on the computational grid, they were partitioned in 6 subdomains as depicted in Figure 8. The resolution of subdomains 2 to 5 remained 1 m, while the resolution of subdomains 1 and 6 was 2 m. In Figure 9 the bathymetry interpolated on the computational grid of the BZS model is shown.

Figure 8 – Outlines of the partitioned bathymetric data map of Lower Sea Scheldt [2018].

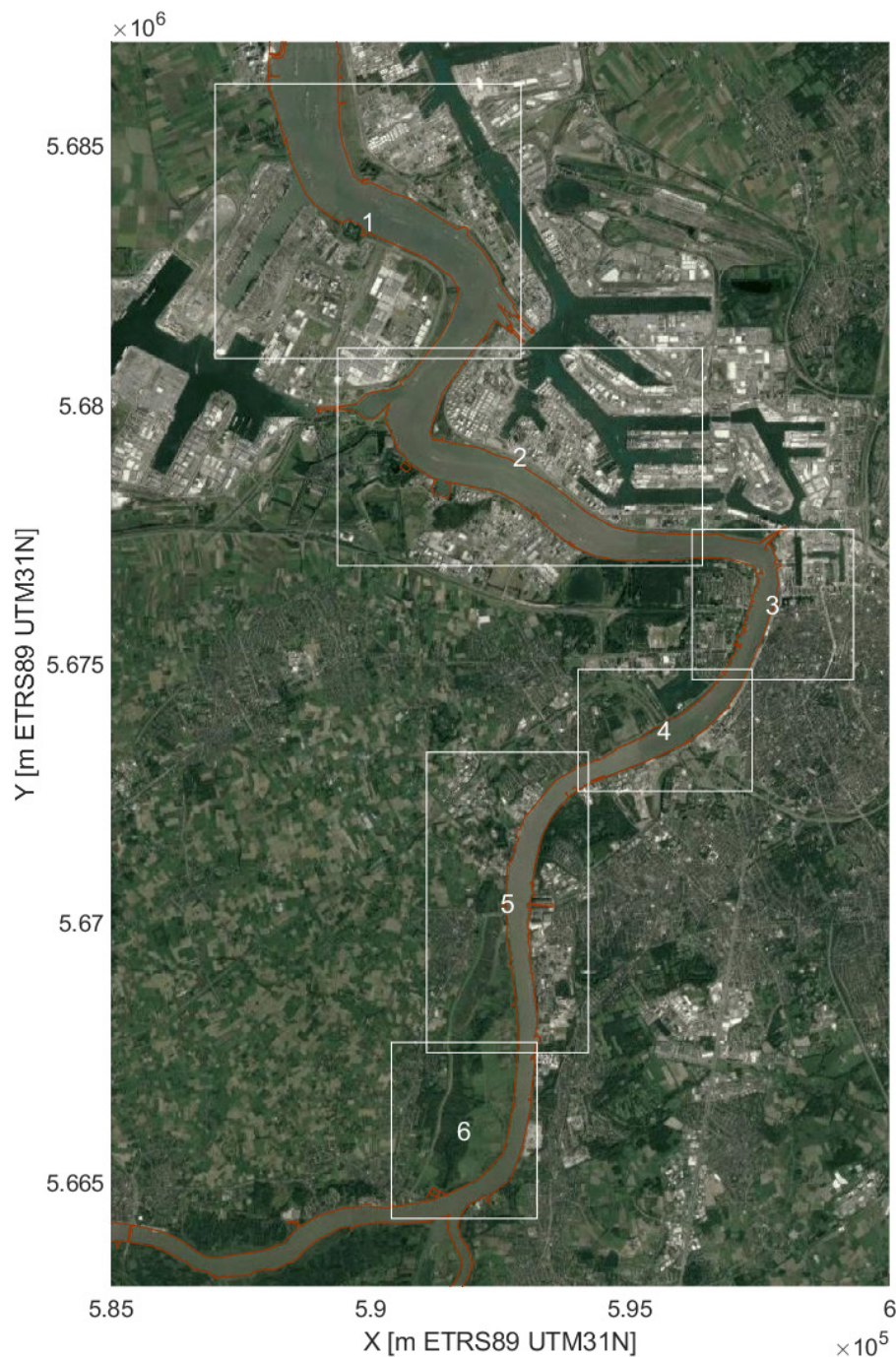
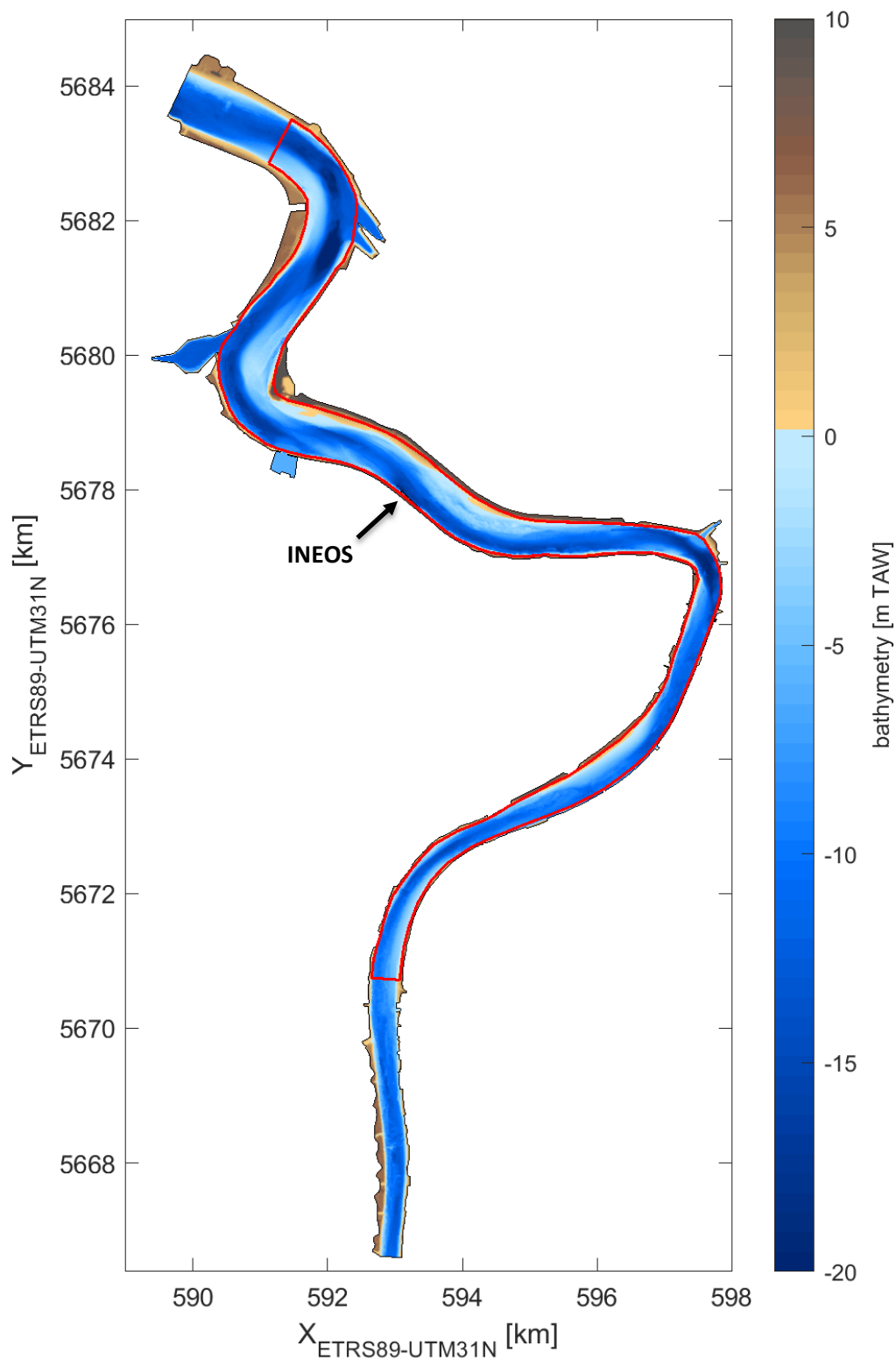


Figure 9 – Bathymetry of the year 2018 interpolated on the computational grid of the BZS model;
Red line denotes the outline of the channel mesh.



3 Calibration

Three sets of sensitivity tests were performed to achieve the optimal calibration of the BZS model. The tests

- Computational grid
- Bed roughness coefficient
- Turbulence model

Before presenting the results of the sensitivity analysis, the main parameters of the BZS model setup are given.

3.1 Simulation period

The model ran for the (spring-tide) period from 10/05/2016 00:00 to 13/05/2016 00:00 (3 days). The simulation period used for the calibration of the model was selected mainly based on the availability of the ADCP-sailed measurements at the closest to the area of interest transect, i.e. Oosterweel. Single-point velocity measurements at Oosterweel were also available at that period (HIC data). The period considered for the analysis of the results was from 11/05/2016 00:00 to 13/05/2016 00:00 (2 days), as the first 24 hours of the simulations were considered as warming-up period.

3.2 Tidal and Wind Forcing

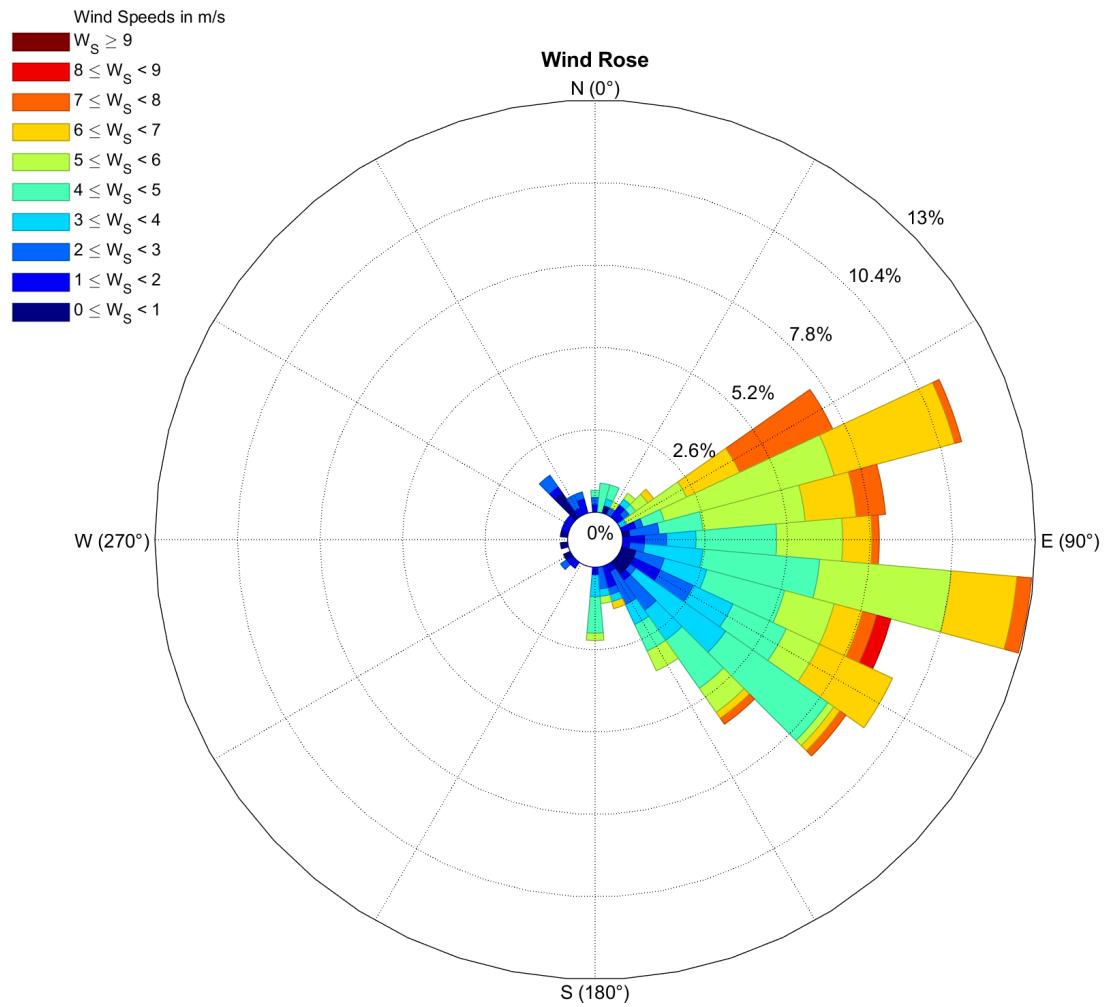
3.2.1 Open boundary conditions

The position of the two open boundaries of the BZS model coincides with the water level measurement stations of Liefkenshoek and Hemiksem (HIC data). The model was forced by water level time-series of interval equal to 5 min measured at the aforementioned stations.

3.2.2 Wind

Wind time-series with a time interval of 10 min, measured at the closest to the model's domain station, i.e. Prosperpolder (HMCZ data), were implemented uniformly in space over the entire computational domain. In Figure 10, the wind rose for the calibration period (10-13 May 2016) is presented. It is shown that the wind conditions for the selected period are in general moderate, wind is blowing mainly from the southeast, east, northeast direction and wind velocities are always less than 9 m/s.

Figure 10 – Wind rose at Prosperpolder [HMCZ] during the calibration period [10-13 May 2016].



3.3 Main Settings

The most important settings of the BZS model are included in Table 1.

The time-step of 2 s was selected after some testing, so that the computational stability and accuracy were ensured. Higher values of time-step led to instabilities at specific dry-wet zones of high spatial resolution (element size ≈ 5 m).

As initial condition, a constant value of water level equal to 0.30 was implemented over the computational domain. The aforementioned value was identical to the water level measured at Hemiksem at $t=0$.

The Manning's law of bottom friction was selected and sensitivity tests led to the finally chosen value for the roughness coefficient $n = 0.018 \text{ m}^{1/3}/\text{s}$ for the whole domain.

Sensitivity tests were also performed for the selection of the turbulence model. Mixing length theory was finally selected together with the implementation of a new technique for the spatiotemporal variation of the eddy viscosity. The latter technique was developed within the present study. More details about how mixing length theory has originally been implemented in TELEMAC-2D for the modelling of turbulence can be found in Dorfmann & Zenz (2016). Detailed presentation of the sensitivity tests is given in the following sections.

Table 1 – Main settings of the BZS model.

Parameter	Value
Time step	2 s
Initial conditions	Constant elevation (0.30 m TAW)
Version of TELEMAC	TELEMAC V7p2r2
Wind	On (spatially uniform)
Coriolis force	No
Salinity transport	Off
Law of bottom friction	Manning ($n = 0.018 \text{ m}^{1/3}/\text{s}$) [<i>Sensitivity tests</i>]
Option for treatment of tidal flats	1: equations solved everywhere with correction on tidal flats
Treatment of negative depths	1: smoothing
Free surface gradient compatibility	0.9
Turbulence model	5: Mixing Length model [<i>Sensitivity tests</i>]
Scheme for advection of velocities	1: method of characteristics
Scheme for advection of water depth	5: conservative scheme
Diffusion of velocity	Yes (implication = 1 is default; 0 cancels the diffusion)
Solver	7: Generalized Minimum Residual (GMRES) method

3.4 Sensitivity tests

3.4.1 Bed roughness coefficient

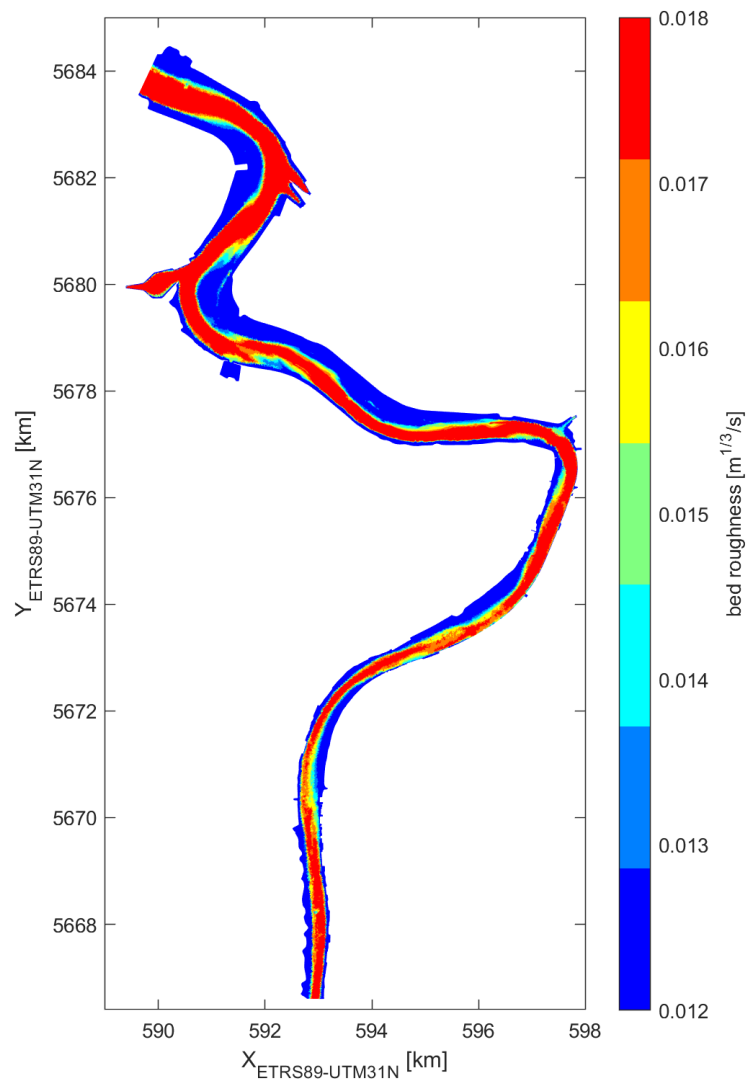
From previous experience of numerical modelling with Telemac it was decided to start the bed roughness coefficient testing considering a uniform Manning's coefficient equal to $0.022 \text{ m}^{1/3}/\text{s}$ over the entire computational domain (run 001B2). Then a lower uniform value $n = 0.018 \text{ m}^{1/3}/\text{s}$ (run 001F) and a spatially-varying distribution (run 001I) with values from 0.012 to $0.018 \text{ m}^{1/3}/\text{s}$ were also considered. The main idea of the latter spatially-varying distribution was to keep low n values at shallow areas close to the banks (see Figure 11), where the model underestimates the velocity magnitude (as we will see later). Specifically, $n = 0.012 \text{ m}^{1/3}/\text{s}$ was considered for bottom levels greater than -5 m TAW and $n = 0.018 \text{ m}^{1/3}/\text{s}$ for bottom levels lower than -10 m TAW . Note that the computational grid for all three test runs is the fully unstructured grid mentioned in Section 2.1.2 and (partially) depicted in Figure 6.

