Sisyphe

Validation Manual

Version v7p3
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Contents

1. Yen

1.1 Purpose

The purpose of this test is to assess the accuracy of SISYPHE at reproducing the bed evolution in an alluvial channel bend under unsteady-flow conditions. The mechanics of sediment transport in channel bends, frequently appearing in natural rivers, are much more complex than that in straight channels. The complexity is twofold. On the one hand, the sediment transport in a channel bend is subject not only to longitudinal transport but also to transverse transport and transverse sorting by the secondary flow inherently associated with bends. On the other hand, the unsteadiness of flow in natural rivers certainly has some effects on the structure of the flow field, thereby affecting the motion of sediment particles.
This test is the experimental setup (RUN 5) proposed by Yen and Lee (1995). In this case, the bed evolution of a 180° channel bed with an initial flat bottom is computed for a triangular-shaped 300 min. hydrograph. Numerical results are validated by measured contours of bed evolution after at the end of the experience and by measured bottom elevations at two different cross sections (90° and 180°). This validation case can be performed for uniform or graded sediment distribution.

1.2 Problem setup

The flume consists of a straight section of 11.5 m long, a 180° bend of 4.0 m radius and a downstream straight section of 11.5 m long, with a constant slope in flow direction equal to 0.002. The width of the flume channel is 1.0 m. A triangular-shaped inflow hydrograph with an initial discharge of $Q = 0.02$ m^3/s, a water depth at the outflow of $h = 0.0544$ m and a peak discharge of equal to 0.053 m^3/s (water depth $h = 0.103$m) at $T = 100$ min is used, see Figure 1.1. After $T = 100$ min, the inflow discharge is reduced linearly until it reached the initial values at the end of the experiment ($T = 300$ min).
The sediment is characterized by a median diameter of $D_{50} = 1$ mm. This value is used for the case uniform sediment. For the case graded sediment, five sediment classes with diameters $D = 0.31, 0.64, 1.03, 1.69$ and 3.36 mm are chosen to reproduce the sediment distribution of the experiment. For this case, an initial distribution of 20% for each class is adopted. The Engelund-Hansen formula is adopted to estimate the sediment transport capacity of the channel. The slope effect and the secondary currents correction are accounted for this test. The influence of the slope effect on the the direction of the bedload transport is accounted through the Talmon formula, with $\beta_2 = 0.85$. The influence of the slope effect on the the magnitude of the bedload

transport is accounted through the Soulsby formula, with an friction angle of 35° The default value of $\alpha = 1$ is used for the secondary currents parameter, therefore the Engelund parameter $A = 7$. For the case graded sediment, two vertical sediment layers with a total thickness equal to 20 cm are assumed.

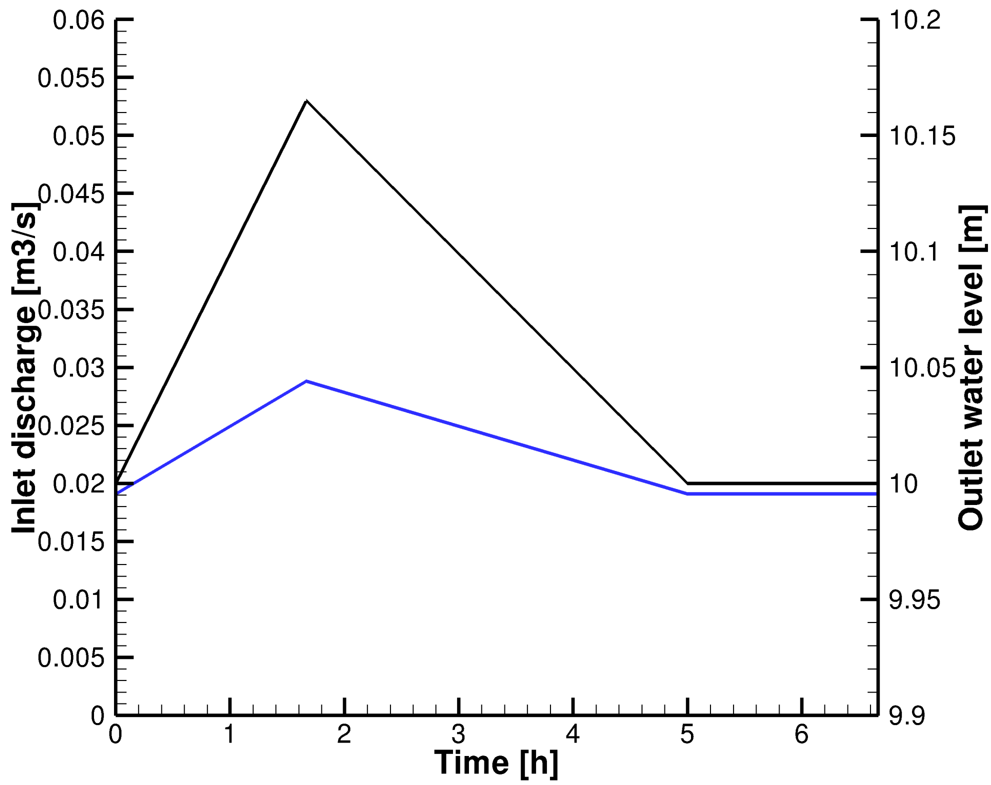


Figure 1.1: Triangular-shaped hydrograph.

A friction closure relationship, based on the Nikuradse roughness length is adopted to account for the bed resistance. For this case, $k_s = 3.5$ mm ($\approx 3 \times D_{50}$) and the Elder model is specified to parameterize the turbulent eddy viscosity. The critical Shields parameter is set at 0.047 and the bed porosity is 0.375.

1.2.1 Numerical setup

Numerical simulations were conducted on an unstructured, triangular finite element mesh with 3230 elements and 1799 nodes and a mean grid size of the order of 0.20 m (Figure 1.2). As initial condition, a fully developed (stationary) flow with a constant water-depth $h = 0.0544$m and discharge 0.02 m^3/s is imposed and the bottom has a constant slope in flow direction equal to 0.002.

The time step is set to 0.5 s. For a mean velocity in the range $[0.37 - 0.53]$ m/s and a mean grid size of the order of 0.2 m, the mean Courant number varies between 0.6 and 1.3.

