E-proceedings of the 36th IAHR World Congress
28 June – 3 July, 2015, The Hague, the Netherlands

Numerical modelling of flood control areas with controlled reduced tide

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ABSTRACT

The present paper focuses on the numerical modelling in TELEMAC-3D of flood control areas with controlled reduced tide structures along the Scheldt estuary and coastal zone for the storm event of December 6th, 2013. A new culvert functionality was implemented in the code to better represent the hydrodynamics of the exchange of water between the Scheldt estuary and these flood control areas with controlled reduced tide. Existing source and sink terms included in the code were paired and used as a culvert. The theoretical background to represent the different kind of flows through the culvert was based on the work of Bodhaine (1968). Additionally different head loss coefficients were introduced according to different geometric features of the culverts. The implementation of these new structures inside the 3D numerical model was validated using measured water levels in the estuary and inside the flooding areas , and using discharges (in and out) through the culverts measured only for one full tidal cycle. For the storm surge only measured water levels were available and these were compared with modelled ones.

Keywords: *Scheldt estuary, Storm surge, TELEMAC-3D, Culvert, Flood control area*

1. INTRODUCTION

Estuaries are transitions between riverine systems and the coastal zone. They provide an important link between overseas trading and local economy since they can shelter important harbors and therefore offer work opportunities. Many people live in these lowlands and climate change is increasing the intensity and frequency of storm surges along coast worldwide, posing an immer growing flood risk for these areas (Webster et al., 2005; Temmerman et al., 2013). For instance, storm surges can propagate far inland and induce flood risk for all those living close. The occurrence of storm surges poses huge challenges for the coastal zone management. In order to prevent a number of hazards associated with these events, it is essential to study and analyze the risk of floods in areas along the coast and estuary.

The Scheldt river originates in the north of France (St. Quentin) at 110 m above sea level and flows after 355 km into the North Sea near Vlissingen (The Netherlands). The estuary is well mixed, presenting small or negligible salinity gradients (Baeyens et al., 1998). A storm surge in February 1953 caused many causalities in England, Belgium and in the Netherlands. This was the start for the so called Delta works in the Netherlands, set up with the purpose of protecting the coastline from storm surges. In Belgium it was only after the storm in 1976, causing large floods, that the development of a new flood protection plan, i.e. the Sigma plan, started. The protection against flooding is combined with restoration of a lot of nature areas (Meire et al., 2014). Over the last centuries high water levels have been increasing in the estuary and therefore dike levels have to follow. Because of the large economic value for the port of Antwerp it was decided not to close the estuary with a storm surge barrier.

Instead of raising the dikes or closing the estuary, Flanders chose the option of building flood control areas (FCA's). The flood control areas are areas along the Scheldt estuary surrounded with dykes, like the rest of the estuary and separated from the river by an overflow dyke with a specific level. When a storm surge enters the estuary, the FCA becomes active only when water levels exceed critical thresholds extracting only critical water volumes at the top of the storm tide. These areas can store large amounts of water from the tidal (storm) wave and therefore protect the hinterland from flooding. When the water level drops after a storm tide, outlet culverts evacuate the water out of the FCA back into the river (Figure 1). These outlet culverts have valves to avoid water flowing from the river into the FCA. Moreover, culverts allow a controlled reduced tide (CRT) installed in the FCA creating a tidal nature and therefore improving the ecological value in some of these areas. The elevation and specific geometry of the culvert determines inflow-outflow patterns within the area creating a tidal ecosystem (Maris et al., 2007)

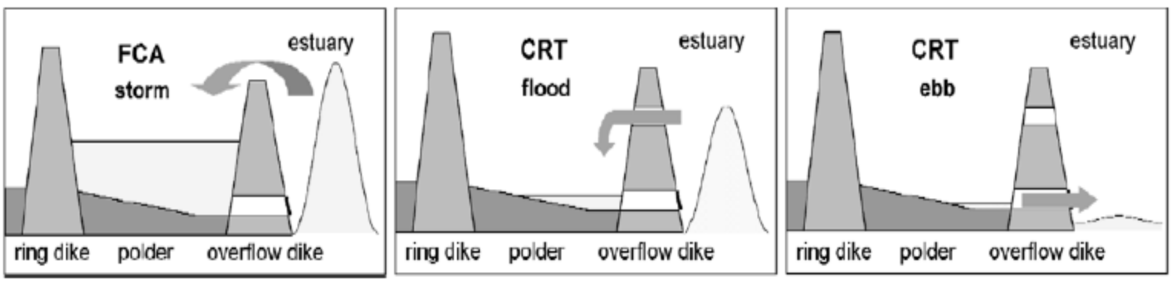

Figure 1 Water overflows the dyke (on the left) and through the culvert (on the middle) from the river into the FCA and evacuates from the FCA into the river through the outlet sluice (on the right). (Source: De Mulder et al. (2013))

