DHI Couplings engine

Scientific Documentation

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PLEASE NOTE

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1 Introduction

This document provides scientific documentation of the DHI Couplings engine. The DHI Couplings engine is a numerical engine built for the purpose of coupling of other numerical engines.

The DHI Couplings engine loads and initializes other engines at runtime; executes time steps; evaluates and carries out exchanges of water; and finally closes simulations and reports on simulation status. The Couplings engine can therefore be regarded as a controller of the other engines.

The DHI Couplings engine is currently used in the following couplings:

- MIKE 1D-MIKE FM
- SWMM-MIKE FM
- MIKE 1D-MIKE1D (for coupling collection system and river models)

2 Links between 1D and 2D models

The DHI Couplings engine controls the exchange of water between 1D and 2D models through four different link types:

- River End/Urban Outlet links
- Lateral links
- Urban links
- Urban-River links

Details for each link type are provided below.

2.1 River End/Urban Outlet links…

A river end or an urban outlet coupling link exchanges water between the end of a river (or an outlet of an urban model) with a 2D surface model, which is typically a Mike21FM model.

From a modelling point of view, the end of the river (or urban outlet) is set up as a water level boundary condition in the 1D model, and the corresponding location is added as a source/sink in the surface model. The exchange mechanism is quite straightforward and can be explained in the following way: if the last computational node of a river node reports a discharge q, that means that a flow q is leaving(entering) the 1D model at the branch end, therefore, the coupling engine will naturally assign an equivalent coupling discharge q into(out of) the surface model, maintaining the mass balance of the model.

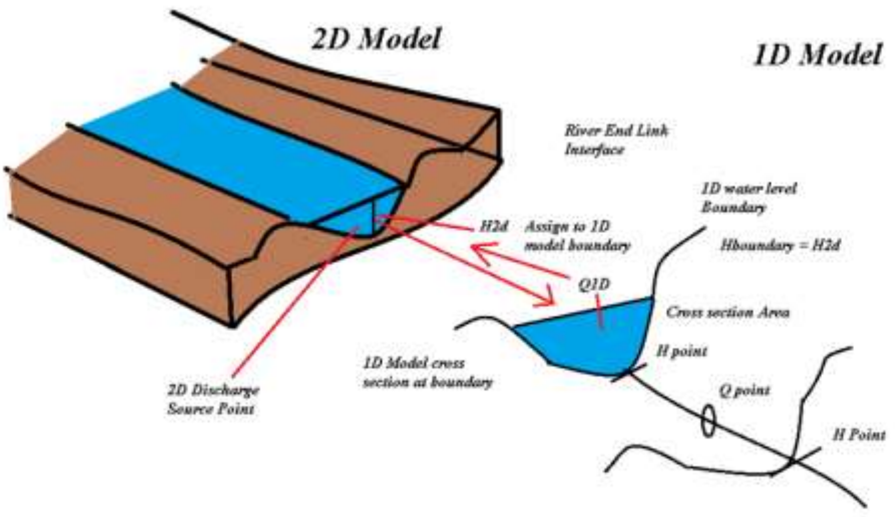

Figure 1: Conceptual diagram of a River End link

The modelling of a river-surface coupling with a river end link requires both a surface and a river model, a specification of the coupling link, and a coordinated way to run the time steps of each model, such that the coupler can keep track of the exchange interactions in the model.

Figure 2 shows how this is properly done: at the beginning of the simulation, the initial water level at the link location in the 2D surface model is computed (h2D). This value is extracted by the coupling engine and passed to the 1D model, making a water level assignment to the 1D water level boundary. The coupling engine then instructs the 1D engine to perform a time step simulation and after this is done, the coupling engine extracts the discharge value q1D at the link location. Since the link location is at an end of the river, this value represents the discharge being exchanged with the 2D model. This discharge is added to the surface model as a source/sink discharge at the link location in the Mike21FM model. The discharge q1D is distributed into one or more element faces in the 2D model and thus, from a Mike21FM point of view, the link functions as a source boundary condition. The coupling engine then instructs the 2D surface model to perform a time step and the coupling engine extracts and passes the water level again, and the process continues for each time step of the coupled simulation.

