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Validation of the STRIVE model for coupling ecological processes and surface water flow

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ABSTRACT

The 1D model package STRIVE is verified for simulating the interaction between ecological processes and surface water flow. The model is general and can be adapted and further developed according to the research question. The hydraulic module, based on the Saint-Venant equations, is the core part. The presence of macrophytes influences the water quality and the discharge due to the flow resistance of the river, expressed by Manning's coefficient, and allows an ecological description of the river processes. Based on the advection–dispersion equation, water quality parameters are incorporated and modelled. Calculation of the water quantity parameters, coupled with water quality and inherent validation and sensitivity analysis, is the main goal of this research. An important study area is the River Aa near Poederlee (Belgium), a lowland river with a wealth of vegetation growth, where discharge and vegetation measurements are carried out on a regular basis. The developed STRIVE model shows good and accurate calculation results. The work highlights the possibility of STRIVE to model flow processes, water quality aspects and ecological interaction combined and separately. Coupling of discharges, water levels, amount of biomass and tracer values provides a powerful prediction modelling tool for the ecological behaviour of lowland rivers.

Key words | ecohydraulics, ecosystem modelling, environmental engineering, Femme, flood routing, vegetated rivers

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INTRODUCTION

The study of integrating ecological processes and surface water flow is situated in a multidisciplinary research, important for a total view on ecosystem development and management. Attention needs to be paid to the interaction of groundwater, surface water and the ecological system in order to describe the transport of matter through river basins (Buis *et al.* 2005).

An integration of the different disciplines is necessary to develop the scientific know-how of ecosystems with also an effective interaction between the different processes, allowing the study of feedback and cascade processes. For this purpose, numerical modelling is a useful tool. Numerical models and studies often consider only a part of the river basin or transport of a limited number of components. However, exchange processes on the basin level ask for a good understanding of land–water areas with special attention to temporal dynamics and spatial heterogeneity ('hot moments' and 'hot spots' according to McClain *et al.* (2003)). The interaction between processes and structures determining the flow of water with dissolved solids and solutes has to be understood. The interaction of physical, chemical and biological processes influence the exchange of water, dissolved solids and particular matter (Fisher *et al.* 1998). In ecosystem studies, not only the river discharge, but also the biochemical processes of the nutrients in the water body are important. The path of these nutrients is connected with the hydrologic variability. Doyle (2005) looked for what discharges were connected with what nutrients were retained. Rivers seem to be important

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corridors for nutrient transport, yet they can also be critical regions where nutrients are removed or transformed (Peterson *et al.* 2001).

An integrated model study of hydraulic, groundwater, biogeochemical and ecological processes is required for the prediction of dynamic ecosystem behaviour, such as retention of matter in a river ecosystem and the associated resilience. However, most of the available models do not allow the integration of surface water flow, groundwater flow and ecosystem processes.

In general, for modelling surface water flow, the groundwater level is taken as a boundary condition. Vice versa, when modelling groundwater flow, the surface water level is taken as a boundary condition. In cases of strong interaction, however, it is useful to couple models. Smits & Hemker (2002) developed a method to couple Duflow, for surface water flow and based on the 1D Saint-Venant equations, and MicroFem (finite elements), for groundwater flow, according to an iterative procedure. This model does not take into account environmental aspects. Whigham & Young (2001) developed a simple water movement model allowing the prediction of the environmental impact of flow scenarios in lowland rivers and their floodplains. The model is a good initial framework, but has its constraints due to its simplification. Querner (1997) combined the regional groundwater flow model Simgro with the surface water flow model Simwat and developed Mogrow, using the simplified Saint-Venant equations (parabolic model) to describe the river flow. Another widely used code is Hec-Ras (Hydrologic Engineering Centers River Analysis System), suitable for 1D, steady and unsteady, surface water flow. Rodriguez *et al.* (2008) describe the coupling of Hec-Ras with Modflow. To study integrated hydrodynamic–ecological modelling, it is not advisable to model complex ecological processes with simplified conceptual hydrodynamic models. In a multidisciplinary approach, different research areas have to be integrated in sufficient detail to properly study the interaction between surface water and vegetation. This is a lack in the existing modelling world. In Prucha (2001), some criteria for code selection are studied in detail.

The coupling of different subsystems and subsystem descriptions forms a methodological challenge. This procedure allows receiving information about a wide range of processes taking place in river ecosystems. We developed a STream–RIVer–Ecosystem package (STRIVE) that enables the construction of integrated river ecosystems to capture cascade effects and feedbacks, along with their effect on retention (Buis *et al.* 2007). This is embedded within the Femme software environment (Soetaert 2002). 'Femme' is a modelling environment for the simulation of time-dependent ecological processes. The program is written in Fortran, is open source and exists of a modular hierarchical structure. 'Femme' consists of a wide range of numerical calculation routines and model manipulations (such as integration functions, forcing functions, linking to observed data, calibration possibilities, etc.). The STRIVE package is similar to the Hec-Ras model for flood routing features, but the novelty is found in the interaction on the time step level with other environmental aspects and modules. Hec-Ras (Hydrologic Engineering Center, River Analysis System) is a free tool developed by the US Army Corps of Engineers (Hydraulic Engineering Center, US Army Corps of Engineers 2004). The program is also based on the Saint-Venant equations, but is not working as an open source.

In the STRIVE package, subsystems of different complexity can be linked to study the dynamic behaviour of water, dissolved and/or particulate matter. The different processes are incorporated in different modules resulting in an integrated model (Buis *et al.* 2007). The following features are incorporated:

- Formulation of geometry of the river (width, bottom slope, etc.).
- Hydraulics of the water system based on the Saint-Venant equations with Manning's coefficient as the imported calibration parameter. This open channel flow module is the core module of the model.
- Transport of dissolved solutes. From upstream to downstream, solutes (Cl^-, NO_3^-, NH_4^+, etc.) are transported by the river. In the study of the solutes, the following parameters are considered: the electrical conductivity (presence of Na, K, Cl and NH_4), Cl, O and minerals (NO_3, NH_4).
- Transport of suspended solids, sedimentation and erosion processes. The transport of sediments by the flow has consequences for the morphology of the river. Concerning the solids; BOD, organic N (N Kjehldahl), detritus (dead organic matter can cause eutrophication) and suspended solids were implemented.

- Macrophyte growth over the year based on temperature and light. Vegetation influences the streamflow and vice versa.
- Reactions in the surface water (algae growth, nitrification).
- Water bottom model with diffusive and advective transport in a vertical way and reactivity of components. It is a connection between groundwater and surface water. Mineralization processes in the bottom and fluxes of nitrate, ammonium, tracers or other components over the edge of the water bottom.
- Output variables are determined, based on the research question. Discharges or water levels, the amount of nutrients in the river, the macrophyte growth and reaction, etc., are potential subjects of interest.

Expertise in the field of geomorphology, hydrodynamics and ecology is generally widespread, but what is critical and typically missing is the integrational aspect of these disciplines, i.e. a synthesis of physical and ecological descriptions in one model structure to analyse land–water interactions. Therefore, this model development is fundamental for integrated stream basin research. This work, in particular, focuses on the influence of the in-stream vegetation on the hydraulic processes.

The presence of vegetation on the riverbed has an influence on the hydrodynamic characteristics of the flow; moreover, a seasonal variation of the vegetation causes variation of the depth of flow and variation of the resistance. Vegetation affects the fluvial processes such as exchange of sediment, nutrients and contaminants (Carollo *et al.* 2006; Schneider *et al.* 2006). The variation of vegetation is expressed as a change in flow resistance characteristics which has consequently a major effect on the flow, i.e. on the hydraulic capacity of the river and the flow velocity profiles.

This paper presents practical examples and applications of the coupled eco-hydraulic modelling. Special attention is paid to the interaction of vegetation and streamflow. At first, the numerical STRIVE model, including a hydraulic module, an ecosystem part and a module calculating water quality aspects, is described. The study area and the performed measurements, to determine boundary and initial conditions, are included in the same section. In the next section, the basic validation of the STRIVE model is performed, with a mass conservation test, a control of system parameters and validation for steady as well as unsteady state conditions. A sensitivity analysis, where the influence of discharge and biomass density on the dispersion of waves and water levels is performed, allows more understanding. Finally, aspects of integrated modelling (tributary inflow, inundations and water quality aspects) are described.

