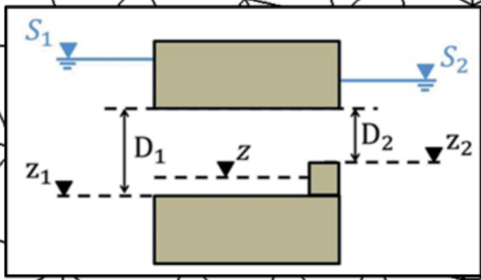
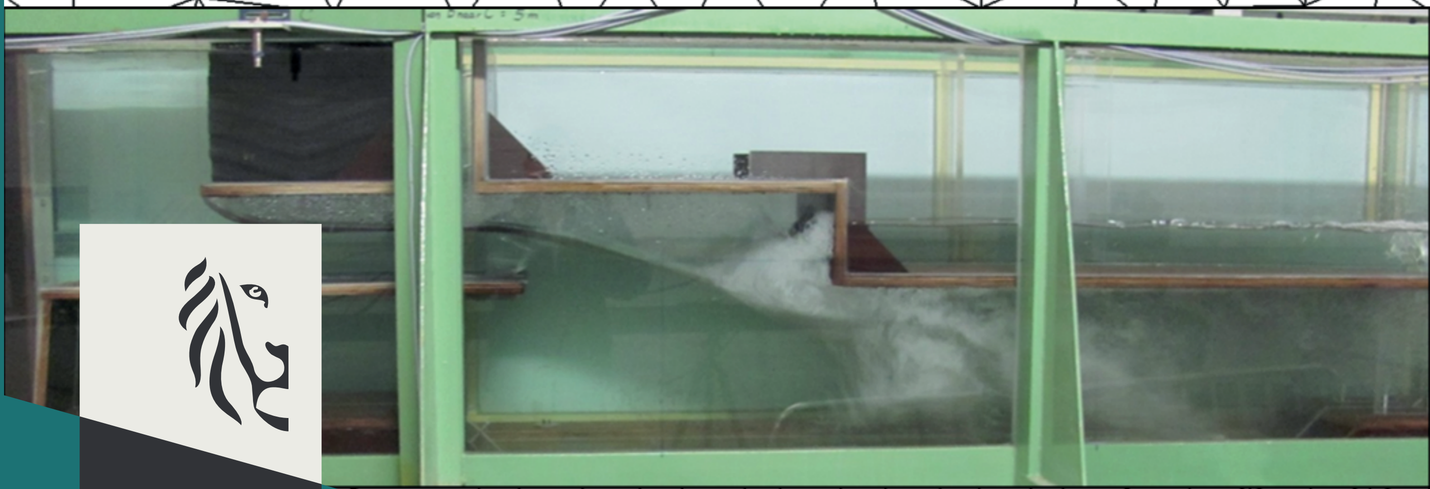
TELEMAC



CULVERTS



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Modelling culverts in TELEMAC

Code validation with scale flume model of Doelpolder

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Abstract

A scale model of part of the inlet culvert construction of Doelpolder was available after project 15_073 (Scale model test on combined in- and outflow construction of Doelpolder). Additional flume tests were done with this scale model in four different setups: without weir logs and without a trash screen, with weir logs and without a trash screen, without weir logs and with a trash screen, and with weir logs and with a trash screen. For eight different sets of fixed water levels before and after the culverts the discharge for each setup was measured. These measured discharges are then compared with calculated discharge, based on the same water levels and construction parameters with a set of discharge through a culvert formulations built in the code of TELEMAC. The calculated discharge are in good agreement with the measured ones. This shows that the culvert formulas present in TELEMAC are capable of reproducing the discharges through the complex culvert structures of the Flemish flood control areas.

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1 Introduction

In the framework of the project 13_131, Navigability of the Upper Sea Scheldt, a new 3D unstructured model of the entire Scheldt estuary was made in TELEMAC-3D (Smolders et al.,2016). Because more and more flood control areas of the Sigmaplan are completed and activated in the estuary, they were all added to this model. For flood control areas with controlled reduced tide water can enter and leave the polder with every tide through a construction in the dike. These kind of constructions behave like a culvert. TELEMAC-3D was not able to simulate culvert flows and so the formulations for 5 types of culvert flows were added to the code (Smolders et al., 2016). The validation of this code was done by using 13 hour measurements of discharges and water levels in and out of the flood control area with controlled reduced tide Bergenmeersen.

In the framework of the project 15_073, Controlled reduced tide at Doelpolder – Physical scale model test on the combined in- and outflow construction, a physical scale model (scale 1/15) was built to test the in- and outflow construction of the controlled reduced tide area Doelpolder (Vercruysse et al., 2016).

Because the physical scale model of the in- and outflow construction of Doelpolder was already built, the additional cost of doing some extra discharge measurements was marginal. These measurements are used as a second validation case for the culvert formulation that was made for TELEMAC-2D and TELEMAC-3D.

This report will describe in detail the physical scale model. The description of which is taken from Vercruysse et al. (2016), but is added here for completeness. The report will further describe the measurement setup and will show the results. The comparison of the results with the discharges calculated by the culvert code present in TELEMAC-2D or TELEMAC-3D will be shown. Good agreement between model results and measurements will further validate the culvert code in the TELEMAC modelling software and will ensure that this code can be used in larger models of the Scheldt estuary to model the water exchange between the river and these flood control areas.

2 Material and methods

2.1 Flood control area (FCA) with controlled reduced tide (CRT)

Flood control areas (FCA'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 and can store storm water. The dike directly between the river and the FCA is lowered so water can flow over the dike into the FCA when it reaches a critical level (Figure 1 nr.1). When the water level drops after a storm tide, outlet culverts evacuate the water out of the FCA back into the river (Figure 1 nr.2). A one way valve prevents water from the river to enter the FCA through these outlet culverts. To restore tidal nature along the estuary, some FCA's also got inlet culverts, which let a reduced amount of water in the FCA every tide (Figure 1 nr.3). This function is called controlled reduced tide or CRT. 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 inlet culverts. So each tide water enters and leaves these areas as a reduced tide compared to the tide in the river/estuary (Figure 1 nr.4). With a storm surge the CRT area can act in the same way as the FCA if a lowered overflow dike is present (Figure 1 nr.5). By recreating a reduced tide in these areas tidal flats and marshes can develop, giving these areas besides a safety function a nature function.

2.2 Culvert geometry Doelpolder

Doelpolder is an area next to the nuclear power plant of Doel and because of safety precautions it cannot serve as an FCA. In the area itself tidal nature will be created by giving this area a CRT function. When this report is written, the structure of this CRT has not yet been built and the geometry is copied from plans made by a study bureau. Figure 2 gives a cross sectional and plan view of the in- and outflow construction of CRT Doelpolder.

Water can flow from the river to the polder through 11 inflow culverts. the bottom elevation of these inflow culverts lies at 3,30 m TAW (= Belgian reference level; 0 m TAW is the average water level at low water at the Belgian coast). Every culvert is 3 m wide but is split in two by a wall in the middle, which has a thickness of 0,40 m. This middle wall is necessary to handle the width of the wooden beams that act as weir logs and the sliding valves. The weir logs are used to fine tune the level at which water is allowed to enter the polder. The sliding valves are used in case of extreme storm surges to close the polder. In front of the culverts trash screens are present to prevent floating debris from clogging the construction. The wall separating a culvert in two ends at 1,50 m from the end of the culvert.

At the end of the inflow culvert the water falls down 4,70 m in a local stilling basin. This basin is slightly lower than the bottom floor of the construction. The floor level of the stilling basin lies at -1,40 m TAW. This local lowered basin ends at 21,60 m after the end of the inflow culvert. At this location the bottom level rises with 1 m over a length of 1 m. A new floor level of – 0,40 m TAW is reached. Here (at 23,30 m from the end of the inflow culvert) the culvert is split in two again by a wall (with a thickness of 0,40 m). Vertical trash screen are present. The ceiling and side walls end at 25,50 m from the end of the inflow culvert and the concrete floor ends at 32,10 m. Bottom reinforcement or protection is foreseen until 52,10 m after the end of the inflow culvert.

Figure 1. – Schematic presentation of the functioning of a FCA 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.