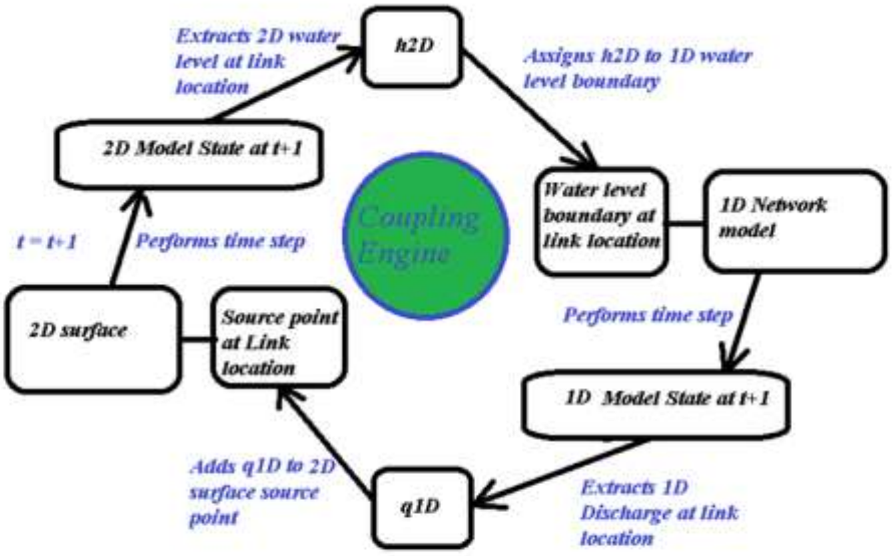

Figure 2: Conceptual diagram of the calculation sequence at a River End or Urban Outlet link

Discharges can be positive or negative, and the sign depends on the perspective of the model in question. A positive discharge from the 1D perspective means that water is leaving the river end (or urban outlet) and spills into the 2D model, whereas that same discharge would be considered negative from the perspective of the 2D model.

2.2 Lateral links

For lateral links, the flow from the river model goes via a lateral boundary, which is then applied onto the 2D surface model. The following is true for lateral links:

- Flow through the link is dependent upon a weir equation and the water levels in the 1D and 2D model.
- Flow through the link is distributed into the 1D computational points (water level points also known as H-points) and several surface model element faces.
- The lateral links do not guarantee momentum conservation in the case of 2D flows crossing a river channel.

A basic lateral coupling structure is required to calculate the flow between the 1D model and the 2D surface model. This lateral coupling structure is typically a weir that represents overtopping of a riverbank or levee with a maximum length so to keep the highest computational efficiency (the larger the weir is, the less number of weirs will be used, and therefore faster computational speed, due to less operations to compute) The geometry of the coupling structure can be determined from cross section bank markers, 2D surface topographical levels, a combination of the highest of each, or from an external file.

For a lateral link, a special discretization of the river extension of the lateral link into short lateral coupling structures is performed. This discretization is performed as a combination of a discretization of each side, the river side and surface side adjacent to the lateral link. The river discretization consists of a division of the lateral link river extension into a series of internal river sections. Each river section corresponds to an H-point in the extension of the lateral link, and it extends until the next Q point (halfway between two H-points), or the limit of the lateral link – whichever one is closer. For an explanation of computational discretization of rivers into H- and Q- points, please refer to the Mike1D reference manual.

Figure 3 provides a conceptual diagram of the discretization of a river lateral link around H-points.

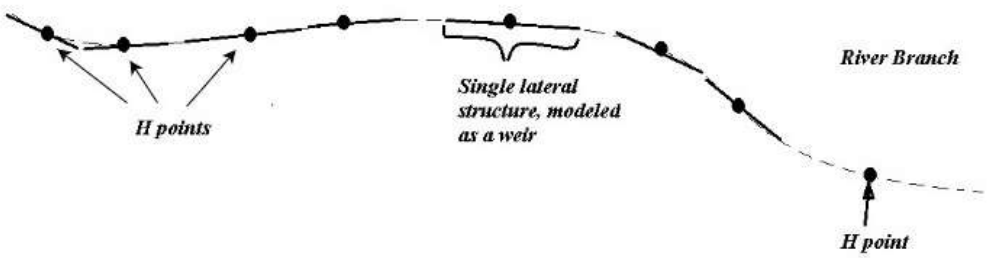

Figure 3: Discretization of river sections along a lateral link

Figure 4 provides a conceptual diagram of 2D model elements along a lateral link.

