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Figure 1 - Bergenmeersen: Flood control area (FCA) with controlled reduced tide (CRT) along the Scheldt estuary, Belgium (picture: www.sigmaplan.be)

***Abstract* – The numerical simulation of flows between (tidal) rivers and floodplains often requires the representation of hydraulic structures such as culverts. The aim of this work is to enable TELEMAC-3D to represent such structures. A model for culverts had already been implemented in TELEMAC-2D, and a new one was added through this work, with discharge configurations. A file describing the culverts positions and characteristics must be provided by the user in TELEMAC-3D in the same way as in TELEMAC-2D. Each culvert is constituted of two source (/sink) points at which the discharge rates have opposite signs. The culvert paires are thus treated exactly like source / sink points in TELEMAC-3D, and the discharge rates are computed by the same subroutine as in TELEMAC-2D. The model was validated on two test-cases with TELEMAC-3D. One of those test-cases is the flood control area, Bergenmeersen (picture here above), situated along the Scheldt estuary. The culverts were calibrated for a data set measured in 2014. The code was validated on water level measurements of this area during the storm of December 6th, 2013. The modelled water levels fitted very well the measured ones.**

I. INTRODUCTION

The Scheldt estuary and its tributaries in Flanders, Belgium, are influenced by the tides of the North Sea until far upstream. In case of extreme weather conditions like storm tides and heavy rainfall, this could lead to extreme water levels and even floods.

In 1976 large parts of the region of the Scheldt estuary in Flanders were flooded after a storm tide. To protect the region from such disaster in the future, the Flemish government presented the Sigma-plan (S for Scheldt) in 1977. Originally this plan aimed at strengthening and elevate the dikes, building flood control areas (FCA) and building a storm surge barrier on the Scheldt near Antwerp. The storm surge barrier was never built, but the flood control areas were. In 2005 the Sigma-plan was updated with the newest scientific developments and to account for climate change and integrated water management. Furthermore, the Sigma-plan wanted to contribute to important European nature goals for the Flanders region.

To create more tidal habitat in the Scheldt estuary, some of the flood control areas were adapted so that the tide could enter and leave these areas in a reduced way (the tide has to be reduced because most of the land surrounding the Scheldt estuary in Flanders has a low elevation)[1]. New structures were built to allow water to flow in and out of these areas every tide.

When in 2013 a new large scale TELEMAC-3D model of the entire Scheldt estuary was made (*i.e.* the Scaldis model [2]), there was a need to model also the water exchange between the river and these flood control areas. At that time there was no subroutine in TELEMAC-3D to model this kind of flow interactions. The Fortran code that was developed for this model in 2014 and presented that year at the TELEMAC-MASCARET User Club conference [3] is now implemented in the main code of TELEMAC so that it is available for all users.

This paper will give a description of how the code was developed and how it was implemented in TELEMAC. It further provides an elaborate description of the parameters of this code by applying it to a test-case called Bergenmeersen (also available amongst the TELEMAC validation test cases).

II. FLOW THROUGH CULVERTS

A number of studies regarding the description of flows through culverts refer to the work of *Bodhaine* [4]. Bodhaine categorized the flow through a culvert into six types, and for each type the discharge is calculated in a different way. The equations are deduced from the continuity and energy equations between the approach section (Figure 2) and the exit (downstream) section of the culvert. The type of flow depends on whether the culvert flows full and whether the flow is controlled by the entrance or exit part of the culvert. Figure 2 shows a sketch for the culvert flow definition. Let z be the elevation of the culvert entrance relative to the datum through the culvert exit. The gravitational constant is given by g and h_{f12} is the head loss due to friction from the approach section to the culvert entrance; h_{f23} is the head loss due to friction inside the culvert, d_2 and d_3 are the water depths at the culvert entrance and exit, respectively; V_1, V_2 and V_3 are the velocities at the approach section, culvert entrance and culvert exit, respectively; D is the culvert height; and h_1 and h_4 are the water depths upstream and downstream of the culvert structure.

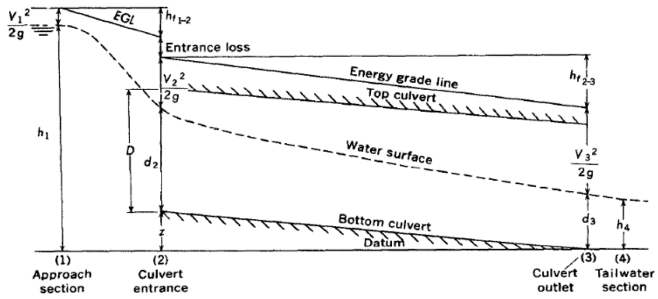

Figure 2 - Sketch of the general flow through a culvert [4].

The six types of flows classified by [4] depend on the water depths upstream and downstream of the culvert. These types are discussed below:

Flow Type 1 – Critical depth at inlet – supercritical flow inside the culvert

In Flow Type 1 the critical depth occurs at the entrance of the culvert and the flow is supercritical inside the culvert. The culvert slope (S_0) has to be greater than the critical slope (S_c) and the culvert flows partially full. For the Froude number $Fr=1$ (which is the case at the entrance section for a Flow Type 1), the discharge coefficient is typically C_D=0.95. The discharge is calculated according to the following formula:

$$Q = C_D A_c \sqrt{2g\left(h_1 - z - h_c - h_{f12} + \alpha \frac{V_1^2}{2g}\right)} \quad (1)$$

with:
- C_D the discharge coefficient
- A_c flow area at critical water depth
- g the gravitational constant
- h_1 upstream water depth
- z elevation of the culvert entrance
- h_c critical water depth
- h_{f12} head loss due to friction from the approach section to the culvert entrance
- α kinetic energy correction coefficient for the approach section
- V_1 average flow velocity at the approach section of the culvert

Flow Type 2 – Critical depth at outlet – subcritical flow inside the culvert

In Flow Type 2 the flow is tranquil (i.e. subcritical) inside the culvert. The critical depth is located at the culvert outlet. The culvert flows partially full. Here the culvert slope S_0 has to be smaller than the critical slope S_c. The discharge coefficient is similar to Flow Type 1. The discharge is calculated according to the following formula:

$$Q = C_D A_c \sqrt{2g * \left(h_1 - h_c - h_{f12} - h_{f23} + \alpha \frac{V_1^2}{2g}\right)} \quad (2)$$

with: h_{f23} head loss due to friction inside the culvert.

Flow Type 3 – Tranquil flow – subcritical flow throughout the culvert

In Flow Type 3 the flow is subcritical throughout the culvert. There is no critical depth. The culvert flows partially full. Like Flow Types 1 and 2, the discharge coefficient varies in function of the Froude number, being typically between C_D=0.82 – 0.95. The discharge is calculated according to the following formula:

$$Q = C_D A_3 \sqrt{2g\left(h_1 - d_3 - h_{f12} - h_{f23} + \alpha \frac{V_1^2}{2g}\right)} \quad (3)$$

with: A_3 flow area at the culvert outlet
d_3 water depth at the culvert outlet

Flow Type 4 – Submerged inlet and outlet

In Flow Type 4 the culvert inlet and outlet are submerged. The culvert flows full. The discharge coefficient varies in function of the culvert geometry, ranging typically between C_D=0.75 and C_D=0.95. The discharge is calculated according to the following formula:

$$Q = C_D A_0 \sqrt{\frac{2g(h_1 - h_4)}{1 + 29 C_D^2 n^2 L / R^{4/3}}} \quad (4)$$

with: A_0 flow area at the culvert entrance
h_4 downstream water depth
n Manning coefficient
L length of the culvert
R hydraulic radius

The user must be aware that, in [4], imperial units are used. In equation 4 here above the factor 29 is an artefact of the imperial units.