The main error statistical parameters Bias, RMSE and RMSE_0 (RMSE_0 corresponds to the bias corrected RMSE) of the modelled water level time-series at the stations of Kallo and Antwerpen are presented in Table 2. It is shown that all the three bed roughness configurations give almost the same statistical values for Kallo station, whereas for the Antwerpen station, the higher Manning's coefficient run (001B2) gives slightly better predictions than the other two runs.

Table 2 – Main error statistics for the modelled water level time series for 3 different Manning's coefficients configurations.

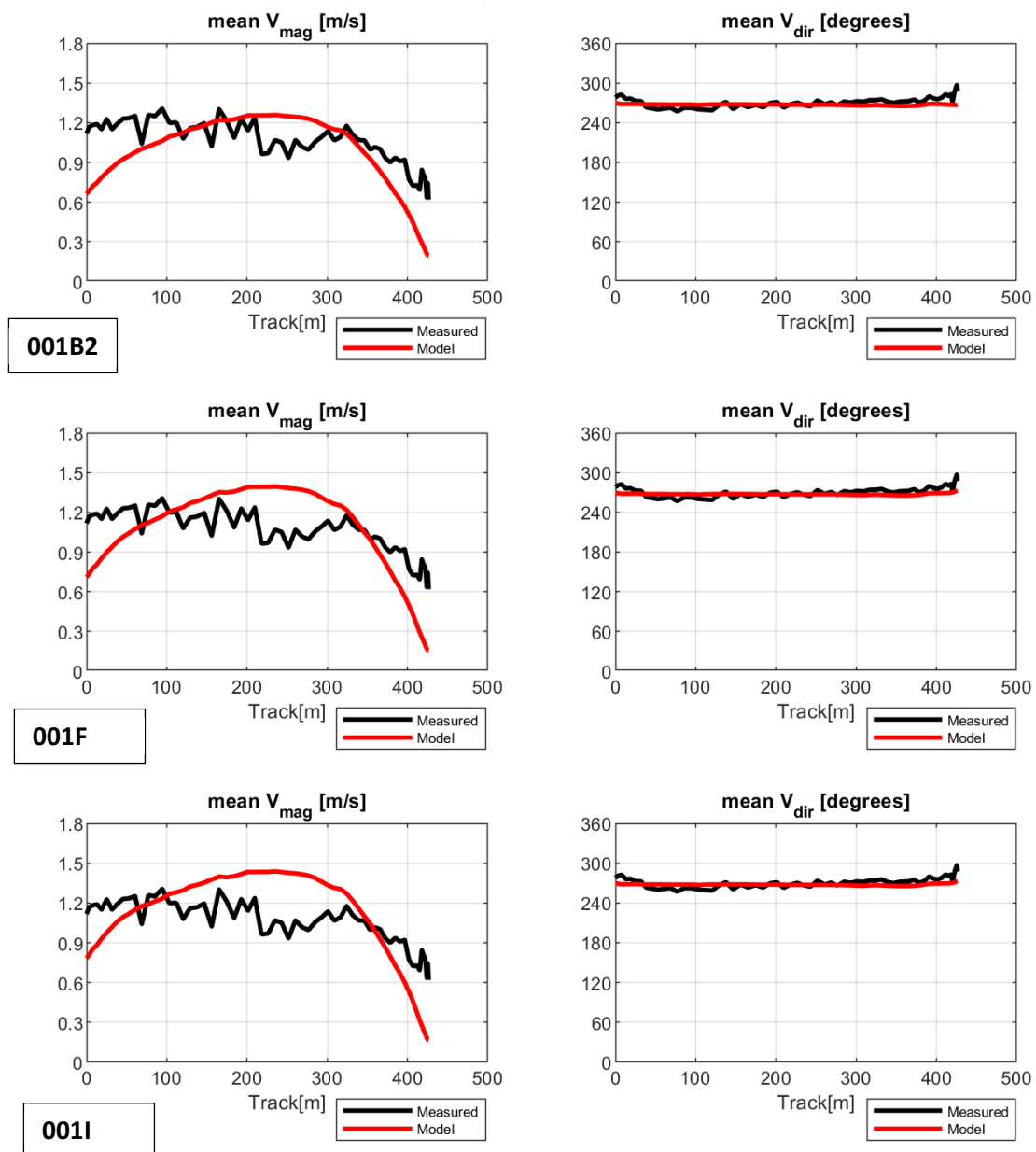
Station	Time Series (11-13 May 2016)		
	BIAS	RMSE	RMSE_0
	[m]	[m]	[m]
001B2 [$n=0.022 \text{ m}^{1/3}/\text{s}$]			
Kallo	-0.02	0.03	0.01
Antwerpen	-0.04	0.05	0.03
001F [$n=0.018 \text{ m}^{1/3}/\text{s}$]			
Kallo	-0.02	0.03	0.01
Antwerpen	-0.04	0.06	0.04
001I [$n=0.012-0.018 \text{ m}^{1/3}/\text{s}$]			
Kallo	-0.02	0.03	0.02
Antwerpen	-0.04	0.06	0.05

Figure 11 – Bed roughness coefficient (Manning) distribution on the computational domain of BZS model for run 0011.



In Figure 12 and Figure 13, the numerically predicted velocities (001B2, 001F, 001I runs) and the measured velocities along the transect 'Oosterweel' during maximum ebb and maximum flood flow, are shown respectively. The left-side panels show the velocity magnitude and the right-side ones the velocity direction. From the maximum ebb flow figure (Figure 12) it is indicated that all three runs clearly underestimate the velocity magnitude close to the banks, while the runs with reduced Manning's n (001F & 001I) overestimate the magnitude at the center of the transect. Note that this behavior characterizes in general the performance of the model in predicting the flow velocities during ebb tide. Furthermore, the reduction of Manning's n causes uniform along the transect increase of the velocity magnitude. The spatially-varying decrease of n at shallow areas (close to the banks) does not improve substantially the local magnitude underestimation.

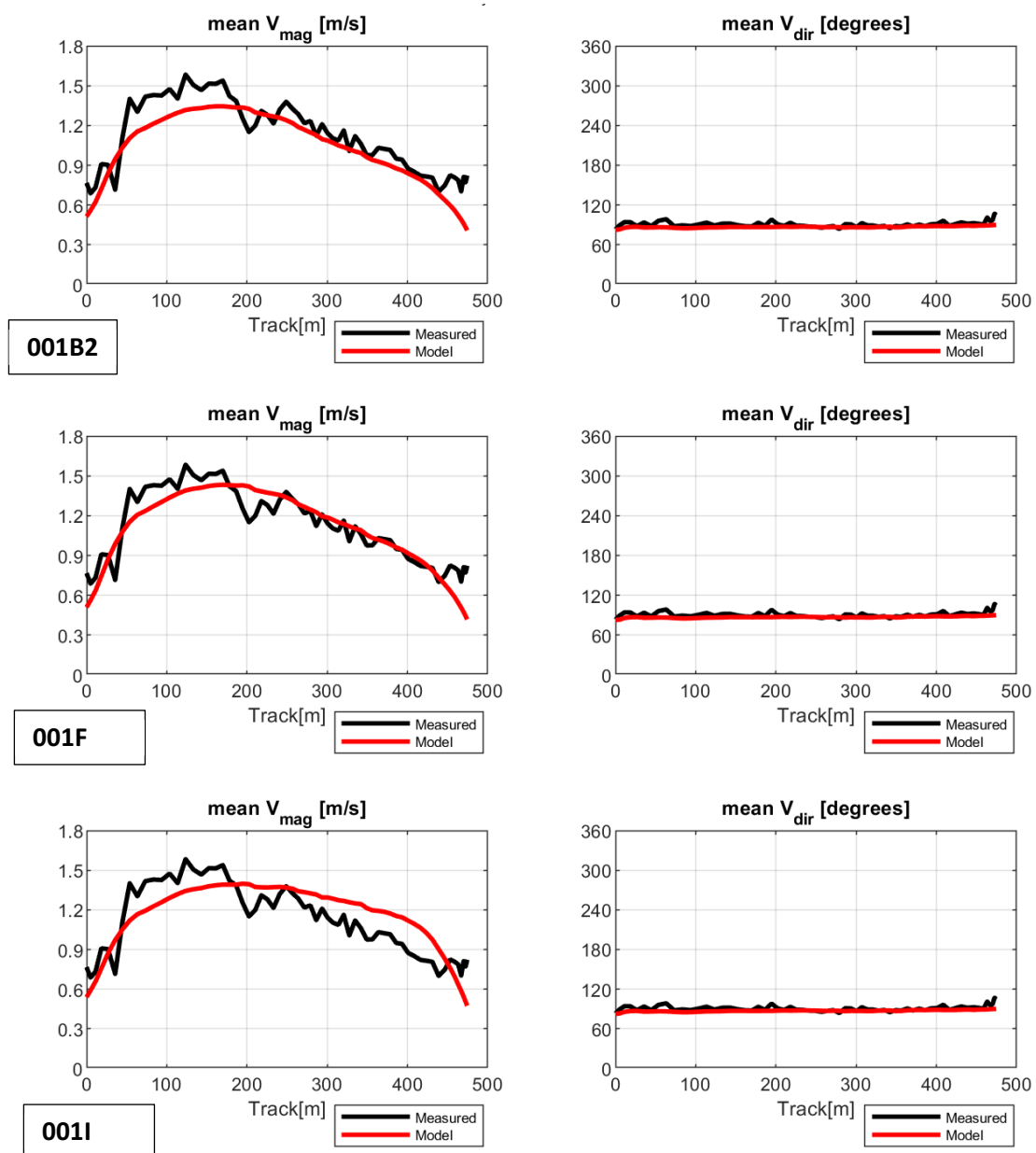
Figure 12 – Numerically predicted magnitude and direction of instantaneous velocities (red lines) versus the corresponding measured ones (black lines) along the transect 'Oosterweel' (0 m at right bank) around maximum ebb flow [11/05/2016 09:40].
Top figures: Manning's $n=0.022 \text{ m}^{1/3}/\text{s}$; Middle figures: Manning's $n=0.018 \text{ m}^{1/3}/\text{s}$; Bottom figures: varying Manning's coef.



As for the maximum flood velocities (Figure 13), the run with the best performance is 001F (constant $n = 0.018 \text{ m}^{1/3}/\text{s}$). Run 001B2 ($n = 0.022 \text{ m}^{1/3}/\text{s}$) clearly underestimates the velocity magnitude, and run 001I (varying n) worsens the predicted velocity profile by increasing the magnitude at the right bank and reducing it at the left bank.

Taking into account all the above, it can be concluded that the bed roughness configuration with the most balanced behavior during the whole tidal cycle is the one used in run 001F, and therefore this coefficient value was finally selected in the basic settings of the BZS model.

Figure 13 – Numerically predicted magnitude and direction of instantaneous velocities (red lines) versus the corresponding measured ones (black lines) along the transect 'Oosterweel' (0 m at left bank) around maximum flood flow [11/05/2016 18:20].
Top figures: Manning's $n=0.022 \text{ m}^{1/3}/\text{s}$; Middle figures: Manning's $n=0.018 \text{ m}^{1/3}/\text{s}$; Bottom figures: varying Manning's coef.



3.4.2 Computational grid

The sensitivity of the numerical results to the grid size and formation was investigated by performing identical hydrodynamic simulations on several variations of the computational grid (17 variations in total). Local refinement up to 2 m along the isobath lines of 0 m or 3 m, extension of the refined area (element size = 5 m) in order to include the Scheldt bending at the area of Antwerp are indicative examples of the variations tested. For a large proportion of the grid variations the results showed no or slight sensitivity and therefore are not presented here. Three main grid variations with main characteristics shown in Table 3, are presented in this report.

Table 3 – Characteristics of the three main computational grid variations of the sensitivity tests.

Run Identifier	# Nodes	Size max/medium/min [m]	Mesh Formation
001F	231 800	20 / - / 5	Fully Unstructured
001F9	327 800	20 / 10 / 5	Fully Unstructured
001F11	168 500	20 / 10 / 5	Unstructured + Channel mesh

The distribution of the grid size in the model's domain has been discussed already in Section 2.1.2. Note that in the variations 001F9 and 001F11, the size of 20 m is limited only close to the open boundaries at distances equal to 1.5 km and 4 km from the downstream and the upstream boundary, respectively. Consequently, For 001F9 variation, the main part of the domain is aperted from elements of size equal to 10 m, except for the area of interest (see Figure 6), where the minimum resolution is 5 m.

The performance of the grid variations was firstly evaluated through two basic error statistical parameters, i.e. bias and $RMSE_0$ (bias corrected), in the water level variation in time at the locations Kallo and Antwerpen where water level is continuously measured. The bias and the RMSE between the numerically predicted and the measured water level time series for the two last days of the simulation period (11-13 may 2016), are presented in Figure 14. The results indicate a good agreement between the predictions of the three grid variations for both stations. The performance of each grid variation compared to the measurements is very good since the bias and the $RMSE_0$ are limited to a couple centimeters at Kallo and to about 4 cm at Antwerpen station.

Figure 14 – Bias (top figure) and RMSE₀ (bottom figure) in the water level time-series between the predictions of the three grid variations and the measurements during the calibration period [11-13 May 2016].

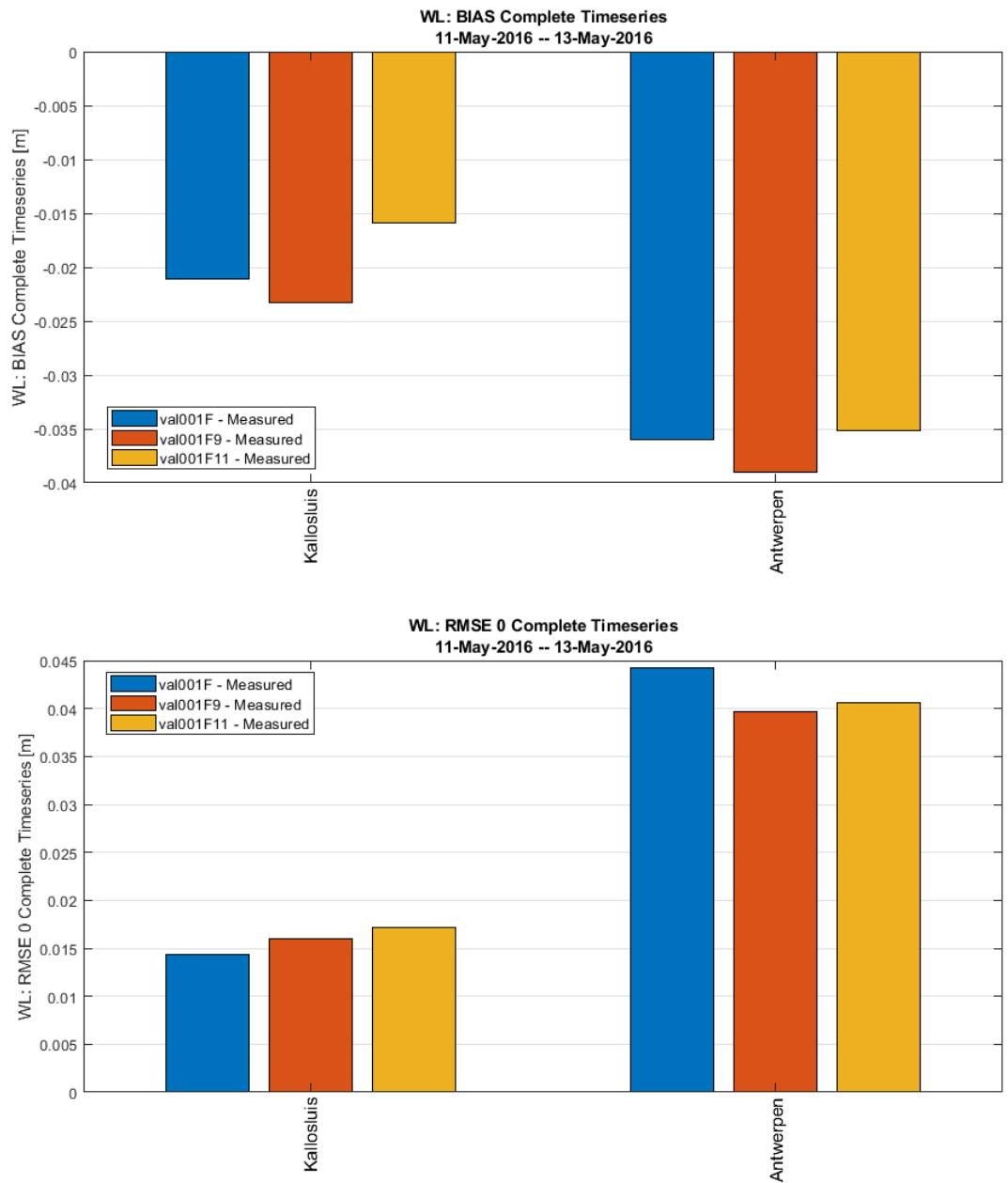
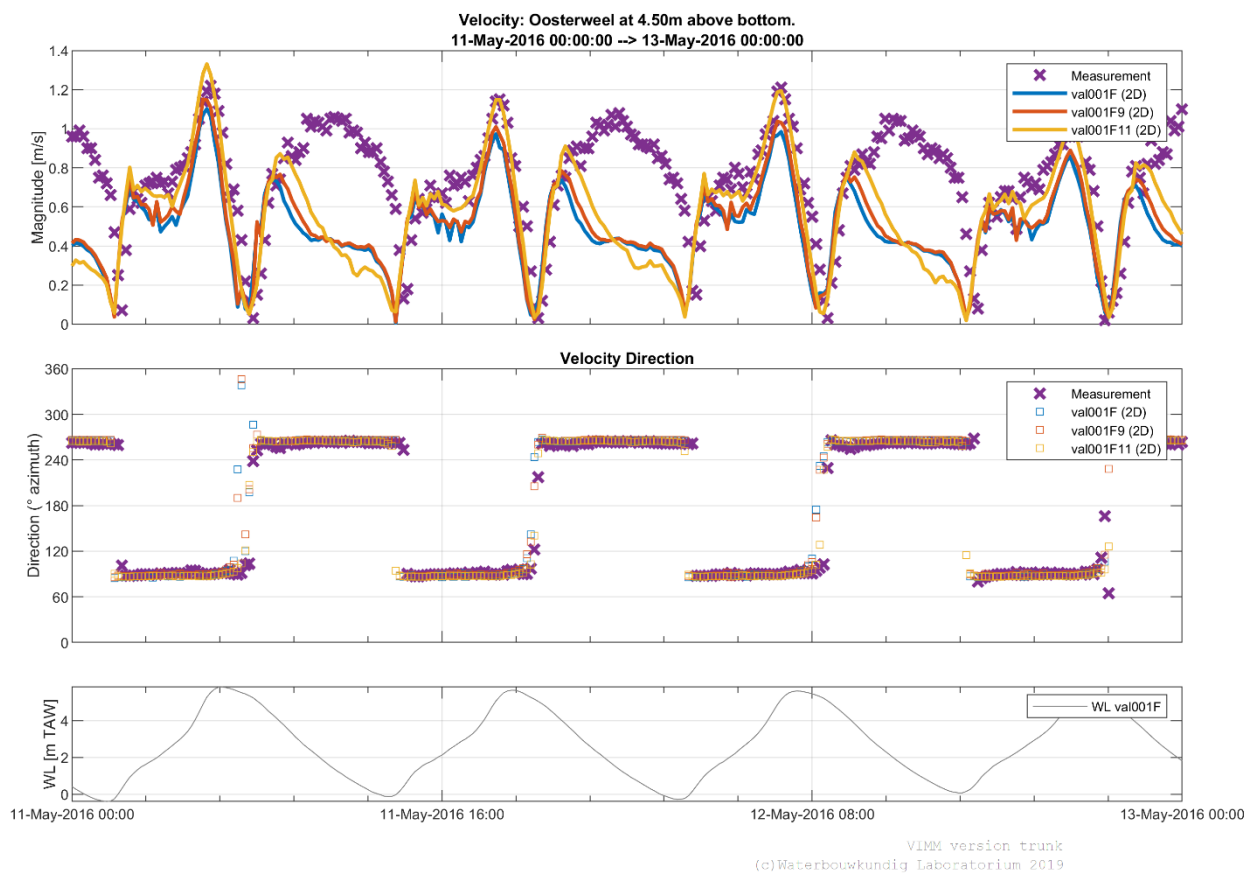