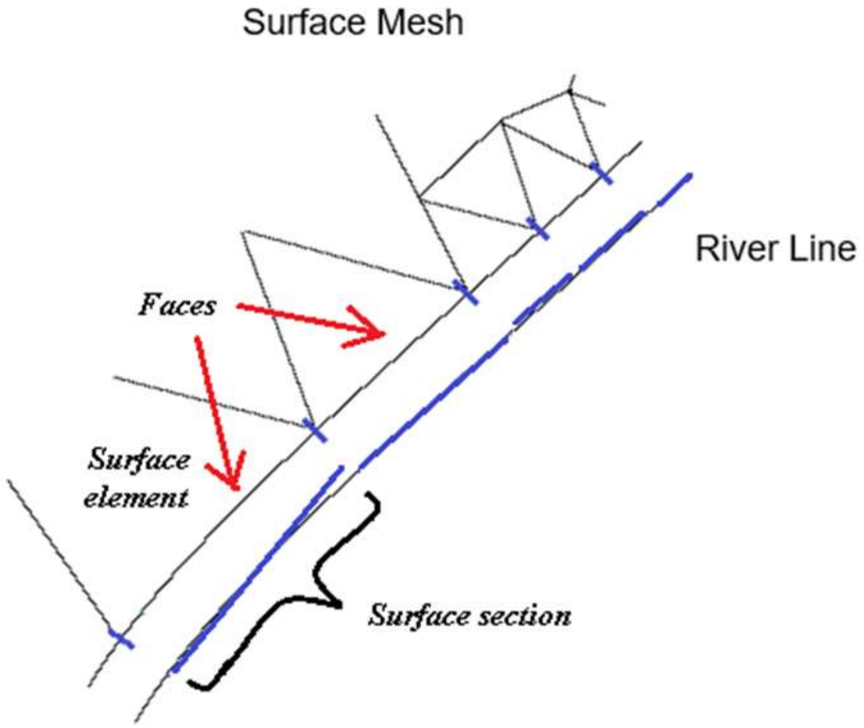

Figure 4: 2D model element faces along a lateral link

Information from the 1D and 2D models is then combined to parameterize lateral coupling structures. The lateral coupling structures have bed levels and widths determined from the resolution of the river and 2D surface discretization defined along the lateral link. The resulting structures are defined such that each structure represents at most one river section and opposing 2D element face. The discretization is outlined conceptually in Figure 5