Flow Type 5 – Rapid flow at inlet

In Flow Type 5, the flow is supercritical at the inlet to the culvert. The culvert flows partially full. Flow Type 5 are rare. When it occurs, the discharge coefficient is in general lower than the other types.

$$Q = C_D A_0 \sqrt{2g(h_1 - z)} \quad (5)$$

Flow Type 6 – Full flow with free outfall

In Flow Type 6 the culvert flows full. The discharge coefficient is similar to the one obtained for the Flow Type 4. The discharge is calculated according to the following formula:

$$Q = C_D A_0 \sqrt{2g\left(h_1 - d_3 - h_{f23}\right)} \quad (6)$$

The indices might seem a bit confusing, but it was chosen to take the formulas from Bodhaine [4] as they are. Bodhaine differentiated between the following six flow type based on conditions given in Table 1.

Table 1. Flow types as defined by Bodhaine (1968).

Flow Type 1	$\frac{h_1 - z}{D} < 1.5$	$\frac{h_4}{h_c} < 1.0$	$S_0 > S_c$
Flow Type 2	$\frac{h_1 - z}{D} < 1.5$	$\frac{h_4}{h_c} < 1.0$	$S_0 < S_c$
Flow Type 3	$\frac{h_1 - z}{D} < 1.5$	$\frac{h_4}{D} \leq 1.0$	
Flow Type 4	$\frac{h_1 - z}{D} > 1.0$	$\frac{h_4}{D} > 1.0$	
Flow Type 5	$\frac{h_1 - z}{D} \geq 1.5$	$\frac{h_4}{D} \leq 1.0$	
Flow Type 6	$\frac{h_1 - z}{D} \geq 1.5$	$\frac{h_4}{D} \leq 1.0$	

Different culvert geometry will affect the choice between Flow Type 5 or 6. To differentiate between both types, Bodhaine suggests to use the relations given in Figure 3, in which r denotes the radius of curvature of a rounded entrance and w is the measure of a chamfered entrance. First a curve corresponding to r/D, w/D is chosen. Then a point is set using the value for the culvert slope and for the ratio between the culvert length and height. If the point lies to the right of the chosen curve, the discharge is of Flow Type 6, if it lies to the left of the curve, the discharge is of Flow Type 5.

The head loss coefficients are topics of different studies made through laboratory experiments. A number of authors have arrived to different values or empirical relationships for the head loss coefficients. [4] suggests different values for the discharge coefficient (C_D) for each Flow Type and depending on a number of culvert geometric features. The discharge coefficients can vary from 0.39 to 0.98. Another example is given by [5] who proposes a non-dimensional coefficient μ, also referred to as a discharge coefficient that, for hydraulic structures made of only one culvert, can be written as follows:

$$\mu = \frac{1}{\sqrt{C_1+C_2+C_3}} \tag{7}$$

with:
- C_1 head loss coefficient at the entrance of the hydraulic structure
- C_2 head loss coefficient in the hydraulic structure
- C_3 head loss coefficient at the exit of the hydraulic structure

If the general expression for the discharge $Q = \mu A\sqrt{2g\Delta H}$ proposed by [5] is compared with the formulae given by [4], it can be seen that the non-dimensional discharge coefficient (μ), incorporates both the effect of the discharge coefficient (C_D) and the continuous and local head losses. ΔH is the head for each type of flow.

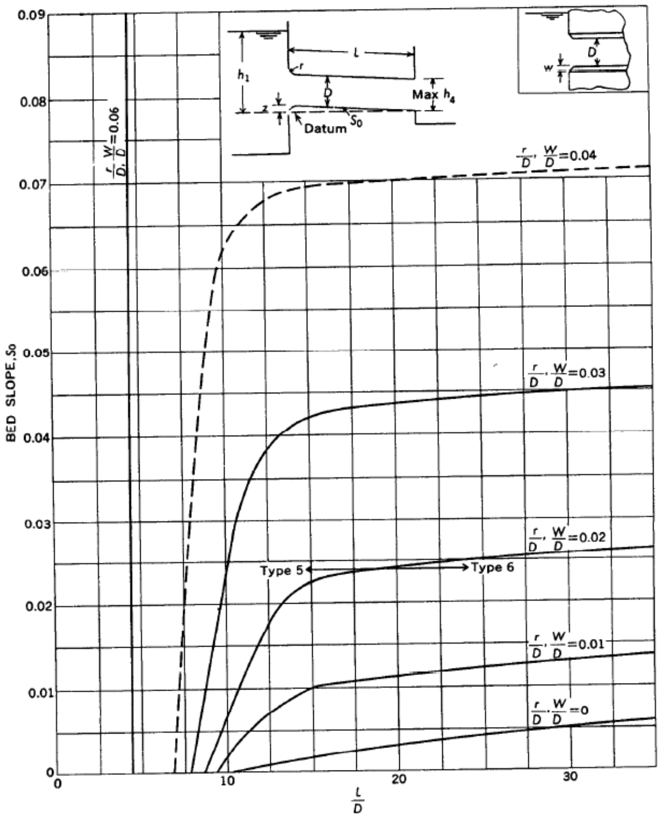

Figure 3 - Criterion for classifying flow types 5 and 6 in concrete box or pipe culverts with square, rounded, or beveled entrances, either with or without wing walls [4]

III. TRANSLATION INTO CODE

Following the proposition of [5] the equations proposed by [4] are translated into equations that could be implemented in the TELEMAC Fortran code. Flow Type 1 was not implemented because it only occurs when the culvert slope is larger than the critical flow slope, which only happens in very rare occasions.

Flow Type 2 – Critical depth at outlet:

$$Q = \mu h_c W\sqrt{2g * (S_1 - (z_2 + h_c))} \tag{8}$$

Flow Type 3 – Tranquil flow:

$$Q = \mu(S_2 - z_2)W\sqrt{2g(S_1 - S_2)} \tag{9}$$

Flow Type 4 – Submerged outlet:

$$Q = \mu DW\sqrt{2g(S_1 - S_2)} \tag{10}$$

Flow Type 5 – Rapid flow at inlet:

$$Q = \mu DW\sqrt{2gh_1} \tag{11}$$

Flow Type 6 – Full flow with free outfall:

$$Q = \mu DW\sqrt{2g * (S_1 - (z_2 + D))} \tag{12}$$

with:
- Q discharge through the culvert,
- W culvert width,
- D culvert height,
- μ total head loss coefficient,
- S_1 water level on side 1,
- S_2 water level on the side 2,
- h_1 water level above the culvert base on side 1,
- h_2 water level above the culvert base on side 2,
- h_c critical water level inside the culvert (this will be assumed to be close to 2/3 of h_1),
- z_1 base level of the culvert at side 1, and
- z_2 base level of the culvert at side 2.

Most of these variables are shown in a schematic representation of the culvert in [6].

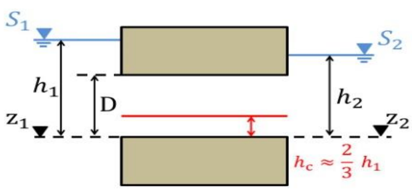

Figure 4 - Schematic representation of a culvert with the different parameters [6]

The conditions for which each type of flow occurs are summarized in Table 2. To distinguish Flow Type 5 from Flow Type 6 a constant C56 is defined that is dependent on the culvert slope and the ratio W/D. Using Figure 3, the curve (W/D) is chosen and the point for which the value of the slope (S_0) encounters the curve will have as abscissa the value C56. Then if L/D < C56, flow type 5 occurs, otherwise Flow Type 6 is used [4].