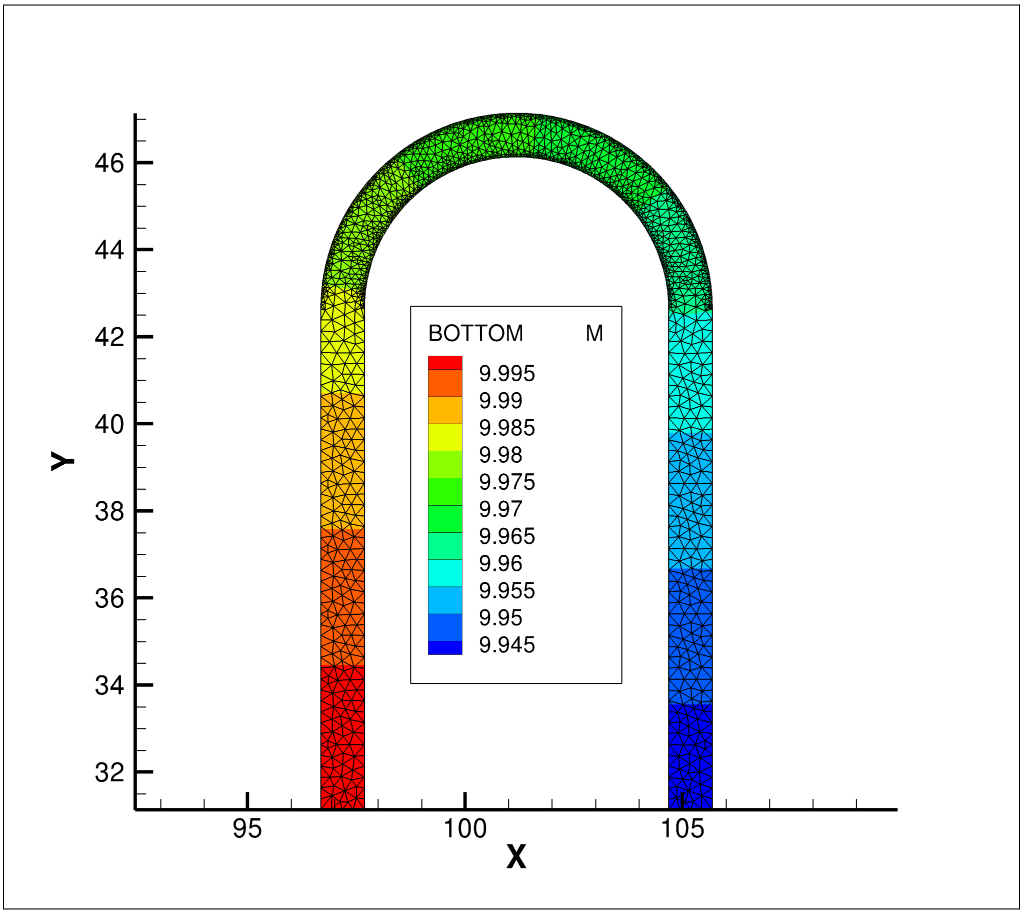


Figure 1.2: Finite element discretization of the bend.

1.3 Results

Numerical results of the normalized bed evolution are shown in Figure 1.5. Morphological changes exhibit the expected patterns of erosion and sedimentation at the channel bend, with the presence of a point bar along the inner-bank and a deeper channel along the outer-bank of the bend. The computed bed changes are in agreement with the measured data. Without accounting for the secondary flow effect, one cannot obtain such reasonable results. Numerical and observed bottom profiles at cross sections 90° and 180° are presented in Figure 1.6 for a total time equal to 5 hs.

1.4 References

Yen, C. and Lee, K.T. (1995) *Bed Topography and Sediment Sorting in Channel Bend with Unsteady Flow*. Journal of Hydraulic Engineering, Vol.121, No. 8.

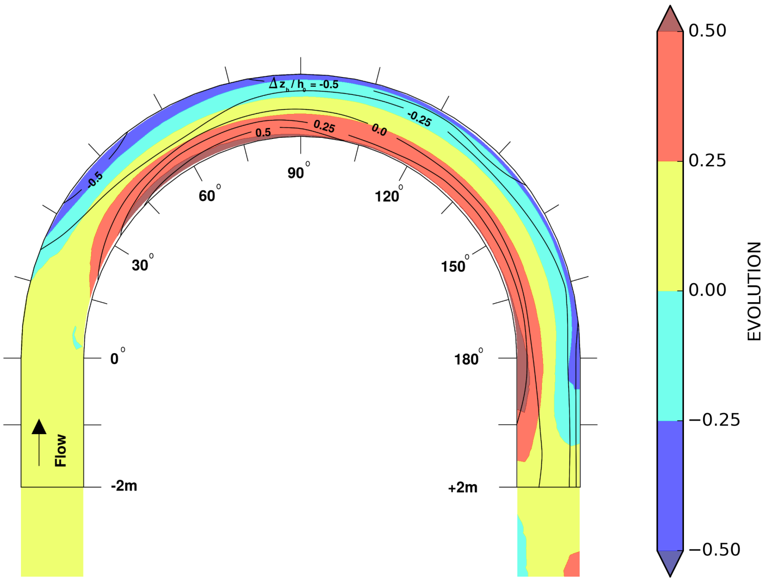


Figure 1.3: Comparison of simulated (coloured) and measured (black contour lines) normalized bed evolution.

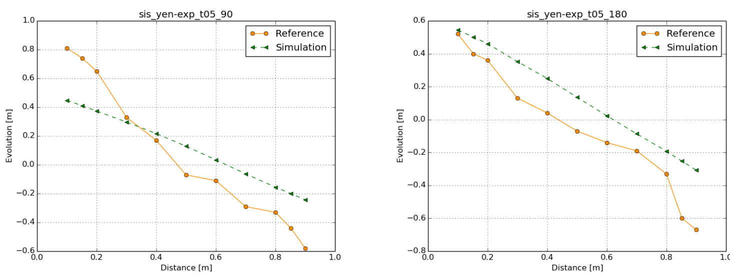


Figure 1.4: Comparison of simulated and measurement bottom elevation at cross section 90° (left) and 180° (right).

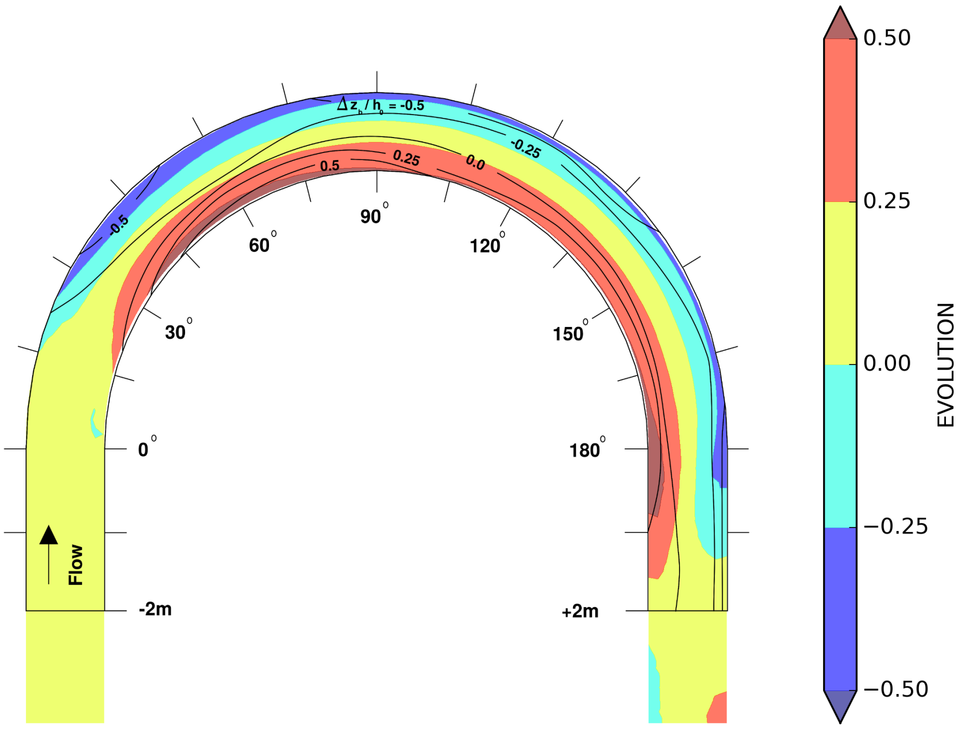


Figure 1.5: Comparison of simulated (coloured) and measured (black contour lines) normalized bed evolution.