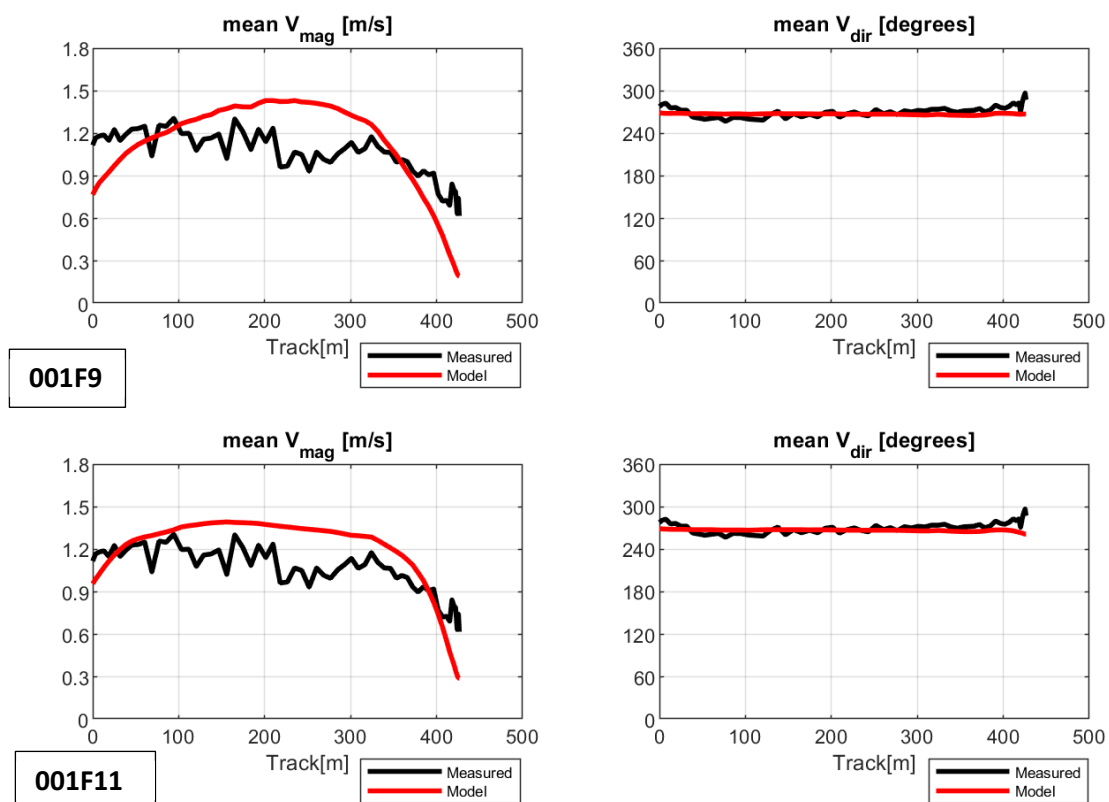
Figure 15 shows the numerically predicted time series of velocity magnitude and direction versus the measured ones at the station of Oosterweel. Note that the measurements are performed by a current meter deployed 4.50 m above the bottom, while the numerical predictions correspond to depth-averaged velocities. The local depth is around -7 m TAW. The behavior of the coarse (001F) and the finer (001F9) grids is similar and the relatively higher peak velocities predicted by the finer one are more or less expected. Noticeably good is the behavior of the channel mesh (001F11) in capturing the measured peak flood velocities, even though it is the coarser of the three. However, all the grid variations perform really bad in capturing the velocity variation during the ebb phase. Further discussion and treatment of this shortcoming of the model follows in the section with the adjustment of the turbulence model.

Figure 15 – Magnitude and direction of the measured (symbols) and modelled velocity time series at Oosterweel predicted from the three grid variations. Single-point velocity measurements are performed at 4.50 m above the bottom ($\approx$ 7m TAW).



In Figure 16 and Figure 17, the numerically predicted velocities (001F9 & 001F11 runs) and the measured velocities along the transect 'Oosterweel' during maximum ebb and maximum flood flow, are shown respectively. The left-side panels show the velocity magnitude and the right-side ones the velocity direction. From the maximum ebb flow figure (Figure 16) it is indicated that the fully unstructured mesh encounters difficulties in capturing the velocity magnitude close to the banks, while it overestimates the magnitude at the center of the transect. The channel mesh gives clearly better prediction of the velocity profile approaching better the 'uniformity' of the measured velocity profile. On the other hand, even though the predictions of the channel mesh very close to the banks are better than those of the unstructured, they are still relatively low compared to the measured ones.

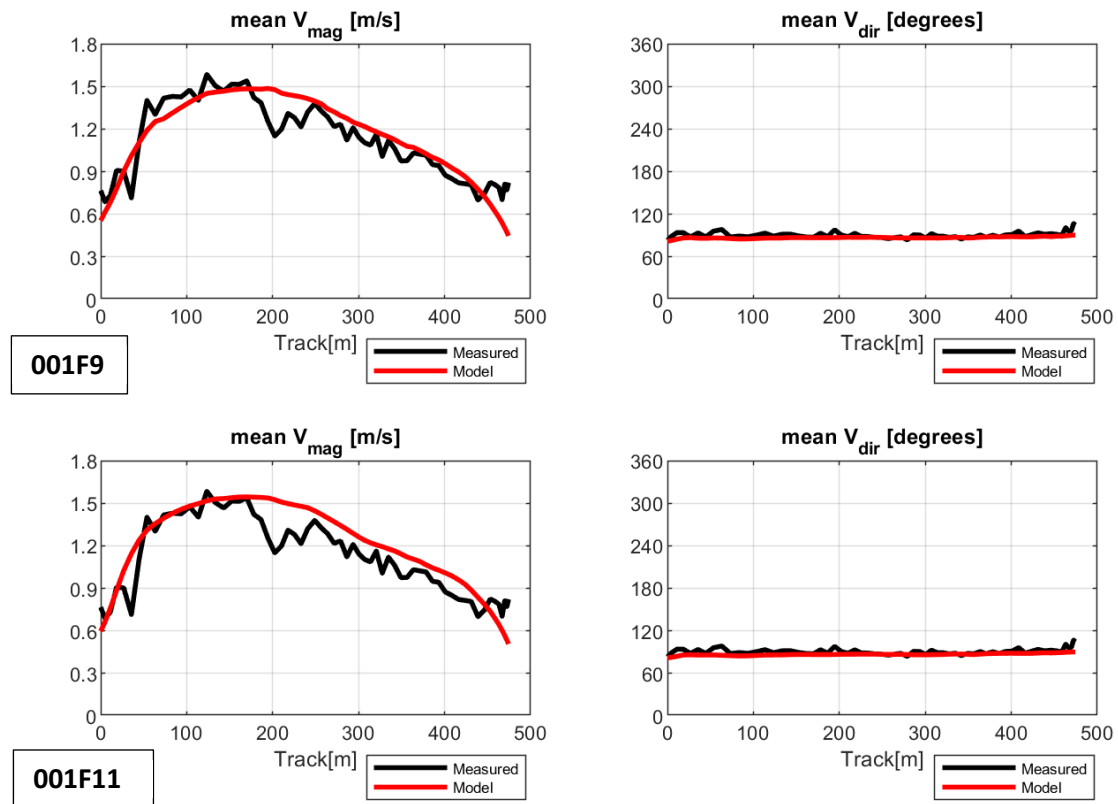
Figure 16 – Numerically predicted magnitude and direction of instantaneous velocities (red lines) versus the corresponding measured ones (black lines) along the transect 'Oosterweel' (0 m at right bank) around maximum ebb flow [11/05/2016 09:40].
Top figures: fully unstructured mesh 001F9; Bottom figures: unstructured-channel mesh 001F11.



As for the maximum flood velocities (Figure 17), the numerical predictions of both mesh formations are obviously closer (than the ebb flow case) to the measured ones. The channel mesh gives slightly enhanced magnitude along the entire transect compared to the unstructured mesh predictions. Taking into account the findings presented in Figure 16 and Figure 17, but also those of Figure 15 with the single-point velocity time-series, it can be said that the model encounters difficulties in predicting the velocities close to the banks (shallow areas) during the ebb phase, while it performs much better in capturing the flow field during flood phase.

Overall, the similar numerical predictions of the two versions of the fully unstructured grids (001F, 001F9) are more or less indicative of the independence of the results from the grid size. On the other hand, it is found that the predictions can be improved by utilizing the combined unstructured-channel mesh (001F11) with even lower resolution than the fully unstructured ones.

Figure 17 – Numerically predicted magnitude and direction of instantaneous velocities (red lines) versus the corresponding measured ones (black lines) along the transect ‘Oosterweel’ (0 m at left bank) around maximum flood flow [11/05/2016 18:20].
Top figures: fully unstructured mesh 001F9; Bottom figures: unstructured-channel mesh 001F11.



3.4.3 Turbulence modelling

TELEMAC-2D offers the user five models for the modelling of different complexity for the modelling of turbulence:

- Elder model
- k-epsilon model
- Smagorinsky model
- Mixing Length model
- Spalart-Almaras model (from v7p3 and on)

All of the above models were tested in a series of runs with main objective to improve the performance of the BZS model at the shallow areas close to the banks mainly during the ebb phase. It is deemed that this objective could be achieved by modelling more accurately the horizontal mixing of turbulence across the river, so that more momentum is transferred from the centerline towards the banks. In other words, the goal is to select or tune the appropriate turbulence model so that the model produces (transversal) velocity profiles with lower velocity peaks in the center and higher velocities close to the banks during ebb and simultaneously preserve its good performance during the flood phase.

In Table 4 the main information about the tests carried out with respect to the turbulence modelling are presented.

Table 4 – Sensitivity tests of turbulence modelling.

Run Identifier	Model	Tuning Coefficients	Mesh	Comments
001F (Ref.)	Smagorinsky	$C_s=0.1$ (default)	unstructured	-
001G	Smagorinsky	$C_s=0.2$	unstructured	Increase eddy viscosity
001G1	Elder	$a_l=6$; $a_t=0.6$ (def.)	unstructured	-
001G2	Mixing Length	$C_L=0.267$ (def.)	unstructured	-
001G4	Elder	$a_l=0.5$; $a_t=6$	unstructured	Increase transverse dispersion Reduce streamwise dispersion
001G5	Spalart-Almaras	default	unstructured	Run in T2D v7p3r1
001G6	Elder	$a_l=6$; $a_t=6$	unstructured	Increase transverse dispersion
001G8	k-epsilon	default	unstructured	-
001G10	Mixing Length	$C_L \times 10$	unstructured	Mixing length (L_m) multiplied by 10
001L	Mixing Length	Varying C_L [1x - 10x]	unstructured + channel mesh	Varying C_L up to 10 times the default value during ebb
001L2	Mixing Length	Varying C_L [1x - 20x]	unstructured + channel mesh	Varying C_L up to 20 times the default value during ebb

The coefficients presented in Table 4 can be used as tuning parameters of turbulence, since they are introduced in the equation of the eddy viscosity ν_t of each model. Briefly,

- C_s is the Smagorinsky constant with default value equal to 0.1 for shear flows.
- a_l and a_t coefficients of the Elder model adjust the streamwise and the transverse dispersion, respectively, which in turn depends on friction velocity and water depth ($K_l = a_l U_* h$ & $K_t = a_t U_* h$).
- C_L is a tuning coefficient of the mixing length $L_m = C_L \kappa h$; where κ is the von Karman constant and h is the water depth.

The first finding of the turbulence sensitivity tests was that the Smagorinsky (both 001F and 001G runs), the Mixing Length (001G2) and the k-epsilon (001G8) models with default settings led to very similar results with respect to the velocity field at the area of Oosterweel. Indicative of the models' behavior are Figure 12 and Figure 13 presented already in Section 3.4.1. On the other hand, the default Elder model (001G1) exhibited the desired to some extent enhancement of the transverse dispersion compared to the aforementioned models. This means that the velocity profile of maximum ebb current at Oosterweel was better captured by Elder model, but, on the other hand, the magnitude of the maximum flood velocity profile was underestimated. Further tuning of the dispersion parameters (001G4, 001G6) introduced more dispersion in the results and did not help in the overall improvement of the model's performance. The Spalart-Almaras model (001G5) presented exaggerated diffusion, i.e. the numerical velocity magnitude profiles at Oosterweel were substantially underestimated compared to the measured ones during the entire tidal cycle. Further investigation of the Spalart-Almaras turbulence model was excluded from the scope of this study.

Increasing the mixing length coefficient 10 times (run 001G10) compared to the default value, $C_L = 0.267$, has a positive effect at the velocity profiles during ebb phase (see Figure 18), but as expected, it worsens the profiles during flood phase (see Figure 19) due to increased velocity diffusivity. For improving the numerical predictions close to the banks during the ebb phase, without 'disturbing' the profiles during flood a modification of the Mixing Length model was implemented. Namely, a spatiotemporal variation of the C_L coefficient was applied in the following manner:

- C_L remained constant and equal to the default value during flood, and gradually increased up to 20 times the default value, i.e. from 0.267 to 5.34 (and vice versa) for free surface elevations lower than 4 m TAW only during the ebb phase (the criterion was regulated by a free surface gradient condition).

As shown in Figure 18 and Figure 19 the modified Mixing Length model (001L2) performs very good in predicting the maximum ebb current profile as much as the maximum flood current profiles at Oosterweel. The predictions has substantially improved especially close to the left bank (LB) during the ebb phase, fact that enhances the confidence in the models results at the area of interest. It has to be noted that 001L2 run uses the combined unstructured-channel mesh, while the two other runs (001G2, 001G10) use the base unstructured mesh. This means that the improvement in the max ebb profile is partially due to the positive impact of the channel mesh (as shown in the previous section). However, the positive effect of the modified model, excluding the influence of the grid, is shown in Figure 20 of the single-point velocity comparison.

Figure 18 – Numerically predicted magnitude and direction of instantaneous velocities (red lines) versus the corresponding measured ones (black lines) along the transect 'Oosterweel' (0 m at right bank) around maximum ebb flow [11/05/2016 09:40].
Top figures: Default mixing length; Middle figures: Mixing length x10; Bottom figures: Varying mixing length (1x to 20x).