Table 2 - Conditions for each type of flow used in TELEMAC

	$\frac{S_1 - z_1}{D}$	$\frac{S_2 - z_2}{D}$	$S_2 - z_2$	L/D
Flow Type 2	<1.5		h_c	
Flow Type 3	<1.5	≤ 1.0	h_c	
Flow Type 4	>1.0	1.0		
Flow Type 5	≥ 1.5	≤ 1.0		C56
Flow Type 6	≥ 1.5	≤ 1.0		≥C56

To use the culverts in our model, additional features had to be incorporated in the code:

- Wooden beams can be placed in front of the culverts to control the timing when water can start flowing in the flood control areas with controlled reduced tide.
- Most culvert structures have trash screens in front and behind them to prevent garbage and drift wood from clogging the hydraulic structure.
- On the outlet culverts there are one-way valves present to prevent the water from entering the FCA's through these culverts.

Most of these structures are incorporated in the code as an extra head loss coefficient, except the wooden beams that act as a small weir. To incorporate these into the code, the geometric features of the culvert presented in Figure 4 are modified and presented in Figure 5. An equivalent culvert bottom elevation was used replacing both the bottom elevations z_1 and z_2 in the formulae described above. The mean between z_1 and z_2 is taken as equivalent bottom elevation of the culvert. The diameter of the culvert used in the equations will be the one corresponding to the entrance of the culvert, *i.e.* like in Figure 5. If the flow goes from left to the right D will be replaced by D_1 and on the opposite direction, the value of D_2 will be used. For the start of the water flow into the FCA the z_1 and z_2 bottom elevations are still used so that the start and end of water flow through the culverts remain as close as possible to reality.

By using this equivalent bottom elevation, the culverts' frictional head losses are overestimated and the local larger head losses due to the presence of the weir are not exactly taken into account. These complicated structures are difficult to model exactly and this assumption keeps things simple. There are many head loss coefficients in the equations and together with the parameters that describe the dimensions of the culverts, the user can tune the modeled discharges.

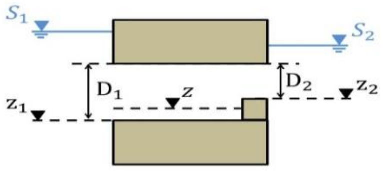

Figure 5 - Representation of the different variables used to calculate the discharges for each type of flow[6]

The head loss coefficient (μ) was adapted from the one computed in TELEMAC-2D, based on [5] and is used as main head loss coefficient. Features of the structure that caused additional head loss, such as one-way valves, trash screens or pillars are added in the computation of (μ), which contributes to the flexibility of the implementation of many types of culvert structures.

The head loss due to singularities can be obtained by the general relation from [7,5]:

$$\Delta H = C\frac{U^2}{2g} \text{or} U = \mu\sqrt{2g\Delta H} \quad (13)$$

with: $\mu = \frac{1}{\sqrt{C}}$ (14)

The coefficient C represents the sum of the different contributions for the head loss due to singularities:

$$C = C_1 + C_p + C_2 + C_3 + C_V + C_{Trash} \quad (15)$$

The different contributions to this head loss coefficient C will be discussed separately and in detail below.

C_1 – the entrance head loss

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 at the culvert entrance that will cause a head loss due to the deceleration of the flow immediately after the vena contracta.

Figure 6 is extracted from [8] and for a culvert between a river and a floodplain the contraction can be seen as very large, so the parameter on the x axis in Figure 6 will be close to zero and are head loss coefficient will be 0.5.

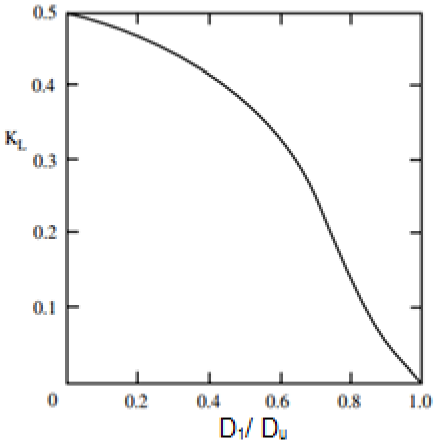

Figure 6 - Local loss coefficient for a sudden contraction as a function of diameter ratio between the diameter after the contraction (D_1) and before the contraction D_u [8].

Bodhaine [4] noticed that the discharge coefficient (C_D) for Flow Type 5 had to be lowered comparatively with the other Flow Types. The calculated discharge seemed to be overestimated when the default equation is used. Therefore a correction coefficient is taken into account. The correction coefficient, C5 is applied to C_1 when Flow Type 5 occurs, such that:

$$\Delta H_1 = C5 * C_1 \frac{U^2}{2g} \tag{16}$$

Bodhaine proposes an interval for the value of this correction coefficient: $4 \leq C5 \leq 10$.

C_p – the head loss due to pillars in the culvert

Sometimes at the entrance of culverts the flow is divided into two sections by a pillar. This pillar causes additional head loss and is taken into account. According to [5] the head loss caused by parallel pillars is given by:

$$\Delta H_p = C_p \frac{U^2}{2g} \tag{17}$$

and $C_p = \beta \left(\frac{L_p}{b}\right)^{4/3} sin\theta$ (18)

with C_p represents the head loss coefficient due to the presence of pillars
L_p thickness of the pillars,
b the distance between two consecutive pillars
β is a coefficient dependent on cross-sectional area of the pillar.

According to [5] β will be 2.42 for rectangular pillars and 1.67 for rounded pillars. θ stands for the angle of the pillar with the horizontal plane. In most cases this will be 90° and $sin\theta$ will be equal to 1. The code does not use this head loss coefficient on its own: its value was added to the C_1 head loss coefficient.

C_2 – the head loss due to internal friction

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

$$\Delta H_2 = C_2 \frac{U^2}{2g} = \frac{2gLn^2}{R^{4/3}} \frac{U^2}{2g} \tag{19}$$

with L length of the structure,
n the Manning Strickler coefficient of the structure (material) and
R the wet cross-sectional area in the structure.

In the code, an assumption was made to calculate the hydraulic radius for each type of flow, since the code does not make any kind of backwater analysis to get the precise water depths that occur in the culvert.

C_3 – the exit head loss

C_3 is the head loss coefficient due to expansion of the flow exiting the culvert. It is given by [7]:

$$\Delta H_3 = \left(1 - \frac{A_s}{A_{s2}}\right)^2 \frac{U^2}{2g} = C_3 \frac{U^2}{2g} \tag{20}$$

with A_s, A_{s2} sections in and just outside at the downstream part of the structure.

Usually C_3 is equal to 1 for a sudden enlargement.

C_V – head loss due to one-way valve

C_V is the head loss coefficient due to the presence of a valve. The head loss due to valves (ΔH_v) is given by:

$$\Delta H_v = C_v \frac{U^2}{2g} \tag{21}$$

with C_V depends on the type of valve and the degree of opening.

For a flap gate valve (rotating around to hinges at its upper edge), some values were obtained experimentally, and they depend on the opening angle of the valve [8]:

Table 3 - Values for the head loss coefficient depending on the opening of a gate valve according to [7].