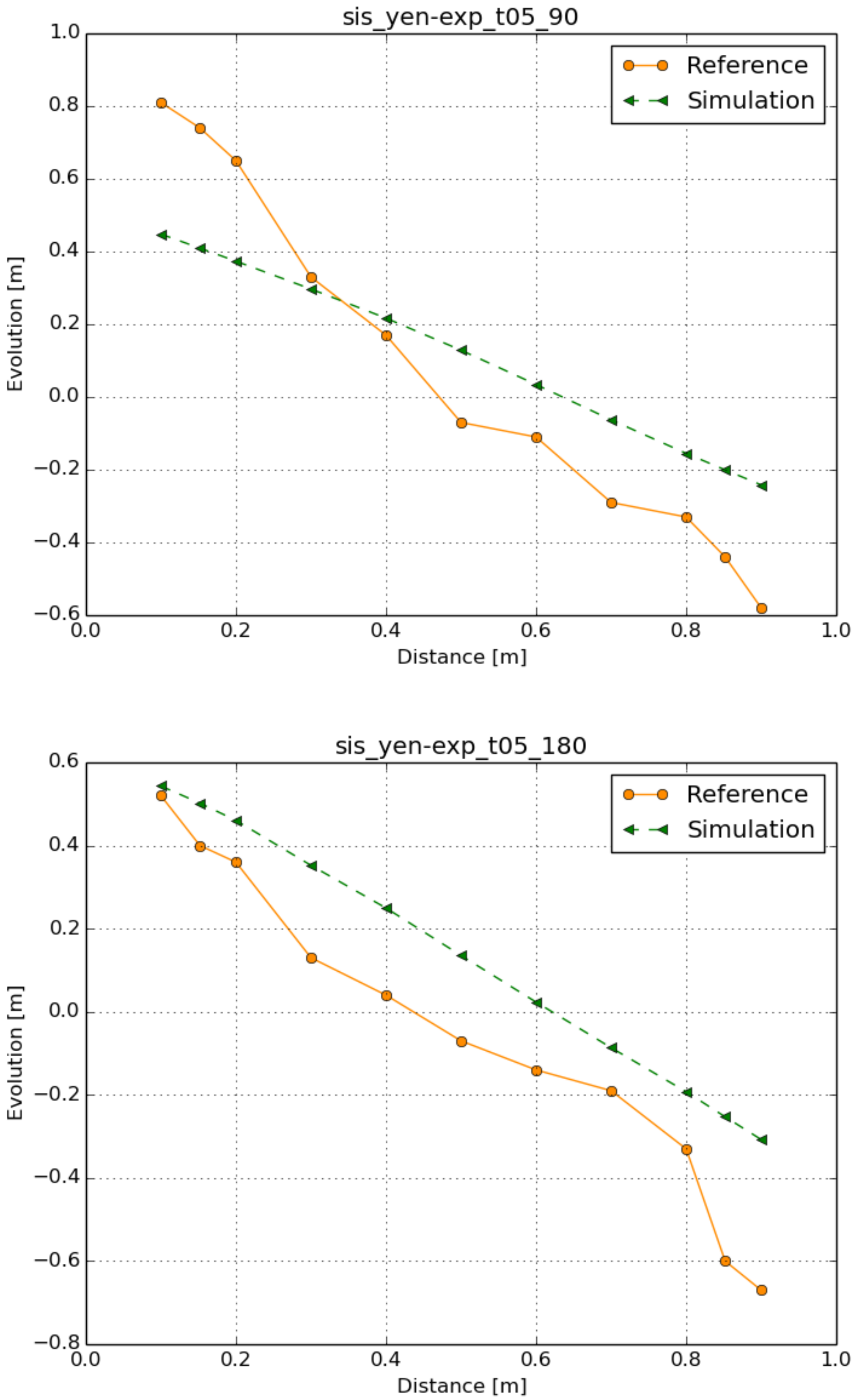


Figure 1.6: Comparison of simulated and measurement bottom elevation at cross section 90° and 180°.

2. bump2d

2.1 Purpose

The evolution of a conical dune is commonly used as a test case for two dimensional morphodynamic models. The flow is almost uniform and sub-critical. This test case was proposed by Hudson (2005). De Vriend (1987) obtained an approximate analytical solution for the spread angle.

2.2 Description of the problem

The sediment dune propagates downstream during the simulation. The formerly conical dune evolves towards a star-shaped pattern expanding in time with a fixed spread angle.

2.2.1 Reference

De Vriend, H.J. *2DH mathematical modelling of morphological evolutions in shallow water*. Coastal Engineering, 11(1):1 – 27, 1987.
Grass, A.J. *Sediment transport by waves and currents*. Technical Report FL29, SERC London Centre for Marine Technology, 1981.
Hudson, J. and Sweby, P.K. *A high-resolution scheme for the equations governing 2D bed-load sediment transport*. International Journal for Numerical Methods in Fluids, 47:1085–1091, 2005.
Siviglia, A., Stecca, G.,Vanzo, Zolezzi, G., Toro, E.F., Tubino, M. *Numerical modelling of two-dimensional morphodynamics with applications to river bars and bifurcations*. Advances in Water Resources, February 2013. DOI:10.1016/j.advwatres.2012.11.010

2.2.2 Physical parameters

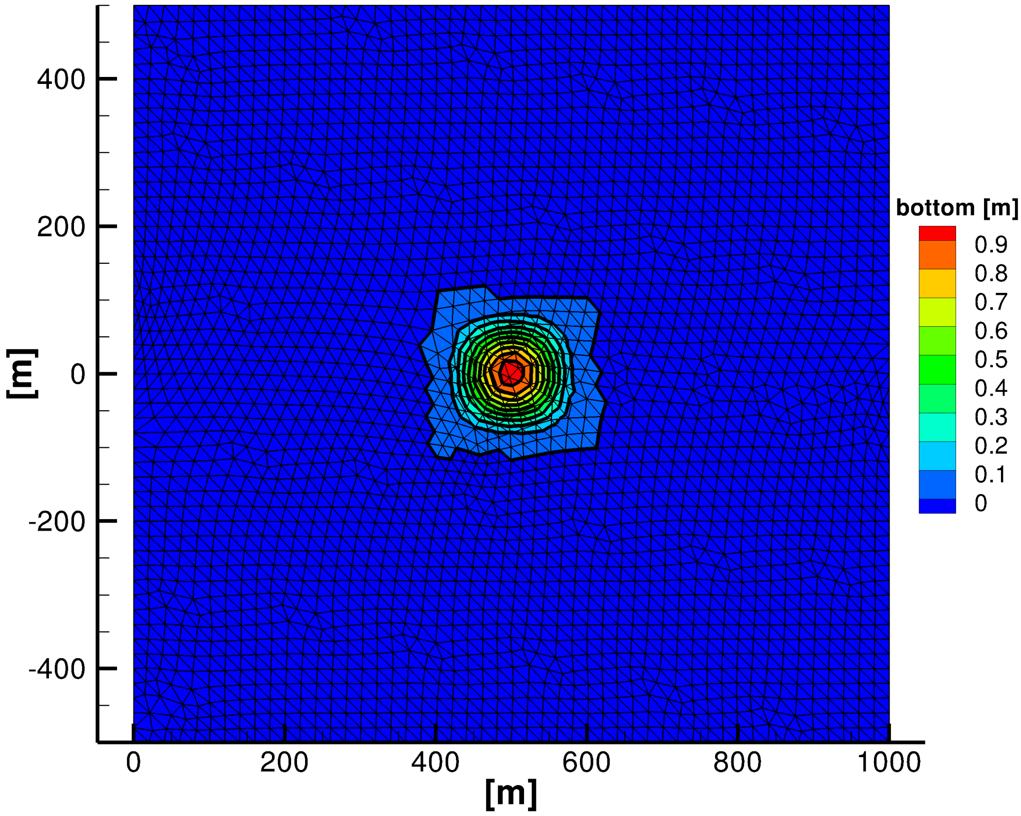
The bed load transport QS is calculated with the velocities u and v using the total load formula of Grass (1981). There are two parameters in the formula: the constant $A_G[s^2/m]$ and the exponent m_g. The first is usually obtained by experimental data and takes into account the grain diameter and the cinematic viscosity. It is set to 0.00167 m^2/s for the simulation. The second parameter is as here usually set to $m_g = 3$. The following formula is implemented in the subroutine qsform.f:

$$\begin{aligned} QS = &\quad A_G = u|u|^{(m_g-1)} \\ QS = &\quad A_G(u^2+v^2)\sqrt{u^2+v^2} \end{aligned} \tag{2.1}$$

No bottom friction, no diffusion, no porosity and no slope effect is included in the simulation.