DESCRIPTION OF THE NUMERICAL MODEL STRIVE

Hydraulic module

A 1D hydrodynamic model for unsteady free surface flow based on the Saint-Venant equations has been implemented, yielding accurate modelling of surface flow characteristics, which subsequently has been coupled to ecological processes to achieve the required interaction between the subsystems. River flow is characterised by its variation of discharges and water levels. Studies on this topic have to take into account this non-permanent character of the flow. When dealing with flood waves, time shift and attenuation of the peak of the wave, both due to storage, are the two main characteristics. This is noticable by studying a wave at two different locations in a river as shown in Figure 1. The hydrograph of the wave is shown in section I as well as in the more downstream section II.

The mathematical formulation of this phenomenon, i.e. non-permanent flow of surface water, is expressed by the Saint-Venant equations which include the continuity Equation (1) and the momentum Equation (2):

$$\frac{\partial Q}{\partial x} + B\frac{\partial h}{\partial t} = q \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\left(\frac{\partial h}{\partial x}\right) - S_o + S_f = q \tag{2}$$

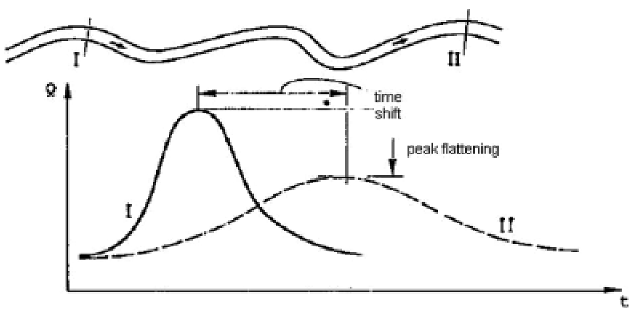

Figure 1 | Propagation and deformation of a hydrograph between two sections of the river.

with Q = discharge [m^3/s], B = section width at water surface [m], h = water depth [m], g = gravity acceleration [m/s^2], A = wetted cross-section area [m^2], $S_o = \tan\alpha$ = bottom slope [m/m], S_f = energy gradient needed to overcome frictional resistance of channel bed and banks in steady flow = friction slope [m/m], q = lateral in- or outflow, discharge per unit length, positive for inflow [m^3/s/m] (Cunge et al. 1980), t = time [s] and x = distance [m].

The friction slope S_f (Equation (3)) is defined by the roughness coefficient of Darcy–Weisbach f [–] (Equation (4)) (Chow et al. 1988):

$$S_f = \frac{fPQ^2}{8gA^3} \tag{3}$$

where

$$f = 8g\frac{n^2}{R^{1/3}} \tag{4}$$

with P = wetted perimeter [m], R = hydraulic radius [m] and n = Manning coefficient [m$^{-1/3}$s].

These equations (Equations (1) and (2)) are the one-dimensional expression (time-averaged and cross-section averaged (Yen 1973) of the Navier–Stokes equations. The integral form of the Saint-Venant equations can be found in the work of Mahmood & Yevjevich (1975), Cunge et al. (1980) and Chow et al. (1988). Here, the differential form is used which assumes that the dependent flow variables (discharge, waterlevel, waterdepth, etc.) are continuous and differentiable functions. The Saint-Venant equations are based upon the following series of assumptions (Cunge et al. 1980):

- The flow is one-dimensional, i.e. the velocity is uniform over the cross section and the water level across the section is horizontal.
- The streamline curvature is small and vertical accelerations are negligible, hence the pressure is hydrostatic.
- The effects of boundary friction and turbulence can be accounted for through resistance laws analogous to those used for steady state flow.
- The average channel bed slope is small so that the cosine of the angle it makes with the horizontal may be replaced by unity.

Solving the Saint-Venant equations for discharge and water level requires boundary conditions and initial conditions. The imposed conditions must reflect the real situation of the river flow that is being modelled (Bates 2005). Therefore, the STRIVE model is validated for one study area (the river Aa in Belgium) where detailed field measurements have been performed (De Doncker 2009). This study is focused on a reach on the downstream part of the Aa over a distance of 1.4 km (Figure 2(a)). The average bottom slope is 0.0002 m/m. The regulation of the water levels is accomplished by use of weirs. Regular measurements of discharge and water level allow us to gather data for the calibration of the model. The sections of the model geometry are represented in Figure 2(b), together with one example of a cross section, i.e. the most downstream measured section (Figure 2(c)). An average water depth of 1 m is measured, while the width of the river is about 15 m.

The study of lowland rivers, which have a rather flat topography and consequently a large gravity part and a low Froude number, implies subcritical flow. Upstream boundary conditions describe the time variability of discharge or water level. Similarly, downstream boundary conditions are a time series for discharge or water level, or a relationship between discharge and water level. For example, the latter can be related to a calibrated weir where the discharge is calculated

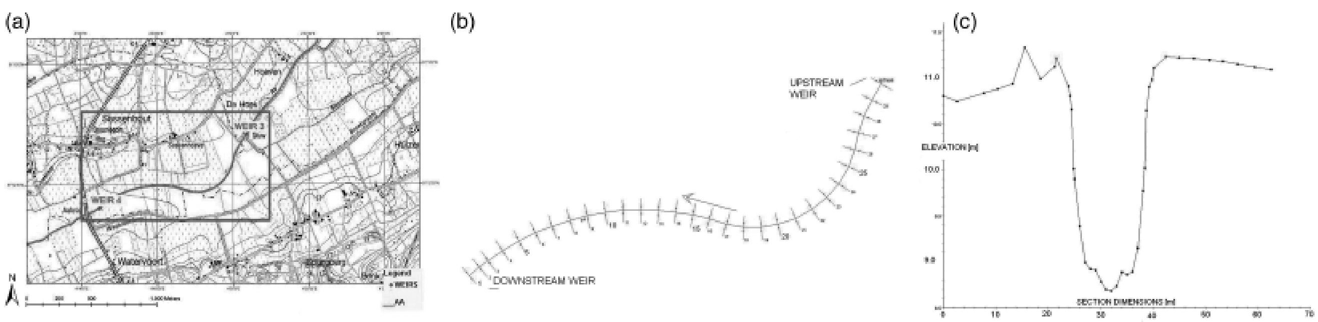

Figure 2 | Study area of the River Aa, Poederlee, province of Antwerp, Belgium (a) and modelled geometry of the River Aa ((b) and (c)): typical cross section and plan view.

from the measured water levels and the weir formula, which relates the water height over the weir with the discharge.

For example, at the downstream section of the reach that will be modelled in section III (cf. study area), the calibration formula of the weir is given by

$$Q = a(Z_{sv} - Z_{cr})^b \tag{5}$$

with Q = discharge [m^3/s], Z_{sv} = water level [m TAW], Z_{cr} = level of the crest of the weir [m TAW], and a and b are coefficients depending on the position of the weir. The formula is only valid for free flow conditions.

After analysis of the values of discharge and water level, measured during during more than 2 years (2005–2006) and comparison with the calibration results of the weir (Van Poucke 1995), reliable values for a, b and Z_{cr} have been obtained.

Ecosystem module

Brock stated that physical, chemical and biological properties of the environment determine if plants can occur and vice versa, these plants influence the environment. Important aspects are the amount of biomass and the production of biomass density [g/m^2]. In our study area, the vegetation type consists of macrophytes characterised by a strong spatial and temporal variation. Macrophytes have influence on the fauna of the river and on the accumulation of organic matter in the bottom. The influence of in-stream vegetation on the environment is large: anorganic carbon (CO_2 and $NaHCO_3$) uptake, uptake and storage of minerals (N, P, K, Na), excretion of nutriments and organic compounds, decomposition of the plants and the material produced by the macrophytes, increase of nutrients due to decomposition, influence on O_2^- concentration, oxidation of the rhizosphere, influence on the water movement, competition for light, influence on the environment temperature, evaporation, etc. (Brock 1988).