	Wide open	**¾ open**	**½ open**	**¼ open**
C_V	0.2	1.	5.6	17

Like with C_1 a correction coefficient ($C_{v,5}$) is applied to this head loss coefficient to take into account the increase of the head loss with Flow Type 5. Through a number of laboratory experiments with a physical scale model at Flanders Hydraulics Research [1], it was clear that when Flow Type 5 occurs, there is a greater influence of the head loss coefficient of the valve.

$$\Delta H_{v,5} = C_{v,5} C_v \frac{U^2}{2g} \tag{22}$$

$\boldsymbol{C_{Trash}}$ – head loss due to trash screen

Trash screens are usually present at the inlet of culverts to prevent garbage from entering or blocking the culvert. The head loss due to the presence of these screens (ΔH_t) can be estimated by its relationship with the velocity head through the net flow area. A number of expressions were obtained in the past by several authors. The expression given by [9] is used:

$$\Delta H_t = \left(1.45 - 0.45 A_{trash} - {A_{trash}}^2\right)\frac{U^2}{2g} = C_{trash}\frac{U^2}{2g} \tag{23}$$

with U net flow velocity

$A_{trash} = \frac{A_{net}}{A_{gross}}$ provides the ratio of net flow area to gross rack area.

The value for C_{trash} can vary between C_{trash}= 0 (for A_{trash} = 1, equivalent to not having any trash screens) to approximately C_{trash}= 1.4 (for A_{trash} = 0, for which the net flow area is negligible small compared to the gross rack area).

IV. IMPLEMENTATION IN TELEMAC

The goal of the implementation of this code in TELEMAC was to use the same subroutine for culverts in TELEMAC-2D and TELEMAC-3D.

Culverts were already implemented in TELEMAC-2D in the subroutine `BUSE` (file buse.f). The existing code has four equations, three of which are similar to the equations presented here (Flow Types 3, 5 and 6). In order to ensure backward compatibility while keeping the implementation within the same subroutine, a new keyword, `OPTION FOR CULVERTS` (`OPTBUSE`), was created to let the user choose between both sets of equations. This keyword can be added to the steering file. If the user wants to use the existing code in TELEMAC-2D or 3D, the value is 1. If the user wants to use the equations presented here, the value is 2.

The existing capability of TELEMAC to set source and sink terms anywhere in the domain was useful to implement a culvert function. The inflow and outflow of a culvert then act as a pair of source / sink points. For instance, when the flow is going from the river to the floodplain side, a source term is added on the floodplain side (positive discharge), and at the same time a sink term is set in the river (opposite discharge). By doing this, the culverts are assumed short and the water that leaves the river enters the floodplain at the same time. The computed discharges in `BUSE` (file buse.f) are simply added at the end of the sources matrix as follows:

```
QSCE2(NPTSCE+I)        =-DBUS%R(I)
QSCE2(NPTSCE+NBUSE+I)= DBUS%R(I)
```

with `NPTSCE` is the number of punctual sources and `DBUS(I)` is the discharge computed for culvert number `I`

In TELEMAC-3D the total number of source points, `NSCE`, is now the sum of the punctual sources (`NPTSCE`, that existed before these developments) and twice the number of culverts.

Equations 8 to 12 are implemented in the code based on the conditions given in Table 2. Figure 7 gives a flow chart of the part of the algorithm once the `OPTION FOR CULVERTS` (`OPTBUSE`) is switched on.

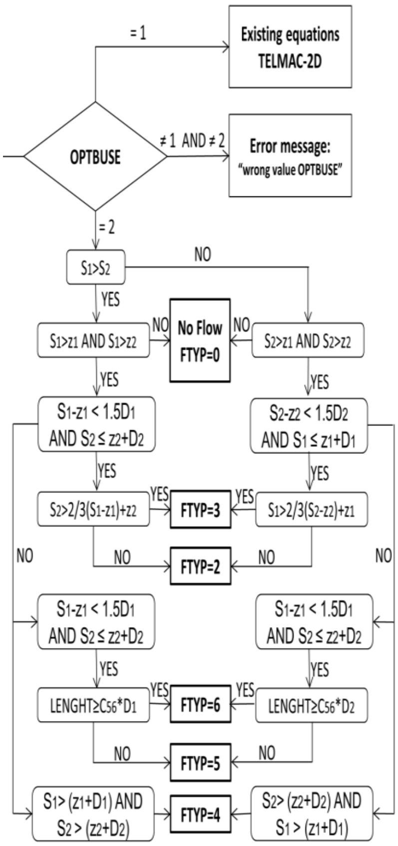

Figure 7 - Flow chart showing the conditions for every type of flow in the BUSE (file buse.f) subroutine.

The user has to specify all the parameters related to the culverts in a text file, through the keyword: CULVERTS DATA FILE. This text file will be read by the subroutine LECBUS (file lecbus.f). The text file and the existing subroutine LECBUS were extended to take extra parameters into account.

The first and third lines of the text file are comment lines and these are not read. On the second line the first variable is the relaxation parameter (RELAXB). This relaxation parameter will give a weight to the discharge calculated at the current time step. This is a value between 0 and 1. The result is a weighted averaged discharge based on the discharge of this and the previous time step. After the relaxation parameter there is a number indicating the number of culverts. The number of culverts needs to be given in the steering file through the keyword NUMBER OF CULVERTS (NBUSE) and this number will be checked with the number in the text file as an extra control parameter. The third line is commented and contains the names of all the parameters used in BUSE. They are separated by a tab. The flow through a culvert can go in both directions.

In the following the index 1 is used for the river side and the index 2 for the floodplain side of the culvert. The following parameters must be listed in the culverts data file:

I1 node number of culvert on side 1 (river)
I2 node number of the culvert on side 2 (floodplain)
CE1 entrance head loss coefficient for the culvert on side 1 (this corresponds to the head loss coefficient C_1)
CE2 entrance head loss coefficient for the culvert on side 2 (this corresponds to the head loss coefficient C_1)
CS1 exit head loss coefficient for the culvert on side 1 (this corresponds to the head loss coefficient C_3)
CS2 exit head loss coefficient for the culvert on side 2 (this corresponds to the head loss coefficient C_3)
LARG the width of the culvert
HAUT1 height of the culvert on side 1
CLP coefficient to restrict the flow direction (0 both directions are possible; 1 = only flow from side 1 to 2; 2 = only flow from side 2 to 1; 3 = no flow)
L linear head loss coefficient used only when OPTBUSE=1; If OPTBUSE=2, **L** is calculated
RD1 culvert bottom elevation on side 1(z_1)
RD2 culvert bottom elevation on side 2(z_2)
CV head loss coefficient when a valve is present
C56 factor to differentiate between flow types 5 and 6
CV5 correction factor for C_V when flow type 5 is used
C5 correction factor for CE_1 and CE_2 with Flow Type 5
TRASH head loss coefficient when trash screen are present
HAUT2 height of the culvert on side 2
FRIC Manning strickler coefficient used in equation 19
LONG length of the culvert (not necessarily based on the position of the source / sink nodes)
CIR parameter to determine if the culvert is rectangular (=0) or circular (=1); in case of a circular culvert the height is taken to calculate the wet section.

The computed discharge is positive for the flow from side 1 to side 2. When the discharge is computed by BUSE based on the above parameters, it undergoes three possible changes.

- First, the relaxation is computed. Based on the weight and the difference with the discharge in the previous time step, this can change the computed discharge of the current time step.
- Then, a test is performed to check if there is enough water present at the current time step to extract the anticipated discharge. A maximum of 90% of the available water is allowed to leave.
- Finally, the code checks that the culvert configuration allows the water to flow in the computed direction, and if not blocks the flow by setting the discharge to zero.