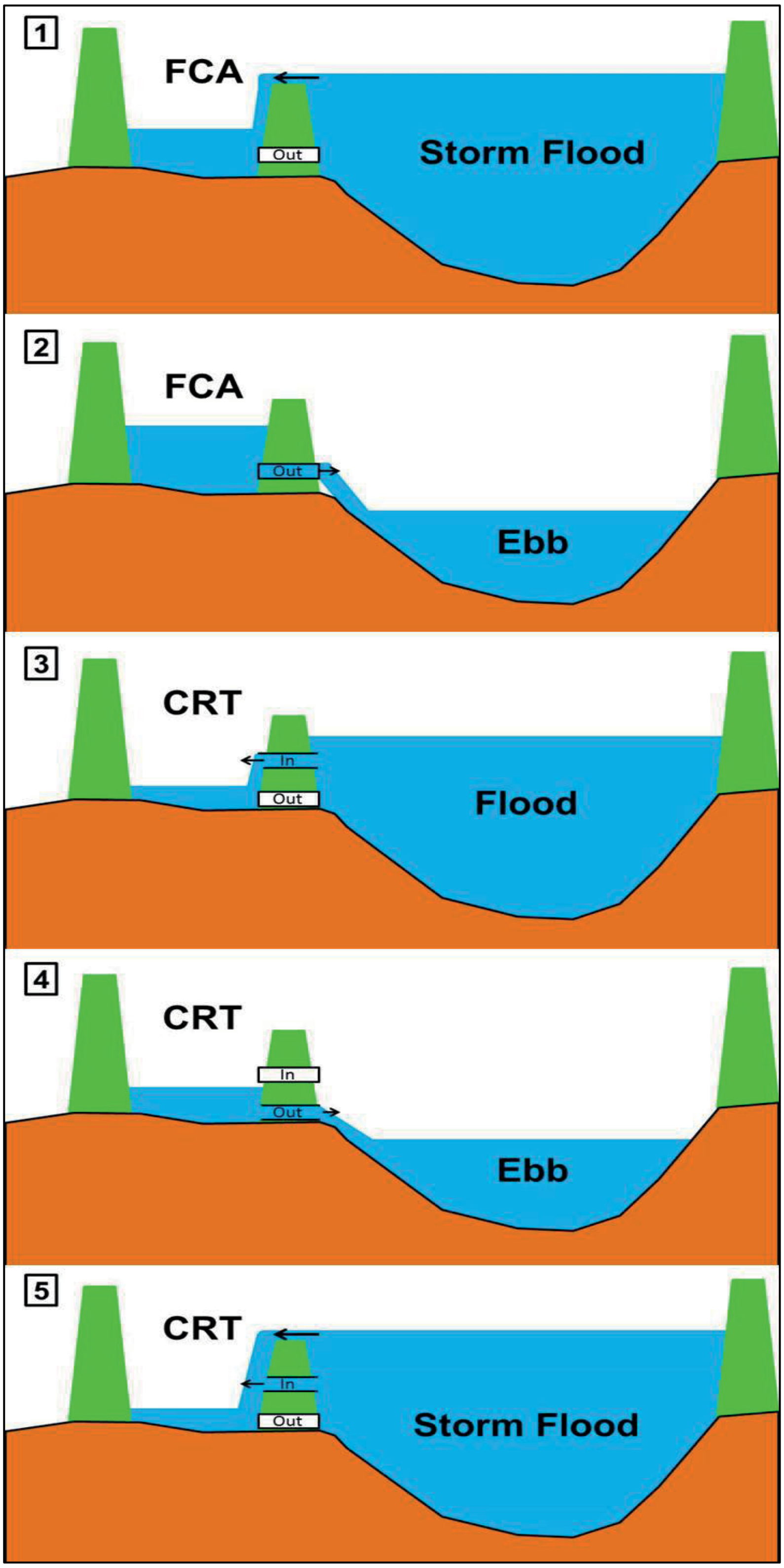
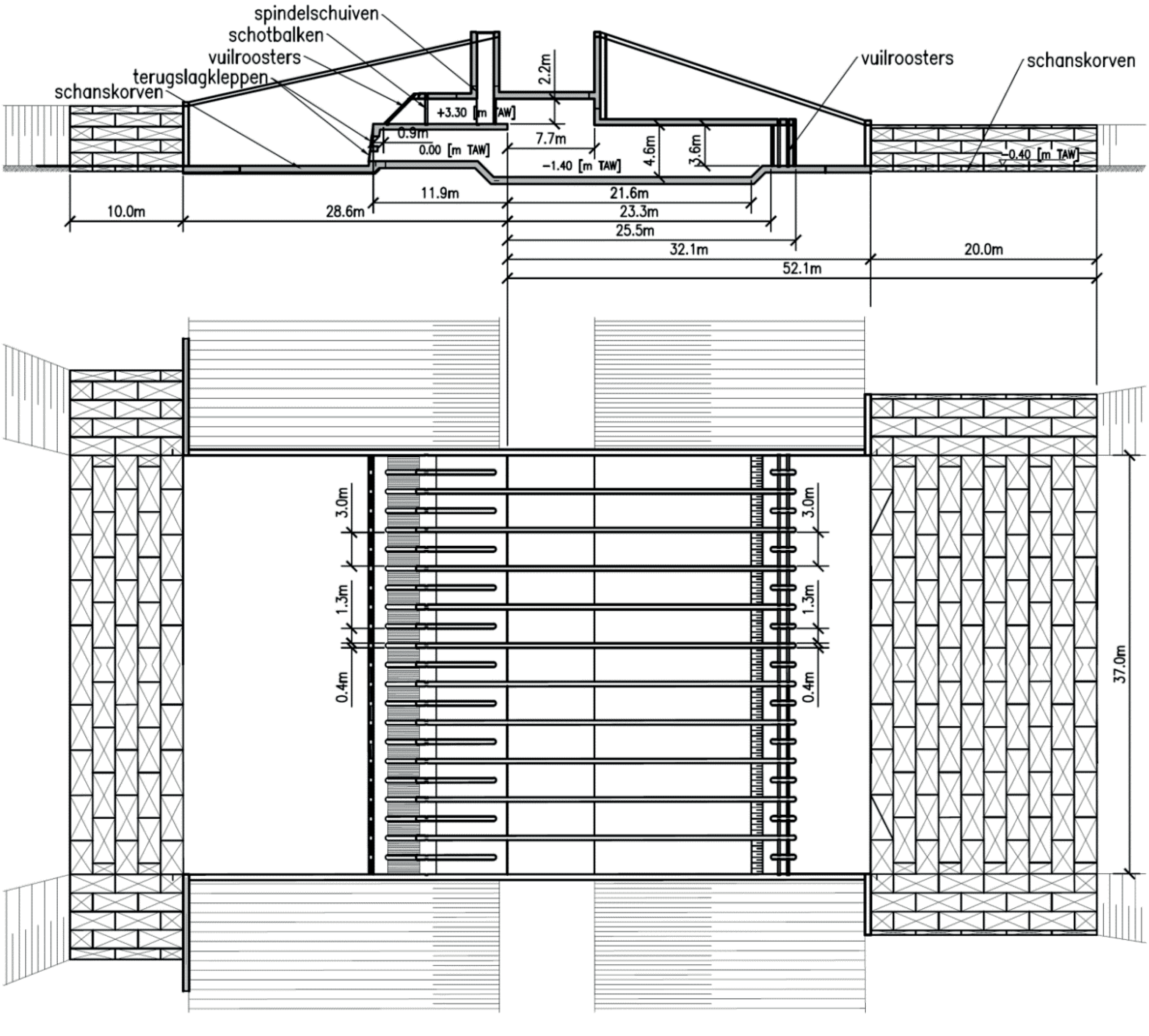


Figure 2 – Cross section and plan view of culverts of Doelpolder.
Notations in figure are in Dutch and can be translated as follows. schanskorven: stone revetments; terugslagkleppen: one-way valves; vuilrooster: trash screen; schotbalken: wooden beam (used as weir); spindelschuiven: sliding valve.
(figure from Arcadis (2014)).



2.3 Scale model geometry

A scale model was built in a flume with a horizontal bottom at Flander s Hydraulics. This scale model was built within the framework of another project to test the in- and outflow construction (Vercruysse et al., 2016b). The water level upstream of the construction is controlled by the discharge of water added to the flume. The water level downstream is controlled by a weir. The height of this weir can be adapted to the desired water level downstream of the structure. The flume has a length of 34.80 m, a height of 0.755 m and a width of 0.560 m.

The scale model was built using the largest possible dimension that fit within the small flume. The height of the flume was the limiting factor and a scale factor of 15 was calculated (see Table 3). Based on this calculated scale factor a comparison between in situ and model dimensions is given in Table 2.

Table 1 – Determining the scale factor of the model based on the height of the construction

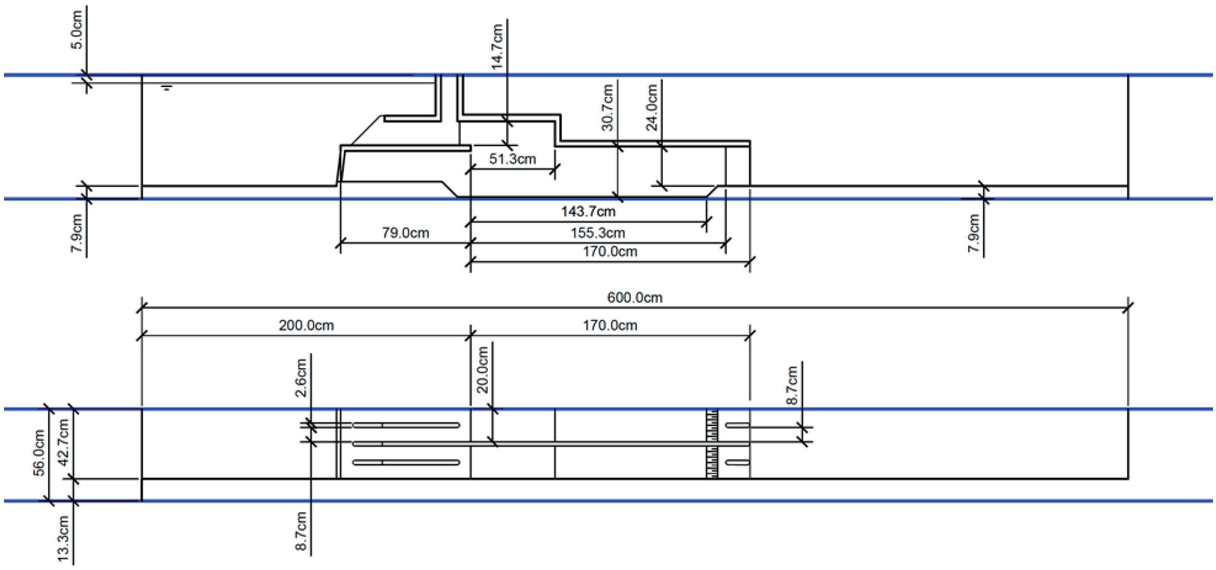
in situ	water level river	TAW + 9.00 m
	Bottom level stilling basin	TAW – 1.40 m
	height	10.40 m
model	height of flume	0.755 m
	reserve on water level	0.050 m
	reserve on possible bottom plate of model	0.012 m
	available height	0.693 m
scale $\left(\frac{\text{height in situ}}{\text{available height}}\right)$		15.007 ≈ 15

Table 2 – Comparison of dimensions of in situ construction and model

	in situ [m]	model [m]
Bottom level inflow culvert	3.30 m TAW	0.313 m above bottom flume
width of culvert	3.00	0.200
width at wooden weir log	1.30	0.087
length floor inflow culvert	11.90	0.793
height inflow culvert	2.20	0.147

For the estimation of friction losses due to the side walls of the culverts, not one but two culverts were built in the scale model, bringing the total width of the model to 0.427 m (2 x culvert width + thickness of wall in between). The total width of the flume is 0.56 m so locally near the culvert model the width of the flume was constraint to 0.427 m. The geometry of the scale model is given in Figure 3.

Figure 3 – geometry scale model. Blue lines are the boundaries of the flume.



For this project the scale model results will be compared with discharge calculations based on mathematical equations. There is no need to upscale the results to real life dimensions and no Froude scaling has to be done.

2.4 Scale model tests setup

Water is added to the flume in an upstream reservoir (see Figure 4). From this reservoir it flows over a sharp crested weir into the main flume compartment where the culvert scale model located. The sharp crested weir makes an accurate estimation of the discharge possible. Discharge is also measured by an electromagnetic discharge meter (type Aquaflux K from Khrone) which is located on the inlet pipe of the reservoir (Figure 4). Also on this inlet pipe a manual butterfly valve is mounted approximately 1.8 m upstream of the discharge meter (Figure 4). A computer controlled butterfly valve is located downstream of the discharge meter, close to the outlet of the pipe into the reservoir of the flume. Because this type of butterfly valve is difficult to control precisely, the manual valve is used to stifle the flow and the electronic valve is used to tune the discharge. Because these butterfly valves close to the discharge meter can influence the results, the sharp crested weir will be used to make the best estimates of the applied discharge.

For the tests water levels are set upstream and downstream of the culvert scale model. The upstream discharge is used to control the upstream water level. The downstream water level is controlled by a weir by changing the crest level. If both water levels (upstream and downstream of the culvert scale model) are set and a steady flow is reached, the water level measured in the reservoir and the dimensions of the sharp crested weir, give an estimate of the discharge flowing through the culverts in the scale model. After 60 seconds of steady flow a next test was started by setting new water level up- and downstream of the culverts scale model.

Water levels in the flume are measured with a water level measuring needle. Seven needle are used to measure water levels in the reservoir (1), upstream (2) and downstream (3) of the scale model and four needles are located just in front of the inlet culverts to measure differences in water level in detail (4 and 5 in the middle and 6 and 7 near the wall of the flume). These locations are presented in Figure 5.