The study area is confronted with a healthy growth of in-stream vegetation which influences the flow. The influence of the amount of vegetation (measured biomass density) on the Manning roughness coefficient as an important modelling parameter is shown in De Doncker et al. (2009a, b). Manning's coefficient is represented in Equation (4) and is calculated using the STRIVE model for steady flow based on the Saint-Venant equations and using monthly measurements of discharges and water levels in the studied reach of the River Aa. A relationship between biomass density, discharge and Manning's coefficient has been established and is described in De Doncker (2009) and De Doncker et al. (2009b). Comparison of Manning's coefficient calculated from measurements and Manning's coefficient predicted by the formula showed good agreement (De Doncker 2009).

Modelling of the measurement campaigns (of the order of days), performed in the studied reach in February, April, May and August in 2005 and 2006, is based on a constant amount of biomass.

Figure 3 shows the variation of biomass density over the year and the relationship between biomass density and Manning coefficient for macrophytes in the studied reach of the River Aa. The biomass density increases in spring and early summer and decreases again in the latter part of the year. Lowest values are measured during the winter months. The biomass growth is related to temperature and light (Desmet et al. 2008). A sigmoidal as well as an exponentional relationship, with little difference, are shown. However, the sigmoidal relationship has advantages for numerical applications. While the exponential relationship will result in extreme values of Manning's coefficient for very low and rather high amounts of biomass density, the sigmoidal relationship is a better approximation of the physical processes where the Manning coefficient reaches a threshold value for low and high biomass density values. Indeed, when a certain amount of macrophytes is obtained, the streamflow is fully influenced by the vegetation and the Manning coefficient will no longer increase, not even with an increase of vegetation. The same situation occurs for low amounts of biomass density where the Manning coefficient reaches a limiting value. The Manning coefficient only increases when a certain amount of vegetation in the river is obtained.

Water quality module

The water quality observed in rivers is determined by input and transformation processes. Neglecting the organic pollutants, heavy metals or other toxic compounds influencing the water quality, the functioning is basically determined by the cycle of life. The process of photosynthesis performed by algae or waterplants combines CO_2, water and sunlight into

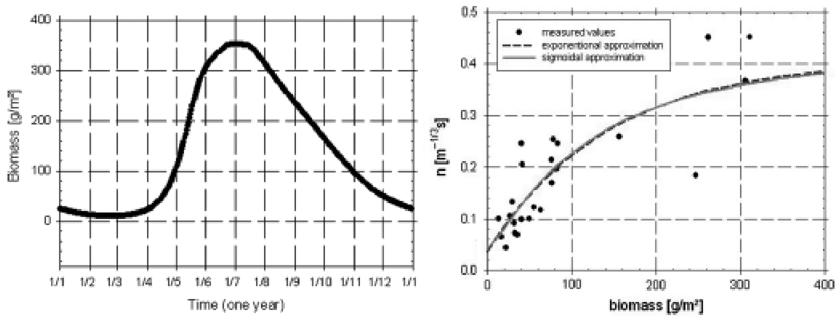

Figure 3 | Variation of biomass density over the year (left) and relationship between Manning's roughness coefficient and biomass density according to a sigmoidal and an exponential approximation (right).

organic matter and oxygen. Modelling water quality means that the components of this reaction (oxygen, pH, organic matter or BOD, and algae or chlorophyll) and the performers of this reaction (algae and bacteria) should be involved (Thomann & Mueller 1997; Allan & Castillo 2007). An extra module, describing the flow of solutes, is incorporated in the STRIVE model. This module describes the transport of tracer or nutrients in the longitudinal direction. The flux of tracer or reactive variables is based on the discharge and the concentration. This leads to updated concentrations based on mass changes in a compartment.

The advection–dispersion–reaction equation (Equation (6)) is the base for water quality modelling, i.e. the transport and reaction of chemicals in surface water (Thomann & Mueller 1997):

$$\frac{\partial C}{\partial t} = -v\frac{\partial C}{\partial x} + D_L\frac{\partial^2 C}{\partial x^2} - \frac{\partial r}{\partial t} \tag{6}$$

where C = concentration in water [mol/kg water], v = water flow velocity [m/s], D_L = hydrodynamic dispersion coefficient [m^2/s] and r = concentration in the solid phase [expressed as mol/kg water]. Considering the right-hand side of the expression (Equation (6)): the first term represents advective transport (transport related to the velocity of the water), the second term represents dispersive transport (fading concentration gradients by diffusion and differences in velocity of the water) and the third term is the change in concentration due to reactions.

Equation (6) states that the change in concentration is the result of a change in input and output (transport) and transformation of the compound (reaction). In this research, tracers are used to check the transport part, i.e. the change in input and output concentration. So chloride and conductivity should be measured, upstream and downstream.

Measurements of discharge, water level, biomass and water quality variables: determination of boundary and initial conditions

The discharge of the River Aa was measured monthly upstream (weir 3) and downstream (weir 4) from the selected reach between September 2004 and April 2007. Velocity measurements are carried out from a bridge and from a boat on several sections along the reach of the River Aa. The method used to calculate the discharge is the integration of the velocity field over the cross section, as is explained in Herschy (1978). An accuracy for the discharge of 2–5% is obtained (De Doncker *et al.* 2008). In general, a limited number (1 or 2) of measurements on each vertical are carried out according to standards, supposing a Prandtl–Von Karman velocity profile. However, this profile is not seen in vegetated rivers (Kouwen 2003), so more intensive gauging in a larger number of measurement points per vertical is needed. 15–20 verticals are measured with 3–8 measurement points at each vertical, depending on the water level.

Two devices are used for measuring the velocity of the water. In the case of open water (no vegetation), hydrometric propellers (type: OTT, C31 Universal Current Meter, with an accuracy of 1% of the measured value) are used. In locations where vegetation might hinder the mechanical functioning of the propeller, electromagnetic instruments (type OTT, Nautilus C2000/SENSA Z300 and Valeport, Type 801, with an accuracy of 1% of the measured value) are applied.

Also, water levels at the weirs upstream and downstream from the reach are registered continuously by the

Hydrological Information Centre of Flanders Hydraulics Research (HIC) using a limnimeter. These water levels were used to calculate the energy slope and to check the influence of the aquatic plant growth on the water level.

The absence of an univocal relationship between discharge and water level at a gauge (HIC) is attributed to seasonal changes in the amount of vegetation, quantitatively represented by the biomass density, and to the presence of the movable weirs. A weir with a well-known calibration formula overcomes this problem (De Doncker *et al.* 2008).

The evolution of macrophyte biomass density in the study area of the River Aa was monitored by quantitative sampling on a monthly base. Samples were collected at three locations along the river reach: upstream, in the middle and downstream. At each location 10 sampling points were randomly selected from a 14 m by 7 m grid and, per point, the aboveground vegetation was sampled from a circular plot of 0.221 m^2 using a mowing device (Marshall & Lee 1994). This instrument has no moving parts; its primary components are a cutting blade to the base of a vertical shaft to shear off plant stems at the substrate surface and a collection rake to allow retrieval of the freed vegetation. The sampler is well suited for the measurements in this study, because a large variety of macrophytes over a range of conditions can be sampled. Also, the instrument is lightweight and easily handled. The fresh vegetation samples were stored in plastic bags and transported to the lab for cleaning (removing debris and mud) and sorting. Finally, all fresh samples were weighed, dried at 70°C and weighed again. Based on fresh and dry weight values the fresh and dry macrophyte biomass density [g/m^2] could be assessed.

Upstream and downstream of the river reach basic water quality characteristics were permanently monitored and water samples were taken at regular time intervals over a 48 h period. The monitored parameters are water temperature (°C), electric conductivity (μS/cm), pH and dissolved oxygen (mg/l, % saturation). For these parameters measurements were automatically recorded at 5 min intervals by multiparameter monitoring instruments (Hydrolab DS3 and YSI 600XLM probes) (De Doncker 2009). Intensive water sampling occurred at 2 or 3 h intervals, conditional upon the water travelling time. Samples were taken less frequently at low discharges (in summer). The water samples were stored in cool boxes and transported to the laboratory as soon as possible. In the laboratory, concentrations of ammonium, nitrate, phosphate and chlorine were determined in the water samples.

