Implementation of controlled reduced tide and flooding areas in the TELEMAC 3D model of the Scheldt Estuary

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***Abstract*—A new model for the Scheldt estuary was built in TELEMAC 3D. The model includes the Belgian coastal area, a small part of the neighbouring French coastal area and a part of the Dutch coastal area, including the Eastern Scheldt; the entire Scheldt estuary with tributaries and even the controlled flooding areas alongside the estuary. It is the first time that all these areas of interest are included in a single schematisation with an appropriate local resolution. The culvert functionality was re-written for this project in order to cover a wider range of flow conditions that can exist through a culvert (relatively to what is currently implemented in TELEMAC 2D). Code development is shared with the community through the system of a subversion. This paper discusses the setup of the model, and describes the new functionalities added to the code.**

I. INTRODUCTION

The Scheldt estuary connects the port of Antwerp to the sea, making the Western Scheldt one the busiest naval traffic routes in the world. To improve the hinterland connection to France, the Flemish government wants to improve the navigability of the Upper Sea Scheldt for inland shipping (Fig. 2). At the moment, the upstream part of the Upper Sea Scheldt is a Class IV fairway. An integrated plan is being developed to increase navigability and make the Upper Sea Scheldt a class Va fairway. Besides navigability other functions like nautical safety, flood protection, naturalness and recreation are also taken into account.

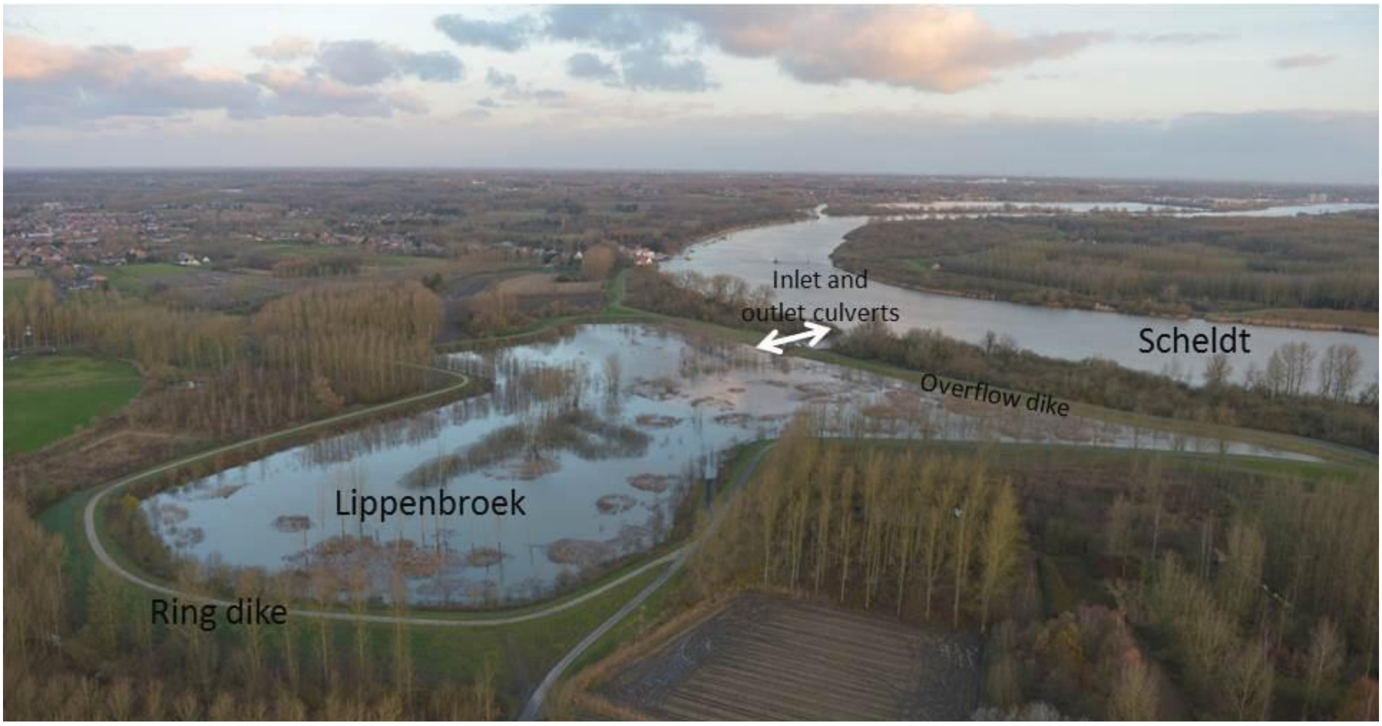

Figure 1. Helicopter view on FCA Lippenbroek taken after storm surge of December 6th 2013.