Figure 4 – flume water supply setup

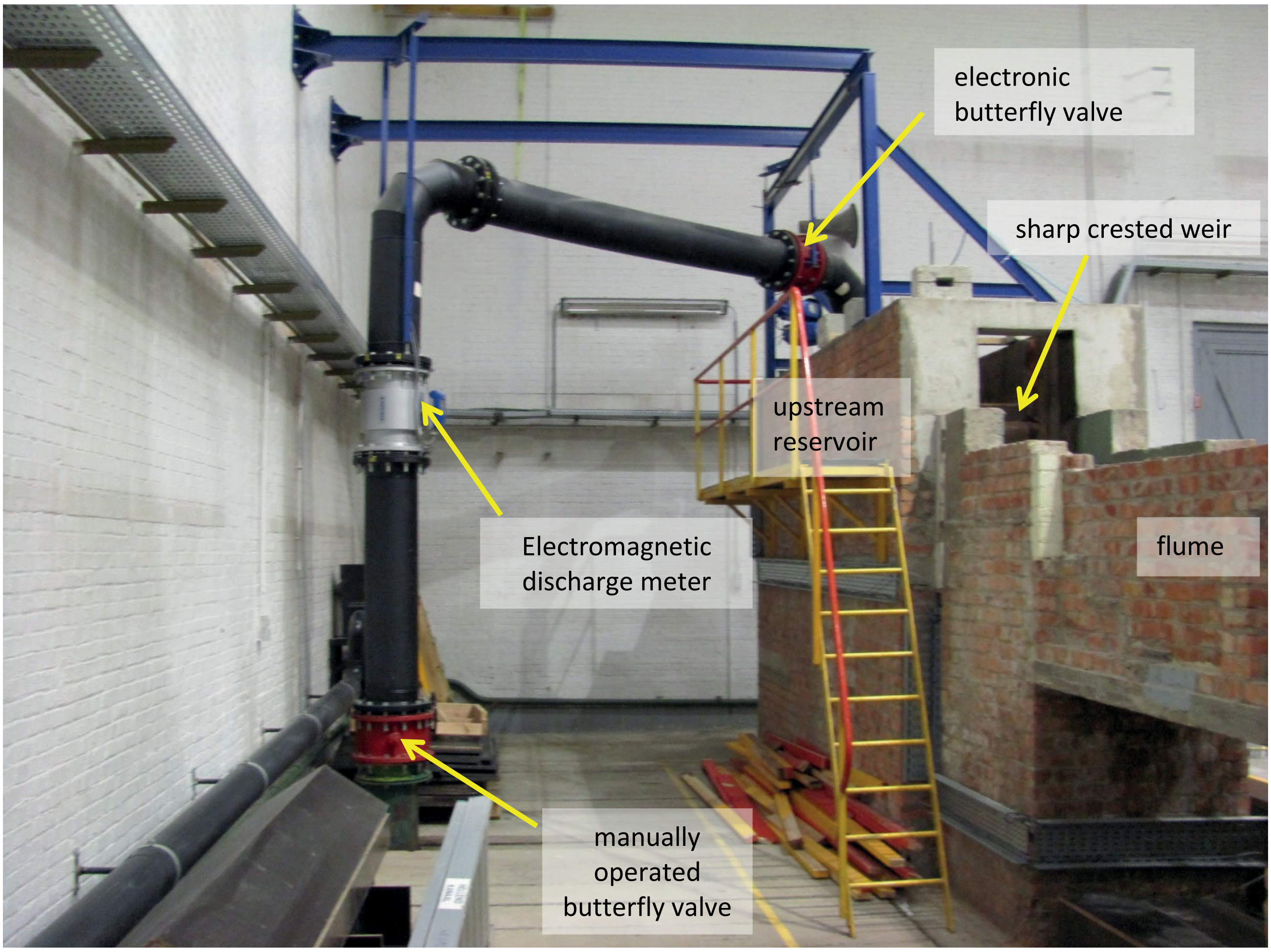
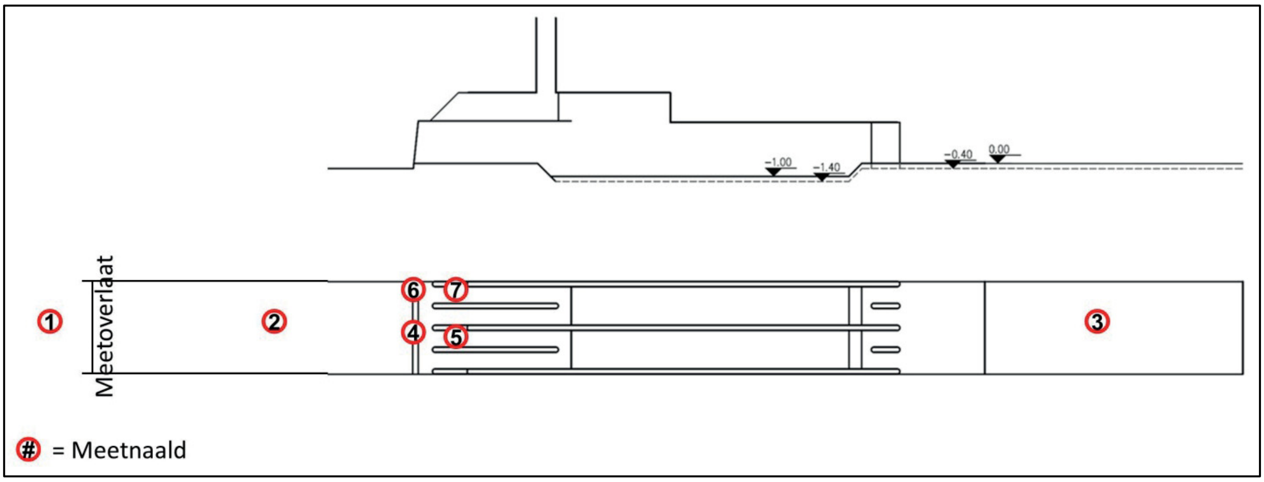


Figure 5 – Location of water level measurements inside the flume scale model setup



Discharge over a sharp crested weir as presented in Figure 6 is calculated according to Bos (1989; pp. 153-158) with the following formula:

$$Q = \frac{2}{3} C_e \sqrt{2g}\, b_c h_1^{3/2} \qquad (1)$$

where: g standard acceleration due to gravity (9.81 m/s²)

b_c width of the weir (m)

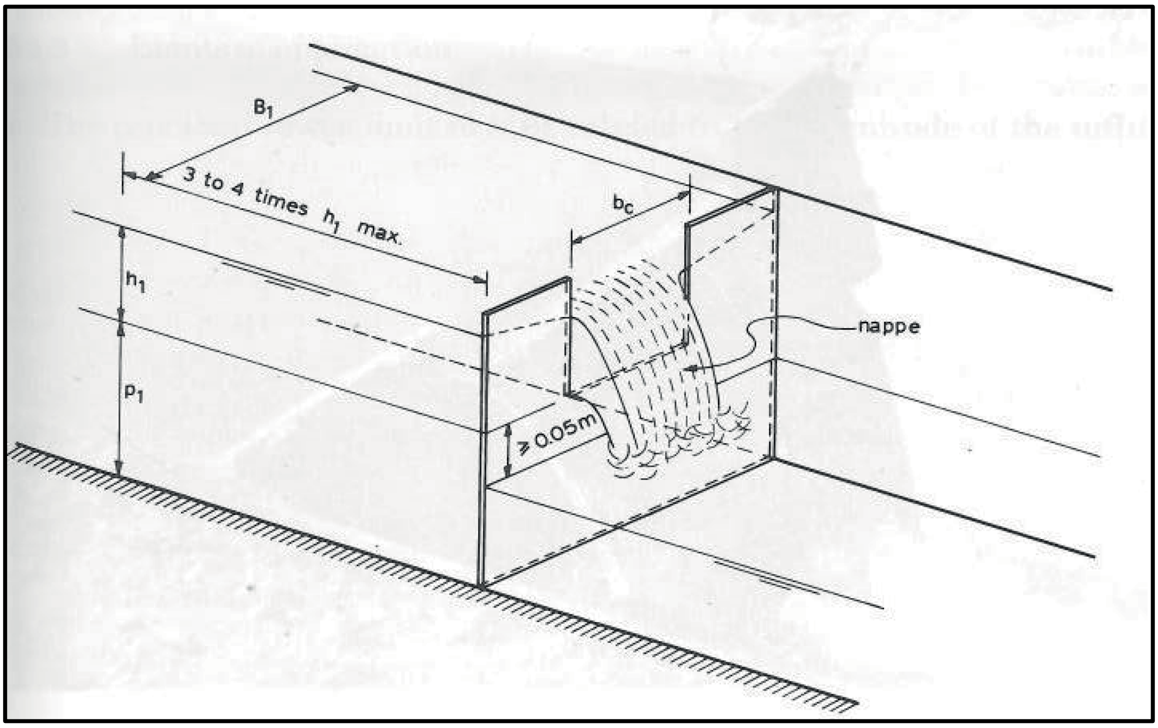
h_1 heigth of water level above crest of the weir (m)

and:

$$C_e = 0.602 + (0.075 * \frac{h_1}{p_1}) \qquad (2)$$

where: p_1 crest height above the reservoir bottom (m)

Figure 6 – Sharp crested weir setup for discharge measurement



The specific dimensions of the sharp crested weir used to determine the discharge in the flume are given in Table 3. With these dimensions and the measured water level in the reservoir, the discharge is accurately estimated by equation 1.

Table 3 – specific dimensions of sharp crested weir used to determine the discharge in the flume

B_1	1	[m]
b_c	0,86	[m]
p_1	0,3	[m]

A side view of the scale model is given in Figure 7. This is the standard setup for the first number of tests. Based on experience in previous tests (Vercruysse et al., 2016) eight sets of constant upstream and downstream water levels were chosen. These water levels were chosen based on the real dimensions of the structure and scaled down to apply in the scale model setup. The list is given in Table 4.

Table 4 – list of up- and downstream water levels forced upon the scale model

test #	river level	polder level	river level	polder level
	m TAW	m TAW	model [m]	model [m]
1	4	3	0,361	0,293
2	5	3	0,429	0,291
3	6	3	0,494	0,293
4	7	3	0,559	0,291
5	7	6	0,560	0,492
6	8	3	0,626	0,292
7	8	6	0,627	0,492
8	8	7	0,626	0,560

These eight tests were executed on the basic scale model as it is shown in Figure 7. Three more sets of tests were done. A second set of tests was done when a small weir log was added to the scale model. This weir log was placed just in front of the inlet culvert (just before the ceiling of the culvert starts), is as wide as the culverts and is placed in front of every inlet. It has a height of 0.027 m. For the third set of tests the weir log was removed again and a grille was placed in front of the inlet culverts. This grille mimics the trash screen that are in front of the real life culvert inlets to keep trash from clogging the culvert function. The grille is made from 1.25 mm thick stainless steel. This grille and the weir logs are shown in Figure 8. The vertical bars in the grille are as wide as they are thick (0.00125 m). The wider vertical bars are placed over the angled culvert walls (made out of plexiglass in the scale model). The bars of the grille cover 17 % of the total opening of the inflow culvert. Finally, the fourth set of tests is done with the weir log and the grille together.

The estimated discharges for each of these for sets of tests will be compared with calculated discharge based on the culvert formulations in the TELEMAC source code.