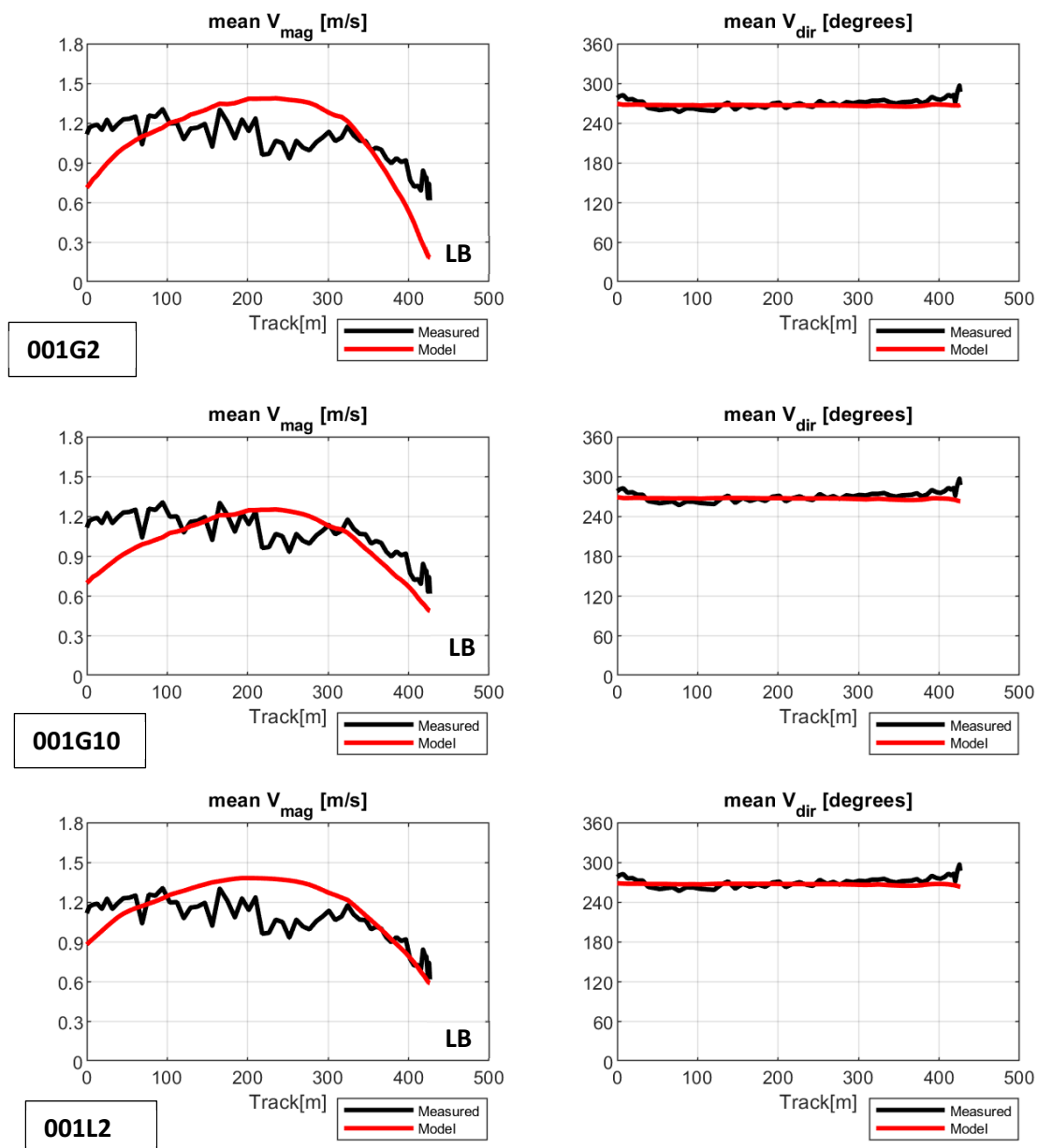
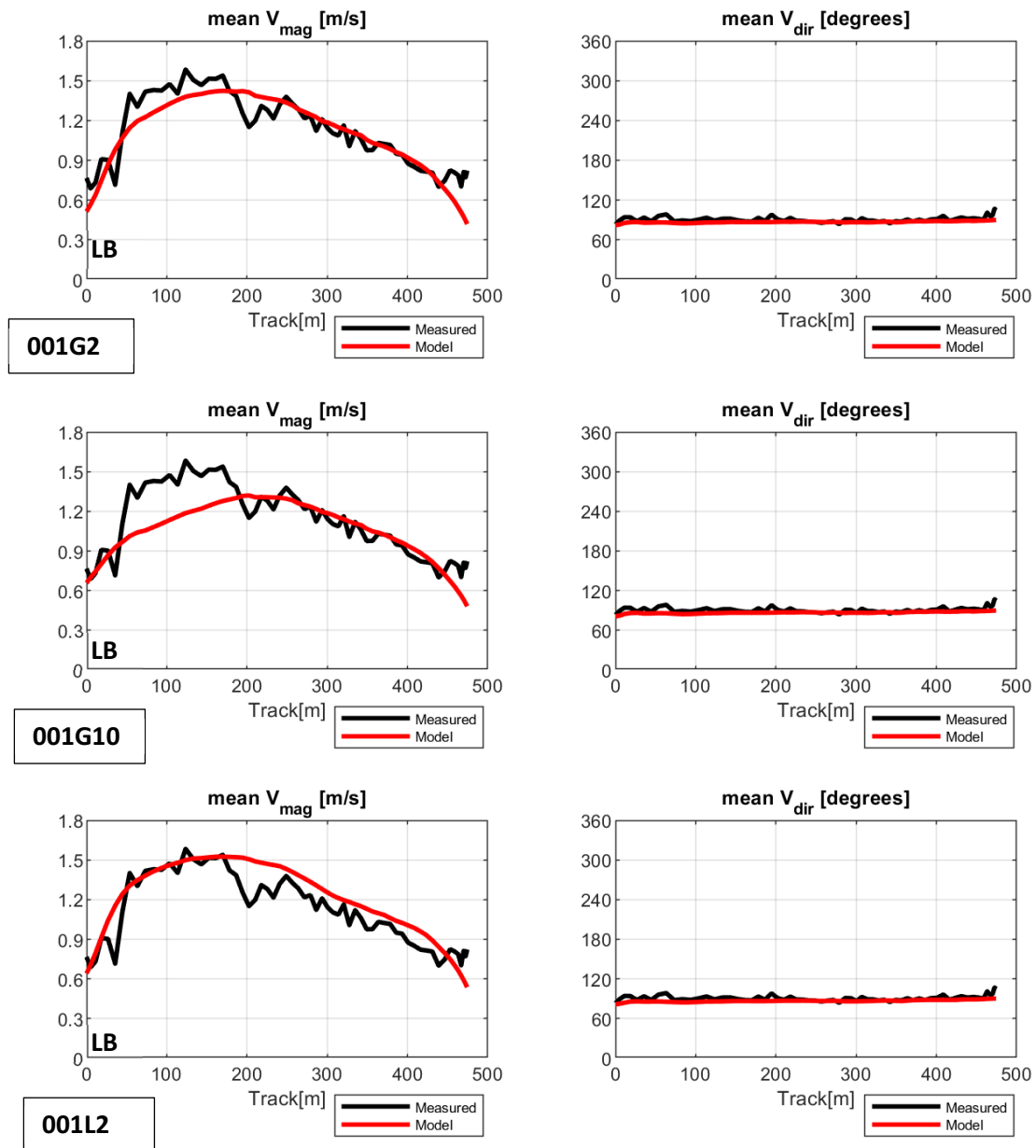
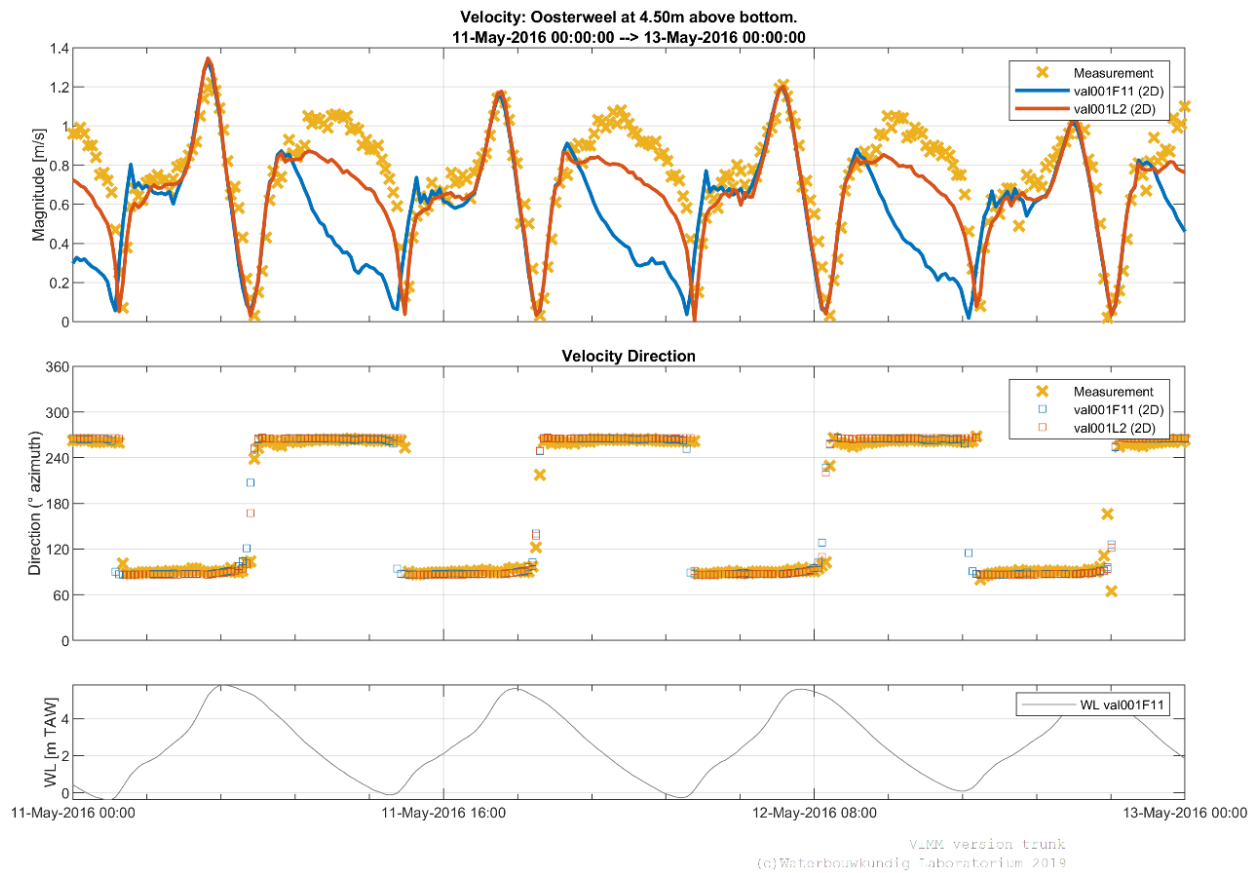


Figure 19 – Numerically predicted magnitude and direction of instantaneous velocities (red lines) versus the corresponding measured ones (black lines) along the transect ‘Oosterweel’ (0 m at left bank) around maximum flood flow [11/05/2016 18:20].
Top figures: Default mixing length; Middle figures: Mixing length x10; Bottom figures: Varying mixing length (1x to 20x).



In Figure 20 the predictions of two runs with identical computational grid (unstructured+channel mesh) and different turbulence models (001F11: Smagorinsky default, 001L2: modified Mixing Length), are presented for the single-point velocity temporal variation at Oosterweel. It is shown that proposed modification in the Mixing Length model leads to substantial improvement of the velocity magnitude during ebb phase. However, a large (maximum) deviation of about 30 cm/s between the modelled and the measured maximum ebb current can be still observed. It is conjectured that this deviation comes partially from the exclusion of the salinity effect from the model. It has been reported that salinity advection can affect the vertical velocity profile during the ebb phase presenting important differences from the one of the flood. In addition to this, the fact that the velocity at Oosterweel is measured by one current meter placed at a position that during low tide is close to the free surface, maybe affects negatively the impression for the model's performance during ebb, in the sense that the measured velocity may not be representative of the depth-averaged one.

Figure 20 – Magnitude and direction of the measured (symbols) and modelled velocity time series at Oosterweel predicted from the default Smagorinsky model (001F11) and the modified Mixing Length model (001L2). Single-point velocity measurements are performed at 4.50 m above the bottom ($\approx 7\text{m TAW}$).



The main error statistical parameters Bias, RMSE and RMSE_0 of the water level time-series predicted by the modified Mixing Length model at the stations of Kallo and Antwerpen are compared to those of the default Smagorinsky model in Table 5. The computational grid in both runs is the combined unstructured channel mesh grid. It is shown that the modified Mixing Length model only slightly affects the statistical indices for both Kallo and Antwerpen stations compared to the default turbulence model.

Table 5 – Main error statistics for the modelled water level time series for 2 different turbulence models.

Station	Time Series (11-13 May 2016)		
	BIAS	RMSE	RMSE_0
	[m]	[m]	[m]
001F11 [Smagorinsky default]			
Kallo	-0.02	0.02	0.02
Antwerpen	-0.04	0.05	0.04
001L2 [Mixing Length modified]			
Kallo	-0.02	0.03	0.02
Antwerpen	-0.04	0.06	0.05

4 Scenarios

The interventions at the existing INEOS jetty and the construction of a new container quay has already been described in the Introduction (see also Figure 3 and Figure 4). Accordingly, three scenarios that correspond to these interventions, summarized in Table 6, are investigated numerically in this chapter along with the Reference scenario (current situation).

Table 6 – Description of the investigated scenarios.

Scenario ID	Description
scenREF	do nothing – bathymetry of the year 2018
scenW	deepening inside jetty to -6.5 m TAW + sheet pile wall
scenQ1	new quay – completely closed sheet pile wall
scenQ2	new quay – part of quay deck supported by piles

4.1 Simulation period

The model ran for a typical full spring-neap tidal cycle of the year 2018 and specifically during the period 01/04/2018 00:00 to 18/04/2018 00:00 (17 days). The model was validated for this simulation period by comparing the numerical results with: (a) measured water level time-series at Kallo and Antwerpen stations, and (b) single-point velocity measurements at Oosterweel. The period considered for the analysis of the results was from 02/04/2018 00:00 to 18/04/2018 00:00 (16 days), as the first 24 hours of the simulations were considered as warming-up period.

The computational time of a typical simulation was around 3 hours on 32 processors of Reynolds queue (WL cluster).

4.2 Tidal and wind forcing

4.2.1 Open boundary conditions

The model was forced by water level time-series of interval equal to 5 min measured at the stations of Liefkenshoek (downstream) and Hemiksem (upstream).

4.2.2 Wind

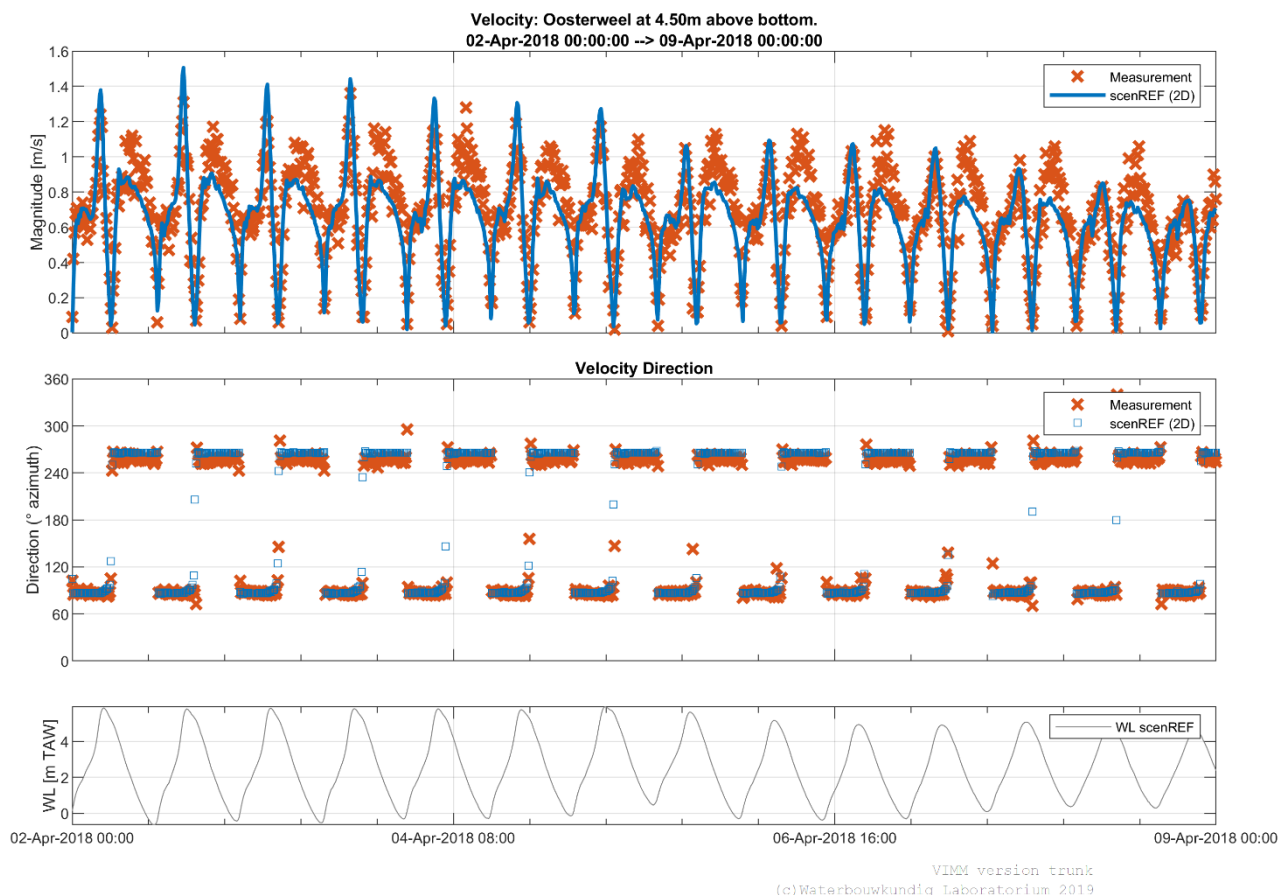
Wind time-series with a time interval of 10 min, measured at Prosperpolder station, were implemented uniformly in space over the entire computational domain. In Figure 21, the wind rose for the simulation period (02-18 April 2018) is presented. It is shown that the wind conditions for the selected period are in general moderate, wind is blowing mainly from the south and the southeast directions and wind velocities are most of the time lower than 10 m/s.

Table 7 – Main error statistics for the modelled water level time-series (full spring-neap cycle) for the Reference scenario (scenREF).

Station	Time Series (02-18 April 2018)		
	BIAS [m]	RMSE [m]	RMSE ₀ [m]
Kallo	0.01	0.02	0.02
Antwerpen	-0.03	0.05	0.03

Figure 22 shows the measured single-point velocity time-series (both magnitude and direction) at Oosterweel for a period of 7 days (half spring-neap tidal cycle) versus the numerical ones. The maximum flood currents can be in general reproduced accurately by the BZS model, but slightly overestimated during spring tide. On the other hand, a deviation of about 30 cm/s between the modelled and the measured maximum ebb current is generally observed (like in calibration runs), which becomes exceptionally larger (about 40 cm/s) at the noon of 04/01/2018 when very large ebb currents were measured (possible errors in measurements..?). It is also reminded that the velocity at Oosterweel is only measured at a position 4.5 m above the bottom. Thus it may not be representative of the depth-averaged one, especially during low tide. Noticeable constant deviation of the modelled velocity direction during ebb phase of about 8 degrees is also observed.

Figure 22 – Magnitude and direction of the measured (symbols) and modelled velocity time series at Oosterweel predicted from the default Smagorinsky model (001F11) and the modified Mixing Length model (001L2). Single-point velocity measurements are performed at 4.50 m above the bottom ($\approx$ 7m TAW).



The absence of the salinity effect may be another reason for this deviation between the numerical results and the measurements, as already explained in section 3.4.3. Note also that uncertainty can be introduced in the predicted results due to differences between the model's bathymetry and the real bathymetry, especially when abrupt bathymetric changes occur. The local bathymetric profiles at Oosterweel present: (a) steep slope in the transverse to the flow direction ($\approx 1:5$), and (b) an abrupt local bump in the flow direction, and this might be another reason for the differences in the results during ebb phase.

The evaluation of the model based on the aforementioned velocity predictions can be done by using the RMAE (Relative Mean Absolute Error) metric, as shown in Table 8. Note that the RMAE includes both the magnitude and direction errors. It is used as a qualification parameter in this study, following the work that has been done by Sutherland *et al.* (2004). For the velocity time-series presented in Figure 22, RMAE = 0.19, which is a quite satisfactory score for the model.