2.2.3 Geometry and Mesh

The problem is solved in the square computational domain [0; 1000] × [-500; 500] m using an unstructured triangle grid with 2601 nodes and 5000 elements (see figure 4.1.



bottom [m]

Figure 2.1: Simulation grid and initial bottom

2.2.4 Initial and Boundary Conditions

The initial condition for the bed elevation z is given by flat horizontal bed with a sediment bump:

$$z(x,y) = \begin{cases} \sin^2\left(\frac{\pi(x-400)}{200}\right)\sin^2\left(\frac{\pi(y+100)}{200}\right) & \text{if } x \in [400,600] \\ & \text{and y} \in [-100,100] \\ 0 & \text{otherwise} \end{cases} \tag{2.2}$$

The initial condition for the hydrodynamic is the steady state computed with the following boundary conditions. At the upstream boundary $x = 0$ a constant discharge of $Q = 1000m^3/s$ is prescribed while a free outflow condition with a fixed water level of 10 m is set at the downstream boundary x = 1000 m. At the side boundaries y = -500 and y = 500 m a slip condition is imposed. For the morphodynamic simulation only bed load transport is taken into account. There is no sediment input to the boundaries.

2.2.5 Numerical parameters

The time step is set to 1 s and the simulation is coupled every time step with sisyphe.

2.3 Results

The analytical solution for the spread angle α^S

$$\alpha^s = \arctan(\frac{3\sqrt{3}(m_G - 1)}{9m_G - 1}) \tag{2.3}$$

is valid under the hypothesis of weak interaction between sediment layer and fluid, which is ensured setting $A_G = 0.00167 < 0.01$ in the Grass formula. With this parameters the spread angle of the analytical solution is $\alpha^s = 21.787$.
The spread anlge from the simulation is calculated by using the 0.1 m bottom isolines after 80 h and 100h simulation time (see figure 2.2). The computed spread angle $\alpha_s = 25.76°$ is a little too high. With a better resolution spread angle will fit better to the analytical solution (e.g. grid with 21738 nodes and about half of the node distances $\alpha_s = 23.15^{circ}$).

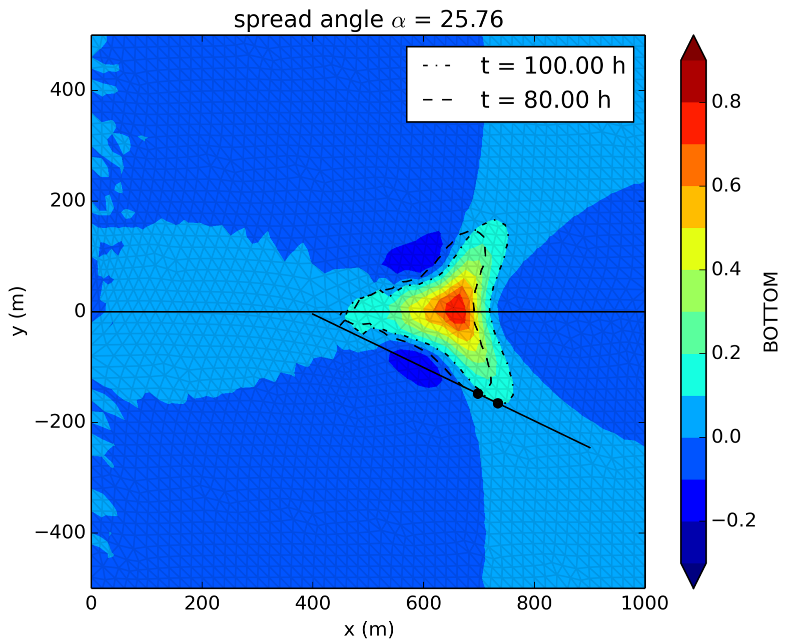


Figure 2.2: Simulation results after 100h

3. littoral

3.1 Purpose

This case test the coupling model between Telemac2dTomawac and sisyphe.

3.2 Description of the problem

This is the classical test case of a rectilinear beach with sloping bed The model allows to calculate the littoral transport. ! This test case illustrates the effect of waves which is :

- to generate the current induced littoral current parallel to the beach
- to increase the sand transport rate using the Bijker sand transport formula.

3.2.1 Geometry and Mesh

The beach is 1000 m long, 200 m wide The beach slope (Y=200m) is 5% and defined in corfon.f
The water depth along the open boundary (Y=0) is h=10m We use a trianglular regular grid
The mesh is as shown on Figure 3.1

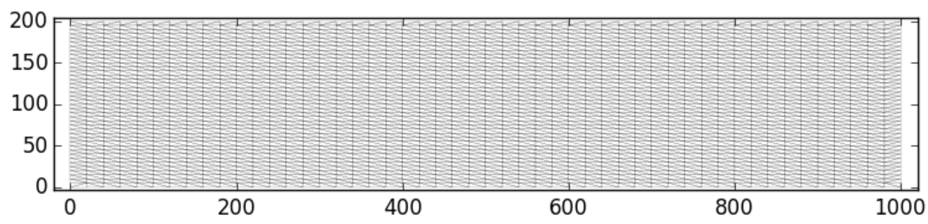


Figure 3.1: mesh of the case littoral

3.2.2 Initial and Boundary Conditions

⇒ Offshore (Y=0): Offshore wave imposed/no littoral current/no set up
Tomawac: The wave height is imposed on the offshore boundary (5 4 4) (Hs=1m), for a wave period (Tp=8s).
Telemac2D: The current and free surface are imposed to 0 along the offshore boundary (5 5 5).
⇒ Left and right hand side of the domain (X=0, X=1000m):recirculation condition

Tomawac: The wave height is imposed on the offshore boudary (5 4 4), based on the model solution, calculated at the center line of the domain. This is done in limwac.f
Telemac : the model solution for the current (4 5 5) on the center line of the model domain are copied on both right and hand side. This is done in bord.f.

3.3 Results

Results (littoral current and transport rates) as well as wave set up/set down are in good agreement with expectations from theoretical classical results (Longuet Higgins).The model is able to reproduce the wave induced current, as well as the effect of set down/set up as the waves dissipate in the breaking zone. The sediment transport rate is located in the near shore breaking zone, where the longshore current is generated. Similar results for the littoral transport could be obtained by using an integrated formula (e.g. CERC formula).
The results are presented Figures 3.2 (Velocity U) 3.3(Wave heigth Hm0) and 3.4 (Bed Shear stress)