BASIC VALIDATION OF THE STRIVE MODEL

In this section the validation of STRIVE is carried out, showing that it is a sound simulation model. A mass conservation test is performed (see the next subsection) and the sensitivity of the system parameters is checked in a channel of 1400 m with a flat bottom (see the second subsection). This theoretical case is used to show that the model works well. Further, as much as possible, the geometry of the River Aa is used (for steady state conditions and for lateral in- or outflow). For unsteady state conditions, the reach is extended to clearly show the studied effects. Also for the sensitivity analysis, for the same reason, a longer reach is used compared to the study area.

Mass conservation test

To check the mass conservation principle, an artificial river channel with a flat bottom and a length of 1400 m is used. The cross section is rectangular and has a bottom width of 5 m. The channel is split into two parts of equal length: the first part has an initial water level of 2.20 m, while in the second part the initial water level is 1.80 m. The upstream and downstream boundaries are closed, so there is no flow into the reach. The Manning coefficient is first set at $0.01 m^{-1/3}$ s and at $0.1\ m^{-1/3}$ s in the second set of calculations. For the higher Manning coefficient, corresponding to summer conditions, it takes about 4 h to come to a stable situation while, for the winter conditions, it takes about 8 h. In the center node, there are large variations of the discharge but small variations for the water level. Finally, the criterion of mass conservation is fulfilled as, at the end, the water level equals 2.0 m over the entire channel reach (see Figure 4).

Influence of system parameters

The numerical solution of the Saint-Venant equations depends on the physical situation, but also on the system

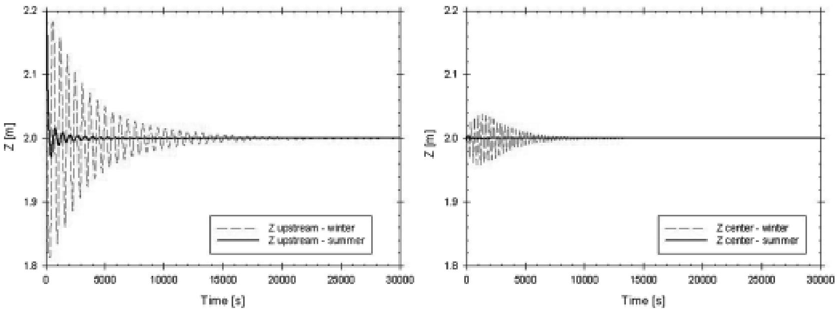

Figure 4 | Upstream water level (left) and water level in the middle of the reach of 1400 m (right) for summer ($n = 0.1$ m$^{-1/3}$ s) and winter ($n = 0.01$ m$^{-1/3}$ s) conditions.

parameters. Here, the effect of spatial and temporal resolution is analysed.

Number of nodes

The number of nodes depends on different aspects. The number has to be sufficiently high to avoid too large calculation cells which are not able to simulate all physical aspects. Also the initial condition is important. The better the initial values agree with a realistic start condition, the fewer boxes are necessary. A small example confirms this statement. A surface water profile is calculated for summer ($n = 0.1$m$^{-1/3}$ s) and winter ($n = 0.01$ m$^{-1/3}$ s) conditions, with a downstream water level of 2 m and a discharge of 20 m^3/s. The artificial river channel as described before (length 1400 m, width 5 m) is used. Using the Bresse equation results in an upstream water level of 2.43 m for winter situations and 7.05 m in summer conditions (theoretically supposing that the river banks are sufficiently high). The same calculation is carried out using STRIVE with a variation in the number of nodes (5, 10, 15,…, 50). For the winter situation, the value of 2.43 m is obtained in any case, due to the small difference between the initial and final situation. For the summer situation, it seems that at least 25 nodes are necessary to come to an accurate result. It has to be added that this example assumes permanent flow.

Timestep

Certainly in non-permanent situations, the time step will be of great importance. In numerical modelling, a good choice of timestep Δt and cell size Δx is necessary. These values have to be sufficiently small to not miss any effects (e.g. peak discharges) and to be sufficiently large to minimize the calculation time. The Courant–Friedrichs–Lewy condition (CFL, Cunge *et al.* 1980) determines a relationship between the time step and the grid size to solve the partial differential equations in a convergent way. For an explicit scheme, it means that the solution will be numerical stable if the CFL condition is fulfilled.

This condition can be avoided by using an implicit scheme (such as the Preissmann scheme) for solving the Saint-Venant equations. Time steps can be taken as being larger, which is useful for long simulation periods, keeping the solution stable.

The calculation carried out in the 'mass-conservation' paragraph is repeated for different time steps (1 s, 10 s, 20 s and 50 s). The water level variations need a larger period to stabilize for smaller time steps so, for this aim, smaller time steps are not necessary, while too large time steps cause instabilities. On the other hand, to simulate specific effects, time steps have to be adapted. For example, when taking into account ecological processes such as conductivity, the time step has to approximate the natural physical process of transportation and dispersion in the river.

Validation for steady state conditions

First, the problem of steady turbulent open channel flow is studied. The results of the STRIVE model are compared with analytical calculation results based on Bresse's equation and with numerical results of the program Hec-Ras.

The water surface profile can also be calculated analytically by the Bresse equation which is the simplification of the Saint-Venant equations for steady flow. This is the case when the calculation of the water surface profile is concerned (Q is

Table 1 | Calculation of surface water profiles (steady state)

	S_0 (m/m)	Width (m)	Length (m)	Angle (°)	Z_{up} (m)	Z_{down} (m)	n (m$^{-1/3}$ s)	Q (Hec-Ras) (m^3/s)	Q (Bresse) (m^3/s)	Q (STRIVE) (m^3/s)
1	0	15	1350	0	1.094	0.56	0.1	2.15	2.14	2.14
2	0	15	1350	30	1.094	0.56	0.1	2.25	2.24	2.24
3	0.0002	15	1350	0	1.094	0.56	0.1	2.83	2.82	2.81
4	0.0002	15	1350	30	1.094	0.56	0.1	2.97	2.96	2.95

a constant value). In the most simple form, in steady state conditions and for uniform flow, the Bresse equation is known as the Manning equation.

Several calculations in permanent conditions have been carried out to evaluate the impact of model parameters. The river has a constant rectangular or trapezoidal cross section. Earlier calculations showed that an average geometry is sufficient for calculation of the Manning coefficient and the water surface profile (De Doncker *et al.* 2005). Further, the slope of the river has been changed and two cases have been considered; a horizontal slope of 0 m/m and a slope of 0.0002 m/m. The width of the river is 15 m and the length is 1350 m. The calculations are carried out with a constant water level upstream (1.094 m) and downstream (0.56 m) as boundary conditions. Keeping the Manning coefficient value constant at a value of 0.1 m$^{-1/3}$ s, the three codes (Hec-Ras model, Bresse equation and STRIVE model) were used to calculate the discharge for both a rectangular and a trapezoidal cross section (Table 1). For a rectangular cross section, discharge values of 2.14 m^3/s (bottom slope 0.000) and 2.82 m^3/s (bottom slope 0.0002) are obtained for each of the three ways of calculation. Similar results for both discharges and evolution of the water surface profile are obtained for the trapezoidal section (Figure 5). By this, it is shown that the Saint-Venant equations, as implemented in the STRIVE model for flow in permanent conditions, deliver accurate results for different values of the bottom slope and bank slope of the cross section.