A feasibility study revealed that rather small measures could ensure the increase in navigability class and also increasing the safety for ships of class IV and lower.

To further develop the conclusions of this feasibility study extensive hydrodynamic and sediment transport modelling is needed. The modelling for this project will focus on the Upper Sea Scheldt part of the Scheldt Estuary (Fig. 2).

The newly developed model has to be able to cope with future project demands as well, asking for a larger model domain than just the Upper Sea Scheldt. Furthermore, the Scheldt Estuary is a complex system where changes made in the mouth can affect hydrodynamics as far inland as Ghent. It was therefore chosen to develop a model of the Scheldt Estuary that would include all zones that could affect each other: the Belgian coastal zone, with the Scheldt Estuary mouth as most important area, the entire Scheldt Estuary and all tributaries under tidal influence.

Existing model schematizations of the Scheldt Estuary lack the grid resolution in the Upper Sea Scheldt necessary for the research to be done or miss the flexibility of an unstructured finite element grid. This new model will also include existing and future controlled flooding areas (CFA) along the estuary. This paper will discuss the TELEMAC 3D hydrodynamic model of the Scheldt Estuary with main focus on the implementation of the flooding areas. The calibration of the model is ongoing so it will not be included in this paper.

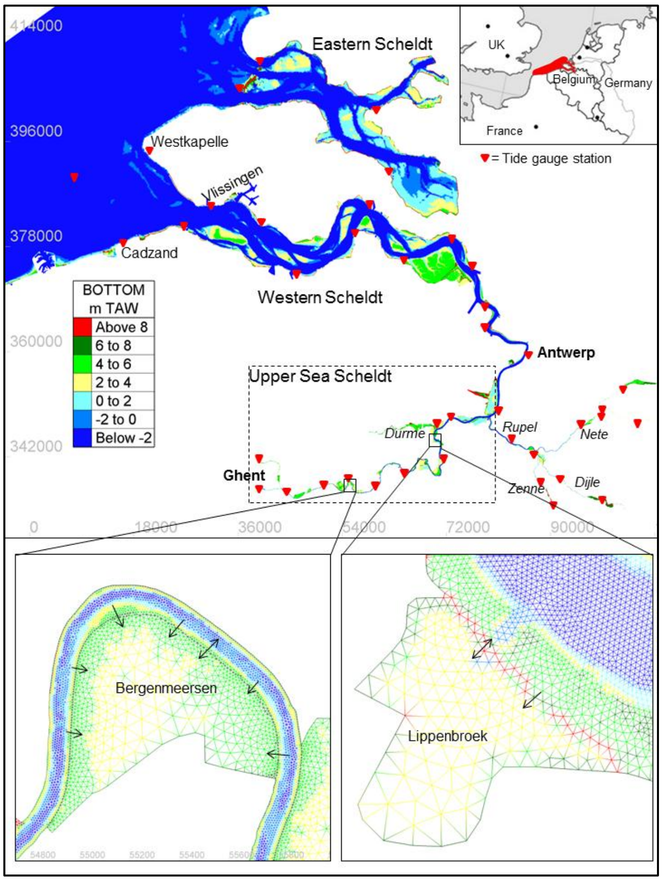

Figure 2. Map of the Scheldt Estuary model. Upper right: extent of the model (red) in the coastal area; Lower left: detail of the flooding area 'Bergenmeersen'; Lower right: detail of the flooding area 'Lippenbroek'.

II. METHODS AND MATERIAL

A. The Scheldt 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 Scheldt estuary extends from Vlissingen (km 0) to Ghent (km 160) (Fig. 2). Upstream at Ghent weirs prevent the tide from penetrating more upstream. The tidal influence also reaches to major tributaries like Durme, Rupel, Nete, Dijle and Zenne. The part of the estuary from the mouth until the Dutch/Belgian border (58 km) is called Western Scheldt and is characterized by different ebb and flood channels surrounding large intertidal sandbars (Fig. 2). The part upstream from the border until the sluices in Ghent is called Sea Scheldt (105 km) and is characterized by a single channel. Flood enters the estuary twice a day with an average flood volume at the mouth of 1.04 10^9 m³ [1].