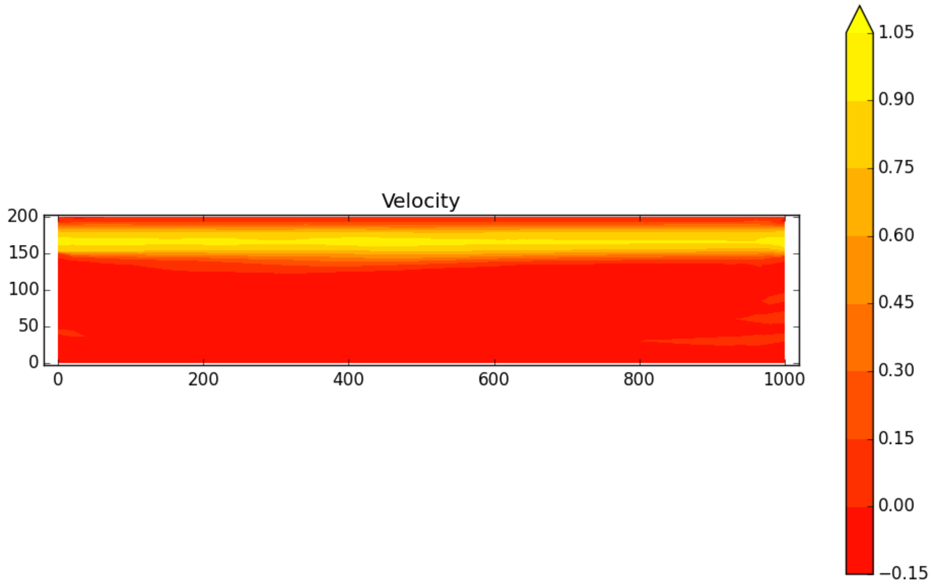


Figure 3.2: Velocity along U of the case littoral

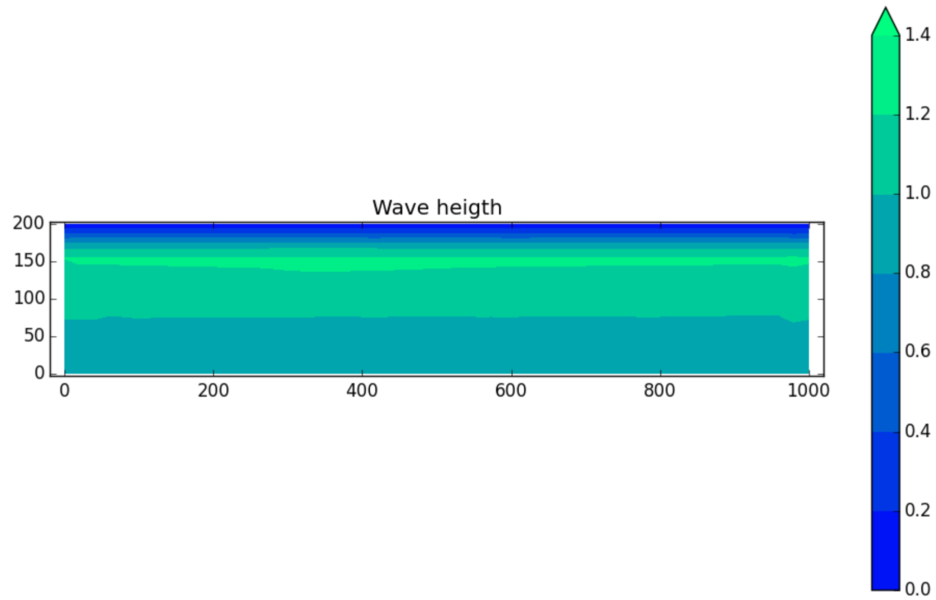


Figure 3.3: Wave heigth Hm0 of the case littoral

Figure 3.4: Bed shear Stress of the case littoral

4. Nestor example 1

4.1 Purpose

First test example for Nestor

4.2 Description of the problem

This example is for testing dredging and dumping of the previously dredged material.
100 m^3 material will be dredged in the polygon named `201_Abschnitt_1_2_1000m**2` defined in `_DigPolys.dat` over a time period of 100 s. The name must start with an integert number between 100 and 999. From position 4 is free text to help the user. Only the number is the identifyer of the polygon. This Dredging area is located between 100,5 and 300,5 m of the flume with an area of 200 x 5 = 1000 m^2. In this part only coarse material (dm=2mm) exists. The dredging starts 50 s after the simulation start (2000.01.01-00:00:50) and ended 100 s later (2000.01.01-00:02:30). This is defined in `_DigActions.dat`. The dredged material will create a final erosion of 0,1 m in the dredging area. The dredging rate (calculeted by Nestor) is the dredging volume devided by the dredging time and the dredging area $\frac{100m^3/s}{100s*1000m^2} = 0,001$ m/s
At the time when the dredging starts the dredged material will be dumped in the polygon named `field 202_Abschnitt_6_7_1000m**2`, which is located between 700,5 and 900,5 m of the flume with an area of 200 x 5 = 1000 m^2. The final sedimentation will be 0,1 m in the dumping field. In this part originally only fine material is located (dm=0,1mm). Due to a small dumping rate (preset to 0,0005 m/s), which is two times smaller than the dredging rate it takes 200 s to dump the material.
The sediment distribution will not be changed in the dredging area. In the dumping area the mean grain size will be increased during the dumping of coarse material. The final mean grain size is only 1,3 mm due to mixing processes of the Hirano layer model. Without mixing processes, the active layer would completely consists of the coarse material and would have a mean grain size of 2 mm as the sedimentation depth is as big as the active layer.

4.2.1 Physical parameters

The simulation has set up with three grain classes (d=0,1 / 0,2 / 2 mm), but only the fine and the coarse material are used. The bottom is discretised with three layers. A constant active layer (10 cm), an underlying stratum (10 cm) and a last layer up to the rigid bed (9.8m).

The Meyer-Peter Mueller transport formula is used but the MPM parameter is set to zero which avoids sediment transport. All bottom changes come from the dredging and dumping processes.

4.2.2 Geometry and Mesh

A 1000 m long flume with three widening parts has been chosen as test geometry (see figure 4.1). The width of the flume is 10 m and increases up to 30 m in the widening parts. Two of the widening parts are 100 m long and the third is 200 m long. The initial bottom has a continuous slope of 0,09 The node area for every node is 1 m^2 in order to calculate dredging and dumping volumes very easily.

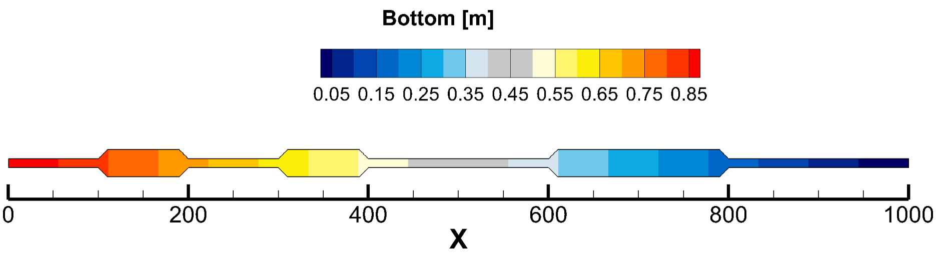


Figure 4.1: Geometry of the test flume with three widening parts

The mesh consists of 18411 nodes and 34800 elements.

4.2.3 Initial and Boundary Conditions

Steady state boundary conditions:

- Discharge at the inlet = 20 m^3/s
- Water depth at outlet = 1 m
- Sedimentological equilibrium at the inlet (zF is constant, QS will be calculated)

Fully developed flow from a previous simulation is used as initial conditions.
With a time step of 1 s a simulation period of 250 s are computed.

4.2.4 Numerical parameters

4.3 Results

Figure 4.2 shows the evolution after 50 s (start of dredging and dumping), after 100 s (end of dredging) and after 250 s (final simulation state and end of dumping process).

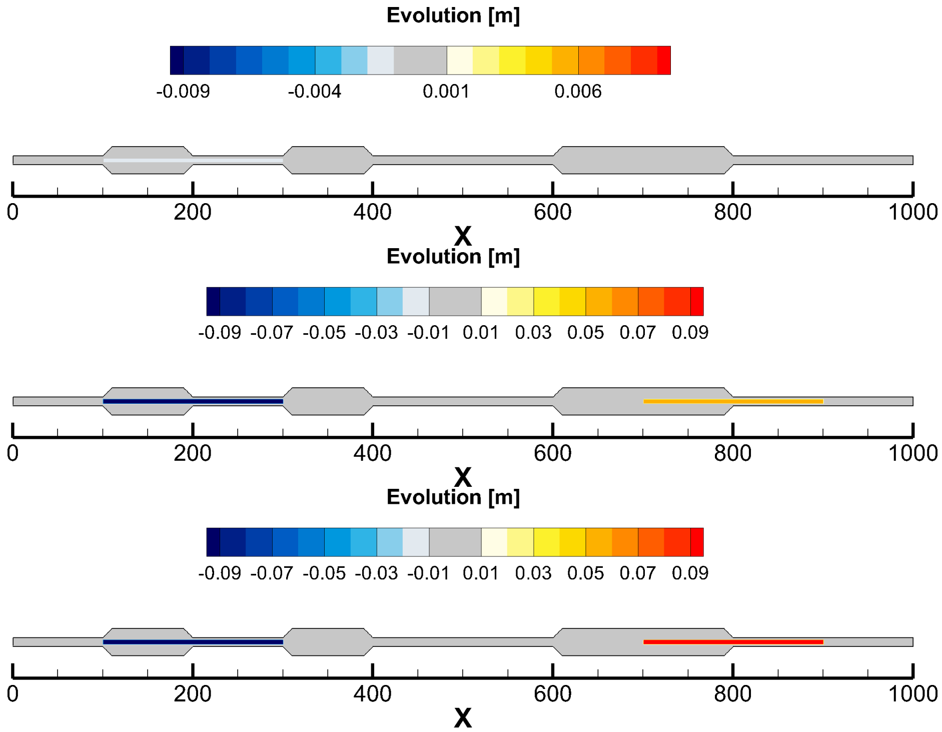


Figure 4.2: Simulated evolution after 50, 100 and 250 s.