When the storm of 6th of December 2013 entered the Scheldt estuary, 13 of these areas were operational and some of them were completely filled (Figure 2). It is expected that by 2030 more than 40 areas will become active.

Figure 2 FCA/CRT 'Bergenmeersen' filled during storm surge of 6th of December 2013 (© Hydrologic Information Centre, Flanders Hydraulics)

The incorporation of these structures in numerical models is essential to better predict and describe the flow hydrodynamics in these areas. The main aim of this work is to analyze how these structures can be taken into account and implemented in the three-dimensional hydrodynamic model TELEMAC-3D.

In the following sections the theoretical background of the different types of flow through a culvert is described. Subsequently, the implementation of this functionality in TELEMAC-3D is discussed. Finally, results of a test case are presented evaluating the numerical model results by comparing them against measurements.

2. NUMERICAL IMPLEMENTATION OF CULVERTS

2.1 Hydrodynamic model – governing equations

The numerical platform TELEMAC-MASCARET was the chosen tool to analyze and study the Scheldt estuarine hydrodynamics. TELEMAC-MASCARET was originally developed by EDF R&D and it includes the three-dimensional circulation model TELEMAC-3D. The model is based on the finite elements method. Assuming the hydrostatic hypothesis, the code solves the three-dimensional RANS equations:

$$\frac{\partial U}{\partial x}+\frac{\partial V}{\partial y}+\frac{\partial W}{\partial z}=0 \qquad [1]$$

$$\frac{\partial U}{\partial t}+U\frac{\partial U}{\partial x}+V\frac{\partial U}{\partial y}+W\frac{\partial U}{\partial z}=-g\frac{\partial \eta}{\partial x}+v\Delta(U)+F_x \qquad [2]$$

$$\frac{\partial V}{\partial t}+U\frac{\partial V}{\partial x}+V\frac{\partial V}{\partial y}+W\frac{\partial V}{\partial z}=-g\frac{\partial \eta}{\partial y}+v\Delta(V)+F_y \qquad [3]$$

$$\frac{\partial h}{\partial t}+\frac{\partial (uh)}{\partial x}+\frac{\partial (vh)}{\partial y}=0 \qquad [4]$$

(U, V, W) are the three components of the flow velocity, (u,v) the depth integrated flow velocities, υ the diffusion coefficient, g the gravitational constant and (F_x, F_y) the source and sink terms of the momentum equations ([2] and [3]).

Additionally, TELEMAC-3D gives the possibility of taking into account passive or active tracers in the model domain. The following equation, describing the evolution of tracer concentration (T), is solved:

$$\frac{\partial T}{\partial t}+U\frac{\partial T}{\partial x}+V\frac{\partial T}{\partial y}+W\frac{\partial T}{\partial z}=v_t\Delta(T)+Q' \qquad [5]$$

The tracer diffusion coefficient is given by υ_t and Q' represents the source terms for tracers.

In TELEMAC-3D (v6p3) hydraulic structures such as culverts or tubes are not implemented. The capability of the model to impose source/sink terms in the domain was used as a basis for the implementation of the culverts. Therefore, the inflow and outflow act as a couple of source/sink points (in the code, new terms are added to the second hand side of the depth integrated continuity equation [4]). For instance, when the flow is going from the river to the floodplain, a source term is implemented on the side of the floodplain, i.e., a discharge is imposed, and at the same time a sink term is imposed on the side of the river with a symmetric value of that discharge (e.g. Q_{river}= - $Q_{floodplain}$). Using this method an assumption was made: it is considered that the discharge occurs at the same time in the river and floodplain. The tracer concentration in the model domain is associated to the discharges and volumes at the source and sinks terms and is as such taken into account.

2.2 Flow through culverts

A number of studies regarding the description of flows through the culverts refer to the work of Bodhaine (1968). In his work, the flow that occurs through a culvert is classified into six types, being the discharge calculated in a different way for each kind of flow. The equations are deduced from the continuity and energy equations between the approach section and the downstream section of the culvert. The type of flow depends on whether the culvert flows full or not and whether the flow is controlled by the inlet or outlet.