The mouth at Vlissingen is 5 km wide and the mean tidal range there is 3.8 m. Because of the funnel shape of the estuary the vertical tide is amplified and the maximum range lies around Schelle (km 91) and reaches 5.2 m at spring tide. Further upstream from Schelle the tidal range decreases again due to increased influence of bottom friction. Near the weirs in Ghent the mean vertical tidal range is still 2 m. The longitudinal salinity profile is primarily determined by the discharge, although discharges of the Scheldt and tributaries (about 75-100 m³/s) are negligible compared to the tidal volume [2,3]. The estuary is well mixed, which means that vertical salinity gradients are small or negligible [1].

In February 1953 a storm caused many casualties in England, Belgium, but mainly in the Netherlands. This was the start for an extensive programme, the Delta works, to protect the coastline from storm tides. In Belgium it was after the big storm of 1976 that a plan, the Sigma plan, was developed to protect the land from floods from the Scheldt estuary and tributaries. Together with the protection against flooding a lot of nature areas were and are being restored. Instead of raising the dikes or closing the estuary, Flanders chose the option of controlled flooding areas (CFA's) and areas with controlled reduced tide (CRT).

B. CFA and CRT

CFA's are areas along the Scheldt estuary and tributaries which have a ring dike at the same protective level as the dikes just alongside the river/estuary. The dike directly between the river and the CFA is lowered so water can flow over the dike into the CFA when it reaches a critical level (Fig. 3 nr.1). When the water level drops after a storm tide, outlet culverts evacuate the water out of the CFA back into the river (Fig. 3 nr.2). A valve prevents the water from the river to enter the CFA through these outlet culverts in normal circumstances. A CRT area is the same as a CFA area, but it also has inlet culverts to let water enter each tide (Fig. 3 nr.3). The elevation of the structure determines how much water can enter the area as it determines the time at which the tide can start entering through the culverts. So each tide water enters and leaves these areas as a reduced tide compared to the tide in the river/estuary (Fig. 3 nr.4). With a storm surge the CRT area acts the same as the FCA (Fig. 3 nr.5). By recreating a reduced tide in these areas a lot of tidal nature is created. Tidal flats and marshes can develop, giving these areas besides a safety function a nature function. The CFA and CRT's are made in such a way that they evacuate only the critical amount of water at the critical time (highest high water levels) from a storm tide. With the storm of 6th of December 2013 some of these areas were completely filled (Fig. 1 and 4).

C. Model grid and boundary conditions

The model grid consists of 467,766 nodes in 2D mesh. In the 3D model we use 5 planes totaling 2,338,830 of nodes. The resolution in the coastal area varies from 200 to 500 m depending on the depth. The resolution in the Eastern Scheldt is 200 m. In the Western Scheldt the resolution is 120 m. In the Sea Scheldt this resolution is increasing slowly towards 30 m near Antwerp, further increasing towards 10 m in the Upper Sea Scheldt. Upstream the tributaries the resolution can reach 4 m. There are 6 upstream boundaries with prescribed discharge. The downstream boundary is at sea. The subroutine bord3d.f was changed to allocate a water level, a flow velocity and a salinity value for each boundary node separately. The Thompson method is applied. The time step is 5 s. Start-up computation is first made which will be used as an initial condition for all future simulations. A constant elevation of 1m is applied as initial condition for the start-up computation.

For this start-up run a linear smoothing function was introduced in the bord3d.f subroutine to let the water levels slowly change from the initial values to the time-varying boundary condition. Time series for this boundary were extracted from the ZUNO model [4].