In the culvert data file, the user can choose the direction of the flow through a culvert by setting the parameter CLP (for example if there is a one-way valve in the culvert).

It is possible to use passive or active tracers when using culverts. The following equation describing the evolution of tracer concentration (T) is solved:

$$\frac{\partial T}{\partial t} + U\frac{\partial T}{\partial x} + V\frac{\partial T}{\partial y} + W\frac{\partial T}{\partial z} = \nu_t \Delta(T) + Q' \qquad (24)$$

The tracer diffusion coefficient is given by ν_t and Q' represents the source terms for tracers. In order to take the transport of tracers into account when using culverts, the user only has to specify the keywords related to the tracers in the steering file.

The subroutines BUSE and LECBUS are both called from TELEMAC-2D and TELEMAC-3D in the same way.

V. Test-case Bergenmeersen

Bergenmeersen is the name of a flood control area with controlled reduced tide. An aerial photograph is shown in. It is located far upstream in the Scheldt estuary. Figure 8 shows the location of Bergenmeersen in the Scheldt estuary, represented as a mesh of the Scaldis model [2,10].

Figure 9 shows Bergenmeersen in more detail. It shows the mesh of the small model that was used to test the culvert code presented in this paper. The mesh size is about 7 m. Bergenmeersen is also available as a TELEMAC-3D validation case, but with a coarser mesh.

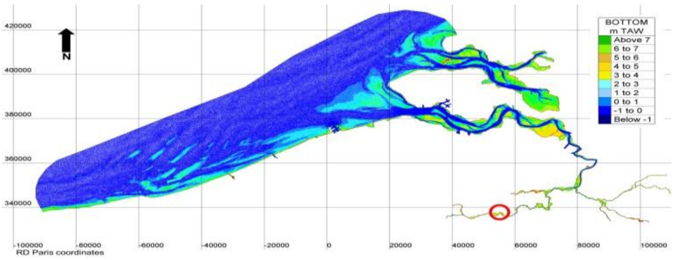

Figure 8 - red circle indicating the location of Bergenmeersen in the Scheldt estuary. The map shown represents the grid of the Scaldis model [2]

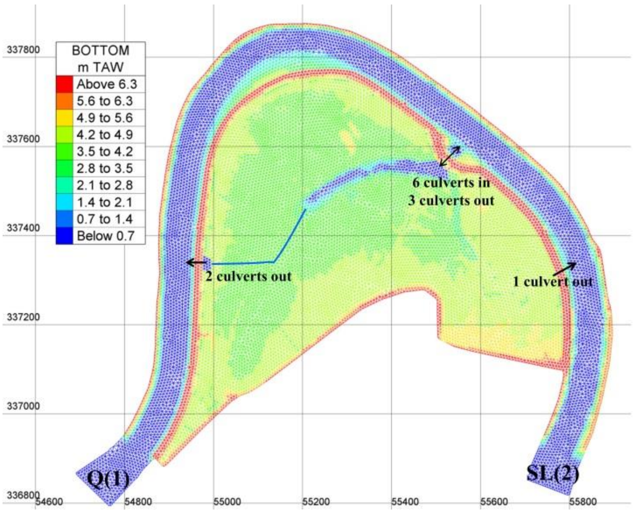

Figure 9 - Detailed view of the mesh and bathymetry of the small model of Bergenmeersen.

Bergenmeersen was first a flood control area only. In the summer of 2013 the inlet culverts became operational making it a flood control area with controlled reduced tide. Twice a day the tide can enter the area, creating new tidal nature. There are 6 inlet culverts, which are built above three outlet culverts.

Figure 10 shows an aerial view on this construction. On two other sides of the area are three old outlet culverts (locations given in Figure 9). The dike between the Scheldt river and the floodplain is an overflow dike. When high storm surges enter the estuary, at a critical level the water can flow over this dike. The level of this overflow dike at Bergenmeersen is 6.4 m TAW (i.e. 0 m TAW is about low water at sea) [11]. The other dikes are 8.0 m TAW. The floodplain is then buffering the excess surge water and prevents floods upstream and downstream from this area.

Every year around September, a 13-hour measurement campaign is executed (13 hours to capture one tidal cycle), measuring the inlet and outlet discharges. Water levels are constantly measured using a diver on the floodplain side and another one at the Scheldt side of the large inlet-outlet structure. Moreover on December 6th 2013 there was a storm surge entering the estuary and water levels inside and outside Bergenmeersen were measured with these divers and a helicopter picture shows that the flood plain was almost completely filled (Figure 10). All these data are available to calibrate the parameters used for the culverts.

Figure 10 - Bergenmeersen at December 6th 2013 filled with water from the storm surge

Figure 11 shows an image of the cross section seen from the side of the Bergenmeersen inlet and outlet structure. In this view the Scheldt river is on the left and the floodplain is on the right. The inlet culverts are built on top of the outlet culverts. The outlet culverts have a one-way valve (nr 1 in Figure 11) to prevent water from entering the floodplain too soon. There are trash screens (nr 2 in Figure 11) on both sides of the construction. They keep large debris and driftwood from clogging the culverts. Just in front of the inlet culvert there is a possibility to place wooden beams (nr 3 in Figure 11) that act as a weir. These are used to fine tune the amount of water entering daily in the flood plain. The height of the weir determines the moment at which water can start flowing in. Inside the inlet culvert there is also a sliding valve (nr 4 in Figure 11) that can close the inlet culvert. These valves were meant to close the inlet culverts in case of a storm surge to keep the full buffer capacity of the floodplain, but they are currently also used to control the amount of water entering the area. Their positions are known for the 13 hour measurement of September 11, 2014. This data set is used to calibrate the culvert parameters.

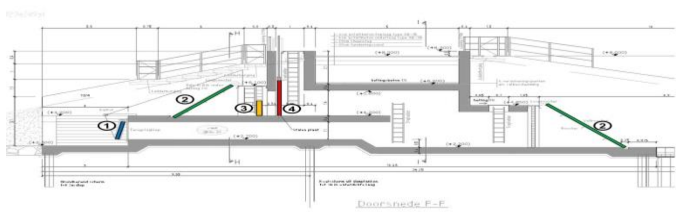

Figure 11 - side section showing the internal construction of the inlet and outlet structure at Bergenmeersen. Number 1 is the one-way valve of the outlet culvert; 2 are the trash screens at both sides of the structure; 3 is the log weir; 4 is a sliding valve that can close the inlet culvert.

Table 4 provides an overview of the parameter values that were used. There are three sets of parameters values given: the first is a start set where a first guess is made for all parameters while remaining as close as possible to the physical values. The second set is the parameter set following calibration against the 13 hour measurement on Bergenmeersen on September 11, 2014. Comparing the first two data sets provides an overview of the changes made to get a better result. Finally set 3 gives the parameters that were used for the simulation of the December 6, 2013 storm surge.

For each set of parameters some variables are not shown in Table 4 because they were not changed or were not dependent on the construction of the structure. Through physical experiments the value of $C_{v,5}$, the correction factor for the head loss due to the presence of a valve when flow type 5 occurs, was set to be 1.5. This value is set to 1.5 only for the outlet culverts who have the one-way valve. The inlet culverts don't have this valve so for them the value of $C_{v,5}$ is set to zero. C_5 was given a value of 6, *i.e.* a value in the interval proposed by [4]. According to Figure 3 the culverts have no bed slope and no wing walls (W=0), so a value of 10 is found for the C_{56} parameter. The parameter `L` is computed with `OPTION FOR CULVERTS`=2 (The user value of `L` will be ignored in this case). These parameters are not listed in Table 4.