4.3.1 Nestor example 3

Purpose

Third test example for Nestor performing the action:
`ActionType = Dig_by_criterion`

Description of the problem

This example performs the maintenance of a fairway. The dredged material is transferred (dumped) to the dumping area. At regular time intervals Nestor tests within the fairway area for each node if the depth criterion is violated. If the criterion is violated the node will be dredged to a specified depth. The depth is defined as elevation of the reference level minus elevation of the node (see Figure 4.3).

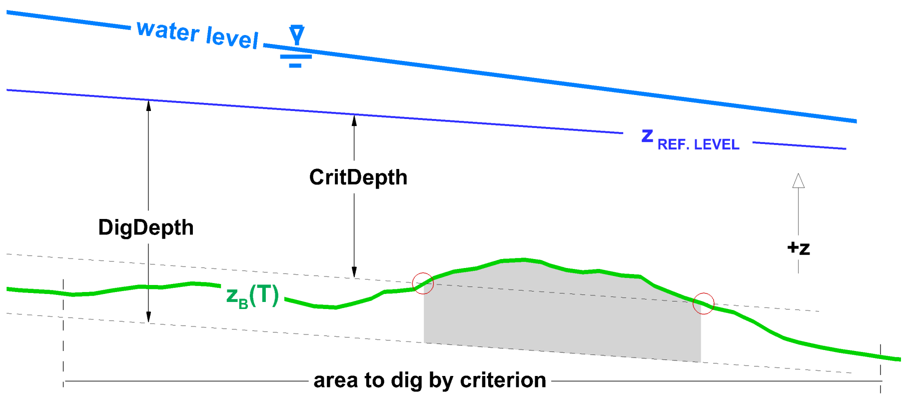


Figure 4.3: Schematic diagram of dredging by criterion

The fairway and the dumping area (see Figure 4.4) are defined by polygons which are defined in file `_DigPolys.dat`.
Convention for naming polygons: The polygon name must start with an integer between 100 and 999. Any text after that is treated as a comment to help the user. Internally only the number is used to identify the polygon.

Reference

Physical parameters

The simulation is set up with 10 grain classes. The bottom is discretized with three layer: A constant active layer (0.1 m), an underlying stratum (0.1 m) and a third layer up to the rigid bed (9.8m).
The Meyer-Peter Mueller (MPM) transport formula is used but the MPM parameter is set to 16 (default 8) to create more dynamics in the bottom elevation. Thus all bottom changes are a result of sediment transport processes and fairway maintenance (digging and dumping).

Geometry and Mesh

A 3.3 km long flume with two 180 degree bends has been chosen as test geometry. The width of the flume is 200 m (see Figure 4.4).
The mesh consists of 1614 nodes and 2948 elements.

Initial and Boundary Conditions

Steady state boundary conditions:

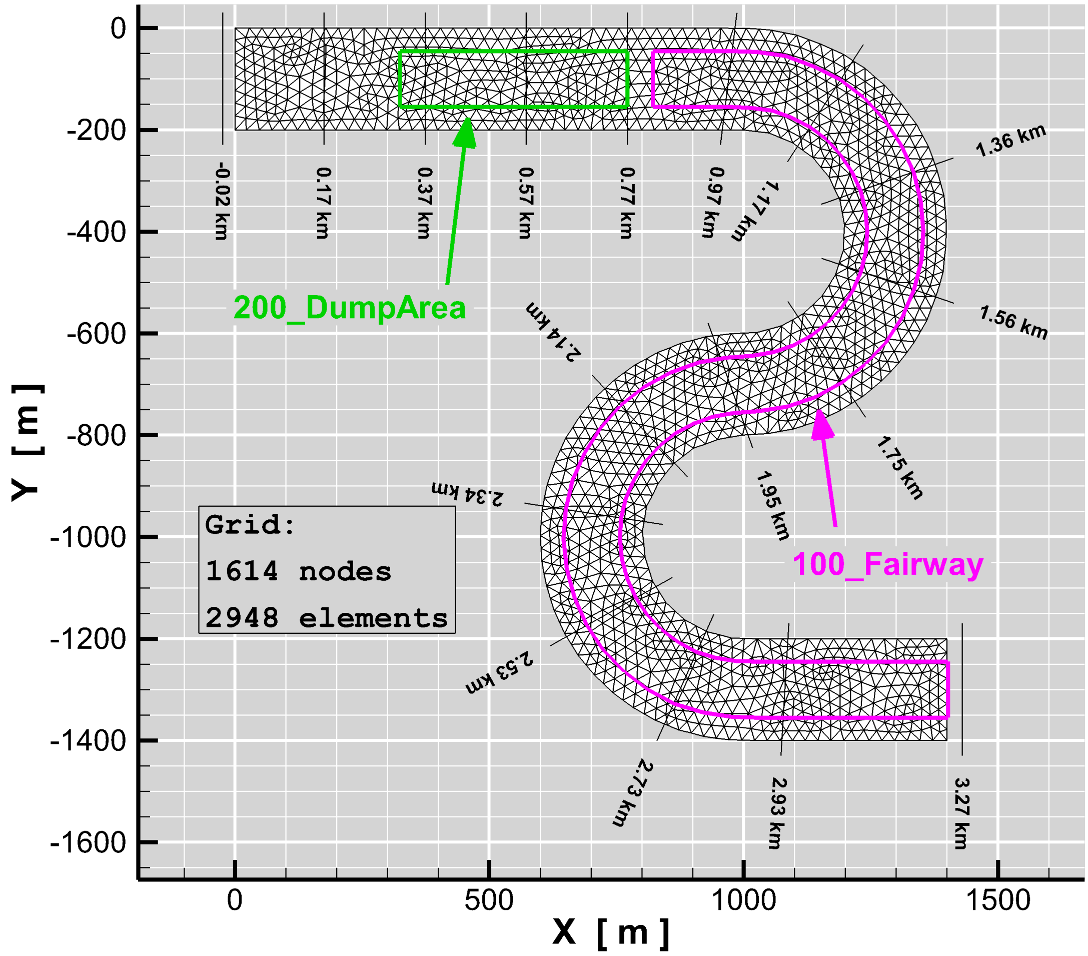


Figure 4.4: Geometry of the test flume with two bends and the two polygons

- Discharge at the inlet = 1000 m^3/s
- Water level at the outlet = 2.2 m
- Sedimentological equilibrium at the inlet (zF is constant, QS is calculated)

Fully developed flow and bottom from a previous simulation are used as initial conditions. The total simulation period is 777600.0000 s (9 days) with a time step of 6 s.

Numerical parameters

Results

Figure 4.5 shows the bottom evolution after 0h and 1h. The evolution after 0h is from a previous computation file. The evolution after 1h is driven by the first maintenance of the fairway and sediment transport. The digging and dumping were executed between 0h and 1h. The fairway was dredged to the `DigDepth` and the dredged material was dumped to the dumping area defined by the polygon `200_DumpArea`.

Figure 4.6 and 4.7 shows the bottom evolution due to sediment transport processes between 1h and 192h. No digging and dumping actions were defined in this time period.

Figure 4.8 shows the bottom evolution between 192h and 216h due to sediment transport processes and the second maintenance of the fairway. The digging and dumping were executed between 192h an 193h. Again the fairway was dredged to the `DigDepth` and the dredged material was dumped to the dumping area, which leaded again to a higher bottom evolution.