Validation for unsteady state conditions

Propagation of waves

The propagation of a triangular hydrograph in a 10,000 m long channel with rectangular cross section as shown in Figure 6(a) (bottom width of 10 m) and a zero bottom slope is modelled in STRIVE using the Saint-Venant equations. The Saint-Venant equations use both boundary conditions (zero upstream and downstream discharge) and need well-balanced initial conditions. The initial water level is 1 m and the initial discharge is 0 m^3/s. Calculation of the surface water level for permanent steady state flow over the total length of the channel can be a good start and can yield initial conditions. Figure 6(a) shows the results at different sections when the Manning coefficient is kept constant ($n = 0.05$ m$^{-1/3}$ s). In Figure 6(b), the influence of this friction coefficient in a specific section ($x = 2990$ m) can be seen. It is indicated that wave propagation is modelled accurately,

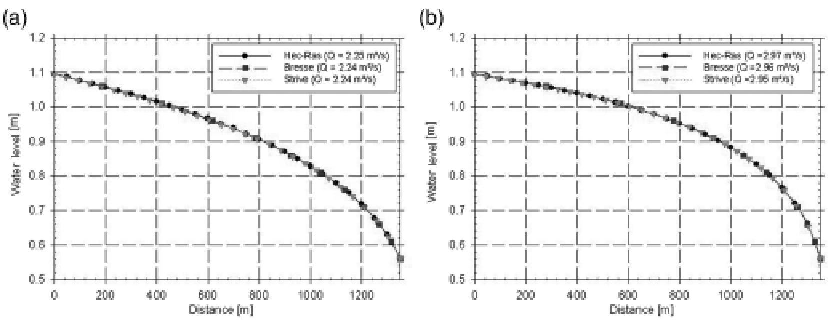

Figure 5 | Calculation of surface water profiles: for a channel with a trapezoidal cross section (angle of 30°) and bottom slope 0 (a) and 0.0002 (b).

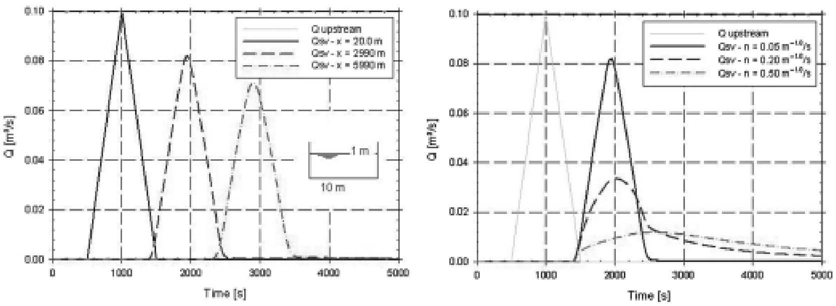

Figure 6 | Numerical results of wave propagation by use of the Saint-Venant equations: result at different locations (left) and result at a single location ($x = 2990$ m) for different values of the Manning coefficient n (right).

including the influence of the roughness coefficient. The results stress the reliability of the STRIVE model. Time shift and peak flattening are seen in both Figure 6(a, b), respectively.

Comparison with analytical solution

A tidal wave in an open channel is modelled. An analytical solution for this problem is described by Ying & Wang (2008) and Bermudez & Vazquez (1994) and is depicted in Figure 7 in comparison with the results of the numerical solution. The figures show the water level and the velocity over the entire length of the reach. The values are results after 7000 s of calculation.

The bed elevation, $Zb(x)$, with a negative slope, is defined by

$$ZB(x) = 10 + \frac{40x}{L} + 10\sin\left(\pi\left(\frac{4\mathrm{x}}{\mathrm{L}} - \frac{1}{2}\right)\right) \tag{7}$$

with $x =$ the coordinate along the river channel and $L = 14{,}000$ m, the channel length. The water level is $Z(x, t)$, so the initial condition is denoted as $Z(x, t = 0) = 60.5$ m and the velocity is $V(x, t)$, with $V(x, t = 0) =$ 0 m/s as the initial condition.

The boundary conditions are

$$Z(0, t) = 64.5 - 4.0\sin\left(\pi\left(\frac{4t}{86400} + \frac{1}{2}\right)\right) \tag{8}$$

$$Q(L, t) = 0.0. \tag{9}$$

The analytical solution is given by Bermudez & Vazquez (1994):

$$Z(x, t) = 64.5 - 4.0\sin\left(\pi\left(\frac{4t}{86400} + \frac{1}{2}\right)\right) \tag{10}$$

$$V(x, t) = \frac{(x - L)\pi}{5400h(x, t)}\cos\left(\pi\left(\frac{4t}{86400} + \frac{1}{2}\right)\right) \tag{11}$$

with h [m] = waterdepth.

In both figures, the numerical solution is in good agreement with the analytical solution, which confirms the good functioning of the numerical model. It is capable of accurately predicting water surface level and flow

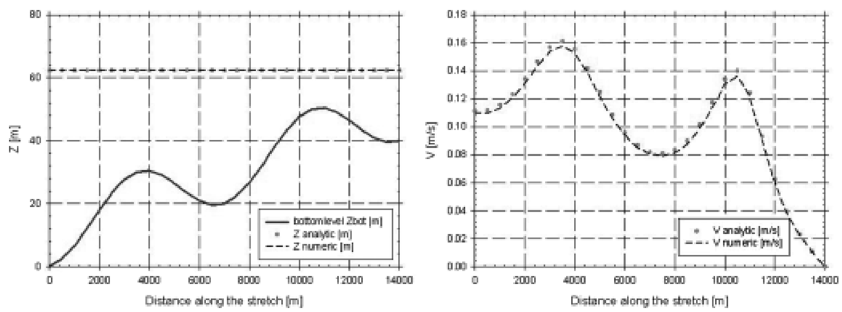

Figure 7 | Analytical and numerical solution of the tidal wave problem: water level (left) and velocity (right) over the entire reach of 14 000 m at time $t = 7000$ s.

velocity. Comparable results are mentioned by Ying & Wang (2008).

Sensitivity analysis

In this subsection, all calculations use an upstream hydrograph $Q(t)$ according to Anderson *et al.* (2006). The resulting hydrograph at the downstream boundary is calculated as well as the water levels along the reach. The total length of the channel is 5000 m. The channel is rectangular, has a bottom width of 12 m and a bottom level of 8.89 m. The cross-sectional characteristics and the range of the values of discharge and water level are derived from the Aa data. The length of the reach is extended to show clearly the effects and influences in the sensitivity analysis.

Influence of discharge and biomass density on celerity and dispersion of waves

Figure 8 shows results for different hydrographs, indicated as $Q_1(t)$ and $Q_2(t)$, with a ratio Q_2/Q_1 as mentioned in Figure 9(b). $Q_1(t)$ has a peak discharge of 2 m^3/s while $Q_2(t)$ reaches peak values of 4 m^3/s. For both, the base flow equals 1 m^3/s. The amount of biomass is expressed by the Manning coefficient. Indeed, analysis in the River Aa showed the relationship between the amount of biomass and the Manning coefficient (De Doncker *et al.* 2009b); in the River Aa, 40 g/m^2 corresponds to a Manning coefficient of 0.1 $m^{-1/3}$ s, while 0.4 $m^{-1/3}$ s is linked to an amount of macrophytes of 400 g/m^2. The upstream hydrograph is a fixed boundary condition and the downstream discharge values are mentioned for comparison. For both hydrographs, it seems that the wave celerity (velocity by which a disturbance travels along the flow path) is smaller and the dispersion (tendency of the disturbance to disperse longitudinally if it travels downstream) (Chow 1959) is larger for higher amount of biomass (higher Manning coefficients, higher roughness). Furthermore, the wave celerity is larger when the discharge increases. This is according to the continuity equation, agrees with larger celerities in streams with larger water levels (Verhoeven 2006) and corresponds with the larger backwater effect for larger roughness coefficients. Not only is the larger dispersion an effect of the larger vegetation growth but also the slower decrease of the peak value of the wave is due to the higher resistance.

Table 2 presents the comparison of downstream discharges for different amounts of vegetation. The value of the peak discharge is mentioned as well as the time after which the peak value occurs.