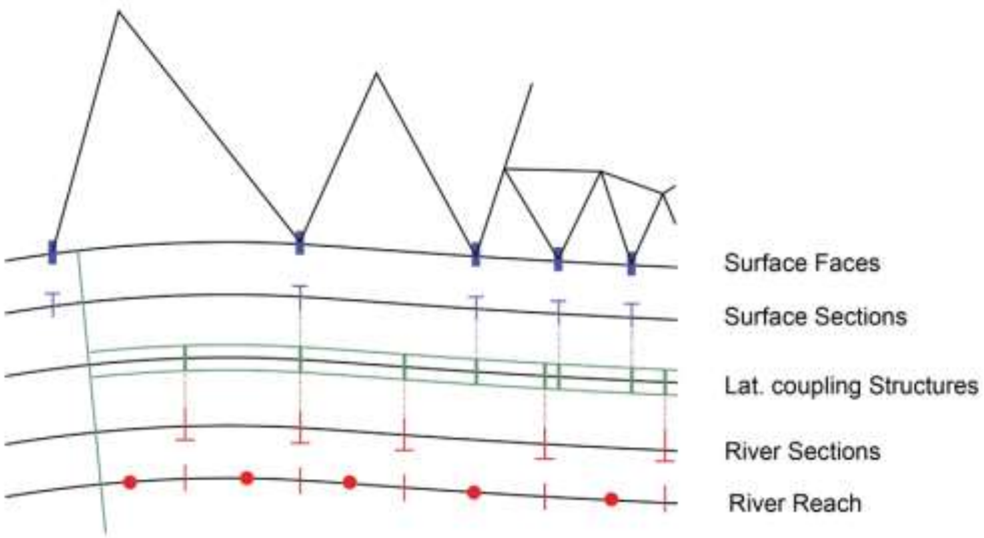

Figure 5: Lateral link coupling structure discretization

During computation, each lateral coupling structure is assigned two water levels, one from the river model and one from the surface model. These values are found by interpolating levels at existing calculation points in both river and surface. The discharge value for each lateral coupling structure depends on the relative comparison between the water levels of each side, and it is calculated according to the structure type parameter as Q = structureType($h1D$, $h2D$), where structureType can be either 1) Villemonte weir, or 2) Honma weir. The discharge through the riverbank structure is computed following the corresponding weir equation.

Figure 6 provides a conceptual diagram of the calculation sequence.

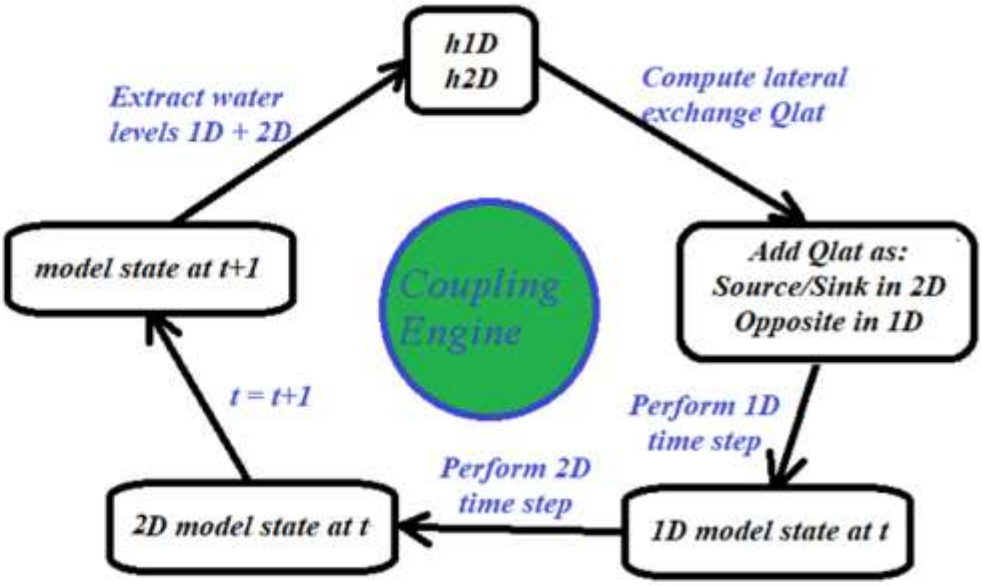

Figure 6: Conceptual diagram of the calculation sequence at a Lateral link

Using the calculated width, bed level and interpolated water levels, the flow across each lateral coupling structure is computed. The flow structure is then distributed to/from the river H-points and the surface faces. This is done by determining the range of influence that a given internal structure has upon each linked river H-point and surface model face. As shown in Figure 5, if river H-points lie within the range of influence of a given coupling structure, flow is distributed across those points according to water depths in each point. The same applies for the corresponding surface element faces. It is important to note that each section related to a lateral coupling structure is associated to one and only one H-point (even if the H-point lies outside of the link extension), and to one and only one element face. Therefore, it will have one and only one water level for surface and river models, and thus the lateral coupling structure discharge can always be computed.

This approach allows for a high level of flexibility when designing the lateral link. It is, however, expected that a similar and uniform distribution of river H-points and surface faces will produce the most accurate solution. This is because when a riverbank link is set up, in principle we don't know where there will be a higher flow of exchange, so a uniform distribution of points will be safer in terms of computational precision. If the locations of the exchange were known beforehand the computational discretization could be concentrated around these areas, but unfortunately this information is unavailable a priori.

2.2.1 Definition of riverbank linkage lines

When a lateral link is created, the coupling module must select the exact location of the interface between the river and surface models. The critical part is the selection of the surface faces, depending on the side of the lateral link, and the chainage limits. The automated face selection process uses either the centre, left levee or right levee line. The face selection only varies in the way the line is defined. The centre line is defined through the digitised points in the river setup. The construction of the levee lines follows through several fully automated steps (see Figure 7).