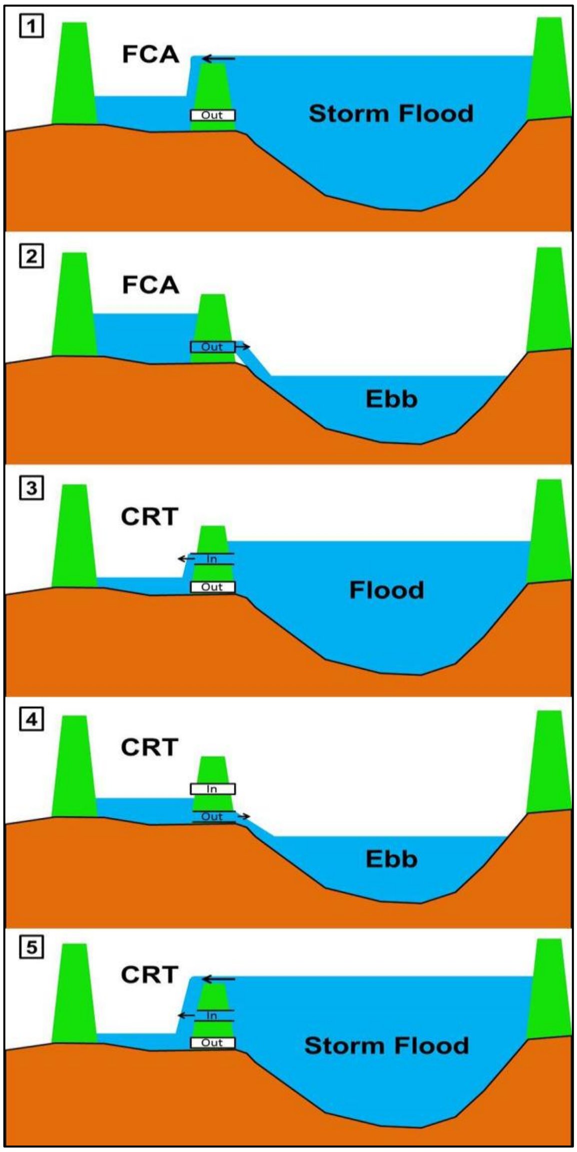

Figure 3. Schematic presentation of the functioning of a CFA and CRT. 1. FCA fills when storm surge enters the estuary. 2. FCA drains at ebb tide. 3. CRT area let a reduced tide in the flooding area. 4. CRT area drains like FCA at ebb tide. 5. With a storm surge the CRT will function like a FCA and CRT together.

Figure 4. FCA/CRT 'Bergenmeersen' filled during storm surge of 6th of December 2013.

Salinity is used as an active tracer. An initial guess for the salinity field in the estuary is read in the geometry file. We made the subroutine fonstr.f read besides the bottom and bottom friction, also the initial salinity. To simulate the structures of the Eastern Scheldt barrier (65 pillars in total) we changed the subroutine source.f from TELEMAC to simulate the dragforce created by the barrier. Including the pillars in the mesh was also an option, but this would locally increase the number of nodes and our computation time. Wind is applied on the coastal zone through the subroutine meteo.f. To include the culvert function in TELEMAC 3D the subroutine t3d_debsce.f was changed.

D. FCA and CRT: culvert implementation

Existing code: In TELEMAC 2D it is possible to model free flow or pressurised flow in a tube. There is a connection made between two points and based on their water levels a type of discharge equation is chosen. In TELEMAC 2D four equations are present following [5]. The flow velocities are deduced from the discharges and then taken into account as source terms both in the continuity and momentum equations.

In TELEMAC 3D (v6p3) hydraulic structures such as culverts or tubes are not implemented. So we implemented a culvert structure in the t3d_debsce.f subroutine. Assuming the hydrostatic hypothesis, TELEMAC 3D solves the following three-dimensional RANS equations:

$$\frac{\partial U}{\partial x}+\frac{\partial V}{\partial y}+\frac{\partial W}{\partial z}=0 \quad (1)$$

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

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

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

(U, V, W) are the three components of the flow velocity, (*u*,*v*) are the depth integrated flow velocities, ν is the diffusion coefficient, g is the gravitational constant and (*Fx*, *Fy*) represent the source and sink terms of the momentum equations (2 and 3). We made use of the capability of the model to impose source/sink terms in the domain. Therefore, the culvert's 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 added to the mesh 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 the symmetric value of that discharge so that always $Q_{river}= -Q_{floodplain}$. Because the culverts in our application are not long structures, the assumption was made that the discharge extracted on one side is added immediatley at the other side.

New equations were incorporated in the code in order to cover a wider range (relatively to what is currently implemented in TELEMAC 2D) of the flow conditions that exist through a culvert. The following equations, correspond to each type of flow that can occur through the culvert.. They are based on the equations proposed by[5] and similar to the ones incorporated in the DELFT 3D model. The flow type 1 conditions from [6] were not incorporated since they only occur when the culvert slope is larger than the critical flow slope.