Figure 7 – Scale model with inner walls in plexi glass material

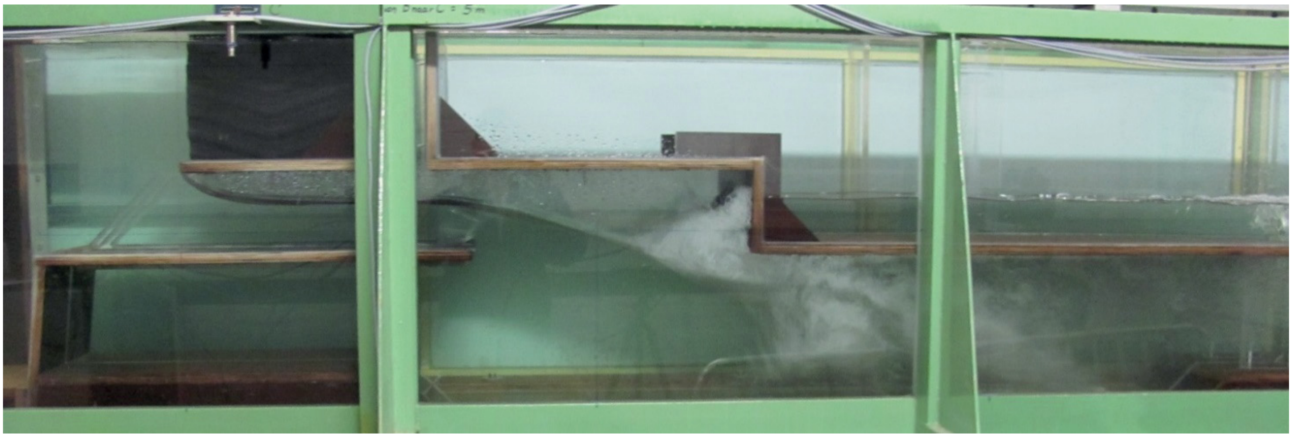
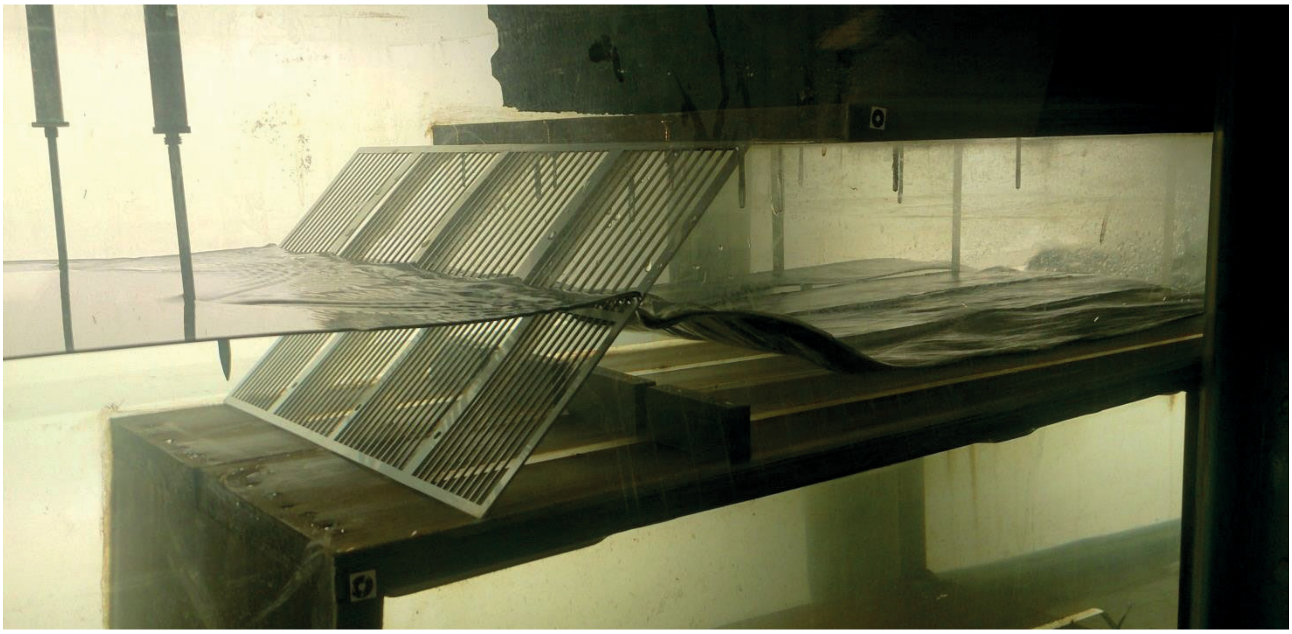


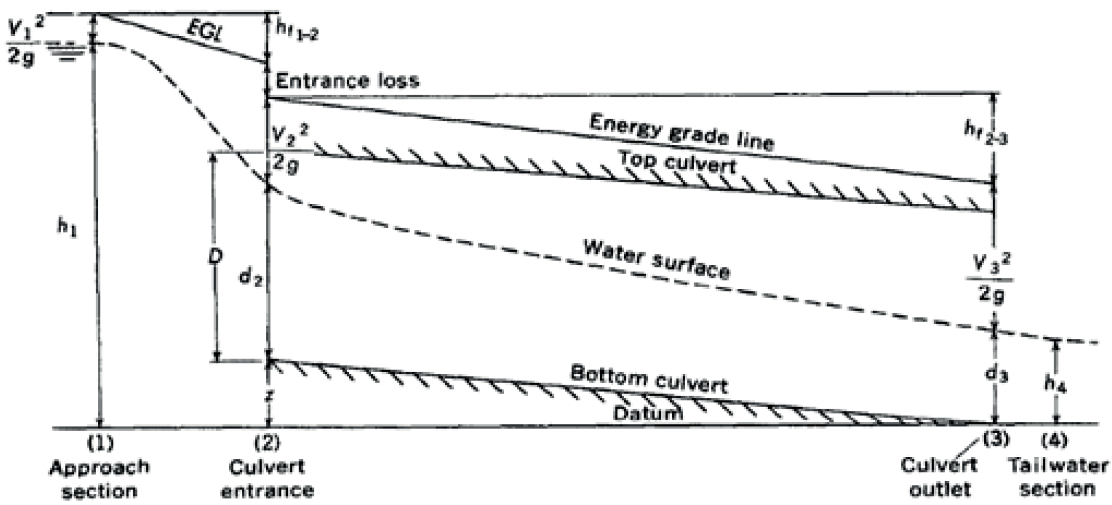
Figure 8 – Scale model detail of trash screen and beam at inflow culverts



2.5 Culvert formulation in TELEMAC code

A number of studies regarding the description of flows through culverts refer to the work of Bodhaine (1968). 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 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 9 shows a sketch for the culvert flow definition. *Z* gives 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 9 – Sketch of general flow through a culvert (Bodhaine, 1968)



The six types of flow classified by Bodhaine (1968) depend on the water depths upstream and downstream of the culvert. We discuss these types here below:

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 of type 1), the discharge coefficient is typically C_D=0,95. The discharge is calculated according to the following formula:

$$Q = C_D A_c \sqrt{2g\left(h_1 - z - h_c - h_{f12} + \alpha \frac{\overline{V_1}^2}{2g}\right)} \tag{3}$$

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

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:

$$Q = C_D A_c \sqrt{2g * \left(h_1 - h_c - h_{f12} - h_{f23} + \alpha \frac{\overline{V_1}^2}{2g}\right)} \tag{4}$$

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

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:

$$Q = C_D A_3 \sqrt{2g\left(h_1 - d_3 - h_{f12} - h_{f23} + \alpha \frac{\overline{V_1}^2}{2g}\right)} \tag{5}$$

with: A_3 flow area at the culvert outlet

d_3 water depth at the culvert outlet

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:

$$Q = C_D A_0 \sqrt{\frac{2g(h_1 - h_4)}{1 + 29 {C_D}^2 n^2 L / R^{4/3}}} \tag{6}$$

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

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. Type 5 flow does not usually occur. When it does, the discharge coefficient is in general lower than the other types.

$$Q = C_D A_0 \sqrt{2g(h_1 - z)} \tag{7}$$

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:

$$Q = C_D A_0 \sqrt{2g\left(h_1 - d_3 - h_{f23}\right)} \tag{8}$$

The indices of the different variables might seem a bit confusing, but it was chosen to take the formulas from Bodhaine as they were and not to make any changes to them. Bodhaine differentiated between these six flow type based on conditions given in Table 5.

Table 5 – Conditions for each type of flow defined by Bodhaine (1968).

Type 1	$\frac{h_1 - z}{D} < 1.5$	$\frac{h_4}{h_c} < 1.0$	$S_0 > S_c$
Type 2	$\frac{h_1 - z}{D} < 1.5$	$\frac{h_4}{h_c} < 1.0$	$S_0 < S_c$
Type 3	$\frac{h_1 - z}{D} < 1.5$	$\frac{h_4}{D} \leq 1.0$	
Type 4	$\frac{h_1 - z}{D} > 1.0$	$\frac{h_4}{D} > 1.0$	
Type 5	$\frac{h_1 - z}{D} \geq 1.5$	$\frac{h_4}{D} \leq 1.0$	
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 10, 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 flow is of type 6, if it lies to the left of the curve, the flow is of type 5.

The head loss coefficients are subject of different studies made in laboratory experiments. A number of authors have arrived to different values or empirical relationships for the head loss coefficients. Bodhaine (1968) suggests different values for the disharge coefficient (C_D) for each type of flow and depending on a number of geometric features from the culvert. The discharge coefficients can vary from 0.39 to 0.98. another example is given by Carlier (1972) 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:

$$\mu = \frac{1}{\sqrt{C_1+C_2+C_3}} \tag{9}$$

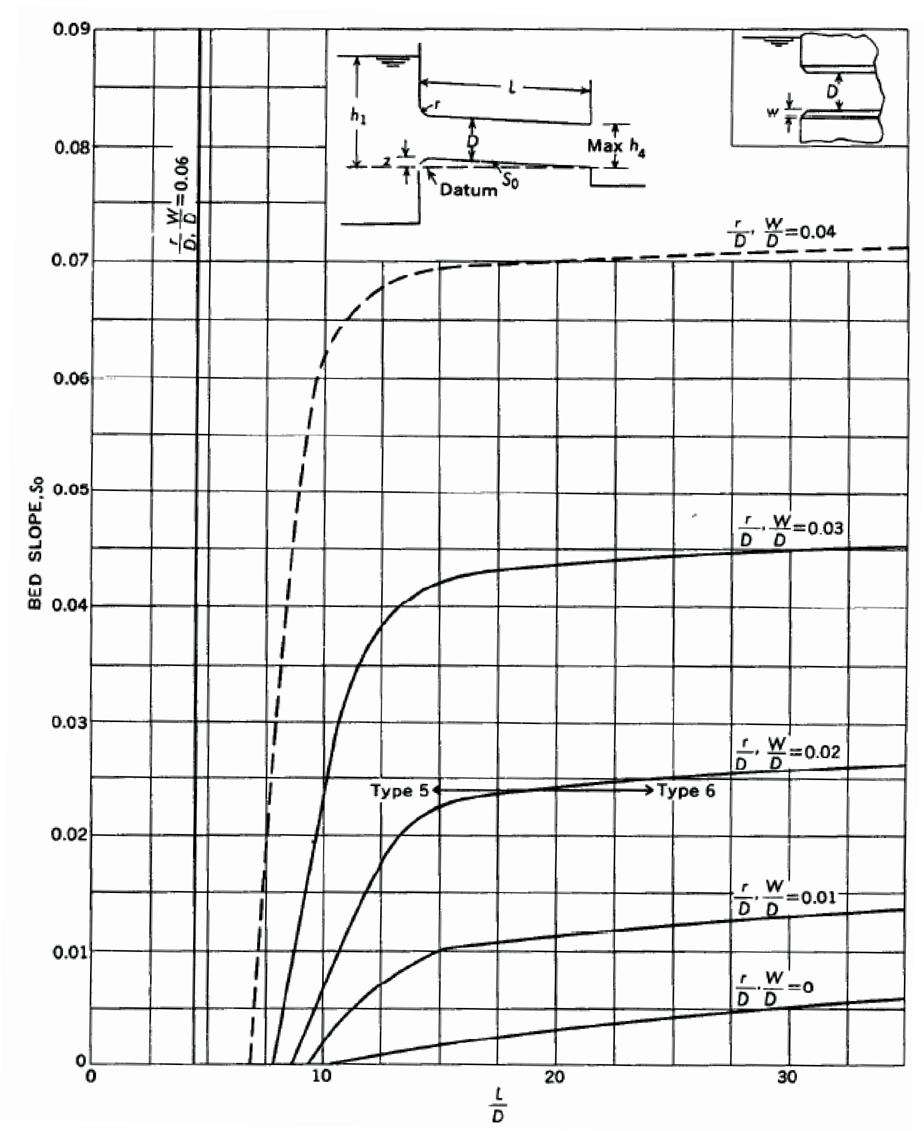
with: C_1 head loss coefficient at the entrance of the hydraulic structure

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 Carlier (1972) is compared with the formulas given by Bodhaine (1968), 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 loss for each type of flow.