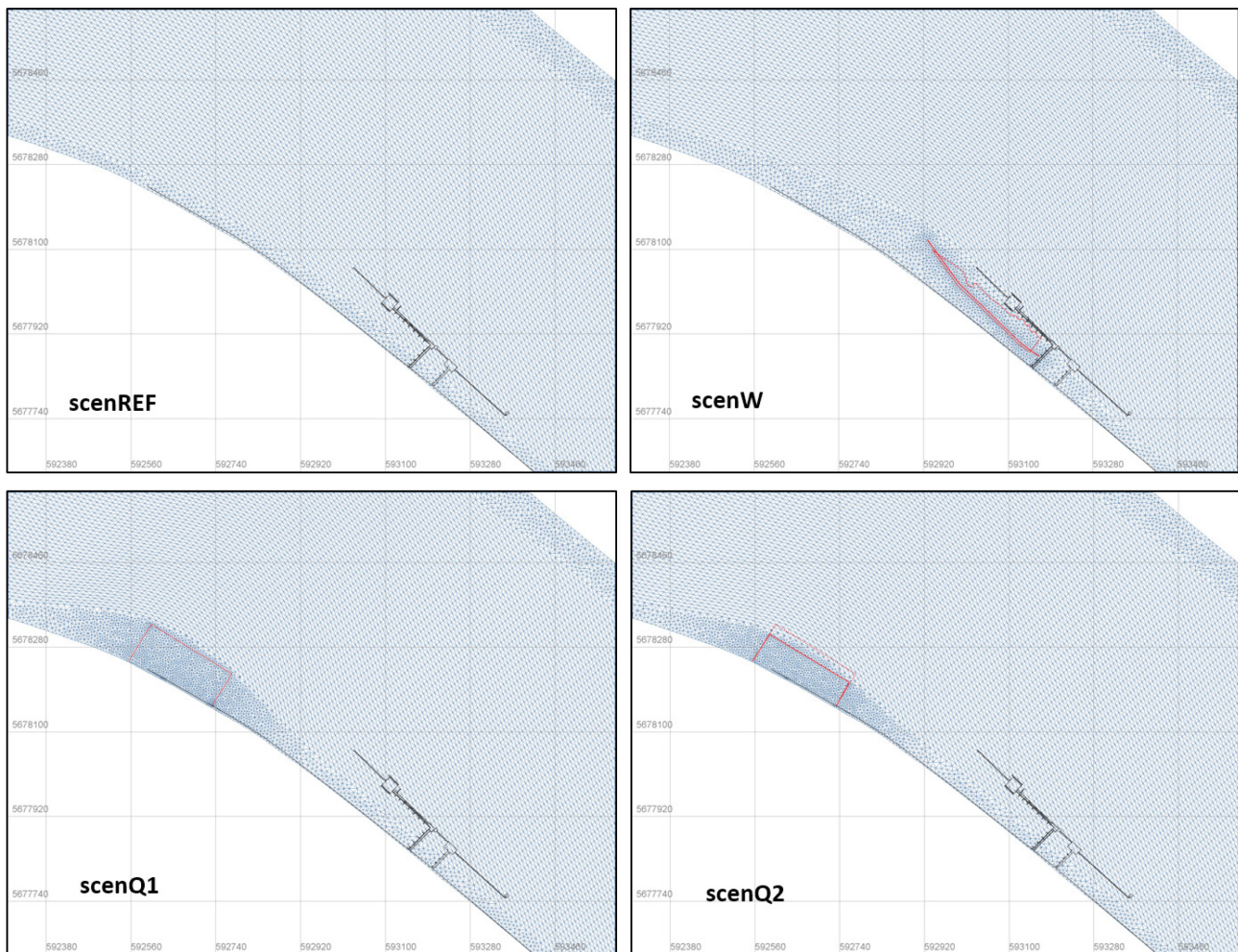
Table 8 – Model qualification based on RMAE (Sutherland *et al.*, 2004).

Model qualification		RMAE [-]
Excellent		<0.2
Good		0.2-0.4
Reasonable/fair		0.4-0.7
Poor		0.7-1.0
Bad		>1.0
Not Applicable		-

4.5 Computational grid adaptation

In Chapter 3 was found that combining a channel mesh, aligned to the flow direction, to the unstructured grid leads to better performance of the model. However, for each of the investigated scenarios, the computational grid had to be properly adapted in the vicinity of the interventions in order to be aligned with the outline of the proposed constructions (Figure 23). Local refinement of the grid took place as well. The effect of the grid adaptations on the numerical results in the vicinity of the adaptations is discussed at the end of this chapter.

Figure 23 – Computational grid adaptation in the vicinity of the proposed interventions for each of the investigated scenarios. Red lines correspond to the outline of the interventions.



4.6 Effect on the maximum flood & ebb currents

The effect of the INEOS interventions (scenW, scenQ1, scenQ2) on the maximum velocity field during flood phase and during ebb phase of a spring tide [02/04/2018 12:00 - 03/04/2018 00:50] is shown in Figure 24 and Figure 26, respectively. The difference of these maximum velocity fields for each of the intervention scenarios against the Reference scenario, are shown in Figure 25 (flood phase) and Figure 27 (ebb phase). Note that the difference plots have resulted by using identical grids for each of the intervention scenarios and the Reference scenario.

According to the numerical results, the maximum flood velocities reach values around 2 m/s in the navigational channel, while maximum velocities of 0.9 m/s can be observed in the inner side of the jetty (Figure 24). Observing the difference velocity maps during flood phase (Figure 25) for the deepening/wall scenario (scenW), the main differences can be observed only locally in the inner side of the jetty, where maximum reduction of 0.3 m/s is found. Substantial differences between the quay-wall scenarios (scenQ1, scenQ2) and the Reference Scenario (scenREF) can be easily noticed in the vicinity of the new construction and the existing jetty. Obviously, velocity drops to zero within and around the outline of the construction, but it is also significantly decreased at close upstream and downstream distances (maximum reduction of about 1 m/s). In the case of scenQ1 simulation this decrease is about 0.7 m/s at the downstream tip of the

jetty and extends (with lower values around 0.3 m/s) along the outer side of the jetty. On the other hand, the maximum flood velocities increase substantially (0.3 – 0.4 m/s) in the inner side of the jetty for both quay-wall scenarios. At the area between the jetty and the quay-wall high increase in the maximum velocities in (0.7 m/s) is found for the scenQ2 simulation. Lastly, increase of maximum velocities around 0.2 – 0.3 m/s is observed in a relatively broad area at the center of the river (deeper of the downstream tip of the jetty) for both quay-wall scenarios.

Figure 24 – Maximum depth-averaged flow velocity (magnitude, m/s) during flood phase of spring tide for the Reference and the INEOS intervention scenarios.

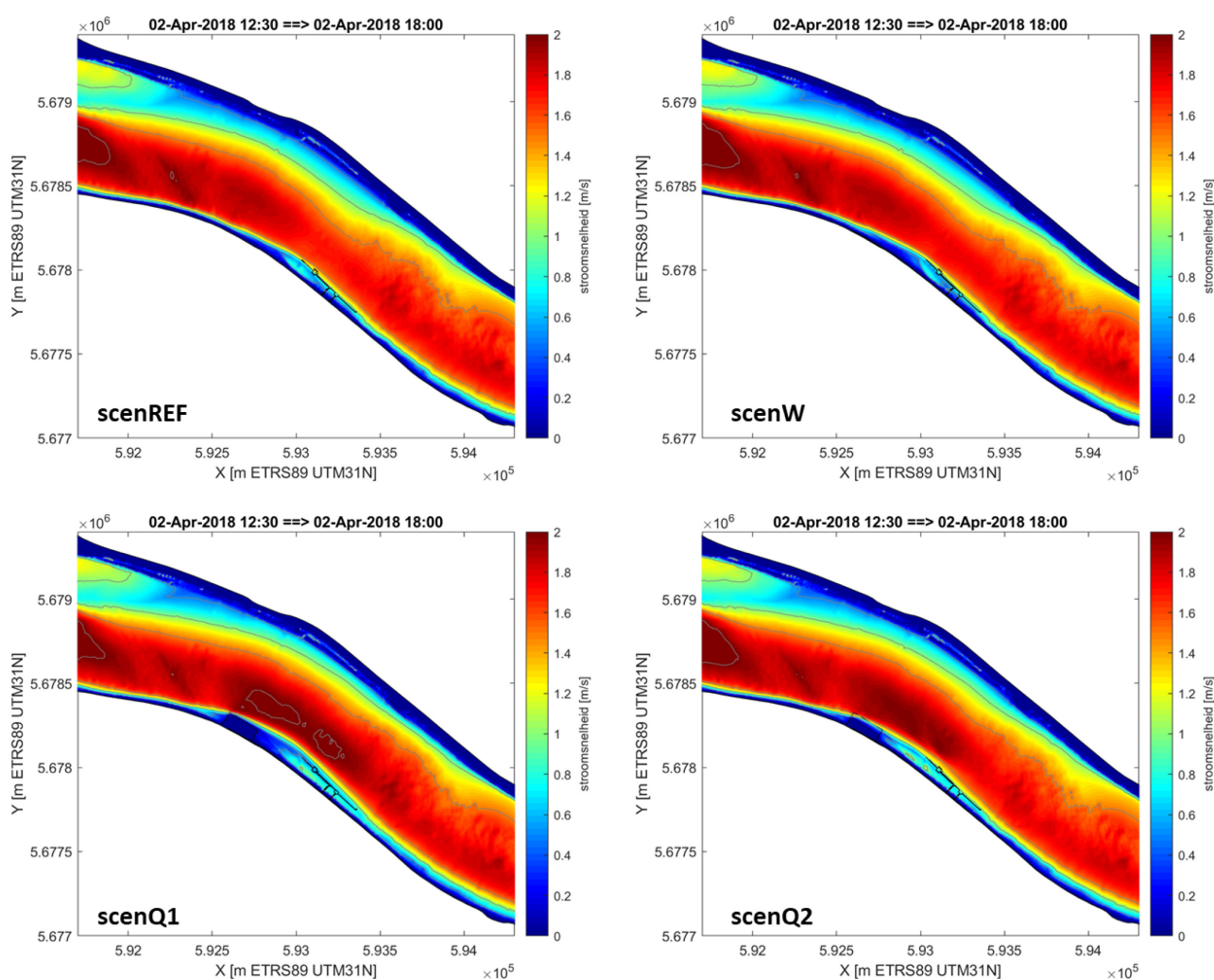
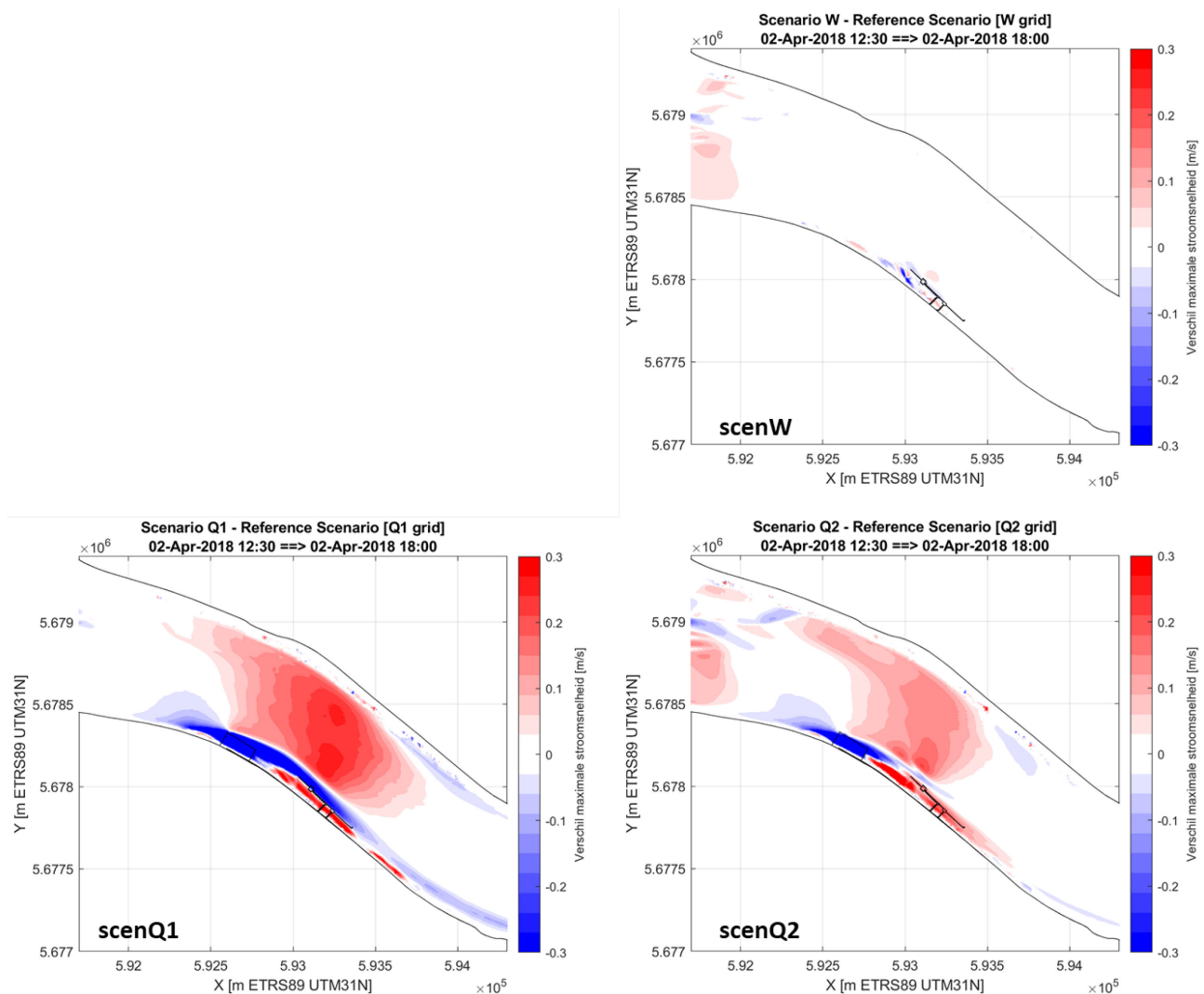


Figure 25 – Difference in maximum depth-averaged flow velocity (magnitude, m/s) during flood phase of spring tide between the INEOS intervention scenarios and the Reference Scenario.



The maximum ebb velocity map of the Reference scenario (Figure 26) shows that the flow velocities reach values around 1.5 m/s in the navigational channel, while maximum velocities of 1 m/s can be observed in the inner side of the jetty. Not many differences are observed in the difference velocity maps during ebb phase (Figure 27) for the deepening/wall scenario (scenW). Only local decrease of about 0.2 m/s in the inner side of the jetty is found. As expected, substantial differences between the quay-wall scenarios (scenQ1, scenQ2) and the Reference Scenario (scenREF) can be easily noticed in the vicinity of the new construction. It is found that the effect of the quay-wall on the velocity field at the jetty is minor during the ebb phase. Obviously, velocity drops to zero within and around the outline of the construction, but it is also significantly decreased at close downstream distances (maximum reduction of about 1 m/s). In the case of scenQ1 simulation this decrease is about 0.7 m/s at a distance of 100 m downstream of the left side of the quay-wall, while for scenQ2 it is relatively lower (-0.5 m/s). The maximum ebb current is decreased about 0.5 m/s and 0.2 m/s at an upstream distance of 50 m from the right side of the quay-wall for scenQ1 and scenQ2 runs, respectively. Lastly, increase of maximum velocities around 0.15 m/s is observed in a relatively broad area at the center of the river (deeper of the quay-wall) for scenQ1 simulation.

Figure 26 – Maximum depth-averaged flow velocity (magnitude, m/s) during ebb phase of spring tide for the Reference and the INEOS intervention scenarios.