Table 2 | Comparison of downstream discharges for different amounts of vegetation

	Q_{up} (m^3/s)	Q_{down} ($n = 0.1$) (m^3/s)	Q_{down} ($n = 0.4$) (m^3/s)
Peak			
Q_1	2	1.898	1.779
Q_2	4	3.844	3.481
Time			
Q_1	0	6 h	11 h
Q_2	0	4 h	9 h

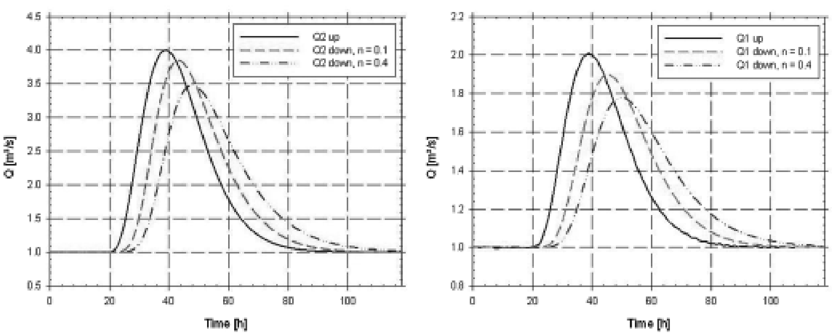

Figure 8 | Influence of discharge and biomass density ($n = 0.4$ $m^{-1/3}$ s and $n = 0.1$ $m^{-1/3}$ s) on celerity and dispersion of waves.

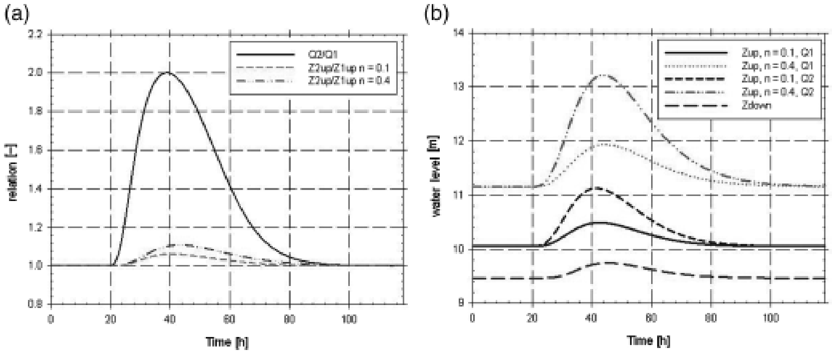

Figure 9 | Influence of discharge and Manning coefficient ($n = 0.4$ m$^{-1/3}$ s and $n = 0.1$ m$^{-1/3}$ s on water levels. (a) shows the water level for varying discharge and Manning coefficient as a function of time. (b) depicts the relation between the discharge Q1 and Q2 used as upstream boundary condition and the resulting relation between the upstream water levels for different Manning coefficients.

Influence of discharge and biomass density on water levels

Figure 9(a) depicts the upstream water level for different discharges (hydrograms $Q_1(t)$ and $Q_2(t)$ as above) and for different values of the Manning coefficients. A higher discharge results in higher upstream water levels and the effect is even larger for higher values of the Manning's coefficient (Figure 9(b)). The backwater effect is higher for more healthy vegetation growth (higher Manning's coefficient) and for higher discharges. It can be seen that peak flows (higher discharge) in summer situations (more vegetation and therefore higher resistance described by a higher Manning coefficient) can cause dangerous situations. In the case of low heights of the dikes, inundations will occur, due to higher water levels. It is seen that a combination of high discharges and high values of Manning's coefficient has an important influence on the water levels.

INTEGRATED MODELLING ASPECTS

Combined influence of discharge and biomass density

Steady state conditions

The influence of the in-stream vegetation (biomass density) is represented by the value of the Manning coefficient (Figure 3). Figure 10 shows the situation in a theoretical reach of 5000 m. The cross section is rectangular and has a bottom width of 12.0 m. There is no slope along the reach and the bottom level is 8.89 m.

Two cases are considered (Figure 10(a)), a lower Manning coefficient of 0.1 m$^{-1/3}$ s, which corresponds to the values in the winter for the River Aa and a higher Manning coefficient of 0.4 m$^{-1/3}$ s (spring and summer values). The Manning coefficient is up to 9 times (0.05–0.45 m$^{-1/3}$ s) higher in spring

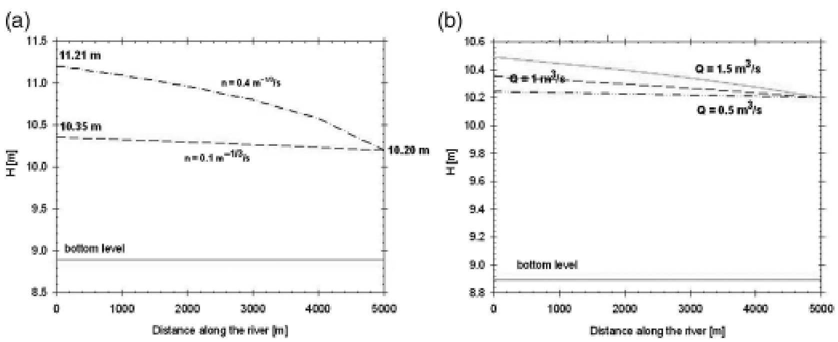

Figure 10 | Backwater influence of the Manning coefficient on the water level for a given discharge of 1 m^3/s (left, (a)) and backwater influence of the discharge on the water level for a given amount of vegetation ($n = 0.1$ m$^{-1/3}$ s) (right, (b)).

when there is a wealth of vegetation (De Doncker *et al.* 2006). Starting from the same downstream water level (10.20 m) and using a discharge of 1 m^3/s, the upstream water level is calculated for both values of n. In spring, a value of 11.21 m for the upstream water level is calculated, while 10.35 m is obtained in winter; this is a difference of 0.90 m due to the presence of vegetation.

Figure 10(b) shows the influence of the discharge on the energy slope S_f. For three different values of the discharge (0.5, 1 and 1.5 m^3/s), the water surface profile is calculated. The Manning coefficient is kept constant at 0.1 $m^{-1/3}$ s. It is shown that tripling the discharge results in an increase of the water level of only 0.26 m (10.24 m for the lowest discharge, 10.35 m for $Q = 1$ m^3/s and 10.50 for the highest value of Q). So, the impact of the vegetation on S_f is much bigger and explains why dangerous situations may occur with regard to inundation during summer floods.

Unsteady state conditions

The impact of a variable amount of vegetation on the stream flow is illustrated. A flood wave, registered in the studied reach of the River Aa, in the period from 12–19 August 2005 is used as the upstream boundary condition for the calculation ($Q_{upstream}$). The downstream boundary condition is the registered water level at the downstream weir ($Z_{downstream}$). The simulated reach has a length of 5000 m, a rectangular cross section and a bottom width of 12.0 m. The bottom level is 8.89 m. Figure 11 depicts the boundary conditions and the calculated results for $Q_{downstream}$ and $Z_{upstream}$ for two values of the biomass density: in winter conditions ($n = 0.1$ $m^{-1/3}$ s) and in summer conditions ($n = 0.4$ $m^{-1/3}$ s). The variation, with the Manning coefficient, of the downstream discharge is limited, but it is clear that the upstream water level ($Z_{upstream}$) is strongly influenced by the dense vegetation growth during summer. With increasing values of the Manning coefficient, the downstream discharge peak shows a small time lag and a substantial attenuation.

In Figure 11(b), the river banks are indicated. For low vegetation growth, the peak discharge and corresponding water level cause no problems. For higher values of the Manning coefficient, the river banks (11.6 m) will be too low for the peak discharge and neighbouring areas will inundate. Therefore, a good knowledge of the impact of biomass density on the roughness of the river is important in building river flood simulation models able to produce reliable results for all seasons of the year. Consequently, knowing the maximum allowable flood water level, it becomes possible to determine the value of the Manning coefficient and by this the amount of biomass that can be kept in the river to safely convey a given flood wave. As can be seen from Figure 11, reducing the n value to 0.205 $m^{-1/3}$ s (which corresponds to an elimination of 75% of the biomass density) keeps the flood wave under concern within the banks of the river. In this way, it becomes possible to define a 'safe' biomass management strategy.