New equations were then incorporated in the code in order to cover a wide range of the flow conditions that exist through a culvert. The following equations, corresponding to each type of flow presented above, were implemented in TELEMAC-3D. They are based on the equations proposed in Bodhaine (1968) and similar to the ones incorporated in DELFT 3D model. The flow type 1 conditions (described also in Bodhaine (1968)) were not incorporated since they only occur when the culvert slope is larger than the critical flow slope, which is never the case for our FCA/CRT application.

Type 2 – Critical depth at outlet:

$$Q = \mu h_c W \sqrt{2g * (S_1 - (z_2 + h_c))} \quad [6]$$

Type 3 – Tranquil flow:

$$Q = \mu (S_2 - z_2) W \sqrt{2g(S_1 - S_2)} \quad [7]$$

Type 4 – Submerged outlet:

$$Q = \mu D W \sqrt{2g(S_1 - S_2)} \quad [8]$$

Type 5 – Rapid flow at inlet:

$$Q = \mu D W \sqrt{2g h_1} \quad [9]$$

Type 6 – Full flow with free outfall:

$$Q = \mu D W \sqrt{2g(S_1 - (z_2 + D))} \quad [10]$$

with: Q the discharge through the culvert, W the culvert width, D the culvert height, μ the total head loss coefficient, S_1 the water level on side 1, S_2 the water level on the other side, h_1 the water level above the culvert base on side 1, h_2 the water level above the culvert base on side 2, h_c the critical water level inside the culvert (this is assumed to be close to 2/3 of h_1), z_1 the base level of the culvert at side 1, and z_2 the base level of the culvert at side 2. Most of these variables are shown in a schematic representation of the culvert in Figure 3.

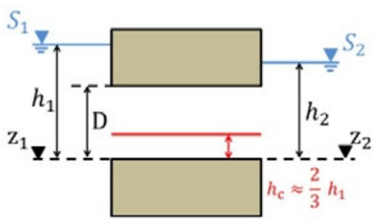

Figure 3 Representation of the different variables used for the culvert equations.

The conditions for each type of flow are given in Table 1. To distinguish between flow type 5 and flow type 6 a constant c, that is dependent on the culvert slope and the ratio culvert W/D, is used. Then if L/D < c, with L the length of the culvert, flow type 5 occurs, otherwise it is flow type 6 (Bodhaine, 1968).

Table 1 Conditions for each type of flow used in TELEMAC-3D

	$\frac{S_1 - z_1}{D}$	$\frac{S_2 - z_2}{D}$	$S_2 - z_2$	L/D
Type 2	<1.5		$< h_c$	
Type 3	<1.5	≤ 1.0	$> h_c$	
Type 4	>1.0	> 1.0		
Type 5	≥ 1.5	≤ 1.0		< c
Type 6	≥ 1.5	≤ 1.0		≥ c

The equations presented above are written to describe flow conditions through a culvert with a single barrel. Nevertheless, additional features are sometimes incorporated in the hydraulic structures, such as weirs in the vicinity of the culvert entrance or exit. Such combined structures have to be taken into account. Then the geometric features of the culvert presented in Figure 3 are modified as shown on Figure 4. The structure becomes more complex. In order to represent these features an equivalent culvert bottom elevation was used, which replaces both the bottom elevations z_1 and z_2 in the formulas described above. The equivalent bottom culvert elevation z is then equal to the mean between z_1 and z_2. The diameter used in the equations will be the one corresponding to the entrance of the culvert, i.e., regarding Figure 4, if the flow goes from left to the right D will be replaced by D_1 and in the opposite direction, the value D_2 will be used.