Type 2 – Critical depth at outlet:

$$Q=\mu h_c W\sqrt{2g*\left(S_1-(z_2+h_c)\right)} \quad (5)$$

Type 3 – Tranquil flow:

$$Q=\mu(S_2-z_2)W\sqrt{2g(S_1-S_2)} \quad (6)$$

Type 4 – Submerged outlet:

$$Q=\mu DW\sqrt{2g(S_1-S_2)} \quad (7)$$

Type 5 – Rapid flow at inlet:

$$\mathrm{Q}=\mu\mathrm{DW}\sqrt{2\mathrm{gh}_1} \quad (8)$$

Type 6 – Full flow with free outfall:

$$Q=\mu DW\sqrt{2g(S_1-(z_2+D))} \quad (9)$$

With Q as the discharge through the culvert, µ the total head loss coefficient, W the culvert width, g the gravitational constant, S_1 the water level on side 1, S_2 the water level on the other side, D the culvert height, 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 Fig.5. The conditions for which each type of flow occurs are described 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 *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 [6]. 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. An example of such an additional structure for CRT culverts is a weir at the entrance or exit. The weir level is used to adapt (increase or decrease) the reduced tide in the flooding area. Such combined structures have to be taken into account in the model. Then the geometric features of the culvert presented in Fig. 5 are modified as given in Fig. 6. Model tests showed that an equivalent culvert bottom elevation should be used, which replaces both the bottom elevations z_1 and z_2 in the formulas decribed above. The equivalent bottom culvert elevation z is then equal to the mean of z_1 and z_2. In terms of the culvert height D: if the flow goes from side 1 to side 2, D_1 is used and vice versa.

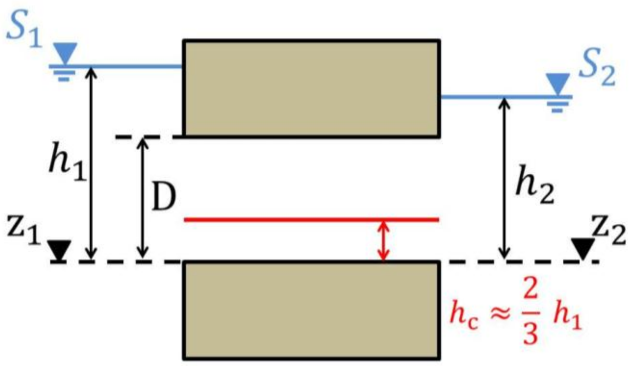

Figure 5. Representation of the different variables used for the culvert equations.

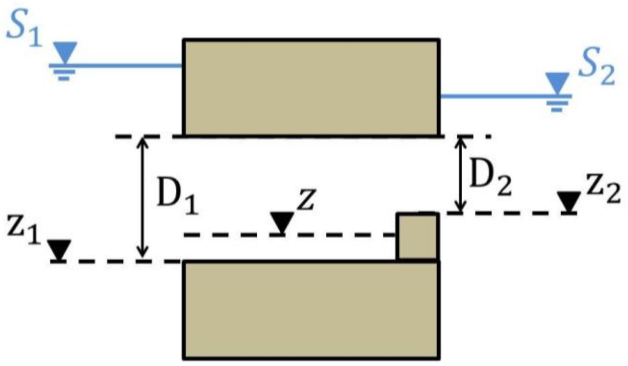

Figure 6. Representation of the different variables used for the combination of a culvert with a weir.

The head loss coefficient μ was adapted from the one calculated in TELEMAC 2D, based on [5]. But more terms were added to account for the extra head loss of additional structures in front and behind the culvert structures in our application, like one-way valves and trash screens. The head loss due to singularities can be obtained by the general relation [7,5]:

$$\Delta H = C\frac{U^2}{2g} \quad \text{or} \quad U = \mu\sqrt{2g\Delta H} \quad \text{with } \mu = \frac{1}{\sqrt{C}} \tag{10}$$

$C = C_1 + C_2 + C_3 + C_v + C_T + C_p$ 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. Usually, there is an abrupt contraction that will cause a head loss due to the desaccelaration of the flow immediately after the culvert entrance. If this is the case, according to [8] C_1 will be 0.5.

TABLE I. PARAMETERS TO DETERMINE THE TYPE OF FLOW THROUGH THE CULVERT.

	$\frac{S_1 - z}{D}$	$\frac{S_2 - z}{D}$	$S_2 - z$	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

Already in the past, [6] noticed that the discharge coefficient (C_D) for type 5 flow had to be lowered comparetively 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:

$$\Delta H_1 = {\alpha_1}^5 C_1 \frac{U^2}{2g} \tag{11}$$

With: $4 \leq {\alpha_1}^5 \leq 10$ according to [6].