1. The cross sections located within the user selected reach are placed according to either the geo-referenced coordinates or the chainage. The latter only if the coordinates are not present or applied.
2. If the cross sections are placed according to the chainage (not geo-referenced), then the distance from the centre line depends on the location of markers 1, 2, and 3. If these markers are not present, they are determined from the raw data with marker 2 normally being the lowest value (at the centre line) and markers 1 and 3 being the first and last point of the cross section, respectively.
3. If cross sections are not present at the end points of the user defined reach, then the widths are extrapolated from existing cross section information at the closest location(s) in the same reach.
4. For all digitised points without a cross section the width is interpolated from the existing neighbouring cross sections.
5. The interpolated widths are placed so that they bisect the angle between the two adjoining line segments.
6. The levee lines are thus made up by joining the respective left or right end points of the combined interpolated widths and cross sections.

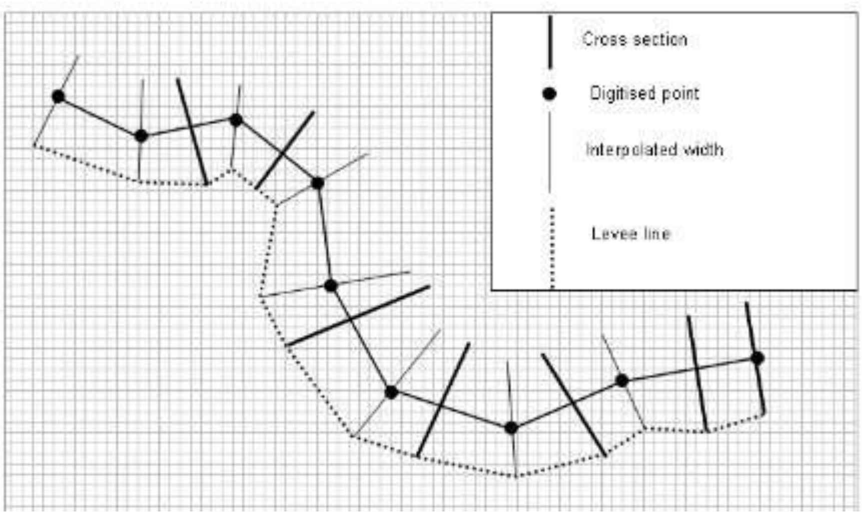

Figure 7: Position of a levee line in relation to the center line and cross section

2.2.2 Automated face selection

The linkage lines are the lines describing the physical location of the water exchange in the surface model, and therefore the line location of the lateral inflows/sinks due to riverbank link exchange. It is important to mention that the individual elements to be coupled are identified at run time and therefore, the linkage line is independent of the element size. The linkage is made up of the coordinates of the end points of the line segments making up the linkage line. At run time the linkage line is mapped onto the element faces.

2.3 Urban links

Urban links refer to the type of coupling link that connects a manhole, inlet, basin, soakaway, or storage unit (SWMM model) with a surface model.

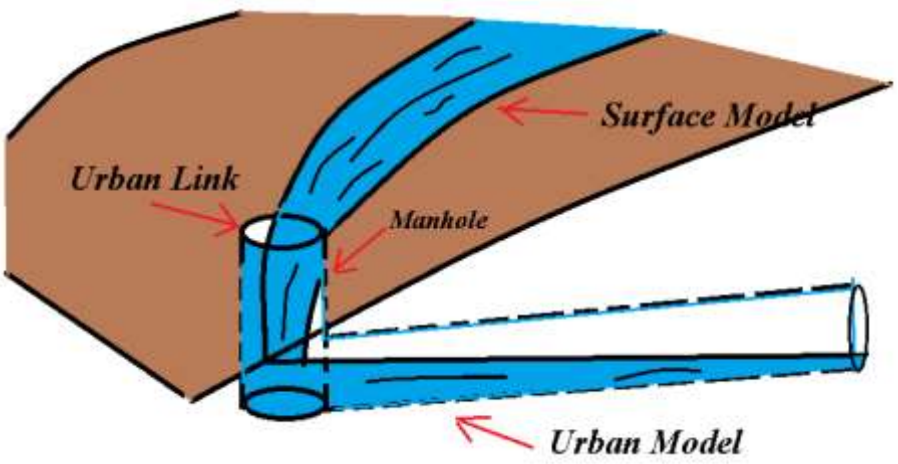

Figure 8: Illustration of a representative urban link (manhole)