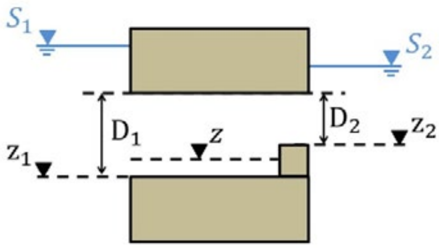

Figure 4 Representation of the different variables used for the combination of a culvert with a weir

The head loss coefficient μ was adapted from the one based on Carlier (1972). New terms were added to account for the extra head loss of additional structures in front and behind the culvert structures, like one-way valves and trash screens (examples in Figure 5 and 6). The head loss due to singularities can be obtained by the general relation (Carlier, 1972; Lencastre, 1961):

$$\Delta H = C\frac{U^2}{2g} \quad \text{or} \quad U = \mu\sqrt{2g\Delta H} \quad \text{with } \mu = \frac{1}{\sqrt{C}} \qquad [11]$$

$C = C_1 + C_2 + C_3 + C_v + C_T + C_p$ represents the sum of the different contributions for the head loss due to singularities.
C_1 represents the head loss due to the contraction of the flow at the entrance of the hydraulic structure which results in a deceleration of the flow immediately after the culvert entrance. Usually C_1 is equal to 0.5 (Larock et al., 2000).

Already in the past, Bodhaine (1968) noticed that the discharge coefficient for type 5 flow had to be lowered comparatively to the other flow types. It seems that the calculated discharge tends to be overestimated when the default equation is applied. In order to take into account that effect, a correction coefficient ($\alpha_1{}^5$) is applied to C_1 when type 5 flow occurs, such that:

$$\Delta H_1 = \alpha_1{}^5 C_1 \frac{U^2}{2g} \qquad [12]$$

With: $4 \leq \alpha_1{}^5 \leq 10$ according to Bodhaine (1968).

C_2 represents the head loss coefficient due to the friction in the structure and is expressed by (Lencastre, 1961):

$$\Delta H_2 = C_2\frac{U^2}{2g} = \frac{2gLn^2}{R^{4/3}}\frac{U^2}{2g} \qquad [13]$$

where *n* the Manning Strickler coefficient for the structures material and *R* the wet cross-shore section in the structure, calculated
in the code for each type of flow. An assumption is made when calculating the hydraulic radius since the code does not make any kind of backwater analysis to get the precise water depths in the culvert. C_3 is the head loss coefficient due to expansion of the flow at the exit of the culvert. According to (Lencastre, 1961):

$$\Delta H_3 = \left(1 - \frac{A_s}{A_{s2}}\right)^2 \frac{U^2}{2g} = C_3 \frac{U^2}{2g}$$ [14]

where A_s and A_{s2} are the sections in and just outside the downstream part of the structure. C_v is the head loss coefficient due to the presence of a valve and depends on the type of valve and the degree of opening. When type 5 of flow occurs a correction coefficient (C_{v5}) is also applied to C_v. C_t is the head loss coefficient due to the incorporation of trash screens. The value for C_t can vary between 0, equivalent of not having any trash screens, to 1.4, for which the net flow area is almost equal to the gross rack area.

Sometimes at the entrance of the culverts the flow is divided into two sections caused by two entrance boxes instead of one but then the flow converges into a single culvert barrel. Following Carlier (1972) the head loss coefficient through parallel pillars is given by:

$$C_p = \beta \left(\frac{Lp}{b}\right)^{4/3} sin\theta$$ [15]

Lp is the thickness of the pillars, b the free thickness between two consecutive pillars and β a coefficient dependent on cross-shore section of the pillar.

3. APPLICATION

3.1 Bergenmeersen FCA – CRT description

In order to test the new culvert functionality implemented in TELEMAC-3D, the new version was applied to a field test case, located in the Bergenmeersen area, where there is a flood control area with a controlled reduced tide function. Two simulation periods were chosen in order to analyze the capability of the code to model exchange of water between these areas and the estuary. The first simulation period makes part of a recent measurement campaign carried out from the 10th to the 12th September 2014. Mean water levels inside the flood control area and in the Scheldt river were measured. Data from the outlet and inlet discharges through the culverts were measured during a full tidal cycle on September 11th. In order to analyze the behavior of the model for a storm surge, a second time period, characterized by the storm of December 6th, 2013, was modeled. For the storm period only water levels in both sides of the flood control area and the Scheldt river were available. There were no discharge measurements for that time period.

The area of Bergenmeersen is characterized by a ring dyke with a crest level of 8 m TAW (Belgian reference level, i.e. equal to mean low water sea level) that surrounds the FCA and by an overflow dyke with the crest level of 6.2 m TAW that separates the river from the FCA.