Figure 10 – 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 (Bodhaine, 1968)



Following the proposition of Carlier (1972) the equations proposed by Bodhaine (1968) 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. This only happens in very rare occasions if the culvert slope is very steep.

Type 2 – Critical depth at outlet:

$$Q = \mu h_c W \sqrt{2g * \left(S_1 - (z_2 + h_c)\right)} \quad (10)$$

Type 3 – Tranquil flow:

$$Q = \mu (S_2 - z_2) W \sqrt{2g(S_1 - S_2)} \quad (11)$$

Type 4 – Submerged outlet:

$$Q = \mu D W \sqrt{2g(S_1 - S_2)} \quad (12)$$

Type 5 – Rapid flow at inlet:

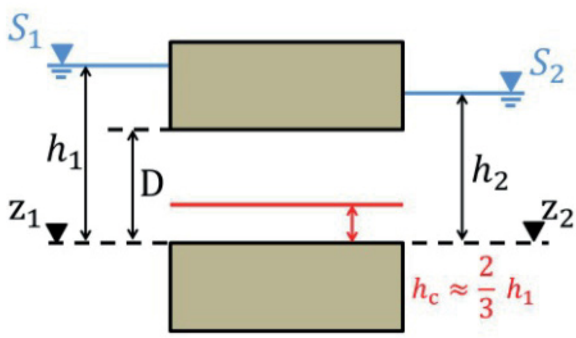
$$Q = \mu D W \sqrt{2g h_1} \quad (13)$$

Type 6 – Full flow with free outfall:

$$Q = \mu D W \sqrt{2g(S_1 - (z_2 + D))} \quad (14)$$

<u>with</u>: 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 side 2, 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 will be 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 11.

Figure 11 – Schematic representation of a culvert with the different parameters [6]



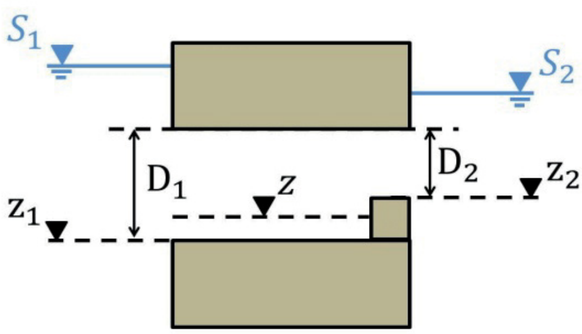
The conditions for which each type of flow occurs are summarized in Table 6. To distinguish flow type 5 from flow type 6 a constant C56 is defined that is dependent on the culvert slope and the ratio w/D. Using Figure 10, the curve (w/D) is chosen and the point for which the value of the slope (S_0) 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 (Bodhaine, 1968).

Table 6 – 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
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		<C56
Type 6	≥ 1.5	≤ 1.0		≥C56

To incorporate the culverts structures present along the Scheldt estuary, 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. At 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 them into the code the geometric features of the culvert presented in Figure 11 are modified and presented in Figure 12. An equivalent culvert bottom elevation z was used, which replaces both the bottom elevations z_1 and z_2 in the formulas decribed 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 12, if the flow goes from left to the right D will be replaced by D_1 and on the opposite direction, the value D_2 will be used. For the start of the water flow into the FCA the z_1 and z_2 bottom elevations are used so that the start and end of water flow through the culverts remains as close as possible to reality. By applying this equivalent bottm elevation assumption, the culverts frictional head losses are overestimated and the local larger head losses due to the presence of the weir are not taken into account exactly. These complicated structures are difficult to model exactly and this assumption will keep things as simple as possible. 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 (or even calibrate!) the modeled discharges.

Figure 12 – Representation of the different variables used to calculate the discharges for each type of flow (Teles et al., 2016)



The head loss coefficient (μ) was adapted from the one calculated in TELEMAC-2D, based on Carlier (1972) and is used as main head loss coefficient. Additional head losses, like one-way valves, trash screens or pillars were added in the calculation of this main head loss coefficient. In this way these additional features (that can be present in culvert structures of different geometric configurations) are taken into account and contribute 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 Lencastre (1961) and Carlier (1972)

$$\Delta H = C\frac{U^2}{2g} \text{ or } U = \mu\sqrt{2g\Delta H} \tag{15}$$

with: $$\mu = \frac{1}{\sqrt{C}} \tag{16}$$

U is the net flow velocity. The coefficient C represents the sum of the different contributions for the head loss due to singularities:

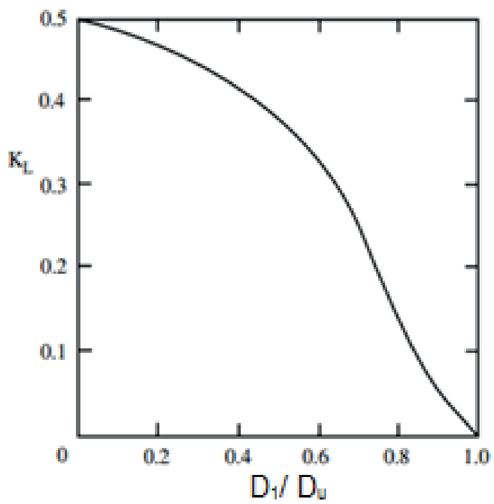
$$C = C_1 + C_p + C_2 + C_3 + C_V + C_{Trash} \tag{17}$$

The different contributions to this head loss coefficient C will be discussed separately and in detail here 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 13 is taken from Larock (2000) 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 13 will be close to zero and the head loss coefficient, value on the y-axis, will be 0.5.

Figure 13 – 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 (Larock, 2000)



Bodhaine (1968) noticed that the discharge coefficient (C_D) for type 5 flow had to be lowered comparetively with the other flow types. The calculated discharge seemed to be overestimated when the default equation was applied. Therefore a correction coefficient is taken into account. The correction coefficient, C5 is applied to C_1 when type 5 flow occurs, such that:

$$\Delta H_1 = \mathrm{C5} * C_1 \frac{U^2}{2g} \tag{18}$$

Bodhaine (1968) proposed 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 Carlier (1972) the head loss caused by parallel pillars is given by:

$$\Delta H_p = C_p \frac{U^2}{2g} \tag{19}$$

and $$C_p = \beta \left(\frac{L_p}{b}\right)^{4/3} \sin\theta \tag{20}$$

C_p represents the head loss coefficient due to the presence of pillars. L_p is the thickness of the pillars, b the distance between two consecutive pillars and β is a coefficient dependent on cross-sectional area of the pillar. According to Carlier (1972) 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 θ will be equal to 1. In the code we don't use this head loss coefficient separately, but it's 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 Lencastre (1971):

$$\Delta H_2 = C_2 \frac{U^2}{2g} = \frac{2gLn^2}{R^{4/3}} \frac{U^2}{2g} \tag{21}$$

where L is the 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 for the estimation of the water depth is 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 Lencastre (1961):

$$\Delta H_3 = \left(1 - \frac{A_s}{A_{s2}}\right)^2 \frac{U^2}{2g} = C_3 \frac{U^2}{2g} \tag{22}$$

where A_s and A_{s2} are the 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:

$$\Delta H_v = C_v \frac{U^2}{2g} \tag{23}$$

where 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 (Larock, 2000). Some values according to Larock (2000) are given in Table 7.

Table 7 – Values for the head loss coefficient depending on the opening of a gate valve according to Larock (2000)

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

Like with C_1 a correction coefficient (CV5) is applied to this head loss coefficient to take into account the increase of the head loss when type 5 flow occurs. Through a number of laboratory experiments with a physical scale model at Flanders Hydraulics Research (Cox et al., 2006), it was clear that when type 5 flow occurs, there is a greater influence of the head loss coefficient of the valve. According to Cox et al. (2006) CV5 can be set as 1.5.

$$\Delta H_{v,5} = C_{v,5} C_v \frac{U^2}{2g} \tag{24}$$

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. We use the expression given by Wahl (1992):

$$\Delta H_t = \left(1.45 - 0.45 A_{trash} - {A_{trash}}^2\right) \frac{U^2}{2g} = C_{trash} \frac{U^2}{2g} \tag{25}$$

where $A_{trash} = \frac{A_{net}}{A_{gross}}$ gives the ratio of net flow area to gross rack area. *U* is the net flow velocity. 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 compared to the gross rack area).

The goal of the implementation of this code in TELEMAC was to use 1 subroutine for culverts in TELEMAC-2D and TELEMAC-3D.

The existing culvert code was only present in TELEMAC-2D in the subroutine BUSE.f. The existing code had four equations with three of them similar to the ones presented here (flow types 3,5 and 6), but our approach is more complete and replaced the existing code.