The exchange of water (Q_US) at the inlet of either one of these options can be computed in three different ways:

1. Orifice equation
2. Weir equation
3. Exponential function
4. Curb Inlet function:

The orifice equation is given by:

$$Q_{US} = sign(H_U - H_S) * C_D * min(A_m, A_i) * \sqrt{2 * g * |H_U - H_S|}, \qquad |Q_{US}| < Q_{max}$$

Equation 1

Where:

Q_US: Flow from sewer to surface grid point.

H_U: Water level in sewer system.

H_S: Average water level on the ground (among the link faces).

A_m: Cross-sectional area of manhole

A_i: Cross-sectional area of inlet (not used for basins)

C_D: Discharge coefficient (typically 1).

Note that if the node's ground level is greater than either the water level in the node or the water level in the surface then the corresponding water level is overridden by the ground level.

The weir equation depends on whether there is flooding of the surface or not. If the surface is not flooded at the time, then it is calculated as a free-flowing weir:

$$Q_{US} = C_D * (H_U - H_S) * W_{crest} * \sqrt{2 * g * |H_U - H_S|}, \qquad |Q_{US}| < Q_{max}$$

Equation 2

where W_crest is the crest width. The same equation is used if the surface is flooded but the water level in the node is below the node's ground level.

If the surface is flooded and the water level in both the surface and the urban model are above the node's ground level, then the weir is calculated as a submerged weir:

$$Q_{US} = C * (H_U - H_S) * W_{crest} * \sqrt{2 * g * |H_U - H_S|} * \left(\frac{|H_U - H_S|}{max(H_S, H_U) - H_g}\right)$$

Equation 3

where H_g is the ground level at the coupling link location.

The exponential formula for computing the flow between the urban and the surface model is given by:

$$Q_{US} = sign(H_U - H_S) * S * \left|max(H_S, H_g) - max\,(H_U, H_g)\right|^{Exp}, \qquad |Q_{US}| < Q_{max}$$

Equation 4

where S is the scaling factor and Exp the exponent.

Note that in the exponential formula, the values of H are given in meters and the discharge is given in $m\hat{}3/s$. If the discharge is needed in another unit system, first obtain it using m and $m\hat{}3/s$ and then convert, as the exponential formula does not have any meaning using another set of units.

For the curb inlet function, the flow between the manhole and the surface is transferred at a grate or inlet from a surface overland flow network to the sub-surface pipe network. The transfer capacity of the connection is specified as a tabular depth-to-flow (DQ) relation. When the water level of the manhole reaches the ground surface minus a freeboard parameter value, the DQ table is no longer considered and outflow from the manhole is simulated using the orifice equation instead.

For situations where the water levels in the sewer system and the surface model are very close to each other oscillations may occur and for that, a suppression factor can be used to dampen such oscillations. The suppression factor can be applied individually at each urban coupling:

$$SuppressionFactor = 1 - \left(\frac{Qdh - dh}{Qdh}\right)^2, \qquad dh < Qdh$$

Equation 5

Where:

Qdh: the threshold for the difference in water levels, dh.

The suppression factor is only applied if the above inequality holds. The threshold is user specified and if set to 0.0 no suppression is applied.

2.4 Urban-river links

Urban-river links couple collection system and river models.