The configuration of the inlet and outlet sluices is quite complex. Three outlet sluices were built with one-way valve installed at the river side. Above these outlet sluices, six smaller inlet sluices were included and at their entrance wooden beams function as a weir at different heights (Figures 5 and 6). The wooden beams control the water level at which water starts flowing in, allowing some space for calibrating the water levels inside the domain and coping with possible future changes in water levels in the estuary. The inlet culverts could be closed by a valve descending from the ceiling. At each inlet and outlet sluice the flow is separated into two parts at the river side and then converges until the FCA side. Additionally, in order to avoid garbage coming into the structure, inlet and outlet structures were provided with trash screens (Figure 6). Table 2 presents an overview of the inlet and outlet sluices geometric characteristics and configuration.

Figure 5 Inlet and outlet sluice configuration in the river side (construction phase) (source: Patrimoniumdatabank W&Z)

Figure 6 View of the inlet sluices from the river side (on the left) and inlet and outlet sluices from the FCA side (on the right)

Table 2 Characteristics of the new inlet and outlet sluices of the new FCA-CRT in Bergenmeersen.

	Inlet (Scheldt side)	**Inlet (FCA side)**	**Outlet (Scheldt side)**	**Outlet (FCA side)**
Number of culverts	6		3	
Culvert width (m)	2.7 2.7		3 3	
Culvert length (m)	9.5		18	
Culvert height (m)	1.6	2.25	1.1	2.25
Level of culvert floor (m TAW)	4.2	2.2	2.7	2.2
Crest level of stop weirs (m TAW)	4.2/ 4.2/ 4.2/ 4.35/ 4.5 / 4.5			

3.2 Model Setup

A mesh resolution of 5 m was set in the river part, while in the floodplain the elements vary from 10 m to 50 m length (Figure 7). Five horizontal planes were imposed in the model. The bathymetric data used in this test case comes from multibeam measurements completed with LIDAR st the shorelines carried out in 2013. The water levels in the Scheldt come from Schoonaarde tidal station. These values were used as boundary condition for the hydrodynamic model.

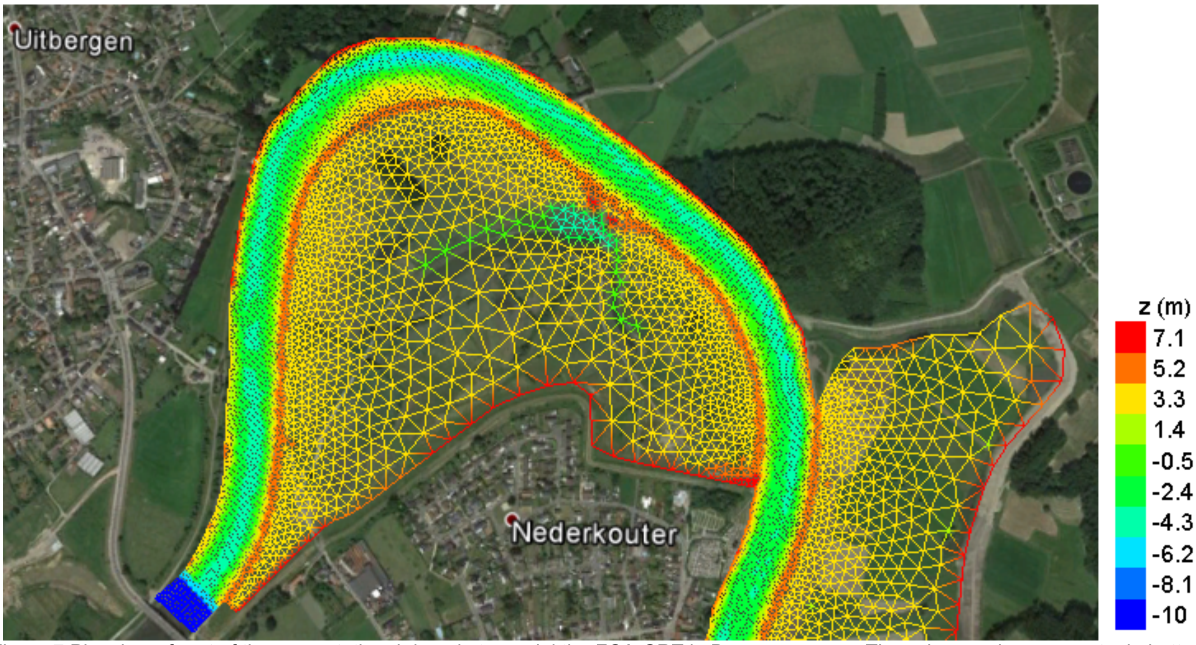

Figure 7 Planview of part of the computational domain to model the FCA-CRT in Bergenmeersen. The colour scale represents de bottom elevation values (m).