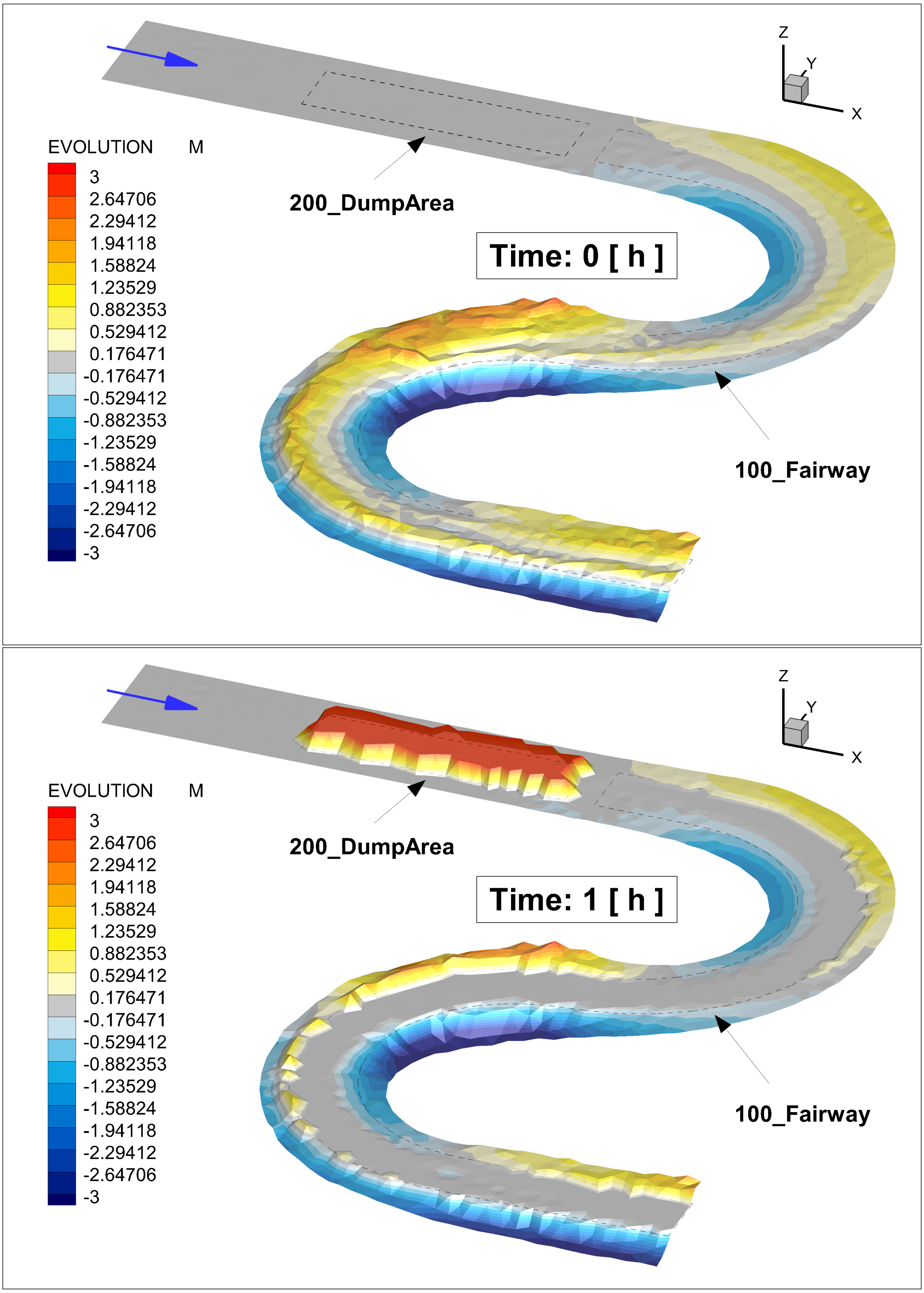


Figure 4.5: Simulated evolution over the time.

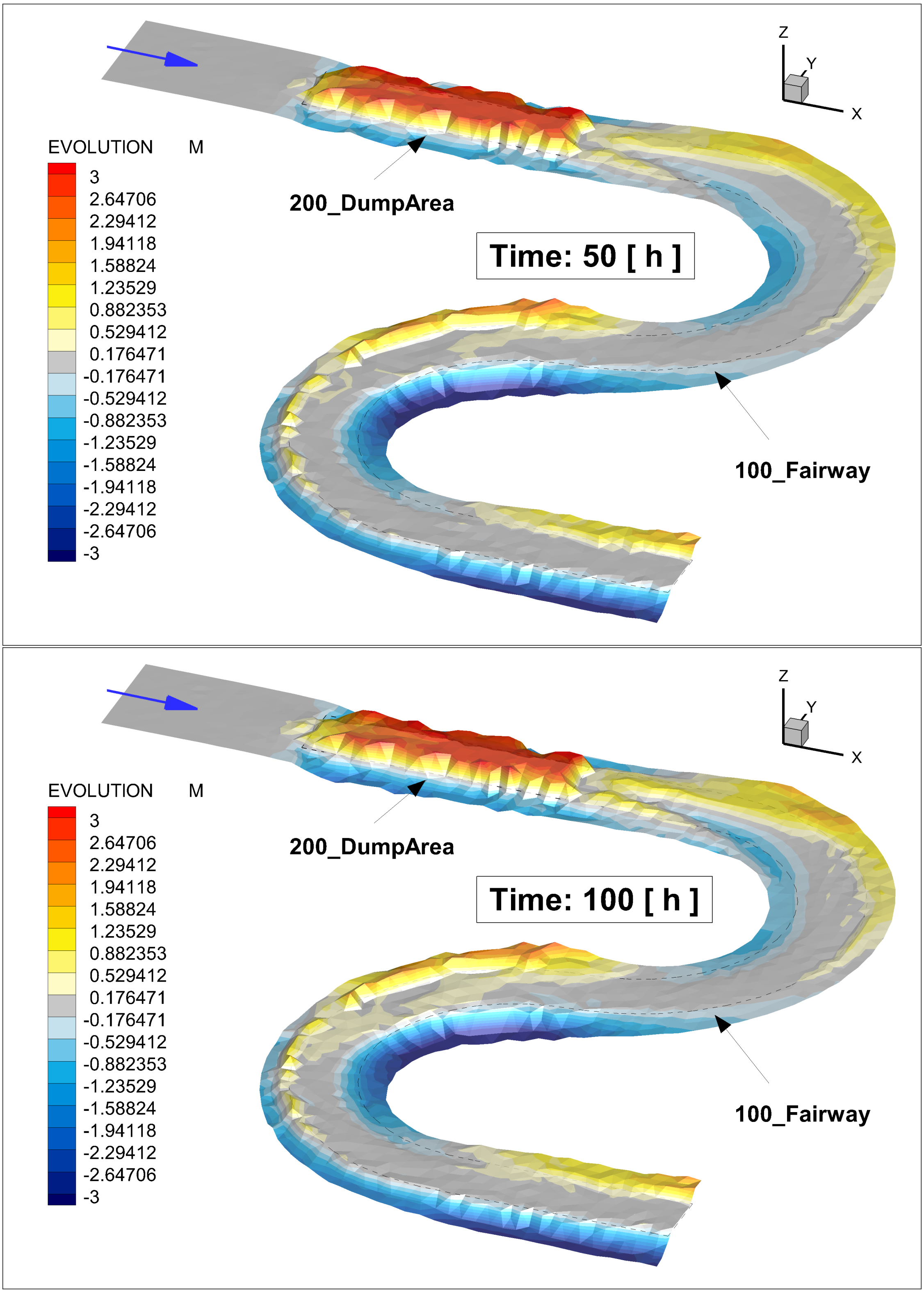


Figure 4.6: Simulated evolution over the time.