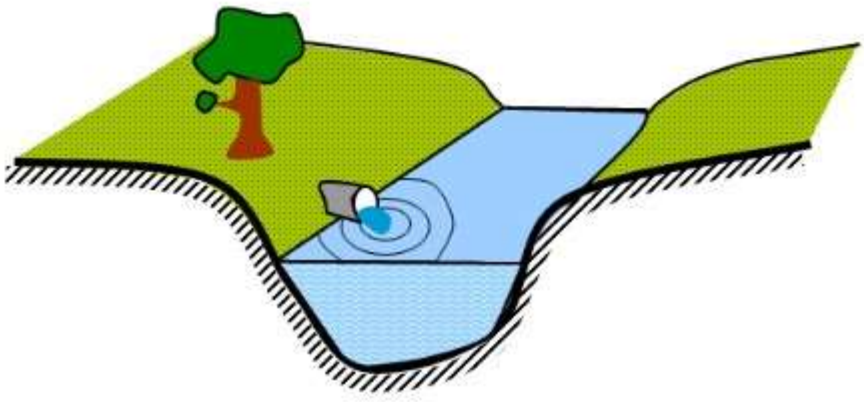

Figure 9: A collection system outlet discharging to a river. This situation is usually modelled using an urban-river link

Urban-river links include both two-way and one-way couplings.

In a two-way coupling, the water level boundary in an outlet of a collection system model is coupled to a location in a river model. In the collection system model, the computational coupling node corresponds to an outlet, a weir, or a pump node, while the computational coupling node in the river is an H-point of the river branch. The mechanism is such that the river model delivers a water level to the urban node and in exchange, the urban model sends a discharge to the river model, which is then added as a source point in the corresponding coupling H-point in the river.

The one-ay coupling does not have the same feedback mechanism (only possible if the urban node is a weir or a pump), so the urban node must compute the coupling discharge without having the river water level passed back.

The default behavior is the two-way coupling as it is a more realistic approach to simulate two interacting sub-models. Two-way coupling links are recommended for most cases, but one-way coupling links could be used when the hydrodynamics of one of the models is independent of the other; however, this rarely the case.

2.5 Hydrodynamics – Exponential Smoothing Factor

The exponential smoothing factor may be used for unstable links, although it will cause a time lag in the corresponding water levels for the coupled models. The exponential smoothing factor works by applying a weight to the transferred water level from the surface model to the river mode.

$$H_r^n = (1 - \alpha) * H_r^{n-1} + \alpha * H_s^n$$

Equation 6

Where α is the smoothing factor, $H_r(n)$ is the water level for the river model at time step n, and $H_s(n)$ is the water level for the surface model at time step n. By applying this expression n times the water level may be given as

$$H_r^n = (1 - \alpha)^n * H_r^0 + \alpha * \sum_{i=0}^{n} (1 - \alpha)^i * H_s^{n-1}$$

Equation 7

Using the above expression, a criterion for selecting the smoothing factor may be obtained. Assuming that the water level in the surface model is constant, then we can expect that after n time steps the river water level at the link location will be within a fraction f of the surface water level (i.e., $H_r \approx f \cdot H_s$), as given by

$$n = \frac{\log f}{\log (1 - \alpha)}$$

Equation 8

Thus, the smoothing factor α should be chosen in relation to the time scale at which the dynamics change in the model (n).

3 Advection-Dispersion

Concentrations of AD components are transferred explicitly between the river and surface depending on the direction of the flow. The surface model handles the coupling exchange of AD components in the following way: for river end links with flow from the river to the surface, the concentration of the AD components in the surface is imposed as with a single inflow source, i.e., as a flux of mass into the surface faces:

$$\Delta\left(\frac{\partial V - C_S^{n+1/2}}{\partial t}\right) = Q^{n+1/2} C_R^n$$

Equation 9

Where $C_S^{n+1/2}$ represents the concentrations in the surface model at the link location for time step $n + 1/2$ and C_R^n is the AD concentration for the river at the link location at time step n.

When the flow direction is from the surface to the river, the modification to the AD equation is:

$$\Delta\left(\frac{\partial V - C_S^{n+1/2}}{\partial t}\right) = Q^{n+1/2} C_S^n$$

Equation 10

It is important to note that the first equation represents an inflow source point in the surface model, where the inflow Q has a concentration given by the river concentration at the link, C_R^n. On the other hand, the second equation represents a sink source point in the surface model (water leaving the surface), where the sink Q has a concentration given by the surface concentration at the link location C_S^n.Another relevant note is the fact that the inflow Q has opposite signs in the two equations.

The corresponding boundary condition in the river can be either a transport boundary or a concentration boundary as usual. Furthermore, a mixing coefficient can be defined as normal. It is strongly recommended to use a transport boundary condition in the river model.