The time step was set to 5 s. For the different simulations the parallel version of TELEMAC-3D was used. The bottom friction was taken into account in the model through the Manning Strickler's parameter n. It was assigned a value of n=0.016 (typical value found in the literature for natural channels (French, 1987)). The Smagorinsky turbulence model was chosen to solve the horizontal viscosity while a mixing length model was applied to estimate the vertical turbulence viscosity.

The different geometric features for the inlet and outlet sluices have to be given to the model together with the direction of the flow through the culvert An outlet sluice only allows the flow to go from the floodplain to the river, because it has a one-way valve, and an inlet sluice allows the flow to go in both directions.

Regarding the head loss coefficients at the exit of the inlet and outlet sluices typical values, found in the literature (Lencastre, 1961) were imposed (C_3=1). At the entrance of the inlet sluices the head losses were increased, comparatively with the typical value of C_1= 0.5, in order to take into account the effect of the flow being split into two parts. Following the expression given by Carlier (1972), the head loss due to the presence of pillars is about C_p = 0.4 and therefore C_1 becomes C_1=C_1+C_p. It was considered that the valve at the outlet sluice was ¾ opened when flow was going out, which corresponds to a value of C_v=1 (Bruce et al., 2000). Since there was also flow separation at the exit of the outlet sluice, the value of C_V was increased to take into account this effect. The coefficient to take into account the trash screens was set to C_t =1 both for the inlet and outlet sluices.

Based on values found in the literature (Bodhaine, 1968), a value of *n*=0.015 (typical value for concrete in smooth conditions) was assigned for the Manning Strickler parameter inside the culvert.

3.3 Analysis of results

In the following, comparisons of the numerical model results with experimental data are shown for the simulation period between the 10th till the 12th September. The comparison between the water level on the Scheldt side computed by the model and obtained from experimental data is shown in Figure 8. In general the evolution of the water level in time is fairly well represented by the model. Nevertheless the model underestimates the water level when the water flows from the flood control area to the river. This feature can be confirmed by the underestimation of the outflow that the model presents in Figure 11.

In Figure 9 it can be confirmed that the model gives good result when computing the water level in the flood control area even if it is slightly underestimated when the water flows from the river to the flood control area. Once again this is confirmed by the underestimation of the inflow calculated by the model and is shown in Figure 10.

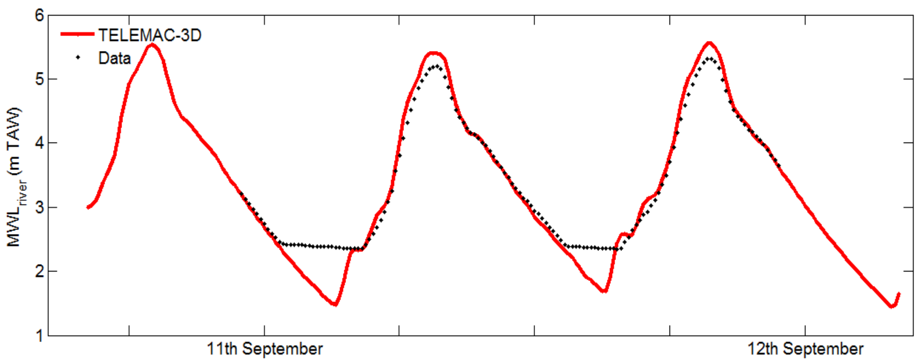

Figure 8 Comparison between water levels measured and computed by the model in the Scheldt river side from the 10th till the 12th September 2014.

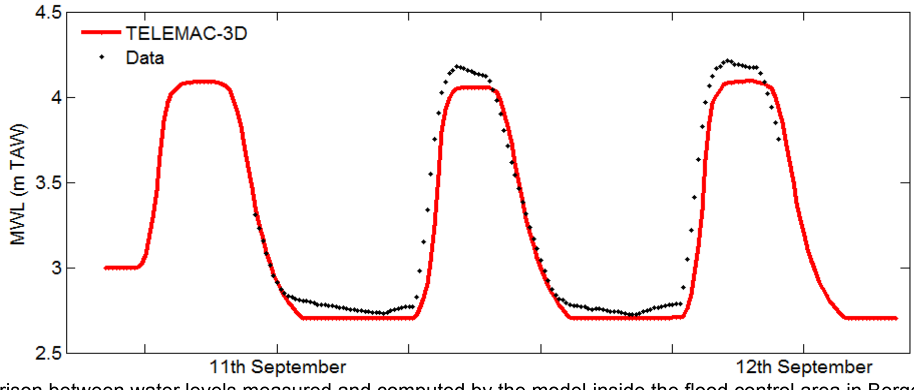

Figure 9 Comparison between water levels measured and computed by the model inside the flood control area in Bergenmeersen from the 10th till the 12th September 2014.