C_2 represents the head loss coefficient due to the friction in the structure and is expressed by [7]:

$$\Delta H_2 = C_2\frac{U^2}{2g} = \frac{2gLn^2}{R^{4/3}}\frac{U^2}{2g} \tag{12}$$

where *n* the Manning Strickler coefficient of the structure 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 [7]:

$$\Delta H_3 = \left(1 - \frac{A_s}{A_{s2}}\right)^2 \frac{U^2}{2g} = C_3\frac{U^2}{2g} \tag{13}$$

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 5 type flow occurs a correction coefficient (C_{V5}) is 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 than the flow converges into a single culvert barrel. Following [5] the head loss coefficient through parallel pillars is given by:

$$C_p = \beta \left(\frac{Lp}{b}\right)^{4/3} sin\theta \qquad (14)$$

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.

Tracers- 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:

$$\frac{\partial T}{\partial t} + U\frac{\partial T}{\partial x} + V\frac{\partial T}{\partial y} + W\frac{\partial T}{\partial z} = \nu_t \Delta(T) + Q' \qquad (15)$$

The tracer diffusion coefficient is given by ν_t and Q' represents the source terms for tracers. For the culvert in 3D the tracer concentration in the model domain is assigned to source and sink terms for tracers (Q'). TELEMAC 3D associates these concentrations to the discharges and volumes at the source terms.

E. Test case 'Lippenbroek'

Lippenbroek was a pilot project for FCA with CRT function. The CRT function was created to restore tidal habitats along the Scheldt Estuary, many of which have disappeared or degraded over the course of time because of agricultural, urban or industrial developments [9]. The location of the rather small (10ha) Lippenbroek area in the Scheldt Estuary is given in Fig.1. Lippenbroek became operational in March 2006. It is located in the freshwater part of the estuary. The FCA is surrounded by a ring dike at 8.35 m TAW, i.e. the Belgian reference level (which is approximately the mean low tide level along the Belgian North Sea coast). 40 m of this dike was lowered (6.8 m TAW) to function as an overflow dike (Fig. 1). There are three inlet culverts and one outlet culvert. The main characteristics are given in table 2.

TABLE II. CHARACTERISTICS OF IN- AND OUTLET OF LIPPENBROEK.

	Inlet sluice	Outlet sluice
Number of culverts	3	1
Culvert width (m)	1	1.5
Culvert height (m)	1.9	1.5
Culvert length (m)	13	40
Culvert floor (m TAW)	4.0	1.5
Crest level weirs (m TAW)	5.3 / 5.0 / 4.7	

On average the tidal range is reduced from 5 m in the river to 1 m in the polder. The natural spring-neap tidal cycle is maintained and the tidal range in the polder varies from a few decimetres at neap tide to over 1.5 m at spring tide [10]. The form of the tidal curve in the polder differs from the one in the estuary, as a stagnant phase is present both at high and low water in the polder, but the crucial characteristics of the tide for ecological purposes are present. Fig. 7 gives an idea of the inlet structure and the wooden log weirs just behind it.

Figure 7. left: inlet structure seen from the FCA side; right: close up of the culverts and the wooden log weirs (photo: [10]).

Modeled water levels in the FCA were compared with measured ones. The modeled discharges through the culverts were compared with measurements. Table 3 presents the different parameters used to model the Lippenbroek FCA with CRT. The different geometric features for the inlet and outlet sluices have to be given and the direction of the flow through the culvert has to be indicated. An outlet sluice only allows the flow to go from the floodplain to the river (CP= 2) because it has a one-way valve, and an inlet sluice allows the flow to go in both directions (CP=0), but water leaving the FCA through an inlet sluice will only occur when the FCA was completely filled in a storm situation.

The different head loss coefficients were assigned and some of them (C_v and C_t) were used to calibrate the model based on comparison with measurements. Regarding the other head loss coefficients typical values [7] were imposed. For the one-way valve of the outlet culvert a C_v value of 1 was given [8]. Trash screen head loss coefficients were also included, since the screens are present at the inlet and outlet culverts in Lippenbroek. Tree branches and leaves may hamper the free flow through these trash screens, hence this parameter. A value of *n*=0.015 (typical value for concrete in smooth conditions) was assigned for the Manning Strickler parameter inside the culvert [6]. For the TELEMAC 3D culvert input the following parameters are equal for the four culverts: C_1 = 0.5; C_3 = 1; c = 10; $\alpha_1{}^5$ = 6. The other parameter values that differ from culvert to culvert are given in table 3.