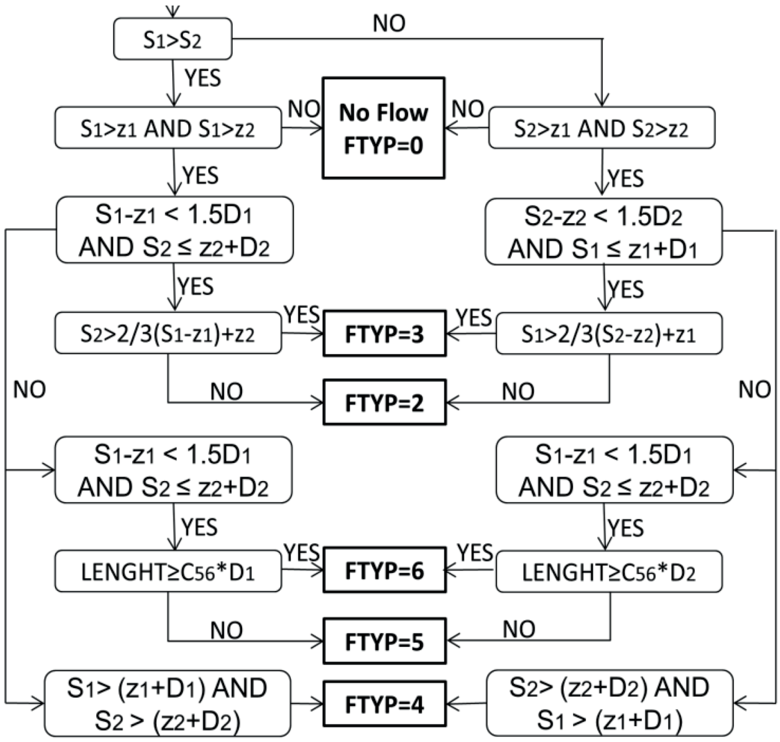
The capability of TELEMAC to impose source and sink terms in the domain was usefull to implement a culvert function. The inflow and outflow of a culvert then act as a couple of source/sink points. For instance, when the flow is going from the river to the flood plain side, a source term is added on that side, i.e., a discharge is imposed on that point, and at the same time a sink term is put in the river with the symmetric value of that discharge. By doing this we assume that the culverts are in general short and that the water that leaves the river, enters the floodplain in the same time step. The culvert subroutine uses sinks and sources and the discharges calculated in this subroutine are also treated like that in the rest of the main code. The calculated discharges in 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)
```

Where NPTSCE is the number of point sources and DBUS(I) is the discharge calculated for culvert number I.

Equations 10 to 14 are implemented in the code based on the conditions given in Table 6. Figure 14 gives a flow chart of what this looks like in the code.

Figure 14 – Flow chart showing the conditions for every type of flow in the BUSE.f subroutine



The user has to specify all the parameters related with the culverts in a text file, through the keyword: CULVERTS DATA FILE. This text file will be read by the subroutine LECBUS.f. The text file and the existing subroutine LECBUS.f were extended to take into account extra parameters. The first and third line 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 parameters 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. On the third line there is in comment the names of all the parameters used in BUSE.f. They are separated by a tab. The flow through a culvert can go in both directions. In the following we agree upon using the index 1 for the river side and 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*

I2 *node number of the culvert on side 2*

CE1 *entrance head loss coefficient for the culvert on side 1 (this corresponds with head loss coefficient C_1)*

CE2 *entrance head loss coefficient for the culvert on side 2 (this corresponds with head loss coefficient C_1)*

CS1 *exit head loss coefficient for the culvert on side 1 (this corresponds with head loss coeffiicent C_3)*

CS2 *exit head loss coefficient for the culvert on side 2 (this corresponds with head loss coeffiicent 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)*

RD1 *culvert bottom elevation on side 1(z1)*

RD2 *culvert bottom elevation on side 2(z2)*

CV *head loss coefficient when a valve is present*

C56 *factor to differentiate between flow type 5 or 6*

CV5 *correction factor for CV when flow type 5 is used*

C5 *correction factor for CE1 and CE2 when flow type 5 is used*

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 *The length of the culvert (! not automatically calculated based on the location of the 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 calculated the wet section.*

When the discharge is calculated by BUSE.f based on the above parameters it undergoes 3 possible changes. First the relaxation is calculated. Based on the weight and the difference with the dischagre in the previous time step, this could change the calculated discharge of this time step a lot. Secondly after relaxation it is tested if there is enough water present in the area around the node at this time step to extract the relaxed discharge. A maximum of 90% of the available water is allowed to leave. In the end the test is done to see if the discharge at this time step is allowed in this direction through the culvert. In the culvert data file the user can choose the direction of flow through a culvert by setting the parameter CLP. For example if there is a one-way valve present on the culvert the user can force this direction of flow using the CLP variable. If the direction of flow is not allowed by the CLP variable given by the user, the discharge is set to zero. The calculated discharge is positive for the flow from side 1 to 2.

3 Results and discussion

3.1 Measured discharges scale model

The scale model experiments were done for four different model setups: no weir log and no trash screen, a weir log and no trash screen, no weir log and a trash screen, and a weir log and a trash screen. For each of these four setups eight tests were done. For each test the water level before and after the culvert was set according to water levels listed in Table 4. For each setup and each test the exact water levels were also recorded at 7 locations (indicated in Figure 5) and a discharge over the sharp crested weir was calculated according to equation 1. The results are listed in Table 8.

Table 8 – Results scale model tests

TELEMAC1: no beams, no trash screen									
water level measuring needle									Q
1	2	3	2	3	4	5	6	7	m^3/s
level at weir model [m]	river level m TAW	polder level m TAW	river level model [m]	polder level model [m]	[m]	[m]	[m]	[m]	
0,2693	4	3	0,361	0,293	0,0421	0,0391	0,0436	0,0378	0,006
0,3053	5	3	0,429	0,291	0,1039	0,0826	0,1047	0,0771	0,025
0,3388	6	3	0,494	0,293	0,1606	0,1306	0,1627	0,1256	0,050
0,353	7	3	0,559	0,291	0,2292	0,2424	0,2338	0,2378	0,063
0,351	7	6	0,560	0,492	0,2312	0,2421	0,2333	0,2388	0,061
0,3678	8	3	0,626	0,292	0,3026	0,3117	0,3043	0,3098	0,076
0,3675	8	6	0,627	0,492	0,3032	0,3119	0,3056	0,3101	0,076
0,3488	8	7	0,626	0,560	0,3072	0,3107	0,3093	0,3108	0,059

TELEMAC2: beam, no trash screen									
water level measuring needle									Q
1	2	3	2	3	4	5	6	7	m^3/s
level at weir model [m]	river level m TAW	polder level m TAW	river level model [m]	polder level model [m]	[m]	[m]	[m]	[m]	
0,2092	4	3	0,364	0,291	0,0505	0,0455	0,0513	0,0449	0
0,2452	5	3	0,427	0,292	0,1062	0,0981	0,1091	0,0948	0,019
0,2802	6	3	0,493	0,293	0,1925	0,1358	0,1943	0,1333	0,044
0,2988	7	3	0,559	0,293	0,2343	0,2399	0,2363	0,2398	0,059
0,2961	7	6	0,560	0,492	0,2355	0,2416	0,2373	0,2399	0,057