Figures 10 and 11 show that the inlet and outlet discharges computed by the model fit well with the experimental data even if the numerical results overestimate slightly the measurements for both cases.

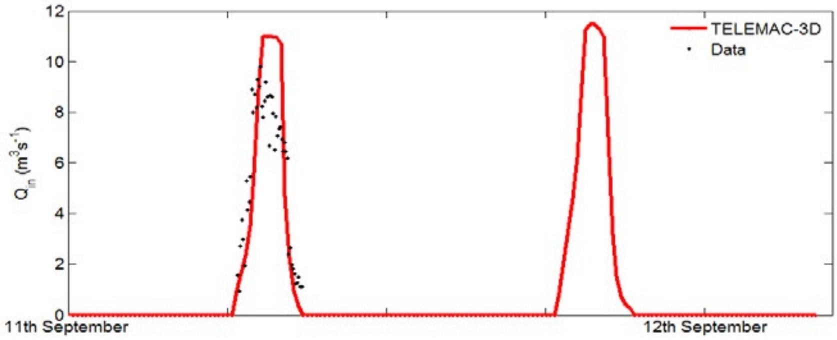

Figure 10 Comparison between inflow measured and computed by the model on September 2014.

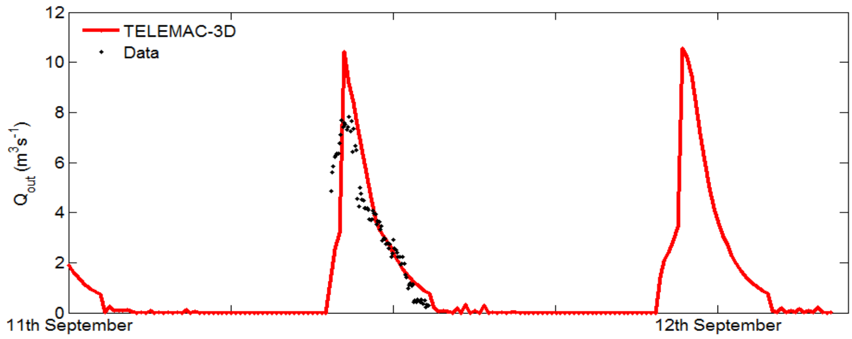

Figure 11 Comparison between ouflow measured and computed by the model on September 2014.

In order to have a more quantitative analysis for the differences found between numerical results and experimental data, the normalized root mean square error value [16] was calculated for the water levels and outlet and inlet discharges (Table 4). It can be verified that the error for the computed water levels is quite low (about 4%). Nevertheless a larger error is found for the inflow and outflow discharges (about 30%).

$$\varphi_i = \sqrt{\frac{\sum_{i=1}^{n}(d_i - m_i)^2}{\sum_{i=1}^{n} {d_i}^2}} \qquad [16]$$

The variable d_j represents the measured values and m_j the computed values.

Table 3 Normalized root mean square error for the water level (MWL_error), outlet discharge (Q_{out}_error) and inlet discharge (Q_{in}_error) computed by TELEMAC-3D.

MWL_error	0.043
Q_{out}_error	0.333
Q_{in}_error	0.306

In order to test the capability of the code to reproduce a storm event, the storm occurred on December 6th, 2013 was modeled. For this time period, measurements of water levels inside the flood control area of Bergenmeersen and in the Scheldt river were available. The model setup was kept the same as for the simulation period of 10th to 12th September, with exception of the imposed water level time series at the boundary

In Figure 12 the water level in the river computed by the hydrodynamic model agrees well with the measurements. In Figure 13 the model underestimates the water level for a normal tide and overestimates it for the peak flow of the storm tide of December 6th. During this period, the water level in the river reached the crest level of the dyke (6.2 m TAW) resulting in the overflow of water inside the flood control area. This behavior can be observed in the model results.