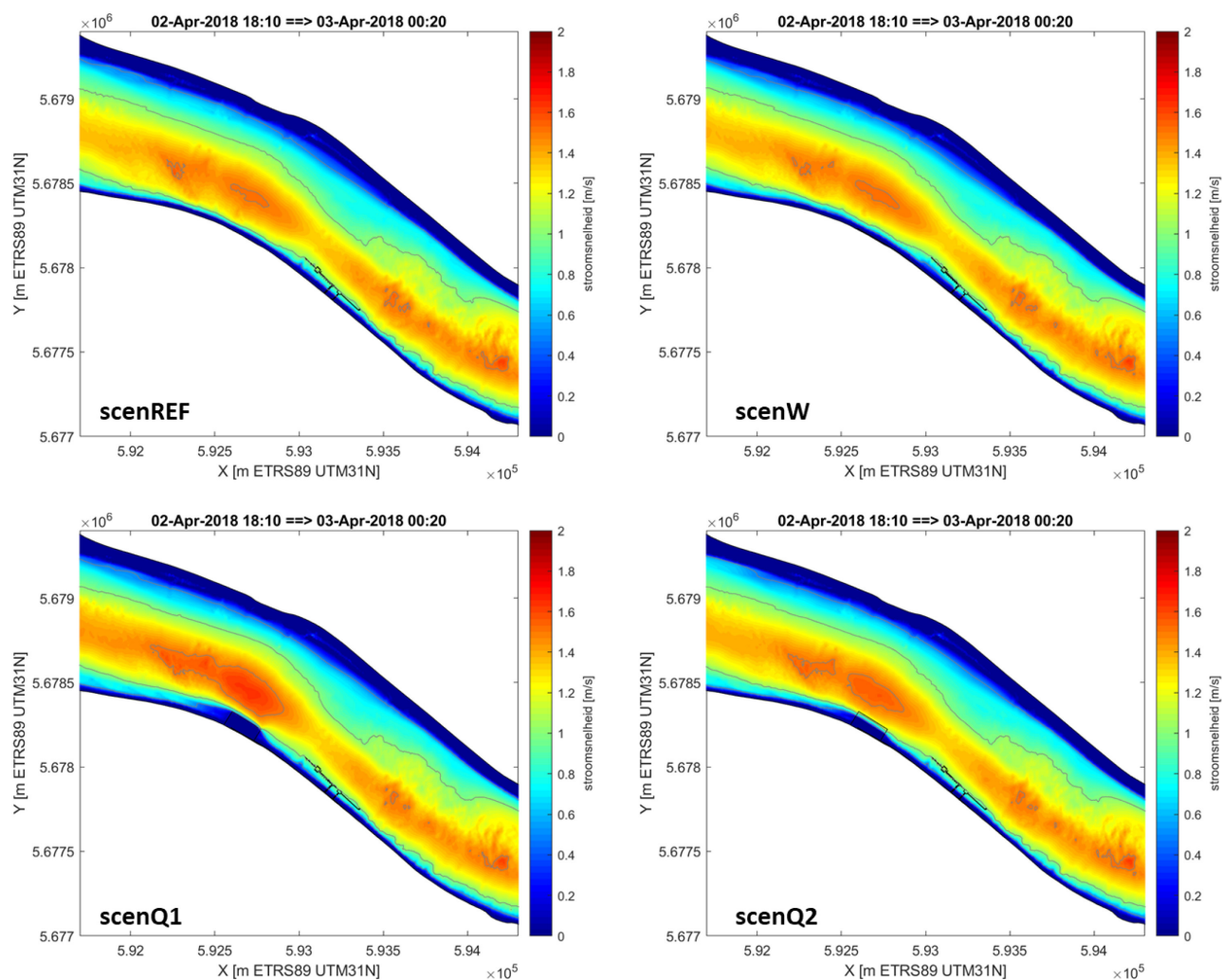
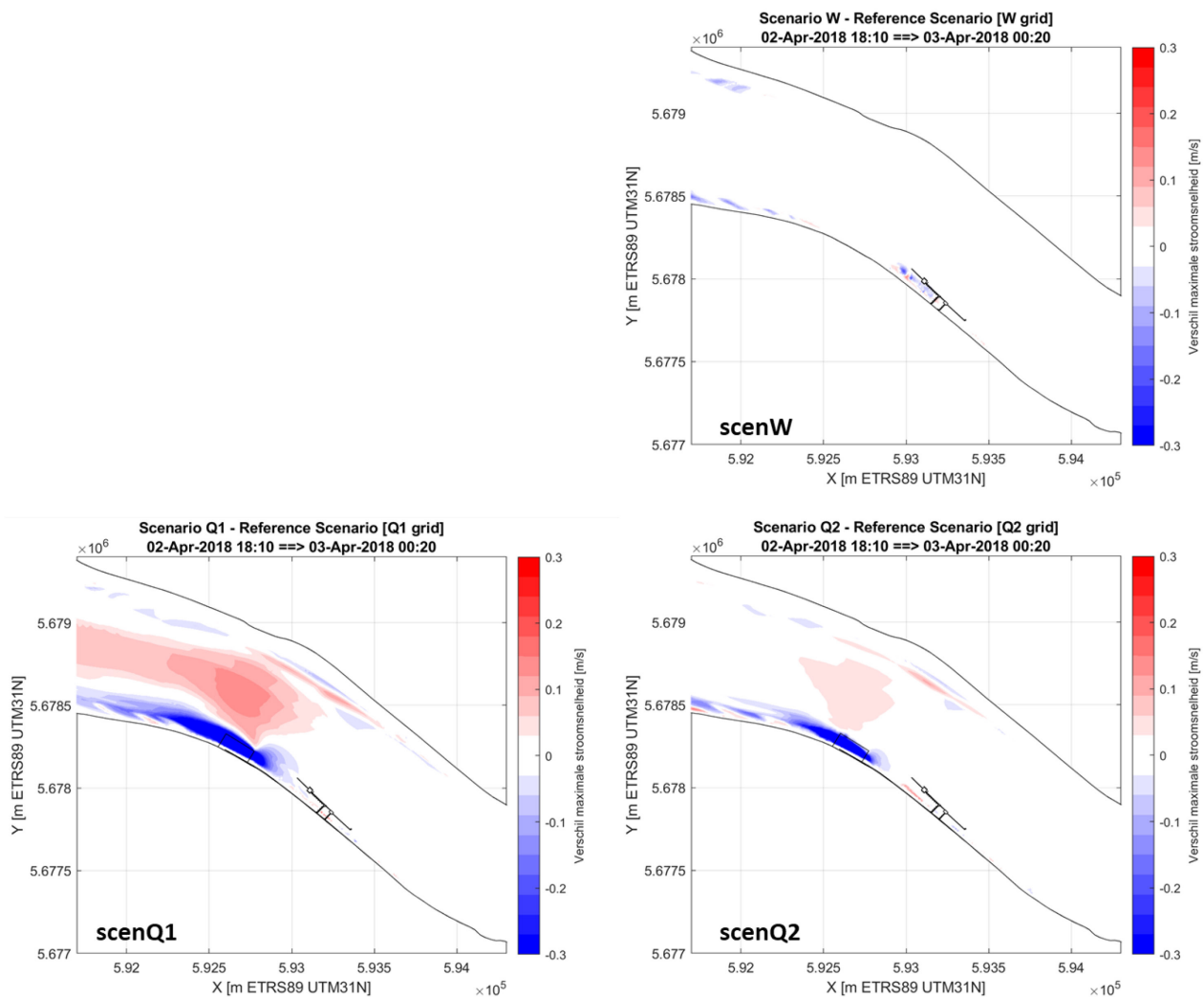


Figure 27 – Difference in maximum depth-averaged flow velocity (magnitude, m/s) during flood phase of spring tide between the INEOS intervention scenarios and the Reference Scenario.



In order to get a picture of the flow direction close to left bank, which is not depicted in the previous figures, the maximum flood and ebb velocity vectors along two cross-sections located at the area of the interventions are shown in Figure 28 and Figure 29. The first cross-section is located close to the downstream tip of the INEOS jetty (Figure 28), while the latter crosses transversally the middle of the new quay-wall (Figure 29). At the jetty cross-section, the maximum flood flow inside the jetty is directed downstream (flow is reversed) for all the scenarios. Apparently, the flow reversal is enhanced for the two quay-wall scenarios, while the maximum flood current (directed upstream) presents also slight increase (0.1 – 0.2 m/s). As expected, no or minor differences between the scenarios are found in the maximum ebb currents. At the quay-wall cross-section, it is shown that the construction of the terminal leads to higher velocities at maximum flood, while the maximum velocities, as already noted, decrease only just the outside of the quay. During the ebb phase, similar decrease just outside the quay-wall front is found, and just a slight increase of the maximum current for scenQ1 run is observed.

Figure 28 – Maximum flow velocity vectors (depth-averaged) during flood and ebb phase of spring tide along a transect at the INEOS jetty for the Reference and the INEOS intervention scenarios.

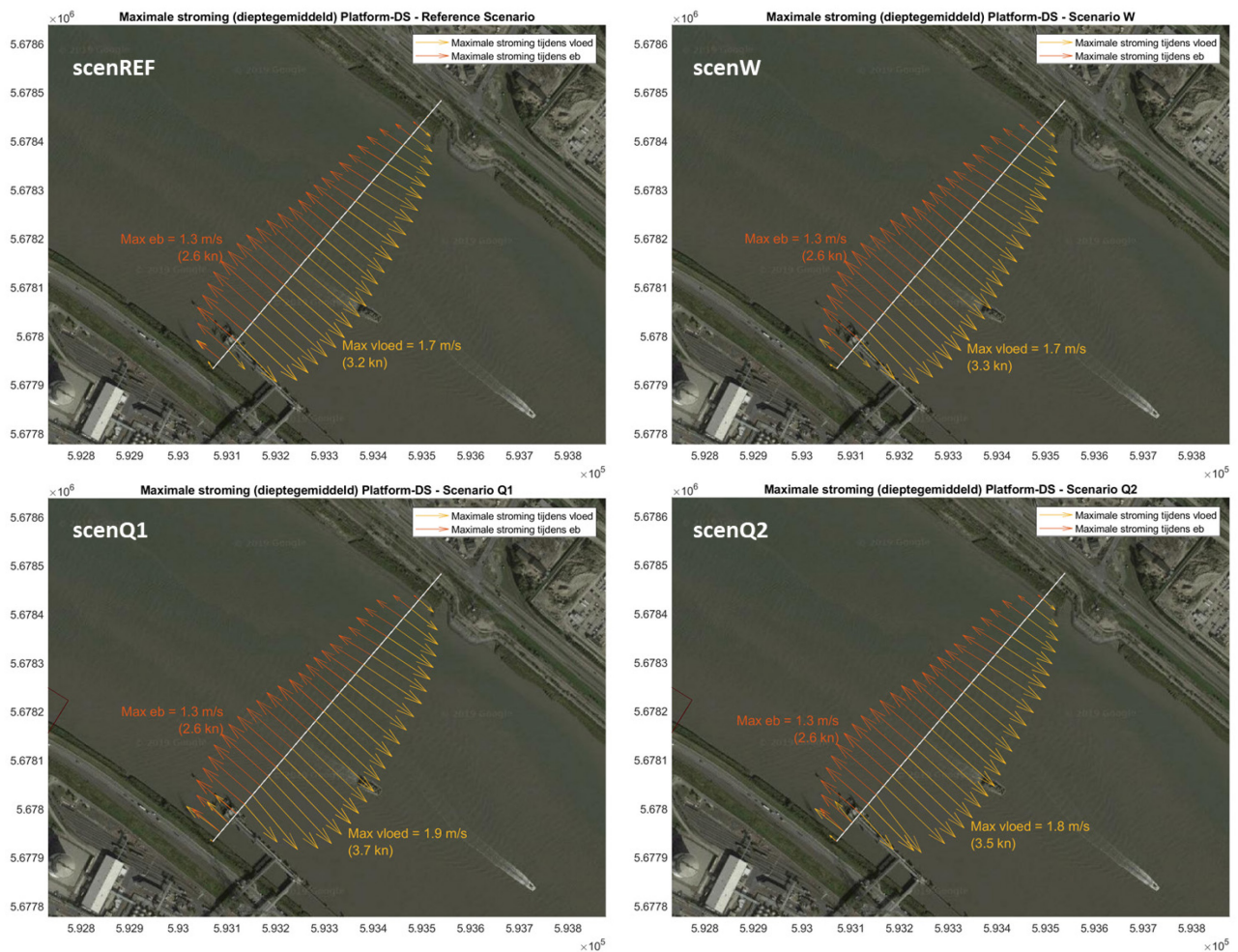
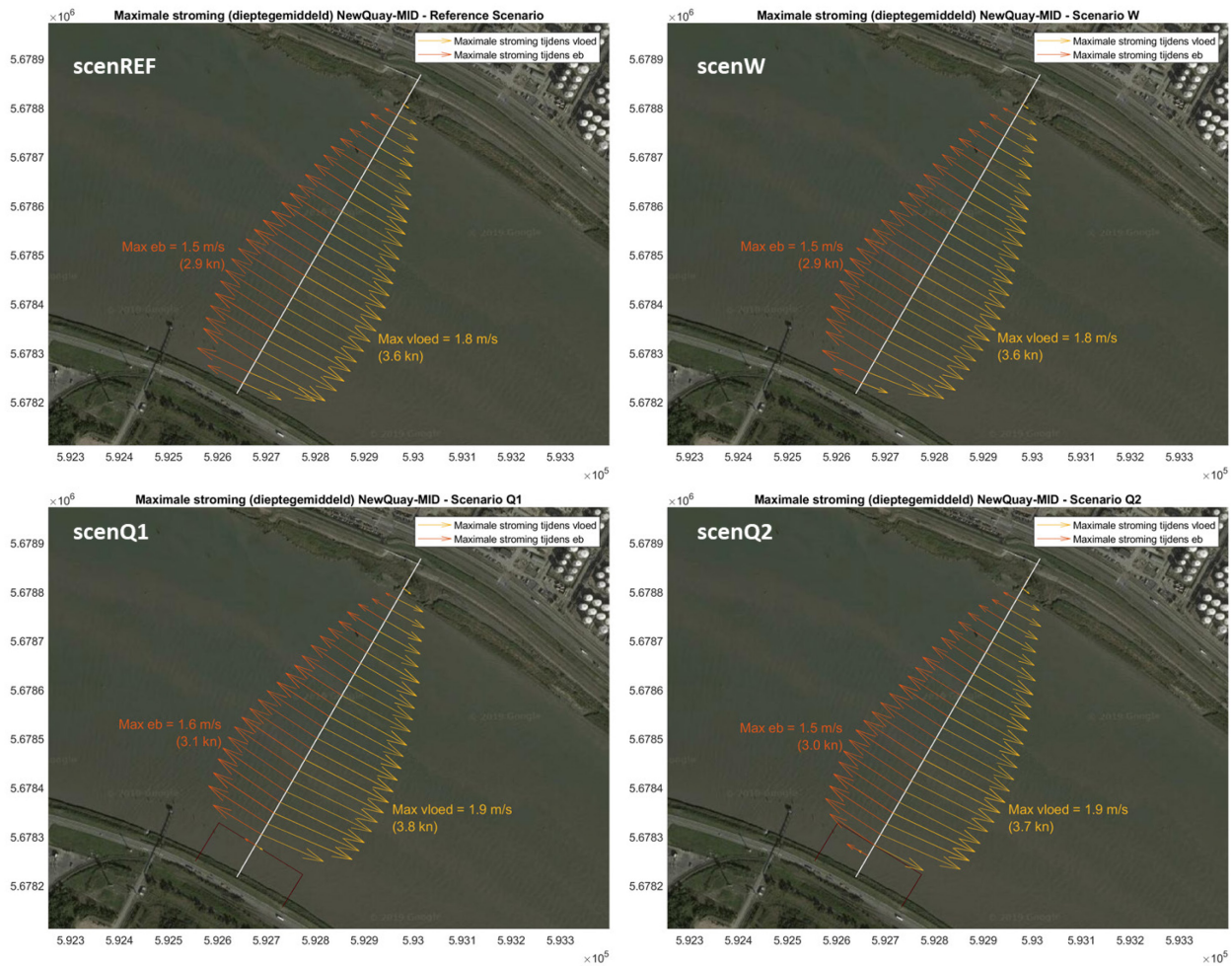
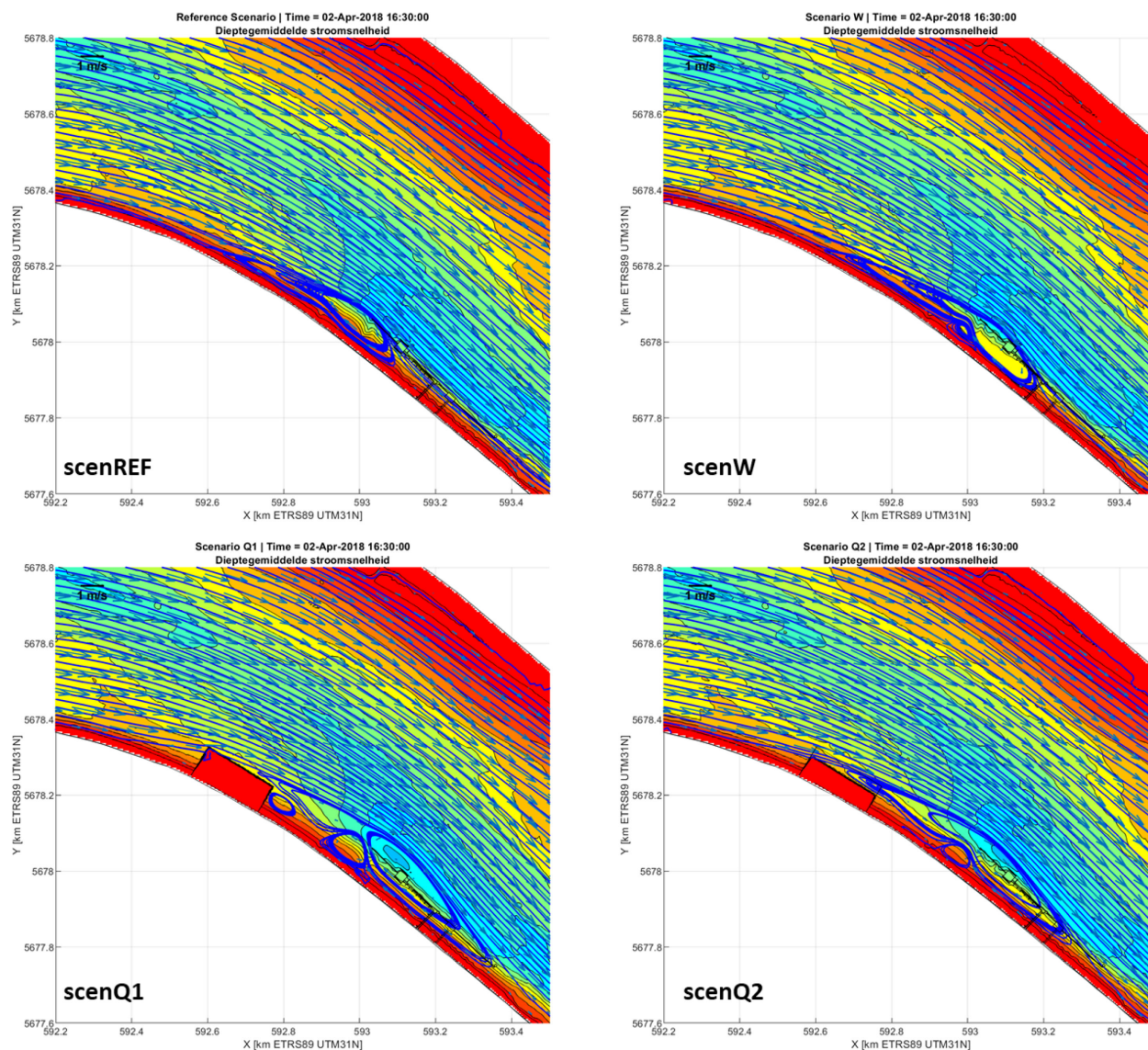


Figure 29 – Maximum flow velocity vectors (depth-averaged) during flood and ebb phase of spring tide along a transect at the new container quay for the Reference and the INEOS intervention scenarios.



The flow reversal at the area of the INEOS jetty during maximum flood, is further investigated by the visualization of the instantaneous flow field accompanied by the flow streamlines at an instant around the maximum flood flow (Figure 30). As indicated by the streamlines of the Reference scenario (scenREF), a relatively large recirculation area is generated during flood at the area downstream of the jetty, even without the construction of the quay-wall. The deepening/wall scenario (scenW) seems to enhance the intensity of the recirculation zone, which now occupies also the area inside the jetty. The construction of the completely closed quay-wall (scenQ1) further enhances the flow reversal, leading to the development of a system of clockwise and counter-clockwise vortices, which extends along the whole length of the jetty. The same behaviour but in less extent is observed at the peak flood current instant for the scenario with the piled quay-wall (scenQ2). It has to be noted, though, that the grid adaptation at the vicinity of the interventions introduces uncertainty in the representation of the flow reversal. Further discussion follows in the last section of the present chapter.

Figure 30 – Instantaneous flow velocity vectors (depth-averaged) and streamlines at maximum flood of spring tide [02/04/2018 16:30] for the Reference and the INEOS intervention scenarios. Bathymetric contours are depicted at the background.