For the start set of the culvert parameters the following values were chosen: for CE_1 and CE_2 the value of 0.5 was selected according to Figure 6. For the 6 inlet culverts for CE_1 the head loss coefficient due to the presence of a pillar was also added. According to equation 18 (with $\theta = 90°$, $\beta = 2.42$, L_p=0.4 and b=1.35, C_p=0.47 which is rounded to 0.5) 0.5 was chosen for CE_1 which gives us 1. There are 6 inlet culverts and 6 outlet culverts, but because the outlet culvert parameters are the same for all 6 culverts only one set of parameters is shown. There are also three old outlet culverts present. Their exact measurements are not known, but the parameters were taken the same for all three of them and are only shown for one. For a sudden enlargement C_S is taken equal to 1. The inlet culverts have a width of 3 m, but because of the presence of a pillar in each culvert, their width is taken equal to 2.7 m (LARG = 2.7 m). For the outlet culverts, LARG = 1.35 m. For the older outlet culverts LARG was taken equal to 1.5 m. The bottom level of the inlet culverts was 4.2 m TAW according to the as built plans. So RD1 and RD2 were set to 4.2, except three inlet culverts had wooden beams in front of them (their height was 0.25 m, 0.25 m and 0.1 m). So RD1 was raised for these three culverts to 4.45, 4.45 and 4.3 respectively. The bottom level of the outlet culverts was 2.7 m TAW and for the old outlet culverts RD1 and RD2 were taken equal to 2.5 m TAW. All inlet culverts had different valve positions. This was taken into account by modifying the height of the river side (side 1). The inlet culverts have a normal height of 1.6 m. HAUT1 and HAUT2 should be equal to this 1.6 m for all inlet culverts, but HAUT1 was adapted for each inlet culvert individually taking the valve position and the wooden beams into account. It represents the amount of opening on side 1 of the structure. The outlet culverts have a height of 1.1 m. The old outlet structures were given a height of 1.5 m. The direction of flow was set: CLP was set to 0 for the inlet culverts, meaning water can flow in both directions through these culverts. For the outlet culverts CLP was set to 2, meaning only outflow due to the presence of the one-way valve.

Table 4 - Parameter values for culvert calibration

		CE1	CE2	CS1	CS2	LARG	HAUT1	CLP	RD1	RD2	CV	TRASH	HAUT2	FRIC	LONG
2014 start	inlet	1	0,5	1	1	2,7	0,35	0	4,45	4,2	0	1,5	1,6	0,015	10
	inlet	1	0,5	1	1	2,7	0,35	0	4,45	4,2	0	1,5	1,6	0,015	10
	inlet	1	0,5	1	1	2,7	0,35	0	4,3	4,2	0	1,5	1,6	0,015	10
	inlet	1	0,5	1	1	2,7	0.45	0	4,2	4,2	0	1,5	1,6	0,015	10
	inlet	1	0,5	1	1	2,7	0.25	0	4,2	4.2	0	1,5	1.6	0,015	10
	inlet	1	0,5	1	1	2,7	0,35	0	4,2	4,2	0	1,5	1,6	0,015	10
	outlet	0,5	0,5	1	1	1,35	1,1	2	2,7	2,7	1	1,5	1,1	0,015	9
	outlet old	0,5	0,5	1	1	1,5	1,5	2	2,5	2,5	1	1	1,5	0,015	30
2014 calibrated	inlet	1	0,5	1	1	2,7	0,35	0	4,45	4,45	0	1	1,3	0,015	1
	inlet	1	0,5	1	1	2,7	0,35	0	4,45	4,45	0	1	1,3	0,015	1
	inlet	1	0,5	1	1	2,7	0,35	0	4,3	4,3	0	1	1,45	0,015	1
	inlet	1	0,5	1	1	2.7	0.45	0	4.15	4,15	0	1	1.6	0,015	1
	inlet	1	0,5	1	1	2.7	0.25	0	4,15	4,15	0	1	1,6	0,015	1
	inlet	1	0,5	1	1	2,7	0,35	0	4,15	4,15	0	1	1,6	0,015	1
	outlet	0,5	0,5	1	1	1,35	1,1	2	2,65	2,65	1,5	1	1,1	0,015	9
	oulet old	0,5	0,5	1	1	1,5	1,5	2	2,5	2,5	1,5	1	1,5	0,015	30
2013 storm	inlet	1	0,5	1	1	2,7	1,3	0	4,45	4,45	0	1	1,3	0,015	1
	inlet	1	0,5	1	1	2,7	1,3	0	4,45	4,45	0	1	1,3	0,015	1
	inlet	1	0,5	1	1	2,7	1.45	0	4,3	4.3	0	1	1,45	0,015	1
	inlet	1	0,5	1	1	2,7	1,6	0	4,15	4,15	0	1	1,6	0,015	1
	inlet	1	0,5	1	1	2,7	1,6	0	4,15	4,15	0	1	1,6	0,015	1
	inlet	1	0,5	1	1	2,7	1,6	0	4,15	4,15	0	1	1,6	0,015	1
	outlet	0,5	0,5	1	1	1,35	1,1	2	2,65	2,65	1,5	1	1,1	0,015	9
	oulet old	0,5	0,5	1	1	1,5	1,5	2	2,5	2,5	1,5	1	1,5	0,015	30
	leak	0,5	0,5	1	1	0,5	0,5	1	2,5	2,5	17	1	0,5	0,015	30

For the one-way valve, it was important to know how far this valve would open with outflowing water to determine its head loss coefficient. According to [7] in Table 3 a head loss coefficient can be found for specific opening angles of the valve. In the framework of one of projects [12], the opening angle of the one-way valve of Bergenmeersen was measured for a 13 hour period (September 2015). In Figure 12 shows the results of this measurement and at full outflow the angle measured reached 70°. So from Table 3 the head loss value is taken according to a ¾ opening of the valve, $C_V = 1$.

For the trash screen head loss equation 23 was used with a net flow area of 77% a head loss coefficient of 0.5 was calculated. This value was tripled because of the presence of debris and driftwood in front of the screens. For the friction coefficient a value was taken for smooth concrete. 0.015 was taken for the Manning Strickler value. For the length of the inlet culverts, 10 m was taken and for the outlet culverts 9 m was taken. For the old outlet culverts 30 m was taken.

With this first set of parameters for the inlet and outlet culverts, a model run can be started. A time step of 2 seconds was used. Private arrays (made available already within TELEMAC) were programmed to give the discharges of the culverts as output. One private array was programmed to give information about the type of flow that was used each time step for all culverts. The steering file used here is the same as the steering file for the Bergenmeersen validation case in the TELEMAC-3D basis.

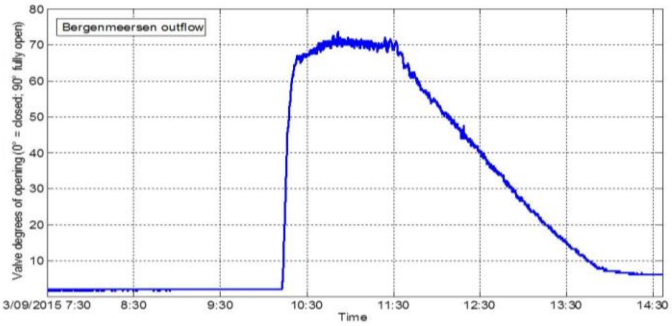

Figure 12 - Measurement of the degree of opening of the one-way valve at the outlet culverts at Bergenmeersen [12].