Tributaries: study of lateral outflow

In the studied reach of the River Aa, there are a few small tributaries. Some of them are only dry canals, not carrying water most of the time. One of them, however, is a small

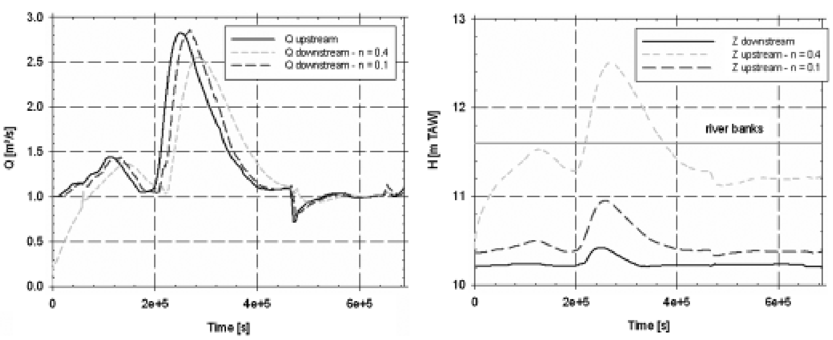

Figure 11 | Boundary conditions $Q_{upstream}$ and calculated $Q_{downstream}$ for $n = 0.4$ $m^{-1/3}$ s and $n = 0.1$ $m^{-1/3}$ s (left) and boundary conditions $Z_{downstream}$ and calculated $Z_{upstream}$ for $n = 0.4$ $m^{-1/3}$ s and $n = m^{-1/3}$ s (right).

creek and adds, just downstream of the upstream weir, a certain amount of water to the main river. Therefore, it is necessary to add the possibility of lateral (in- or) outflow in the model.

Two possibilities are incorporated: first, the case of lateral outflow at a certain location and, second, distributed in- or outflow over a certain distance along the river (e.g. groundwater in- or ouflow). Further, this lateral connection introduces the aspect of water exchange between river and inundation areas.

A measured flood wave (period from 12–20 August 2005) is chosen as the boundary condition. The measured upstream discharge is the upstream boundary condition and the calibration formula of the weir is the downstream boundary condition. Similar results are seen with other types of boundary conditions (e.g. use of water level upstream, etc.). Further, the geometry of the Aa is used for the modelling. A measured depth, which includes a varying bottom profile, and an averaged width (i.e. the same width for all the sections) is used. Figure 12(a, b) shows the discharge and water level respectively at different sections along the River Aa. These figures have to be compared to both Figure 13(a, b).

Figure 12(a) shows the discharges over the reach at seven sections (distributed over 1350 m). Time shift and peak flattening of the discharge is only small, while the water level decreases from upstream (section 1) to downstream (section 30) (Figure 12(b)). In Figure 13(a), a lateral outflow at location $x = 800$ m (node 18) is implemented. Consequently, the water level is also lower over the entire reach (Figure 13(b)). Looking to the graphs of the discharges, it can be seen that extracting a discharge of 0.01 $m^3/s/m$ along one cell leads to a decrease in discharge of almost 0.5 m^3/s, which is as expected because cross sections are taken every 50 m (corresponding to the distance of a 'cell' or 'box' in the model). Comparing the water levels, it seems that, due to the lateral extraction of water, a general decrease in water level can be remarked over the entire reach. When the volume of water decreases, the water level will also decrease.

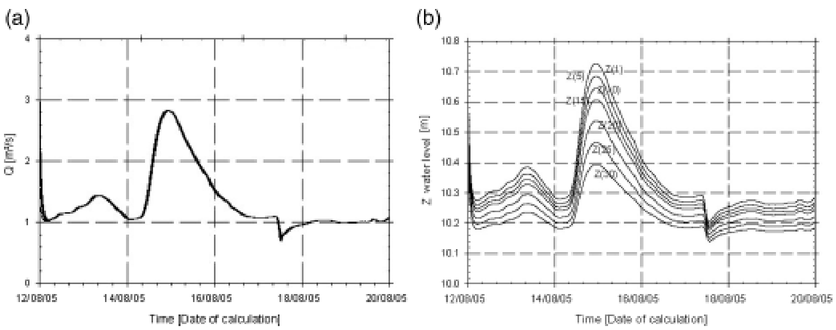

Figure 12 | Calculation of discharge (a) and water level (b) at different sections of the River Aa. The discharge in seven sections hardly changes, while the water level decreases from upstream (1) to downstream (30).

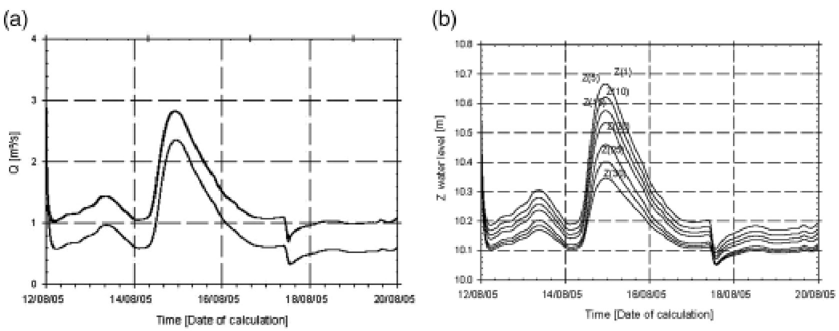

Figure 13 | Calculation of discharge (a) and water level (b) at different sections of the River Aa, extracting a lateral outflow at a distance of 800 m (section 18). The discharge decreases in all sections downstream from section 18, while the water level decreases over the entire reach.

Flooding: study of inundation areas

The river water is stored in the main channel as long as the water level does not exceed the river bank crest. During a flood, the water flows laterally over the banks into the floodplains. When the river is saturated the water is stored in the external floodplain. The storage in the floodplain causes flattening of the peak discharges in the wave. During periods of lower discharge, this volume of water slowly returns to the main channel, so the water stored externally flows laterally from the floodplains to the main channel. The total volume of the water flow is consequently spread out in time.

The described physical process is implemented in the numerical model. The water flow between a river and its floodplain ('storage cells') is modelled by a weir. A calibration of all the banks along a river (all the 'weirs' from the river to its storage cells and between those cells) is an intensive task and therefore a disadvantage of the method. Besides, these banks change all the time due to the processes of erosion and sedimentation. By using this technique, one can simulate floods within all kinds of configurations, including those with multiple storage cells connected to each other. So, the one-dimensional model is extended to a quasi-two-dimensional model by studying the interaction between the main channel and the floodplain. Storage cells reflect the retention and storage capacity of the floodplains. The water flow between the main channel and storage cells is modelled by weirs (Troch 1991; Van Lysebettens 2006). The storage cells, as an internal boundary condition, are implemented in the Saint-Venant equations (Declercq 2007). This set of nonlinear differential equations is converted into a set of linear difference equations using the implicit differential scheme of Preissmann. The coefficients in the expressions of the continuity equation and momentum equation include the effects of lateral in-or outflow in the storage cells.For example, for free flow over a weir, the formula of Poleni is used (Berlamont 2004). Consequently, the Double Sweep algorithm is used to come to a numerical solution.

Numerical calculations are carried out and validated based on measurements in an experimental set-up. Figure 14 shows a definition sketch and an overview photo of the flume in the lab. The model flume includes a rectangular channel, width 40 cm, height 43 cm and length 12.41 m. An upstream weir and downstream gate are added in the channel. Discharges up to 32 l/s are studied. The upstream weir allows an accurate determination of the discharge, while the downstream gate allows regulation of the water level in the flume. Next to the channel, one large inundation area is added. This flooding area can be divided into three parts and also three lateral weirs, width 30 cm, are included in the flume construction.

A wave is sent through the main channel of the flume. In the first case (Figure 15(a)), the floodplain is closed and the water follows the channel. The upstream boundary condition is based on the measured discharge series ($Q_{\text{up-measured}}$), while downstream an equation is set up linking the discharge to the opening height of the gate. The Manning coefficient of the flume equals $0.012\ \mathrm{m}^{-1/3\}}$ s. This value is determined by measuring the surface water level for different steady state conditions and for different heights of the downstream gate.