TABLE III. CULVERT PARAMETER VALUES FOR LIPPENBROEK TEST CASE.

	C_v	C_t	C_{V5}	W	D1	D2	L	CLP
	[-]	[-]	[-]	[m]	[m]	[m]	[m]	[-]
Inlet1	0	0.8	0	1	1.9	0.6	13	0
Inlet2	0	0.8	0	1	1.9	0.9	13	0
Inlet3	0	0.8	0	1	1.9	1.2	13	0
Out	1	0.1	1.5	1.5	1.5	1.5	40	2

Following what was done in TELEMAC 2D, the flow direction is also imposed through the keyword CLP and a relaxation parameter is incorporated in the code. For the Lippenbroek test case the modelled water levels in the FCA were compared with measured ones. The discharge through the outlet culvert and the combined discharge through the inlet culverts is also compared with measured values.

F. *Test case 'Bergenmeersen'*

Field measurements were also available for a second FCA with CRT function: Bergenmeersen. The location of this area is even further upstream compared to Lippenbroek (Fig. 2). Bergenmeersen has a longer dike, separating it from the river and this dike is completely used as overflow dike with a crest level of 6.8 m TAW. The configuration of the inlet and outlet culverts for this area is quite different compared to Lippenbroek. If the model can reproduce water levels and discharge for both areas we proved that the model can handle a wide range of structures. In this case six inlet culverts were built on top of three outlet culverts. Each culvert was divided into two pieces at the entrance at the river side. Also at the river side there were wooden log weirs at different heights and trash screens. The outlet culverts had one-way valves and the inlet culverts could be closed by a valve descending from the ceiling. Table 4 gives an overview of the culvert dimensions.

TABLE IV. DIMENSIONS OF CULVERTS BERGENMEERSEN.

	Inlet (Scheldt)	Inlet (FCA)	Outlet (Scheldt)	Outlet (FCA)
Culvert width (m)	2.7		3	
Culvert length (m)	9.5		18	
Culvert height (m)	1.6	2.25	1.1	2.25
Culvert floor (m TAW)	4.2	2.2	2.7	2.2
Crest of weirs (m TAW)	4.2/ 4.2/ 4.2/ 4.35/ 4.5 / 4.5			

For the different head loss coefficients we tried to keep as much of them the same as in the Lippenbroek test case, but also because of the different configuration of the culvert structure, some changes were necessary. The head loss coefficients at the entrance of the inlet culverts were increased in order to take into account the effect of the flow being split into two parts by pillars. According to [5] the head loss due to pillars is $C_p \approx 0.4$ and therefore C_1becomes C_1=C_1+C_p. At the exit of the outlet culverts the same split by pillars was present, but the head loss of that was taken into account in the head loss of the valve (C_v=12). Also during the measurement campaign, the trash screens at the culverts were not cleaned and therefore this coefficient was increased both for the inlet and outlet culverts (C_t=1).

III. RESULTS

A. *Lippenbroek*

By only using the head loss coefficients for the trash screens and for the one-way valve as calibration parameters good results were obtained. The water levels inside the area agreed well with the measurements (Fig. 8). Only at the low water levels there was a discrepancy. Creeks drain the area and these are not so well represented in the mesh. Also the discharges for the outlet culvert compared well with the measured values (Fig. 9), as did the discharges for the inlet culverts (Fig. 10). To show that the culvert function does indeed switch between different flow types through the culvert based on water levels in the river and the flooding area Fig. 11 is added. This figure gives the different flow types as discussed before in the right hand y-axis while we can follow the water levels in the river and the flooding area in the left hand y-axis.

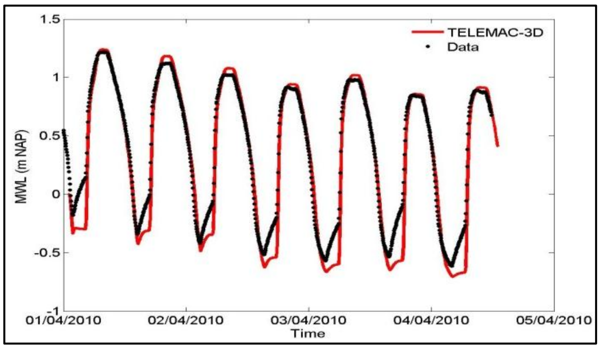

Figure 8. Modelled (red line) and measured (black dots) water level in the FCA Lippenbroek.