For lateral links the mass of the AD component being transferred is calculated from the lateral discharge and the concentration in the river or surface model (depending on the flow direction). This is then applied as a source or sink term in the branches and the elements The source term into the surface is as for the standard links. For the river model, the one-dimensional advection dispersion equation is:

$$\frac{\partial AC}{\partial t} + \frac{\partial QC}{\partial x} - \frac{\partial}{\partial x}\left(A * D * \frac{\partial C}{\partial x}\right) = -A * K * C + C_2 * q$$

Equation 11

Where:

C: Concentration

D: Dispersion coefficient

A: Cross-sectional area

K: Linear decay

C_2: Source/sink concentration

q: Lateral inflow per river length

4 Flow distribution by depth

Flow is distributed according to the Chezy equation for resistance:

$$Q = A * C * \sqrt{R * S}$$

Equation 12

Where:

Q: Flow.

A: Area ($= width \times depth = wh$).

C: Chezy coefficient.

R: Hydraulics radius (approx. depth = h)

S: Slope of channel/river.

Rearranging the equation yields:

$$Q = whCh^{0.5}S^{0.5} = h^{1.5}wCS^{0.5}$$

Equation 13

Thus, proportionality can be derived between flow and water depth h

$$Q \sim h^{1.5}$$

Equation 14

Flow is then distributed across n surface element faces and for each one, the distributed flow Q_i is calculated from the total flow Q_{TOT}.

5 Inclusion of Friction Term in Weir Formulae

The Villemonte equation (Weir Formula 1) is given as:

$$Q = w * C * h_1^k * \left[1 - \left(\frac{h_2}{h_1}\right)^k\right]^{0.385}$$

Equation 15

where w is the width, C is the weir coefficient, k the exponential coefficient (accepted value is generally k = 1.5), $h_1$is the depth of water above the weir level downstream ($H_{us} - H_w$) and h_2 is the water depth above weir level downstream ($H_{ds} - H_w$). This equation is a free overflow term ($w * C * h^k$) combined with a scaling term for submergence ($[\dots]^{0.385}$) that approaches 0 as $h_1$approaches h_2.

The Honma equation (Weir Formula 2) is given as:

$$Q = w * C * h_1 * \sqrt{h_1}, \qquad \frac{h_2}{h_1} \leq \frac{2}{3}$$

Equation 16

$$Q = w * \frac{3}{2} * \sqrt{3} * C * h_2 * \sqrt{h_1 - h_2}, \qquad \frac{h_2}{h_1} > \frac{2}{3}$$

Equation 17

The first equation is for free overflow (identical to Weir Formula 1 with free overflow), while the second is for submerged conditions. Both equations are derived from the equation for form loss across a structure:

$$\Delta h = \ z * \frac{v^2}{2g}$$

Equation 18

Where v represents the flow velocity. If a friction loss is added this becomes:

$$\Delta h = (z \ + F) * \frac{v^2}{2g}$$

Equation 19

Where:

$$F = \frac{2 * g * L * n^2}{h_1^{4/3}}$$

Equation 20

and n is Manning's number. The length L is calculated as the length between opposite corners in a surface element:

$$= \sqrt{(\Delta X_S)^2 + \ (\Delta Y_S)^2}$$

Equation 21

Using the fact that area = wh_1 as well as Q = area · velocity, we obtain:

$$Q = w * \sqrt{\frac{2g}{z \ + F}} * h_1 * \sqrt{\Delta h}$$

Equation 22

This equation is similar in form to Weir Formulae 1 and 2 for free overflow, except for the weir coefficient C and the additional friction:

$$C = \sqrt{\frac{2 * g}{z}} \Leftrightarrow \mathrm{z} = \frac{2g}{C^2}$$

Equation 23

$$C_f = \sqrt{\frac{2 * g}{z \ + F}} = \sqrt{\frac{2 * g}{\frac{2 * g}{C^2} + F}}$$

Equation 24

So, the existing equations for weir flow calculations can be used, but the weir coefficient is modified according to:

$$C_f = \sqrt{\frac{2 * g}{\frac{2 * g}{C^2} + \frac{2 * g * L * n^2}{h_1^{4/3}}}}$$

Equation 25

This equation is applied to the weir calculations in the lateral link specifications.