The red line in Figure 12 shows the water levels inside the floodplain after the first run without any calibration. The black dots indicate the measured water level in the floodplain. The squares give the water level that was measured outside the floodplain near the outlet structure. Calibration can be done in three ways: 1) the total head loss can be increased or decreased. All parameters contributing to the total head loss are always summed in the calculations. So there is little difference between increasing the head loss coefficient of the valve or the trash screen. The only difference is that not all head loss coefficients are always used. The outlet culverts use the head loss due to the one-way valve for example and the inlet culverts do not. 2) A second way to alter the discharges is to adapt the physical parameters like RD1 and RD2 or HAUT1 and HAUT2. 3) A third way is to alter the boundary conditions, like the floodplain. Because small bathymetric elements are not always very well represented in the mesh, this might cause differences in water levels inside the floodplain. To get better results, the mesh was refined only in the floodplain area, with a discretisation of 0.1 m, and a ditch that was not well represented in the model was created by changing the bottom level of some points by hand. This ditch made a connection between the two old outlet culverts on the left side (Figure 9) and the main channel in the floodplain. Creating this ditch caused a better drainage of the floodplain.

To get the best result, the head loss of the outlet culverts (blue line in Figure 13) was increased by 0.5 by increasing C_V. The bottom level of the whole construction was lowered with 0.05 m resulting in different RD values, the RD2 values of the inlet culverts were brought to the level of the wooden beams like their RD1 values (also decreasing their HAUT2 values accordingly), and the length of the inlet culverts was reduced to 1 m because of the small openings at side 1 due to the valve positions. The latter made the code switch in some situations from flow type 6 to flow type 5 resulting in different discharge values. When the model automatically modifies the discharges towards the river, the discharges towards the floodplain will change also because of a different amount of water present in the floodplain.

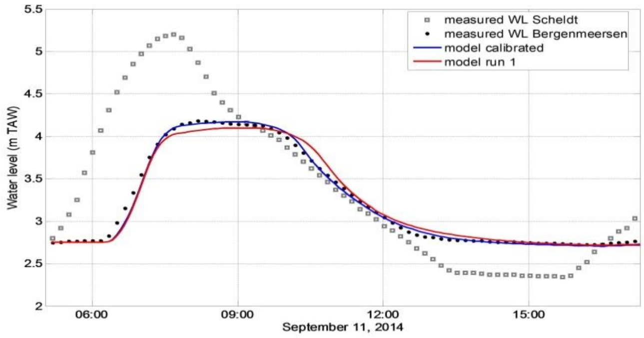

Figure 13 - measured and calibrated water levels for Bergenmeersen at September 11, 2014.

Figure 14 shows the measured (dots) and modelled (blue line) discharge going in the floodplain through the inlet culverts. The results show a very good agreement between the simulation and the measurements. Only between 7:45 and 8:15 the modelled discharges are too high.

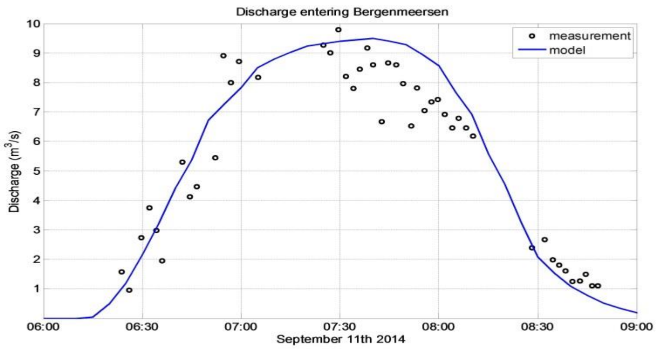

Figure 14 - Measured and modelled total discharge entering Bergenmeersen floodplain at September 11, 2014. The model results are the ones after calibration.

Figure 15 shows the measured (dots) and the modelled (blue line) discharge leaving the floodplain at the six new outlet culverts. The shape of the modeled curve fits well with the measured one, but a small time lag can be seen. The modelled curve was significantly improved by the 0.1 m increase in bottom level of the floodplain.

The second set of parameters in Table 4 is the one after the calibration effort resulting in the better results in Figure 13 (blue line), Figure 14 and Figure 15. To test if this parameter set could also deliver good results with other boundary conditions, the water levels of a storm surge of December 6, 2013 were taken as boundary conditions. For this data set, the only available measurements are the water levels, no measurements of the discharge having been done. All other settings and parameters remained unchanged. The red line in Figure 16 shows the simulation results, while the black dots show the measured values. The model fails to reproduce the water levels in the floodplain. For the inlet culverts the wooden beams in front of them were there from the beginning, but the position of the valve was not known. The valve at that time were assumed fully opened, so this was changed in the parameter set for the inlet culverts. It was also noticed by the people who performed the measurements that the water level in the floodplain was already rising slightly before the water level outside had reached the bottom level of the inlet culverts. So, a leakage was also assumed through the old outlet culverts. So one extra culvert was added with only flow in to represent the leakage flow. This resulted in the third parameter set given in Table 4 and gave a very good result for the modelled water levels inside the floodplain as given by the blue line in Figure 16.

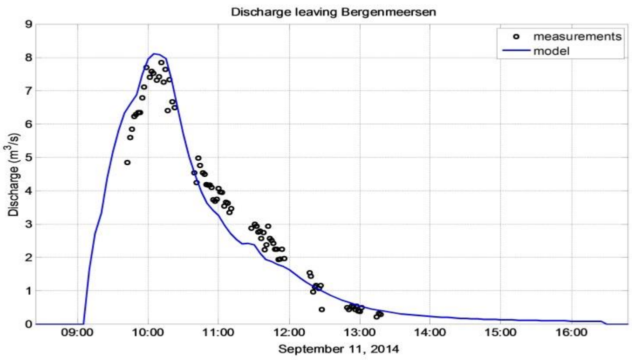

Figure 15 - Measured and modelled discharge leaving the Bergenmeersen floodplain through the new outlet structure (6 outlet culverts) at September 11, 2014. The model results are the ones after calibration.

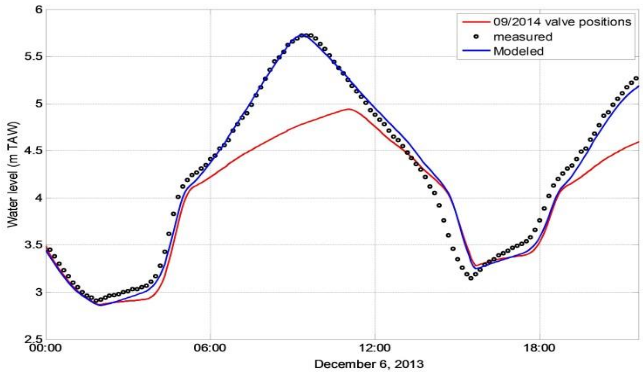

Figure 16 - Measured and modelled water levels for the storm of December 6, 2013. The Red line indicates the modelled water level when using the September 2014 valve positions in the culvert file instead of all open valves in December 2013.

The results are not perfect yet, but are coming very close. Because it is unknown if all valves were really fully open, in the absence of detailed information about the three old culverts and about the possible leakage and where it's coming from, further improvement of the results will be difficult.