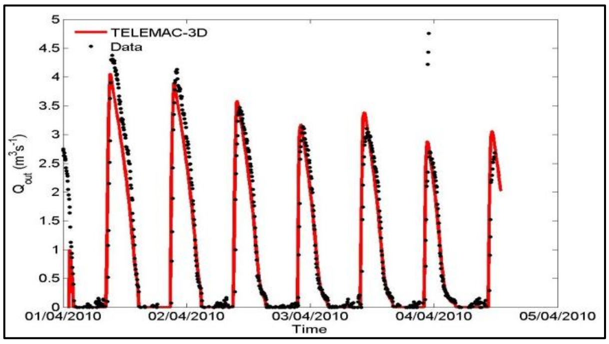

Figure 9. Comparing discharges from the outlet culvert: modelled (red line) vs. measured (black dots).

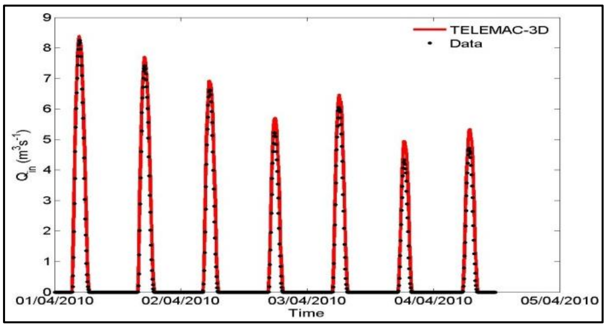

Figure 10. Comparing discharges of the combined inlet culverts: modelled (red line) vs. measured (black dots).

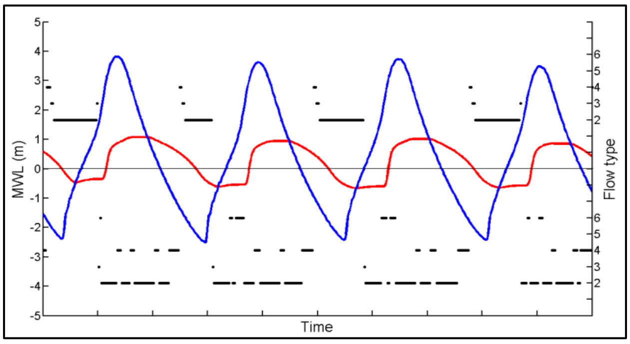

Figure 11. Model results: blue line indicating the water level in the river. The red line gives the reduced tide in the CRT 'Lippenbroek'. The right y axis shows the flow types that occurred through the inlet culvert (below) and the outlet culvert (up) (black dots).

B. *Bergenmeersen*

The results for Bergenmeersen were not so good as for Lippenbroek, but still good, taken into account the very complex in- and outlet structure. A comparison between modelled (red line) and measured (black dots) water levels is given in Fig. 12. Discharges were also modelled and compared well with the measured values (no figures shown).

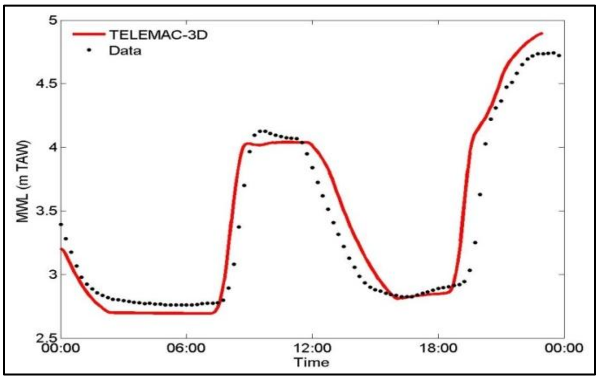

Figure 12. Comparing modelled (red line) and measured (black dots) water levels in Bergenmeersen.

IV. Conclusions

Both Lippenbroek and Bergenmeersen have very different in- and outlet structures. The model results of both areas are very good to conclude that the culvert function can handle these different types of structures or combination of structures (weirs, valves, trash screens). With the experience of these two areas we try to find a set of parameters for all FCA/CRT areas in the estuary (13 active in 2013; >50 in the future).

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