4.7 Effect on sediment transport

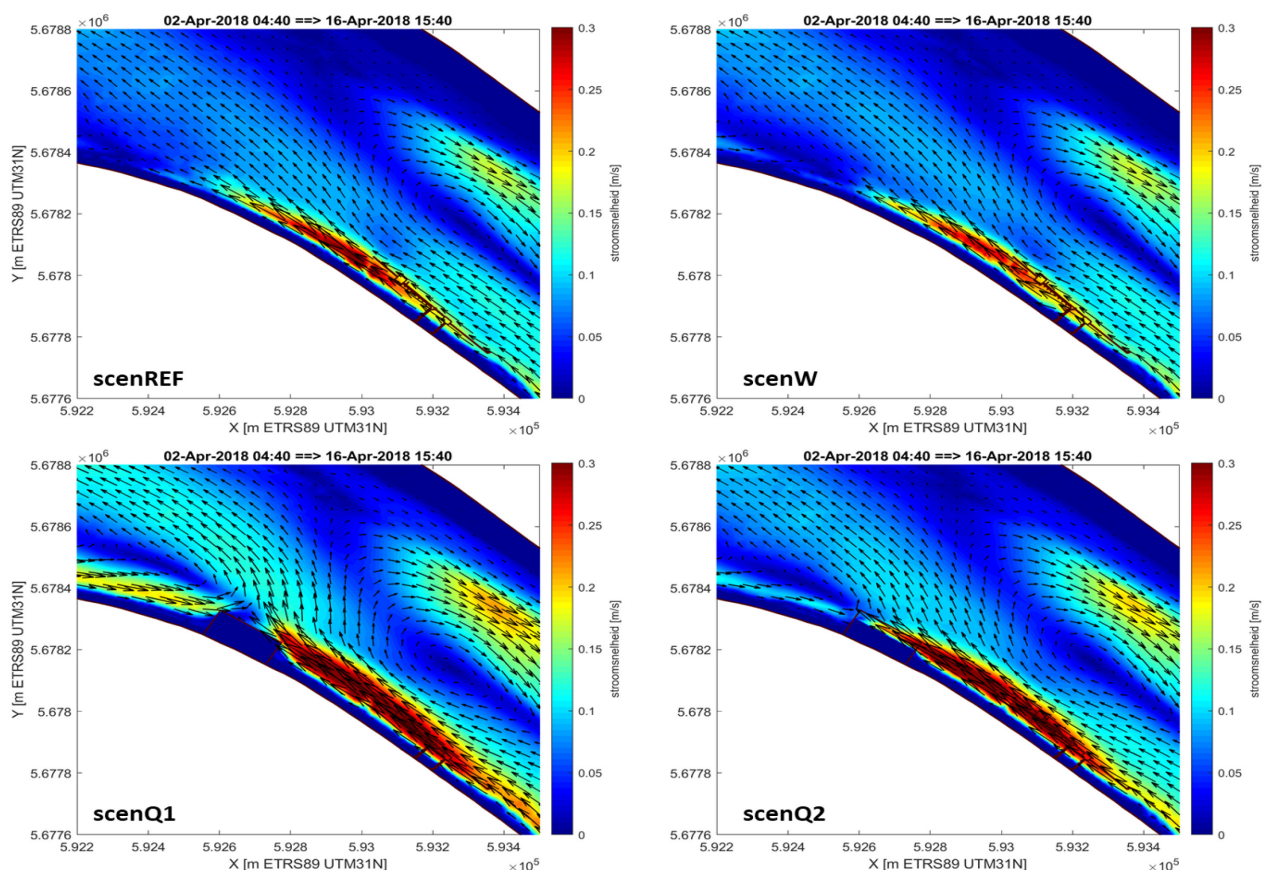
The effect of the interventions on sediment transport is investigated using variables that come from the hydrodynamic simulations. These are the following:

- Residual flow velocity
- Bed shear stress
- Potential bottom evolution

4.7.1 Residual velocity field

The residual flow velocities at the area of interest, calculated as the averaged velocities over one spring-neap tidal cycle (02/04/2018 04:40 – 16/04/2018 15:40), are presented in Figure 31 for all the investigated scenarios. The residual velocities are important as they affect the net sediment transport and thus the morphological development of the bottom. In the plot of the Reference scenario (scenREF) it is shown that relatively high residual velocities (~ 35 cm/s), directed downstream, occupy the area close to the left bank downstream of the INEOS jetty. The residual patterns are only slightly affected for the deepening/wall scenario (scenW), while the magnitude appears slightly reduced (~ 0.30 m/s) in this case. The construction of the new quay-wall is expected to enhance the downstream-directed residual velocity pattern at the area of the jetty, as shown in the plots of the two scenarios scenQ1 and scenQ2. The strengthening of the latter pattern regards more to the dimensions rather than the intensity of the current. Especially for the scenQ1 simulation the upstream-directed residual velocities downstream of the new quay-wall (scenQ1) are increased reaching values up to 0.2 m/s. The upstream-directed residual current outside the quay-wall is also enhanced compared to the current situation.

Figure 31 – Residual depth-averaged flow velocity (magnitude contours & vectors, m/s) averaged over a full spring-neap tidal cycle for the Reference and the INEOS intervention scenarios.



4.7.2 Bed shear stress

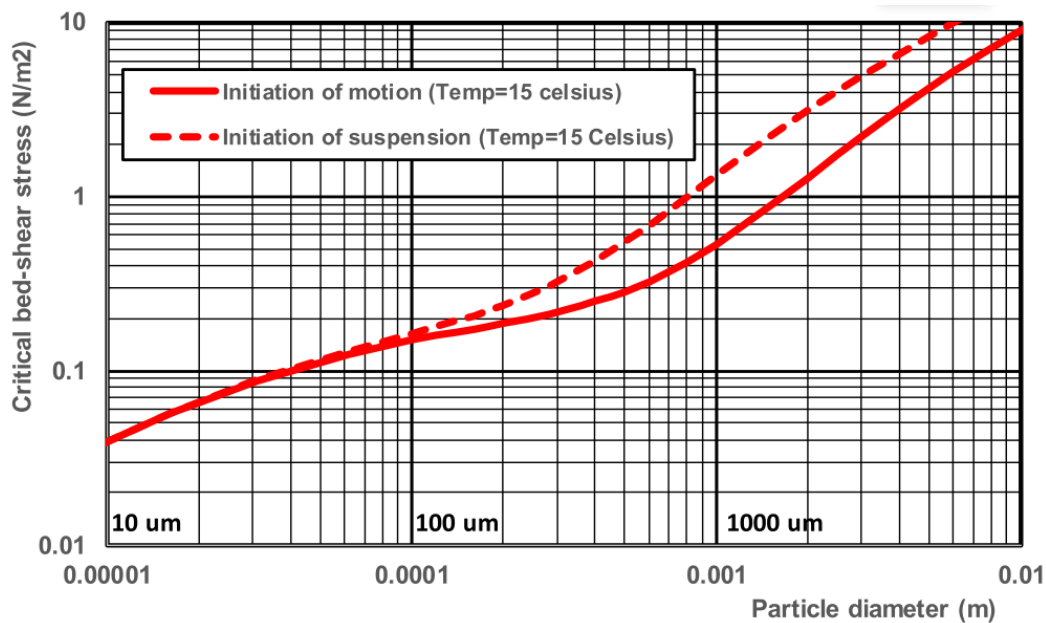
The bed shear stress is calculated by using the following formula:

$$\tau_b = \frac{1}{2} \rho C_f V^2$$

where ρ is the water density, C_f is the friction coefficient of the flow (based on the Manning's roughness) and V is the velocity magnitude.

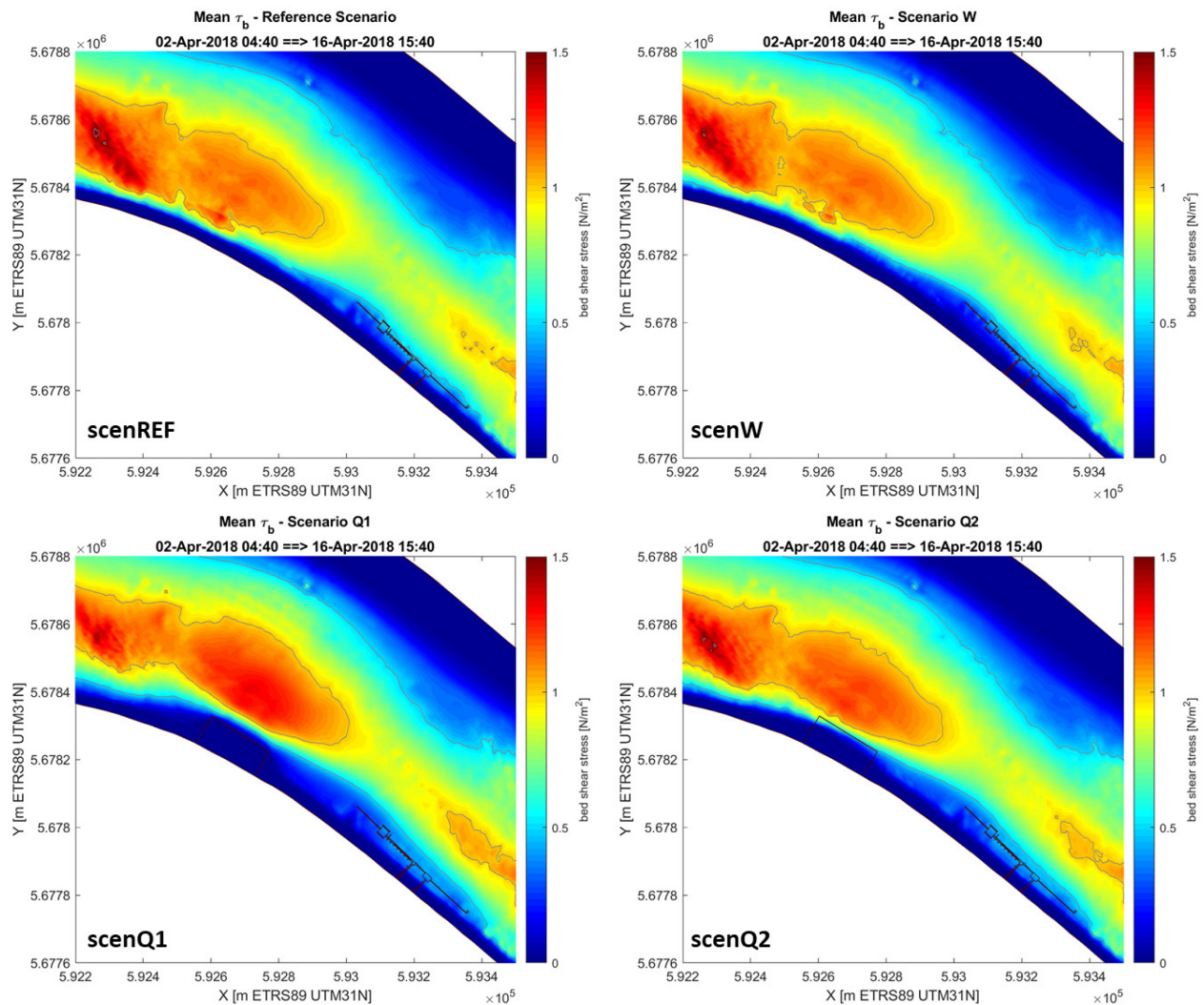
Bed shear stress is the driving mechanism to set the sediment at the bottom in motion. The critical bed shear stress for the initiation of motion and suspension of sediment (silt or sand) as a function of the mean grain diameter is given in Figure 32.

Figure 32 – Critical bed shear stress for the initiation of motion and suspension of sediment versus the mean grain diameter (Van Rijn, 2016).



The full spring-neap averaged bed shear stress for the simulated scenarios is shown in Figure 33. It is found that deepening the bottom in the inner side of the jetty (scenW) leads to local decrease of the mean bottom stress at the entrance of the jetty, while the τ_b distribution at the wider area remains unchanged compared to the Reference scenario. For the scenario of the close quay-wall (scenQ1) it is shown that τ_b maximum values outside the quay-wall front are increased substantially compared to the reference ones. On the contrary they are decreased close and around the terminal outline following the great reduction of mean velocity magnitude due to the blocking of the flow. The mean τ_b at the area between the quay and the jetty is slightly attenuated compared to the reference one, even though the maximum reverse flow between the quay and the jetty was presented enhanced during flood (Figure 25). Apparently the reduction of the flow velocities along the left bank during ebb, due to blocking of the flow by the quay, has stronger influence than the flow reversal strengthening during flood. For the piled quay-wall scenario (scenQ2) the reduction of the mean τ_b at the entrance of the jetty is almost negligible compared to the reference one. In general the disturbance of the mean flow characteristics is now less extensive compared to that of the completely closed terminal.

Figure 33 – Mean bed shear stress (N/m^2) averaged over a full spring-neap tidal cycle for the Reference and the INEOS intervention scenarios.



Another characteristic indication of how active is the area of interest in terms of sediment motion, is given in Figure 34 and Figure 35, where the percentage of time period during a full spring-neap tidal cycle that bed shear stress is greater than 0.1 N/m^2 and 0.5 N/m^2 , respectively, is presented. The specific τ_b values are selected based on the diagram of Figure 32, as estimated thresholds for the initiation of motion/suspension of silty material (0.1 N/m^2) and medium-size sand (0.5 N/m^2). Comparing the plots for all the investigated scenarios it is found that: (a) for the deepening/wall scenario, only the excavated area becomes less active compared to the current situation, (b) for the scenario with the closed quay-wall (Q1) the affected area is obviously larger and consequently sediment motion will become less active downstream and upstream of the new terminal, at the area between the jetty and the terminal but also at the downstream half of the jetty. For the 0.5 N/m^2 threshold case, the weakening of the sediment mobility extends to a broader zone along the left bank, which includes the entire area of the jetty but also an area further downstream of the new terminal. (c) For the scenario with the piled wall (Q2), very similar behavior to the scenQ1 scenario is observed, with the difference that the sediment mobility attenuation is now relatively smaller in magnitude and in extension.

Figure 34 – Percentage of time period during a full spring-neap tidal cycle that bed shear stress is greater than 0.1 N/m^2 for the Reference and the INEOS intervention scenarios.

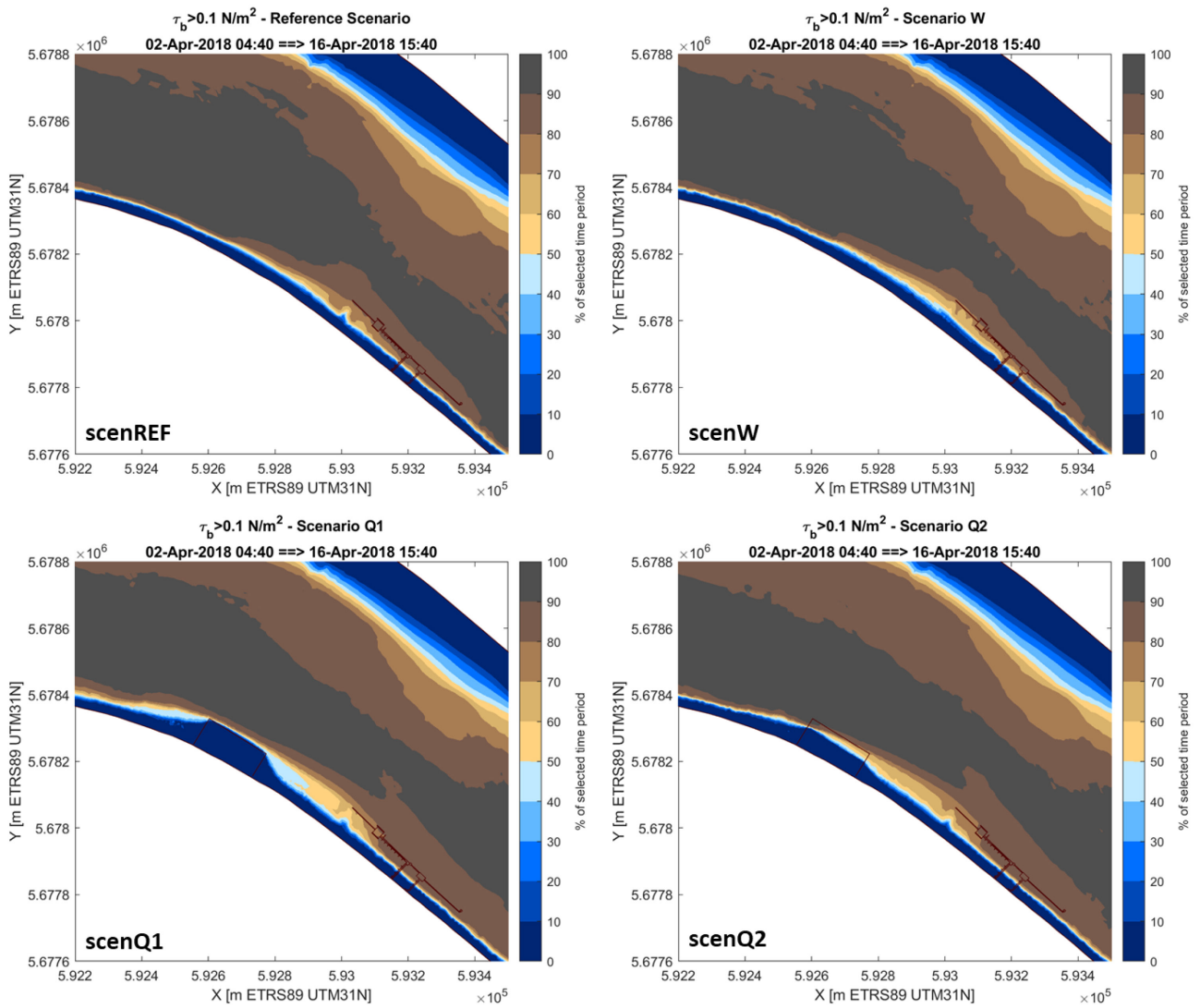
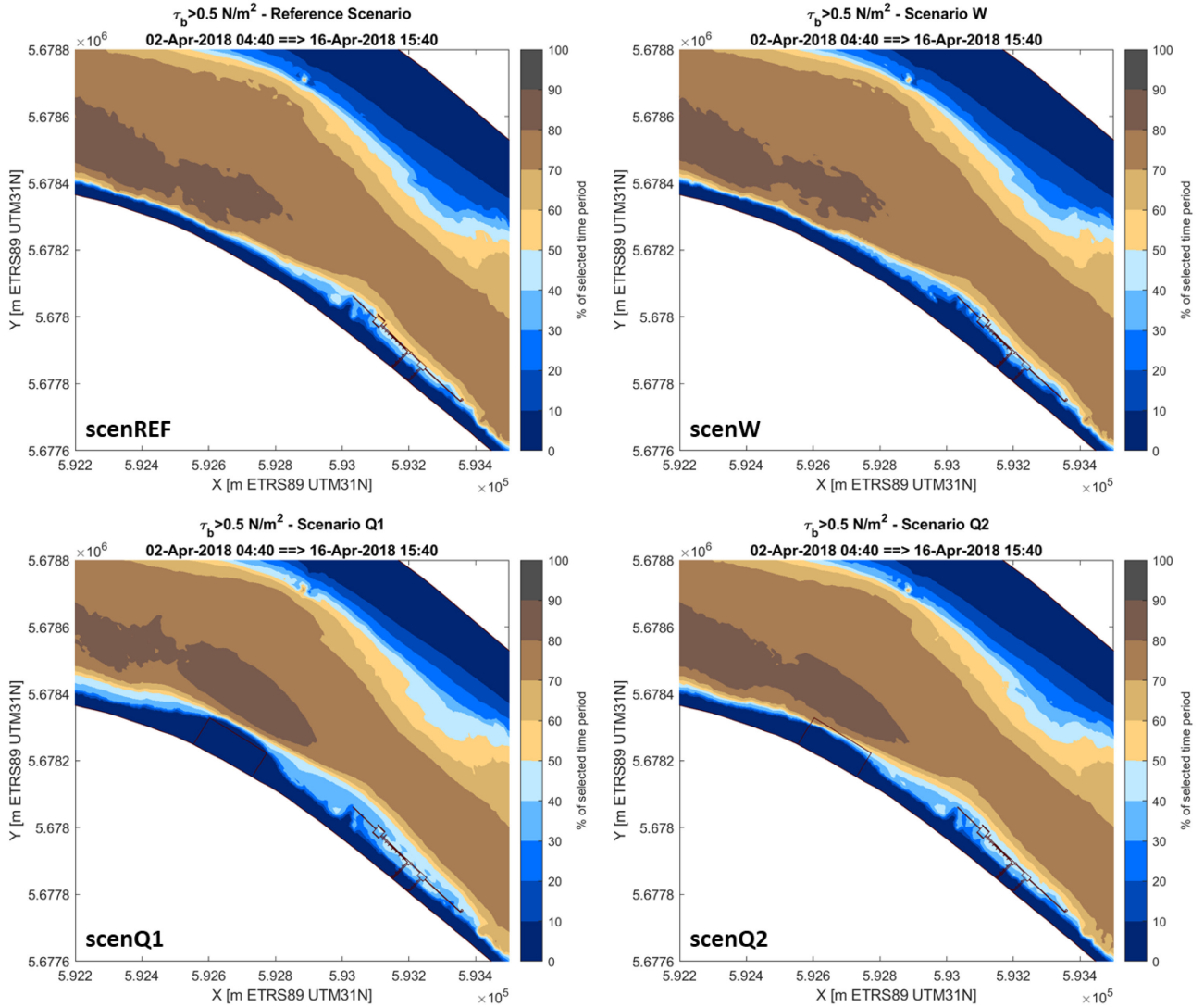


Figure 35 – Percentage of time period during a full spring-neap tidal cycle that bed shear stress is greater than 0.5 N/m² for the Reference and the INEOS intervention scenarios.