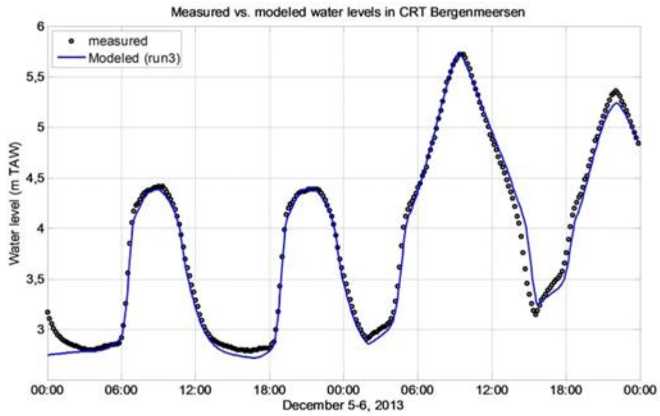

Figure 17 - Modelled and measured water levels for the period of December 5-6, 2013.

Figure 17 shows a longer time series where modelled and measured water levels are compared for the period of December 5-6, 2013. These results show the capability of the code to model the flow through these structures very well, even with trash screen, valves and wooden beams: the results are very good.

Finally, Figure 18 shows again the water levels inside and outside the floodplain on December 6, 2013, but on the bottom part of the Figure the types of flow that occur through the inlet and outlet culverts were added. This figure clearly shows that most types of flow occur during the simulation (remember that Flow Type 1 through a steep culvert was not added in the code). Negative Flow Types indicate flow going from the floodplain to the river Scheldt and vice versa for positive flow types. The figure also shows positive Flow Types for outlet culverts, but the CLP parameter, making that outlet culverts are only allowed to have flow from the floodplain to the river and not vice versa, makes sure the calculated discharge is set to zero in this case. Because of the exceptionally high water level with this storm surge we can also see negative flow types for the inlet culverts. This means that water is also leaving the floodplain through the inlet culverts. This was designed in this way to foresee extra capacity to drain the floodplain in case room is needed for a subsequent storm surge.

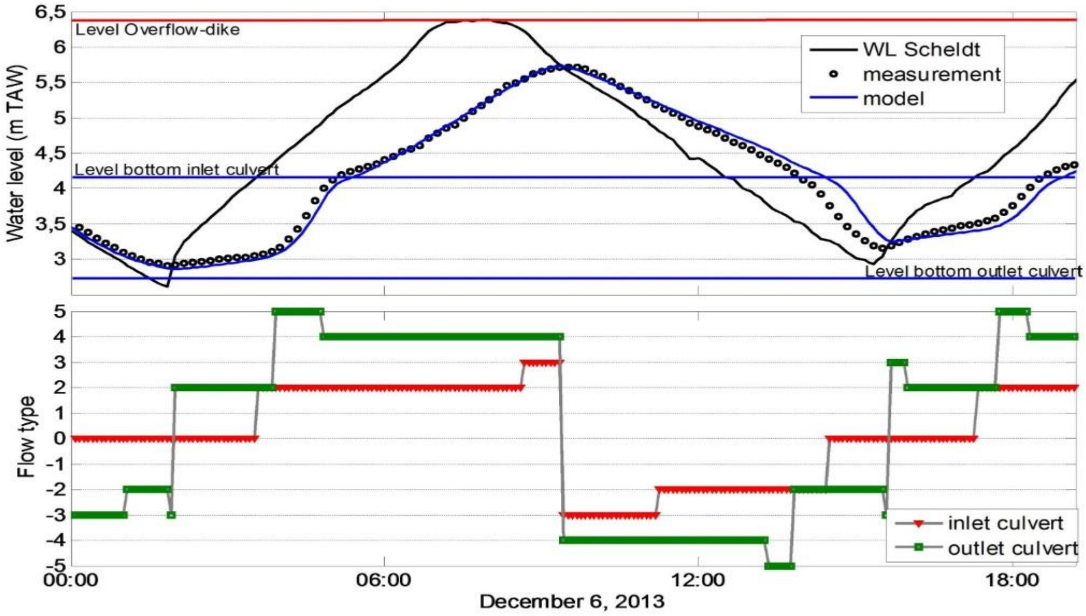

Figure 18 - modelled and measured water levels for the storm of December 6, 2013 and the type of flows that occurred through in- and outlet culverts

VI. CONCLUSIONS

This work aimed at adding a culvert functionality in TELEMAC-3D. The framework available in TELEMAC-2D was used so that the two modules are consistent. A new formulation for culverts was also added in this framework, and successfully validated against field measurements on the Bergenmeersen case. The latter is described in details in the paper.

Due to the complexity of the culvert modelling, there is a high number of parameters to provide in the formulation. The process of fitting of these parameters (head losses in the culverts, etc.) on the Bergenmeersen test-case was described. All the developments, documentation and validation cases will be available in release V7P2 of the open TELEMAC system.

REFERENCES

[1] T. Cox, T. Maris, P. De Vleeschauwer, T. De Mulder, K. Soetaert, P. Meire, "Flood control areas as an opportunity to restore estuarine habitat." Ecological Engineering, Vol. 28, Issue 1, 2006, pp. 55-63.

[2] Smolders, S.; Maximova, T.; Vanlede, J.; Plancke, Y.; Verwaest, T.; Mostaert, F. (2016). Integraal Plan Bovenzeeschelde: Subreport 1 – SCALDIS: a 3D Hydrodynamic Model for the Scheldt Estuary. Version 5.0. WL Rapporten, 13_131. Flanders Hydraulics Research: Antwerp, Belgium.

[3] W. Baeyens, B. van Eck, C. Lambert, R. Wollast, L. Goeyens, "General description of the Scheldt estuary." *Hydrobiologia* 34,1998,83-107.

[4] G.L. Bodhaine, " Measurement of peak discharge at culverts by indirect methods", U.S. Geological Survey, Techniques of Water-Resources Investigations, 1968, book 3, chapter A3 60p.

[5] M. Carlier, "Hydraulique générale et appliquéé", 1972, Paris, Eyrolles

[6] Teles M. J., Smolders S., Maximova T., Rocabado I., Vanlede J., 2015. Numerical modelling of flood control areas with controlled reduced tide. Scheldt estuary physics and integrated management : proceedings of the 36th IAHR World Congress, 28 June - 3 July 2015, Delft & The Hague, the Netherlands- p. 90-99

[7] A. Lencastre, "Manuel d'hydraulique générale", 1961, Paris, Eyrolles

[8] E. Bruce Larock, W. Roland Jeppson, Z. Gary Watters, "Hydraulics of Pipeline Systems", 2000, CRC Press

[9] Wahl T.L., 1992. Trash control structures and equipment: a literature review and survey of bureau of reclamation experience. United States Department of the Interior, Bureau of Reclamation, Hydraulics Branch, Research and Laboratory Services Division, Denver, Colorado, 1992.

[10] Smolders S., Teles M.J., Maximova T., Vanlede J., 2014. Implementation of controlled reduced tide and flooding areas in the TELEMAC-3D model of the Scheldt Estuary. Proceedings of the 21st TELEMAC-MASCARET User Conference, 15th-17th October 2014. Grenoble, France, pp. 111-118. ISBN: 978-2-9548911-0-1

[11] De Beukelaer-Dossche, M.; Decleyre, D. (Ed.) (2013). Bergenmeersen: construction of a Flood Control Area with Controlled Reduced Tide as part of the Sigma Plan. Waterways and Sea Canal (W&SC)/Agency for Nature and Forest (ANB): Antwerp/Brussels. ISBN 9789040303432. 136 pp.

[12] Vercruysse, Jeroen; Visser, Klaas Pieter; Peeters, Patrik; Mostaert, Frank (2016). Vismigratie GOG/GGG: Opmeting van helling terugslagklep en debiet te Bergenmeersen. Versie 4.0. WL Rapporten, 15_034. Waterbouwkundig Laboratorium: Antwerpen, België.