0,3119	8	3	0,625	0,293	0,3043	0,3104	0,3053	0,3073	0,071
0,3119	8	6	0,628	0,493	0,3042	0,3104	0,3053	0,308	0,071
0,2938	8	7	0,626	0,560	0,3064	0,3109	0,3063	0,309	0,055

TELEMAC3: no beam, trash screen									
water level measuring needle									Q
1	2	3	2	3	4	5	6	7	m³/s
level at weir model [m]	river level m TAW	polder level m TAW	river level model [m]	polder level model [m]	[m]	[m]	[m]	[m]	
0,2175	4	3	0,358	0,292	0,0412		0,0441		0,005
0,2512	5	3	0,426	0,293	0,1032		0,1064		0,023
0,2833	6	3	0,493	0,293	0,1631		0,166		0,046
0,2999	7	3	0,560	0,292	0,232		0,2353		0,060
0,2968	7	6	0,558	0,492	0,2346		0,2385		0,057
0,3143	8	3	0,629	0,293	0,3062		0,3064		0,073
0,3139	8	6	0,628	0,493	0,3061		0,3066		0,073
0,2899	8	7	0,627	0,560	0,3082		0,3098		0,055

TELEMAC4: beam and trash screen									
water level measuring needle									Q
1	2	3	2	3	4	5	6	7	m³/s
level at weir model [m]	river level m TAW	polder level m TAW	river level model [m]	polder level model [m]	[m]	[m]	[m]	[m]	
0,2092	4	3	0,360	0,292	0,0492		0,0504		0,000
0,2425	5	3	0,428	0,293	0,1085		0,1105		0,018
0,2788	6	3	0,493	0,293	0,1672		0,1692		0,042
0,295	7	3	0,558	0,293	0,2332		0,2336		0,055
0,2933	7	6	0,561	0,491	0,2392		0,2409		0,054
0,3077	8	3	0,627	0,293	0,3052		0,3063		0,067
0,3108	8	6	0,625	0,491	0,3072		0,3083		0,070
0,2895	8	7	0,627	0,560	0,3082		0,3098		0,050

3.2 Calculated discharges

Although there were only two culverts built in the scale model, each divided in two by an internal wall, for the culvert code these are actually four culverts. So for each of these culverts a set of parameters (as listed in section 2.5) is given in Table 9 for each scale model setup. The entrance head loss (CE1 and CE2) is estimated to be 0.5 m based on interpretation of Figure 13. The exit head loss CS is calculated based on formula 22 and is set to 0.2 m. LARG is the width of the culverts and this was 0.087 m. The height of the culvert at the entrance and exit remained the same (so HAUT1 = HAUT2) and this was 0.147 m. CLP is set to 1, because there is only flow from one side to the other side in this scale model setup. RD1 and RD2 give the relative height of the bottom of the culvert at the inlet and outlet respectively. This was 0.313 m above the flume bottom, which is taken as a reference level. There is no valve present because these are inlet culverts of Doelpolder, so CV = 0. C56 is the factor to differentiate between flow type 5 or 6. CV5 is set to 1.5 according to Cox et al. (2006). C5 is estimated 6 based on Bodhaine (1968). Trash was set to zero for the setups when no trash screen was present. FRIC gives the Manning friction coefficient for the internal culvert walls. For the scale model this was very smooth material. The value is set to 0.016 s/m$^{1/3}$. LONG gives the length of the culvert and this was 0.5 m. CIRC was set to 0 because the culverts were rectangular. The last three parameters are used to calculate the head loss due to internal friction on the walls. When a weir log was present in front of the culverts the parameter RD1 was increased with the weir log height (0.027 m). When a trash screen was present the parameter TRASH was estimated based on equation 25 and for the scale model this resulted in an extra head loss of 0.3 m.

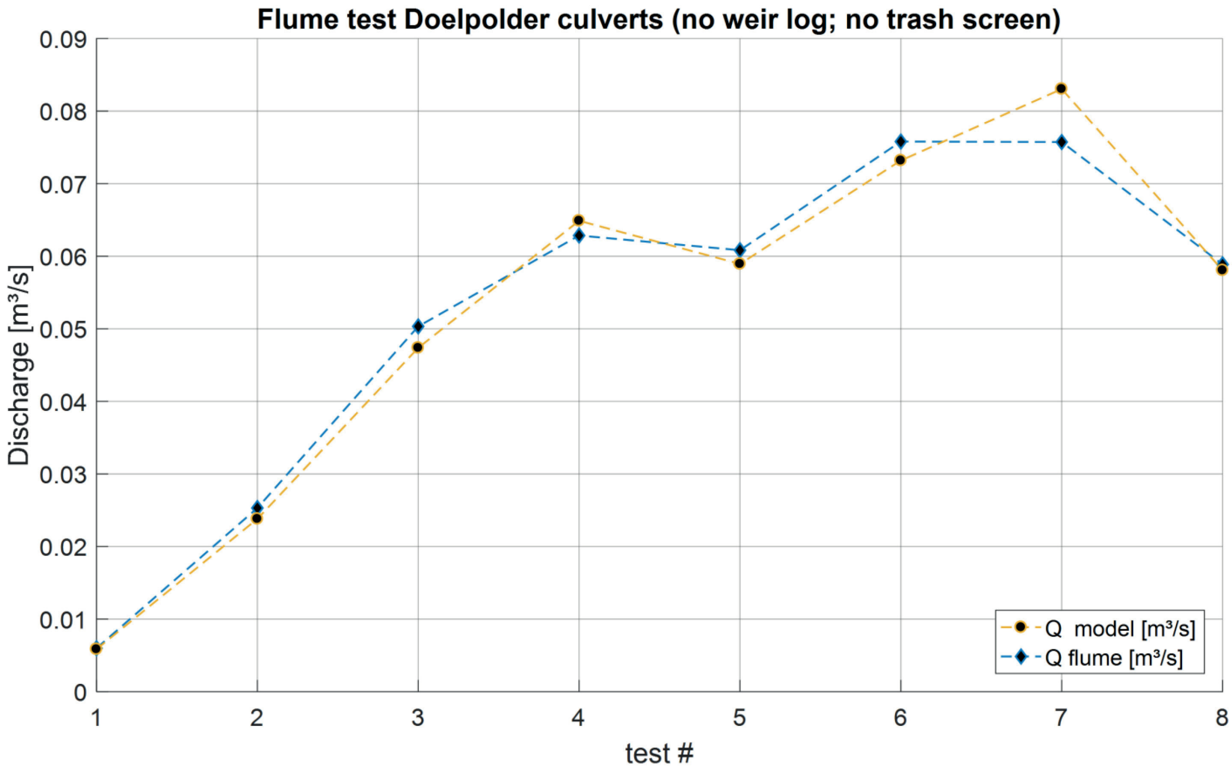
Table 9 – culvert characteristics for discharge calculation for the four different scale model setups

CE1	CE2	CS1	CS2	LARG	HAUT1	CLP	RD1	RD2	CV	C56	CV5	C5	TRASH	HAUT2	FRIC	LONG	CIRC
no weir log; no trash screen																	
0,5	0,5	0,2	0,2	0,087	0,147	1	0,313	0,313	0	10	1,5	6	0	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,313	0,313	0	10	1,5	6	0	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,313	0,313	0	10	1,5	6	0	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,313	0,313	0	10	1,5	6	0	0,147	0,016	0,5	0
weir log; no trash screen																	
0,5	0,5	0,2	0,2	0,087	0,147	1	0,34	0,313	0	10	1,5	6	0	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,34	0,313	0	10	1,5	6	0	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,34	0,313	0	10	1,5	6	0	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,34	0,313	0	10	1,5	6	0	0,147	0,016	0,5	0
no weir log; trash screen																	
0,5	0,5	0,2	0,2	0,087	0,147	1	0,313	0,313	0	10	1,5	6	0,3	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,313	0,313	0	10	1,5	6	0,3	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,313	0,313	0	10	1,5	6	0,3	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,313	0,313	0	10	1,5	6	0,3	0,147	0,016	0,5	0
weir log and trash screen																	
0,5	0,5	0,2	0,2	0,087	0,147	1	0,34	0,313	0	10	1,5	6	0,3	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,34	0,313	0	10	1,5	6	0,3	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,34	0,313	0	10	1,5	6	0,3	0,147	0,016	0,5	0
0,5	0,5	0,2	0,2	0,087	0,147	1	0,34	0,313	0	10	1,5	6	0,3	0,147	0,016	0,5	0

No weir logs and no trash screen

The measured discharge in the flume and the calculated discharges are in good agreement. For the eight different tests the results are shown in Figure 15. The difference between measured and calculated discharge is larger for test 7. The culvert formulations in the TELEMAC code don't take into account more complicated culvert structures like the in- and outlet construction of CRT areas. By being creative with the input parameter set weir logs and trash screens can be taken into account. And as long as there is free outflow the culvert code performs very well, but in test 7 when the outflow side of the construction is fully submerged and there is still a strong slope between both sides of the construction, the code doesn't take into account some additional head losses.

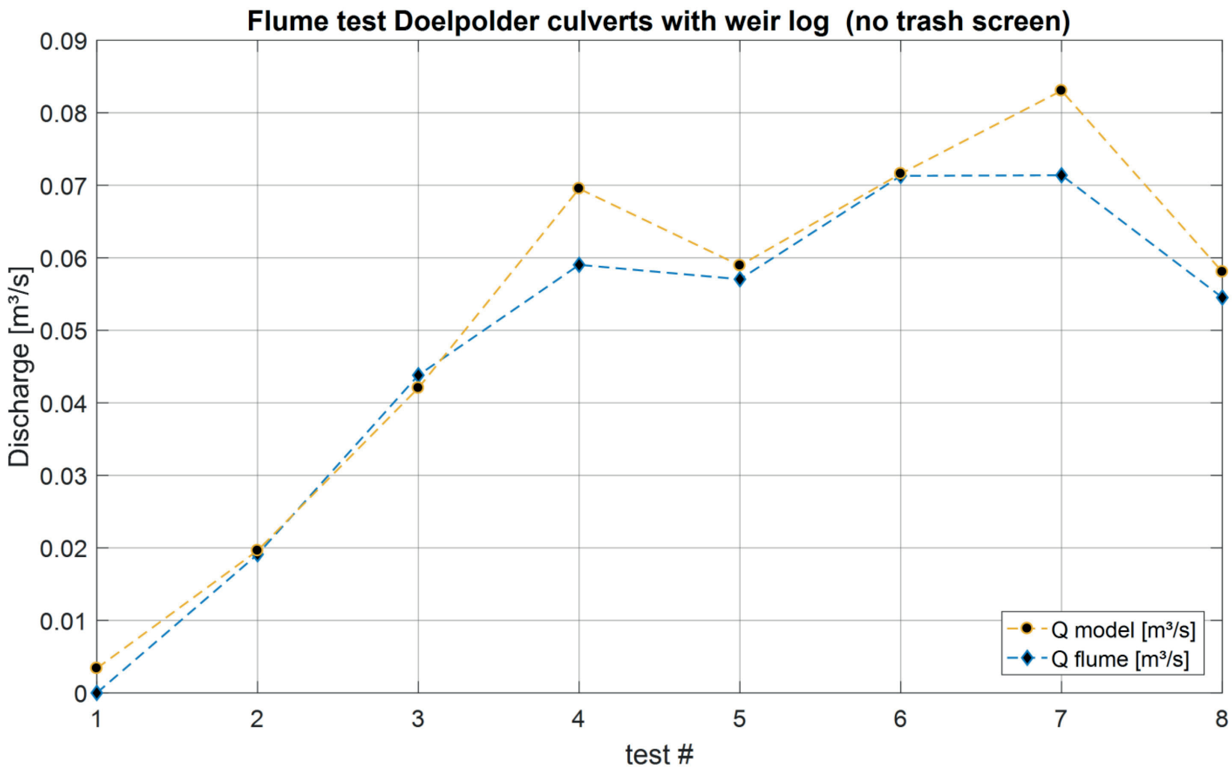
Figure 15 – Discharge measured in the flume and calculated with the culvert code for the scale model setup without weir logs and trash screen.



Weir logs but no trash screen