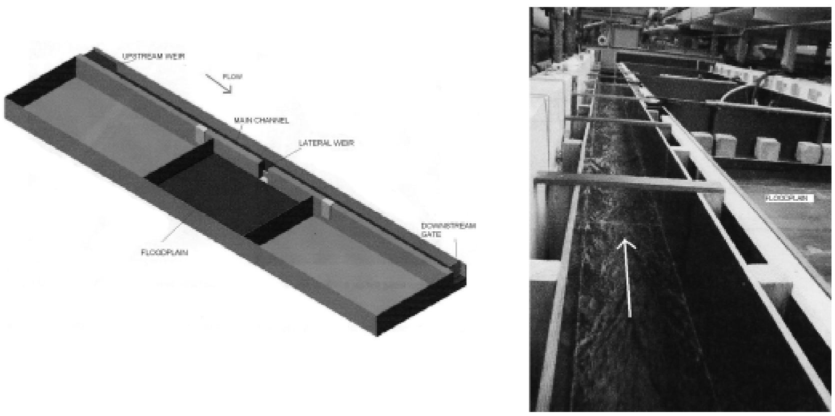

Figure 14 | Laboratory test flume for inundations: definition sketch (left) and overview photo (right).

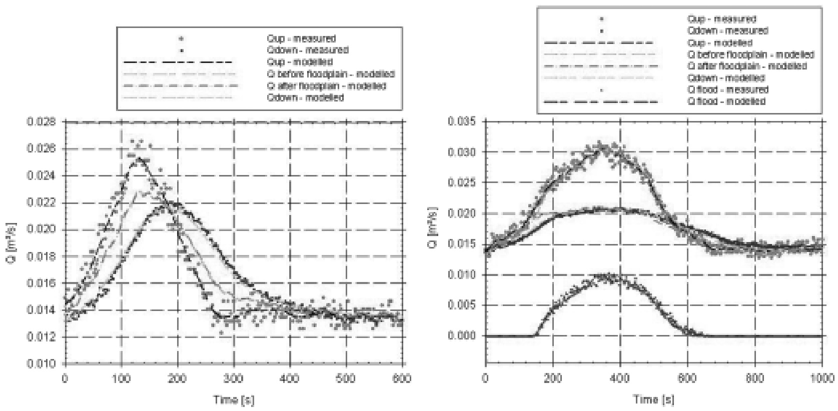

Figure 15 | Wave in a river channel without (left) and with (right) a floodplain.

The STRIVE model presents analoguous simulation results as the values obtained in the experimental set-up. Figure 15 depicts some first results. The simulated discharge downstream ($Q_{\mathrm{down-modelled}}$) corresponds to the measured discharge ($Q_{\mathrm{down-measured}}$). Due to friction, a time shift and flattening of the discharge peak is remarked. Further peak flattening is obtained by introducing a floodplain, which causes storage of the water. Figure 15(b) shows the result; the floodplain is empty at the beginning of the measurement and calculation. Measured and modelled values are in good agreement. Deviations are due to measuring errors.

Water quality aspects: transport of electrical conductivity

River variables as discharge, water level and electrical conductivity (EC) are related. In the River Aa, upstream and downstream water levels are measured continuously. Discharges are measured during measurement campaigns as well as the EC value. In the following, the EC is measured upstream and downstream. With this information and an approximation of the volume in the studied reach, an estimation of the discharge is carried out. Next to that, the value of EC is useful as a calibration parameter or as a boundary condition. EC is a measure for the amount of total dissolved solids (TDS), which is an indication of the water quality, TDS is an aggregate indicator of presence of a broad array of chemical contaminants. A dataset of February 2006 (6–13 February) is selected for model calibration. Boundary conditions are the upstream water level and the downstream discharge-water level relationship. Discharges cannot be used for calibration due to the limited amount of data available.

The simulation is rather complicated due to the uncertainty on a lot of parameters: the position of the downstream weir is not known, the bottom depth, bottom width and wetted cross section in the reach are not known exactly, calculated Manning coefficients are approximate, etc. All parameters have to be determined to achieve an accurate solution. Another important parameter is the Manning coefficient, based on biomass density and discharges, which has consequences for the modelling of base and peak discharges. The Manning coefficient is calibrated for the dataset and based on the measurements (De Doncker *et al.* 2009b).

Figure 16(a) shows the discharge, water level and EC values in the River Aa. In Figure 16, measured discharges upstream as well as downstream are close to the modelled values. In Figure 16(b), the upstream water level (boundary condition) and the downstream water level are plotted. The peak value is simulated very well. The differences in the first and the last part of the values are due to changes in the position of the downstream weir. Results are very sensitive to this weir position. Figure 16(c) shows modelled and measured conductivity values. Comparing peak values upstream and downstream allows us to estimate the travelling time of the tracer. Together with values of the water volume in the reach, the discharge can be calculated. As sometimes discharge determination in rivers is difficult, measurement of EC and Z allows use of these variables as boundary conditions.

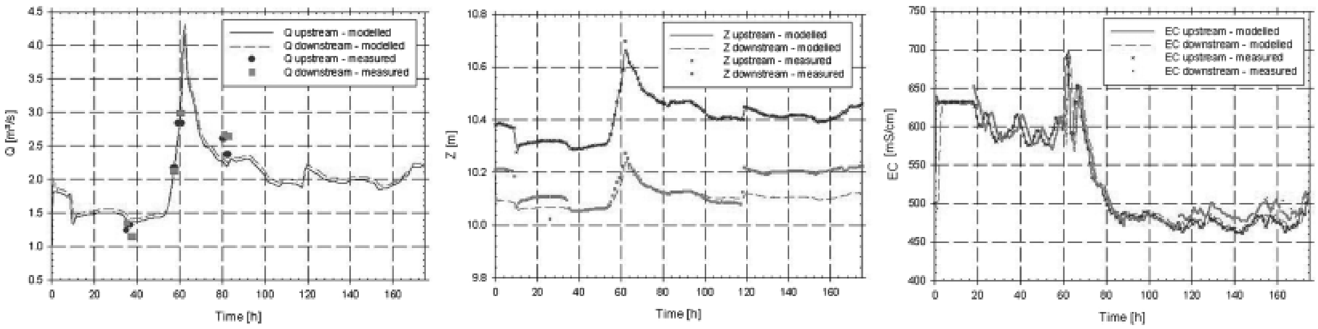

Figure 16 | Modelled and measured values of discharge, water level and electrical conductivity from 6–13 February, upstream and downstream of the studied reach in the River Aa.

CONCLUSIONS

The development of the integrated STRIVE package allows simulation of water quantity and water quality aspects, using dynamic boundary conditions. The model is general and can be adapted and further developed according to the research question.

Data analysis showed the large influence of in-stream vegetation (biomass density) on the roughness parameter in the Saint-Venant equations. Data collection was carried out in the River Aa over three years. This extended dataset, based on geometrical and hydraulic as well as biological and chemical parameters, allows calibration of the STRIVE model, which results in a well-tested code with accurate and reliable results. The model is tested in steady state as well as in unsteady state conditions and is compared to analytical solutions as well as to other numerical (Hec-Ras) solutions. A sensitivity analysis is carried out to get familiar with the interaction of biomass density and the hydraulic parameters. The interaction between discharge, water level and electrical conductivity is shown. The 1D STRIVE model returns accurate results of the different variables, including the large impact of the presence of biomass density.

Use of the described modules (hydraulic module, ecosystem module, water quality module) in the STRIVE package allows integrated modelling. Also, modules can be used seperately. The hydraulic module results in values of discharge and water level, while use of the ecosystem module includes the variation of biomass over the year, described by Manning's coefficient. The water quality module couples transport of tracers and nutrients to the quantity variables of the river. As an integrated example, discharge can be derived from quality measurements and simulations.

Over the year, the amount of vegetation is linked to the seasonal cycle. The combination of a healthy vegetation growth and summer storms can cause flood problems. A well-considered integrated river management needs to balance the requirements from the ecosystem with regard to water quality and the need for a safe flood protection policy. Calculation results show the influence of the resistance on both flow and water levels. Taking into account the environmental conditions (living area, agricultural land, etc.), peak values of the discharge have to be reduced, e.g. for safety reasons. Therefore, a sound vegetation control policy can contribute to control flood water levels, at the same time guaranteeing the quality of the ecosystem.

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