4.7.3 Potential bottom evolution

Since the online coupling of hydrodynamics with bed morphology evolution morphological modelling was out of scope of the present study, the potential bottom evolution was calculated after the completion of the simulations by introducing the calculated hydrodynamic variables in the Exner equation:

$$\frac{\partial \eta}{\partial t} = -\frac{1}{1-p} \nabla q_s$$

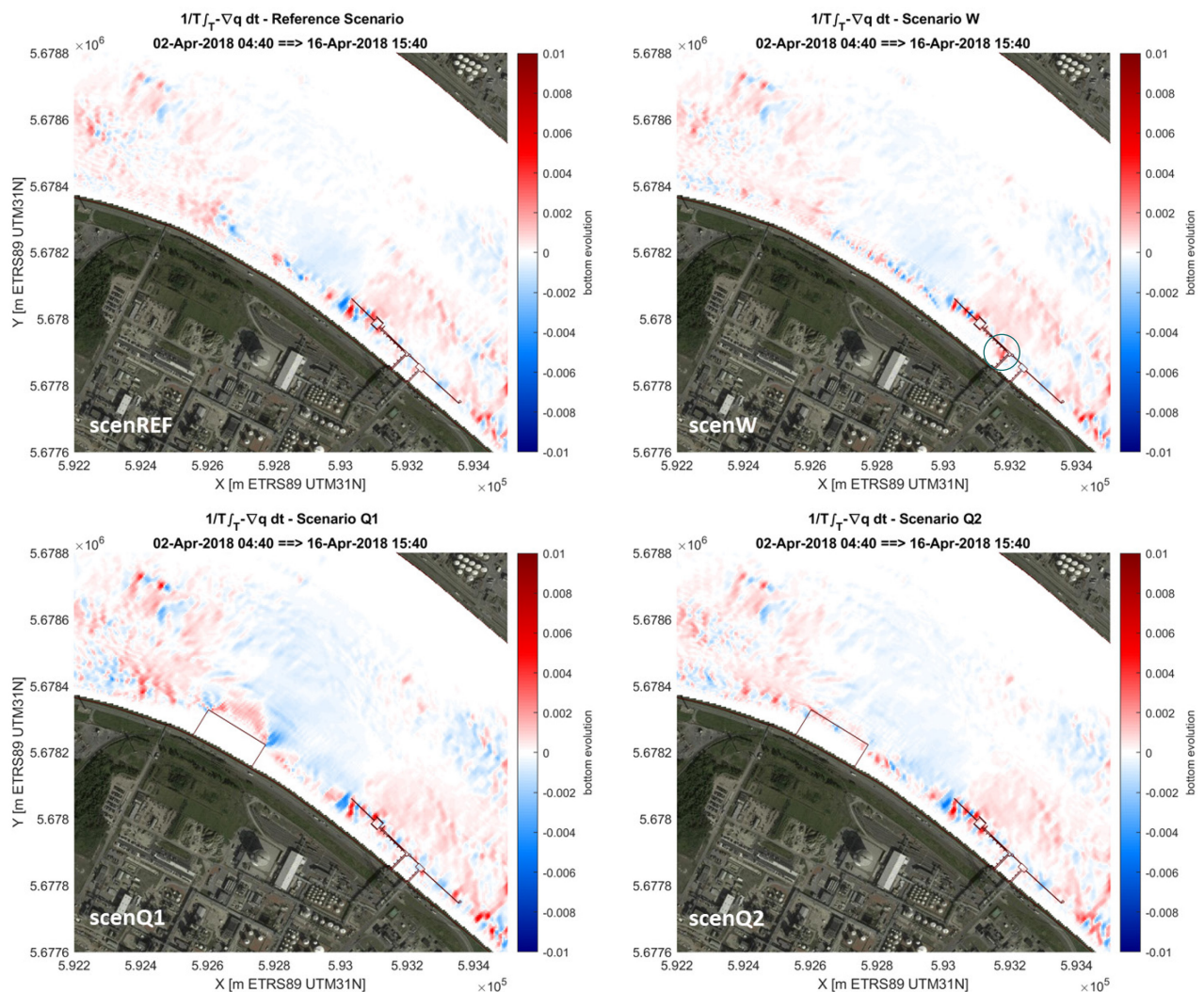
where η is the bottom level, p is the sediment porosity and q_s is the sediment flux. The sediment flux can be expressed according to Bagnold (1966) (see also (Levy *et al.*, 2017)) as

$$q_s = c \|u\|^2 u$$

where c is a calibration factor independent of the grain size, and u is the velocity. The sediment flux equation is applied separately for each of the horizontal dimension x and y .

In our study the sediment flux components (calculated every 10 minutes) were averaged for a full spring-neap cycle and then the divergence of the mean fluxes was calculated (in Matlab). According to the Exner equation this term is indicative of the mean bottom evolution. In the manner that it is used in this study, though, it only shows the trends of bottom change and it doesn't give quantitative estimations. The aforementioned bottom change trends for all the investigated scenarios are given in Figure 36. The main findings are: (a) for the deepening/wall scenario (W) not many differences are observed with regard to the reference scenario. However, it can be noted that no bottom change patterns appear at the excavated area whereas sediment is accreted at the corner inside the jetty just upstream of the excavation (see circled area). (b) For the scenario with the closed quay-wall (Q1) the model predicts sedimentation in front of the terminal, erosion at the upstream corner and less activity in the area between the jetty and the terminal. Additionally the bottom change patterns downstream of the quay seem to be more pronounced than those of the reference scenario. (c) For the scenario with the piled quay-wall (Q2) the impact on the sedimentation-erosion patterns is similar but less strong than that of scenario Q1.

Figure 36 – Mean potential bottom evolution averaged over a full spring-neap tidal cycle
for the Reference and the INEOS intervention scenarios.



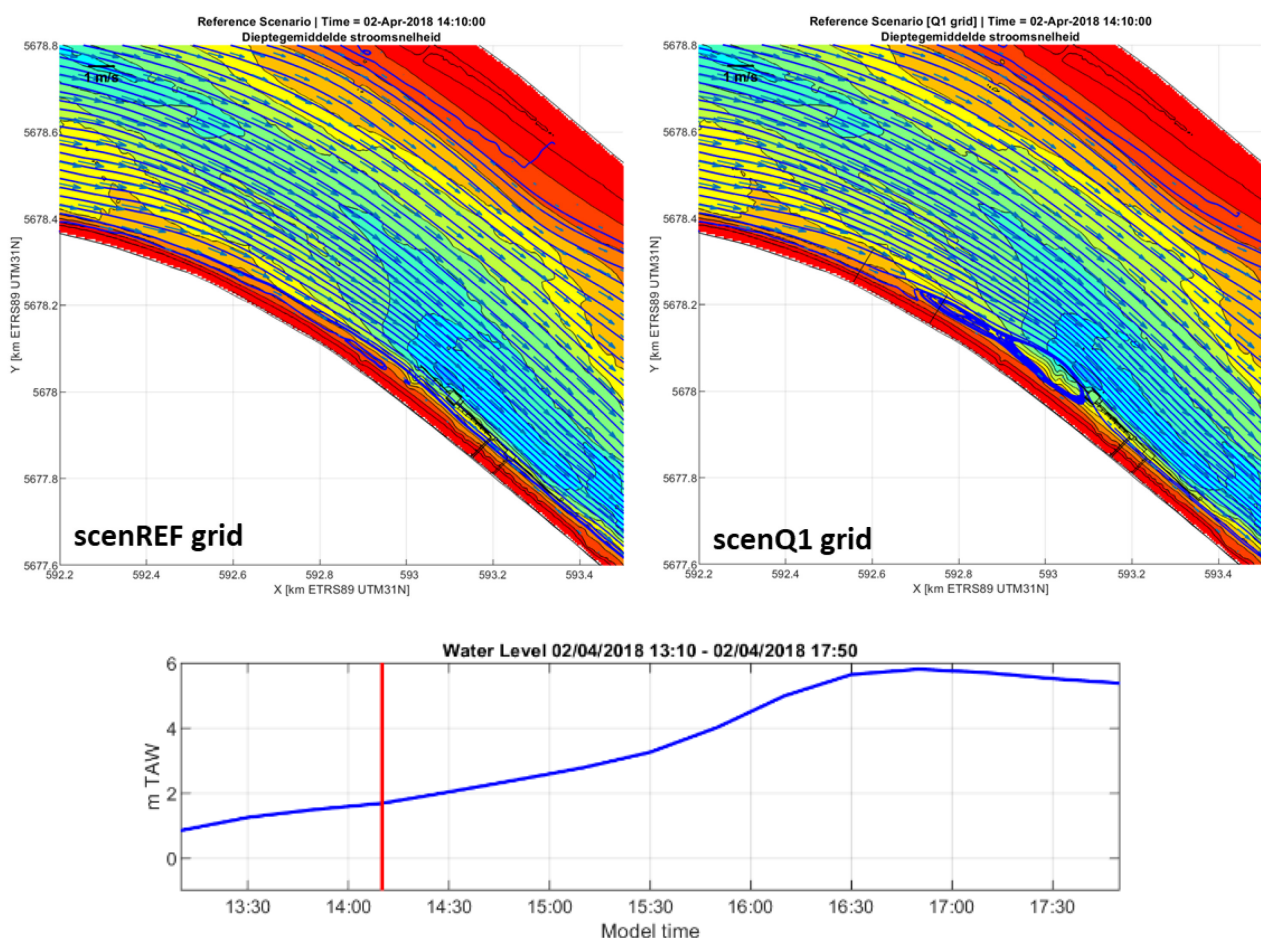
4.8 Discussion

In this section the effect of the local grid adaptation (presented in section 4.5) on the numerical results in the vicinity of the adaptations is further discussed. For this reason, the results of two different simulations of the Reference scenario, with identical model settings and forcing, but with different computational grids are investigated. In Figure 37, snapshots of the velocity field corresponding to the scenREF grid (left figure) and the scenQ1 grid (right figure), are presented. It is clearly shown that the scenQ1 grid predicts the formation of a recirculation zone along the left bank, while in the computation with scenREF grid (at the same instant) this zone is not present. It is conjectured that this local variation of the flow field, with the enhancement of the flow reversal for scenQ1 grid, is related with the bending of channel mesh around the new quay (see Figure 23). This bending of the channel mesh is accompanied with the use of unstructured elements around the location of the quay, a formation which apparently affects the direction of numerical diffusion compared to the case where the channel mesh strictly follows the flow direction. More details about the effect of flow-aligned unstructured grids (finite element method) on the numerical diffusion can be found in Holleman *et al.* (2013).

Figure 37 – Flow velocity vectors and streamlines at an instant early in the flood phase of spring tide [02/04/2018 14:10] for the reference scenario using the scenREF grid (left figure) and the scenQ1 grid (right figure).

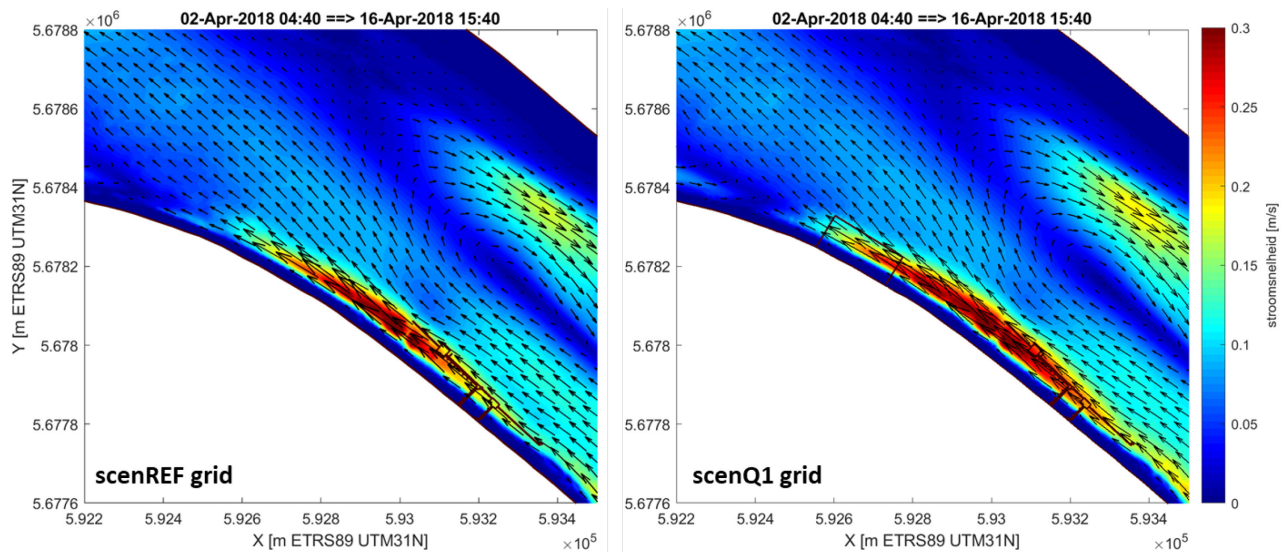
Bathymetric contours are depicted at the background.

Bottom figure: Water level variation in time (red line denotes the time of the flow field snapshots).



The grid adaptation has a local (limited) effect on the mean flow characteristics as shown in Figure 38, where the residual flow velocity over the full spring-neap cycle for the two grid variations, is presented. Obviously, the enhancement of the flow reversal during flood phase, by the scenQ1 grid formation, results into increased magnitude of the downstream-directed residual velocity along the left bank compared to the one of the scenREF grid.

Figure 38 – Residual depth-averaged flow velocity (magnitude contours & vectors, m/s) averaged over a full spring-neap tidal cycle] for the reference scenario using the scenREF grid (left figure) and the scenQ1 grid (right figure).



5 Conclusions

In the present study, a detailed TELEMAC-2D hydrodynamic model was developed for the modelling of three scenarios of interventions at the area of the INEOS facilities, i.e. the deepening of the existing jetty and a new quay wall construction (two variations). The main objective was to provide insight into the future maintenance requirements by making use of the numerical modelling results. Apart from this, the present study focus on how selected numerical and physical parameters (grid formation, bed roughness, turbulence) can affect the performance of the numerical model when simulating the hydrodynamics of tidal estuaries, through a series of sensitivity tests. The effect of local grid adaptation on the numerical diffusion and consequently on the results is further discussed.

For the model's calibration, the following measurements were used: (a) ADCP transect measurements at the closest to the area of interest transect, i.e. Oosterweel, (b) single-point velocity measurements at Oosterweel and (c) water level measurements at Kallo and Antwerp stations. Then, after the validation of the model the results of the scenarios' simulations were presented and evaluated.

For the effect of the interventions on the flood and ebb currents, it was found that:

- the deepening scenario has small effect on the maximum flood and ebb currents limited at the excavated area.
- for both variations of the new quay-wall (completely closed & half-piled) the maximum flood velocities increase substantially (0.3 – 0.4 m/s) in the inner side of the jetty and at the area between the quay and the jetty. The maximum ebb currents are substantially reduced at the downstream, lee side of the quay.
- a relatively large recirculation area is generated during flood at the area downstream of the jetty, even without the construction of the quay-wall. All the scenarios enhance the intensity of the recirculation zone, but mostly closed quay-wall, which leads to the development of a system of clockwise and counter-clockwise vortices, which extents along the whole length of the jetty.

For the effect of the interventions on sediment transport, it was found that:

- for all the investigated scenarios (Reference included) relatively high residual velocities (~30 cm/s), downstream directed, occupy the area close to the left bank downstream of the INEOS jetty. The residual patterns are enhanced (mainly in dimensions rather than in intensity) in the case of the two quay-wall scenarios.
- the mean bottom stress (τ_b) at the entrance of the jetty is slightly decreased for the deepening scenario. For the closed quay-wall scenario, the mean τ_b at the area between the quay and the jetty is slightly attenuated compared to the reference one, while for the piled quay scenario results are closer to the reference ones.
- the mild effect of the interventions on the mean τ_b distribution is reflected on the calculated potential bottom evolution, with slight differences in the sedimentation/erosion patterns. Major differences are spotted only at limited distance from the new construction's outline.

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