The results are shown in Figure 16. In test 1 there was no flow of water through the culvert construction because of the additional height of the weir log at the entrance of the culverts. But according to the culvert code and the measured water levels there should be some flow. This means that the height of the bottom of the inlet culverts + the height of the weir logs in the flume was larger than the height given in the parameters to calculate the discharge. And this could also explain the discrepancy between the calculated and measured discharge in test 4. The culvert code predicts here a flow type 2 (Critical depth at outlet – subcritical flow inside the culvert) based on the water level and the height of the weir logs, where as it probably should be flow type 5 (rapid flow at inlet). The difference in measured and calculated discharge for test 7 is again attributed due to not accounted for friction head losses. The other tests gave results in good agreement between measurements and calculations.

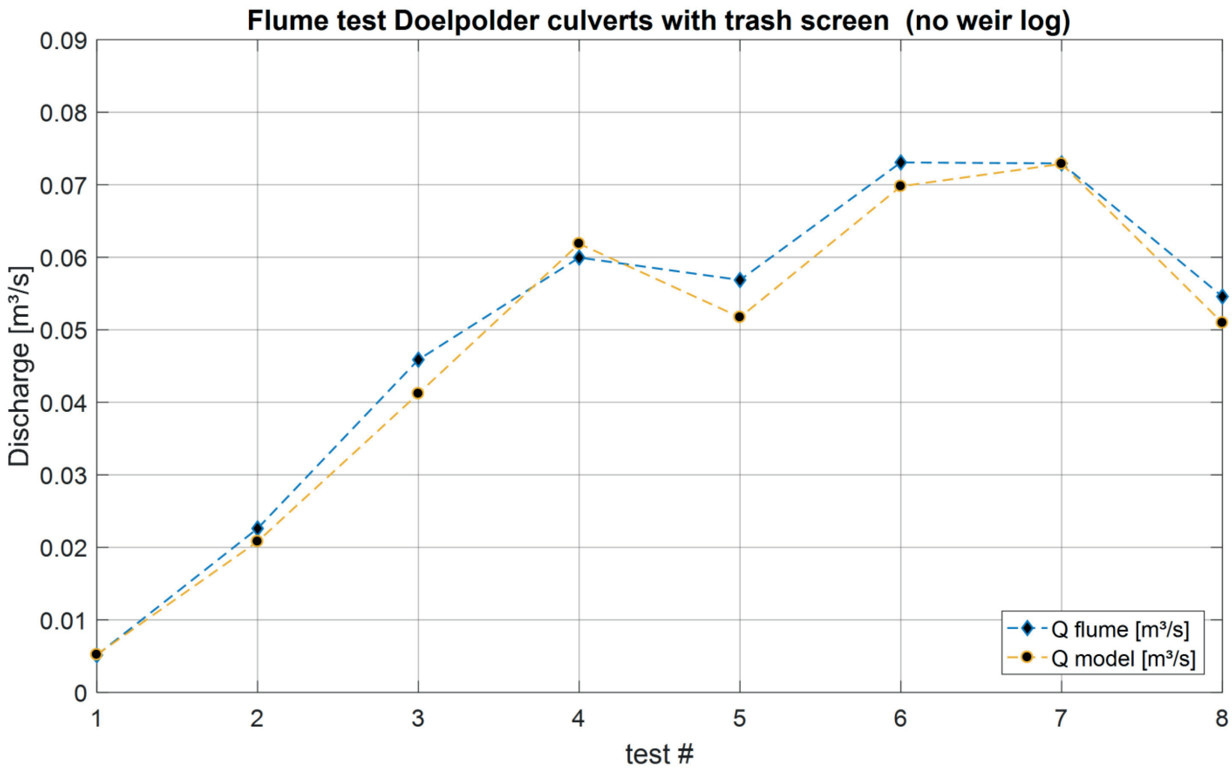
Figure 16 – Discharge measured in the flume and calculated with the culvert code for the scale model setup with weir logs but without a trash screen.



No weir logs, but a trash screen

The results of both measurements and calculations are shown in Figure 17. The results show a good agreement between the measured discharges and the calculated ones. For test 7 the results are identical, but for the tests 5,6 and 8 the calculated discharges are a little lower than the measured ones.

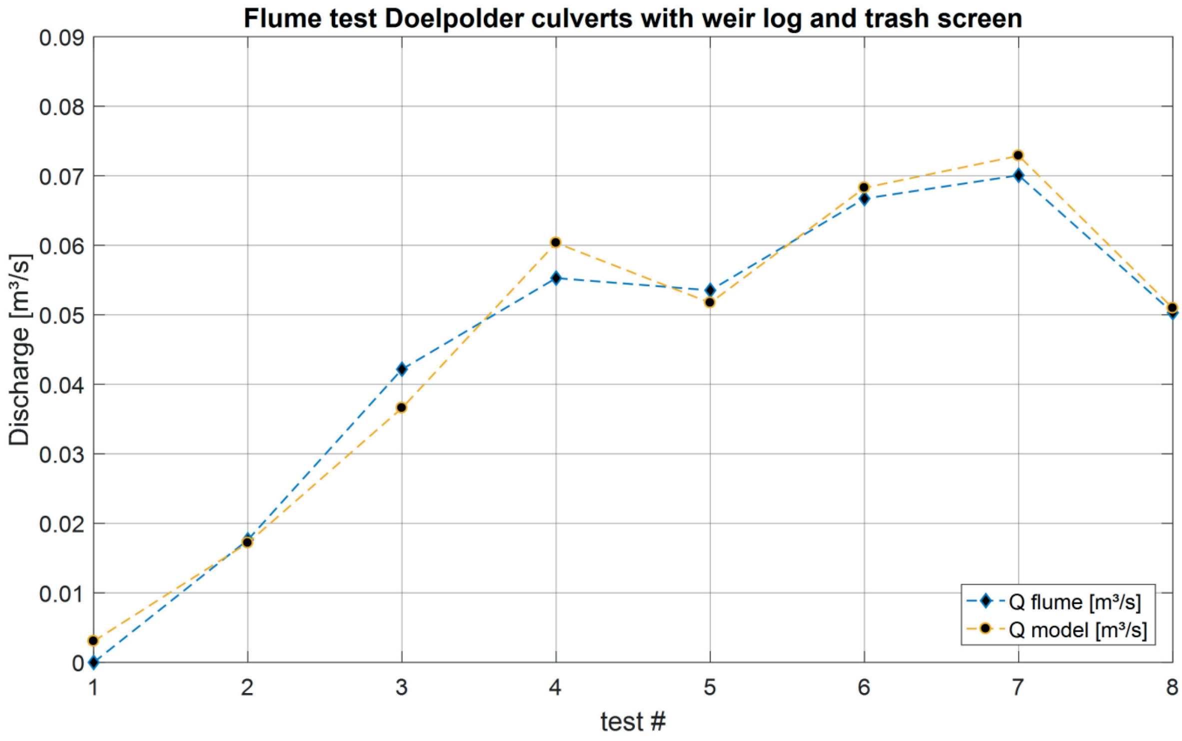
Figure 17 – Discharge measured in the flume and calculated with the culvert code for the scale model setup without weir logs but with a trash screen.



Weir logs and a trash screen

The results are a combination of the two previous model setups. They are shown in Figure 18. The wrong height of the weir in the calculations predicts a discharge where there was no discharge in the flume for test 1. For tests 4 and 7 the same remarks can be made as previously stated. In general, the approximation of the calculated discharges for the measured discharges is good given the complicated structure of the culverts.

Figure 18 – Discharge measured in the flume and calculated with the culvert code for the scale model setup with weir logs and a trash screen.



Comparing the four different setups

The lowest discharges are found for the setup with weir logs and trash screen. This setup has the largest head losses. Figure 19 shows the calculated discharges for the four different setups. the setup without the weir logs and trash screen has the lowest amount of head losses and gives thus the highest calculated discharges. Except for test 4 where the setup with only the weir logs shows higher calculated discharge. This is not seen in the measurements and this is probably caused by a not correctly given input height of the weir logs (correct height is unknown) and this resulted in a different flow type, resulting in an overestimation of the discharge. The flow types are given in Figure 20. Flow type 2, 4 and 5 are used. There is only a difference in test 4 where the flow type differs between type 2 and 5. The difference lies in the following condition in the code:

WLriver – RD1 > 1.5 * HAUT1

For the setup without the weir RD1 equals 0.313 m. For test 4 WL river equals 0.559 m and HAUT1 is 0.147 m. This means that the condition is met when there is a weir (0.559-0.34 > 0.2205) and not when there is no weir (0.559 – 0.313 < 0.2205). And the formula to calculate the discharge in case of flow type 2 gives a higher discharge with the same input parameters compared to the formulation in case of flow type 5. If the weir logs are positioned in the opening of the culvert itself, they restrict the inlet area of the culvert and flow type 2 is a good estimation. In this case the weir logs were positioned in front of the culvert entrance, leaving the inlet area of the culvert as big as without the weir logs. In this case maybe it was better to use RD2 or the bottom level of the culvert in the above condition so flow type 5 would be used.

Figure 19 – Comparing the calculated discharges of the four different scale model setups

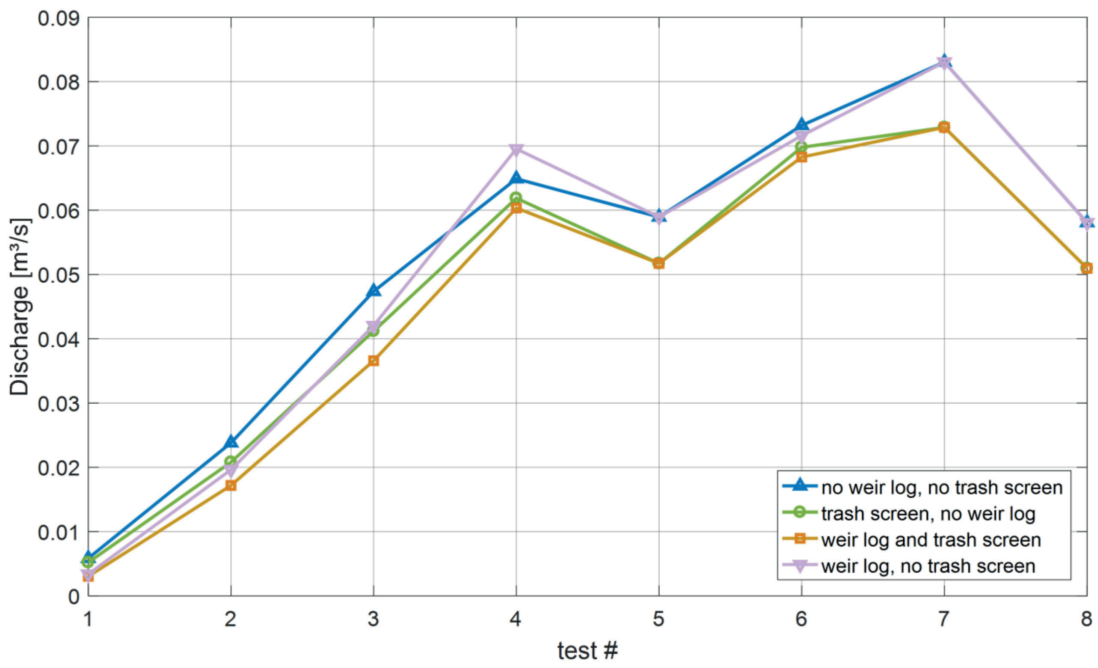
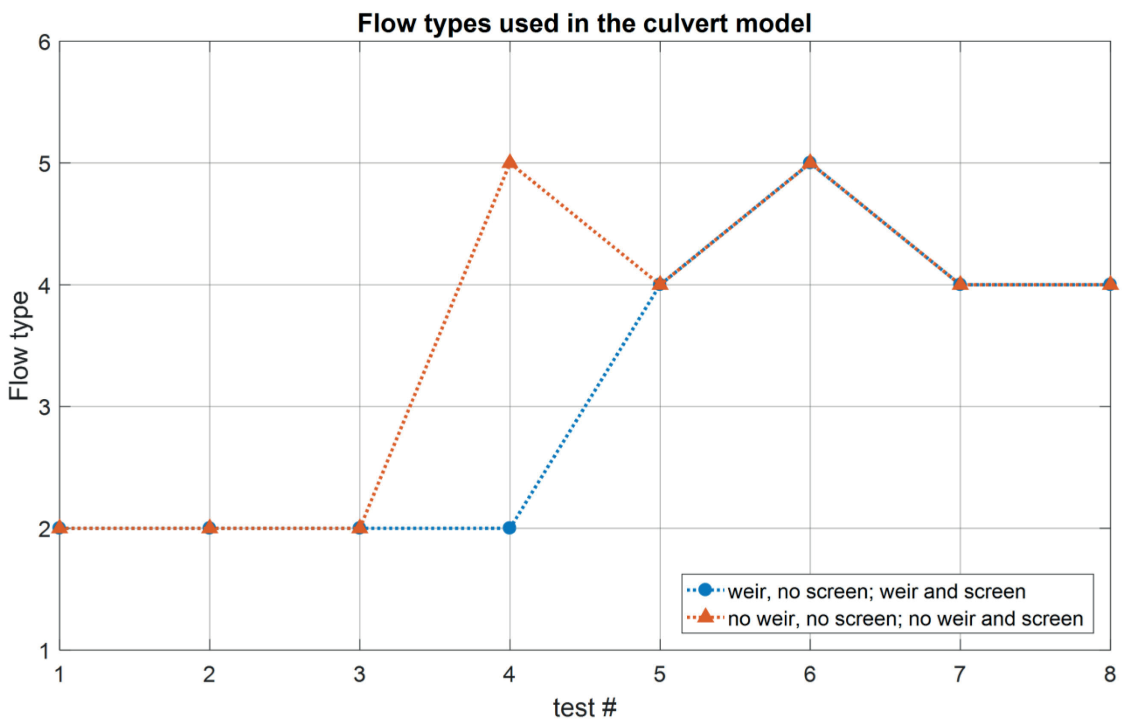


Figure 20 – Comparing the calculated flow types for the four different scale model setups.



4 Conclusions

Four different setups of the inflow culverts of CRT Doelpolder were tested in a flume: without weir logs and trash screen, with weir logs and without trash screen, without weir logs but with a trash screen, and with weir logs and with a trash screen. For each setup eight sets of water levels before and after the culvert construction were imposed on the model. For each test the discharge was calculated based on the water level before a sharp crested weir. The results are shown in this report.

The discharges from the scale model tests were recalculated based on a piece of code to calculate flow through culverts, implemented in TELEMAC (Smolders et al., 2016). The results of these calculations were presented in this report and show in general a good agreement with the measured discharges from the scale model in the flume. The results show that the culvert code, implemented in TELEMAC, can be used to model the water exchange between river and flood control area in TELEMAC.

This report further shows that with a good estimation of the culvert parameters and head loss coefficients a good approximation is found of the actual discharges through complex culvert structures, like the one that is used for CRT Doelpolder. When modelling such an area inside TELEMAC-2D or TELEMAC-3D it is an option to calibrate the set of input parameter with a 13-hour measurement campaign, like in Smolders et al. (2016) and which gives very good results. If such measurements are not available this reports shows that with a good initial estimation of the parameters good results can be obtained.

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