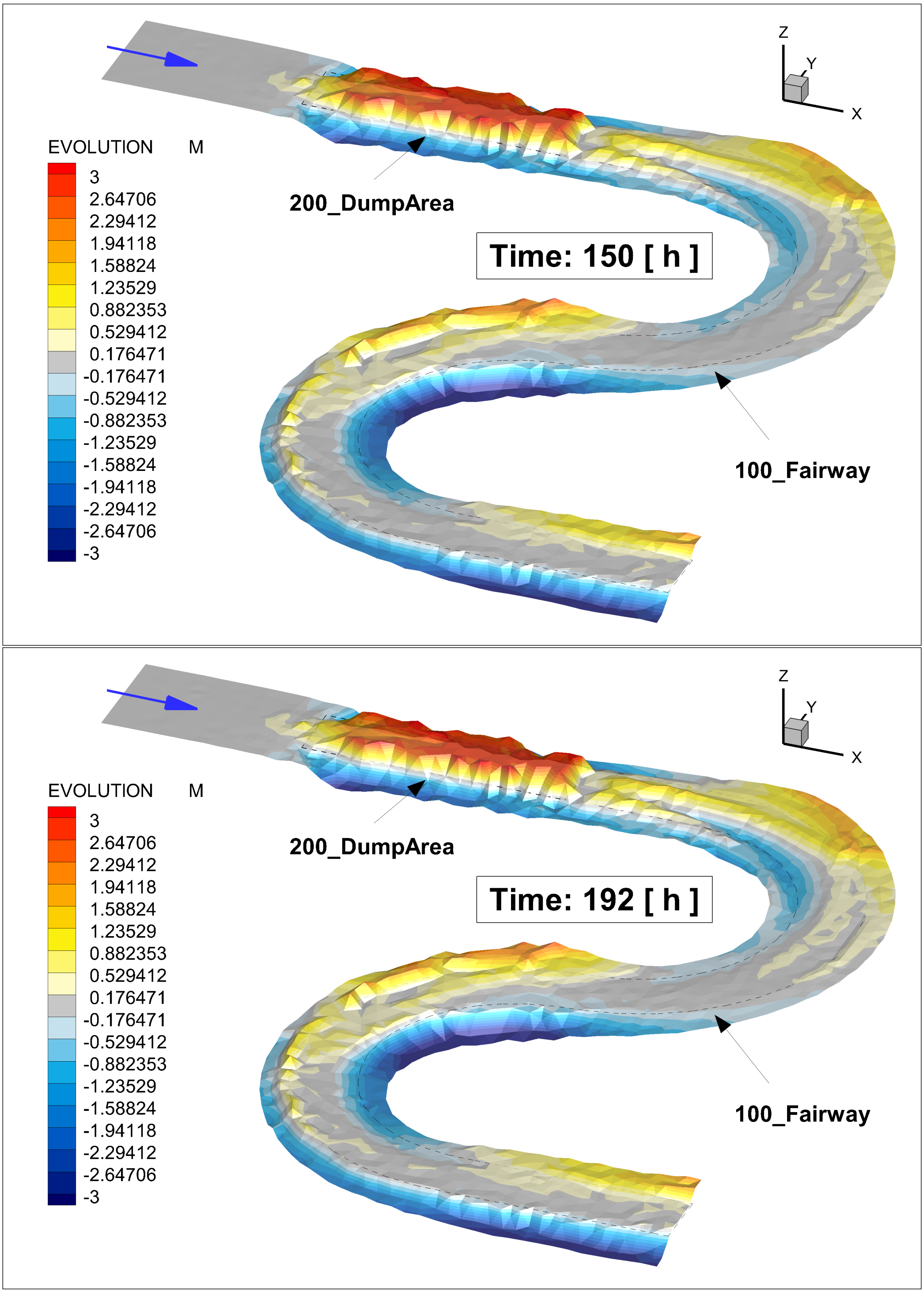


Figure 4.7: Simulated evolution over the time.

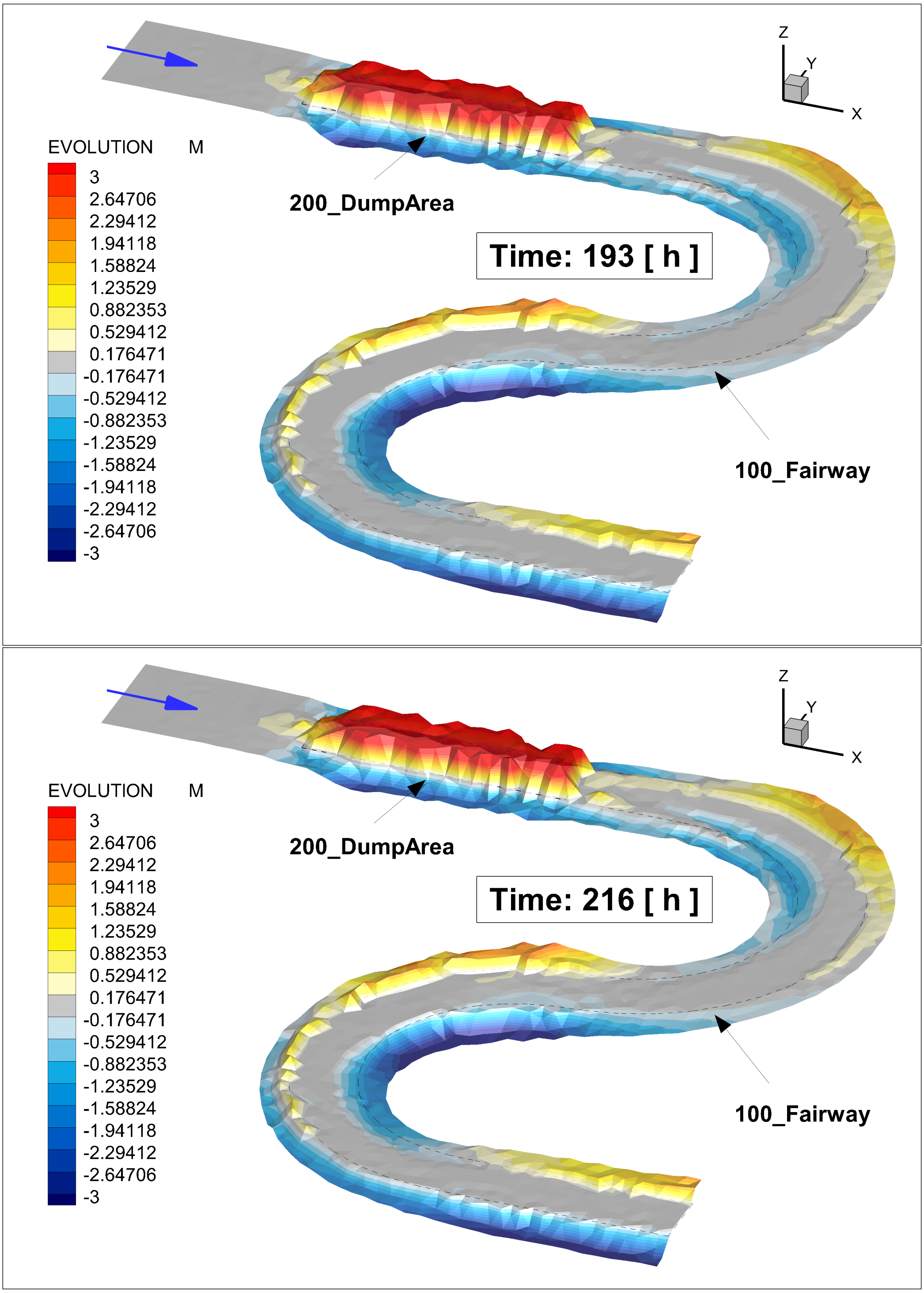


Figure 4.8: Simulated evolution over the time.

[1]

[1] HERVOUET J.-M. *Hydrodynamics of Free Surface Flows. Modelling with the finite element method*. Wiley, 2007.