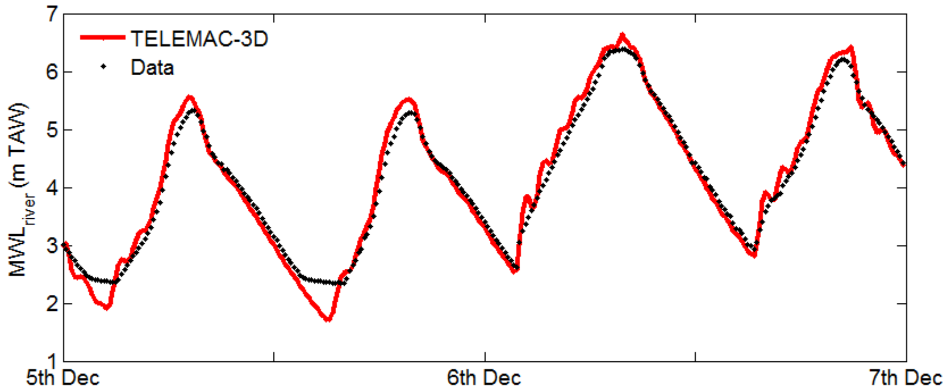

Figure 12 Comparison between water levels measured and computed by the model in the Scheldt river side from the 5th till the 7th December 2013.

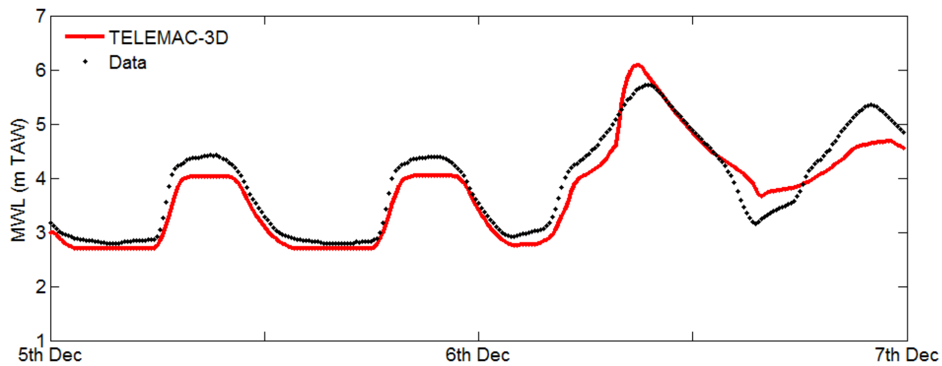

Figure 13 Comparison between water levels measured and computed by the model inside the flood control area in Bergenmeersen from the 5th till the 7th December 2013.

4. CONCLUSIONS

The scope of this paper was the implementation of a new culvert functionality in the three-dimensional hydrodynamic model TELEMAC-3D and its performance in both a normal tide and storm tide situation.

After the implementation in TELEMAC-3D. The culvert works as a couple of source and sink terms in which the discharges are calculated depending on the water levels between the source/sink terms, i.e., on the water levels in the flood control area and in the river. The theoretical framework was based on the work of Bodhaine (1968) together with Lencastre (1961). The model presents some limitations, such as the fact of making approximations on the calculation of water levels inside the culvert and not making a backwater analysis to know exactly the water levels at the entrance, inside and at the exit of the culvert.

A test case was applied to verify numerical results. A flood control area with controlled reduced tide, located in Bergenmeersen, Belgium, was chosen since a set of measurements were made during two time periods: one during the storm event of December 6th, 2013 where water levels were obtained inside the FCA and in the Scheldt river and a second one, made recently, from the 10th till 12th September where not only water levels were assessed but also a 13h measurement of inlet and outlet discharges that occurred through the culverts.

Despite the fact that the model approximates the real bathymetry by a grid (which is coarser inside the FCA) and the possible differences between the sluice construction on paper planview and the real built thing, the numerical results give a good agreement with the measured data. Both the water levels and inlet and outlet discharges during the time period of September and the storm event on December are well reproduced. This shows the potential of the model to simulate this kind of water flows both at a normal spring-neap tidal cycle and for storm surges.

ACKNOWLEDGMENTS

The authors would like to thank the people from the Hydrological Information Centre of Flanders Hydraulics for the measurement data of water levels and discharges. They like to thank The Maritime Access division of the Flemish government for the bathymetry data.

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