AALTO UNIVERSITY SCHOOL OF SCIENCE AND TECHNOLOGY
Faculty of Engineering and Architecture

Tero Niemi

DEVELOPMENT OF A HYDRAULIC MODEL AND ITS APPLICATION TO A SMALL URBAN STREAM

A Master of Science Thesis submitted for inspection in Espoo on May 17, 2010

Supervisor: Professor Ari Jolma
Instructor: Ioan Ferencik, M.Sc. (Tech.)

AALTO UNIVERSITY SCHOOL OF SCIENCE AND TECHNOLOGY | **ABSTRACT OF THE MASTER'S THESIS**

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Understanding flow in urban streams is important, for example to prevent damages caused by flooding or to be able to estimate pollution transport in a stream. In this thesis a one-dimensional hydraulic model was developed to simulate gradually varied subcritical steady or unsteady open channel flow in a small urban stream or a branched stream network. The model was implemented using Fortran 95 programming language and it was connected to PostgreSQL/PostGIS spatial database using a middleware program implemented in Perl. The database stores the input data and the simulation results of the model, whereas the middleware program enables the interaction between these components and provides a user interface for the system.

It was observed that simulating flow in a small urban stream is challenging for many reasons. In small streams estimation of flow resistance is difficult, since the flow varies on a large scale and the variations are usually rapid, the methods commonly used to determine resistance are not suitable for small streams, and local energy losses have a significant impact. Lateral inflow to the stream might also be so significant that is should be taken account in modeling, but quantifying its amount is difficult. Furthermore, low flows are rather common in small streams from the computational point of view, and these may cause numerical difficulties in simulation.

The developed model was applied to a small Ridalinpuro stream in Nummela, Southern Finland, to verify that it works. To confirm that the model works also in a branched channel network and in other situations it is intended to, more tests are needed with appropriate data. However, in its present state the model provides a tool that can be used to simulate flow in single reaches of open channel, and which can easily be modified and extended to suit further needs.

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Veden liikkeiden ymmärtäminen kaupunkiuomissa on tärkeää esimerkiksi tulvavahinkojen ehkäisemiseksi ja veden mukana kulkeutuvien saasteiden leviämisen arvioimiseksi. Tässä diplomityössä kehitettiin yksiulotteinen hydraulinen malli simuloimaan tasaisesti muuttuvaa stationaarista tai epästationaarista verkasvirtausta pienessä kaupunkiuomassa tai haarautuvassa uomaverkostossa. Malli toteutettiin Fortran 95-ohjelmointikielellä ja se yhdistettiin PostgreSQL/PostGIS-tietokantaan käyttäen Perl-kielellä ohjelmoitua väliohjelmistoa. Tietokantaa käytetään mallin tarvitsemien lähtötietojen ja simulointitulosten tallennuskohteena, kun taas väliohjelmiston tarkoituksena on yhdistää malli tietokantaan ja toimia käyttöliittymänä kehitetylle malli-tietokanta-systeemille.

Pienten kaupunkiuomien virtausmallinnuksen havaittiin olevan hankalaa monista eri syistä johtuen. Ensinnäkin virtausvastusten arviointi on vaikeaa, sillä virtaaman muutokset ovat yleensä suuria ja nopeita, tavallisesti käytetyt virtausvastusten arviointimenetelmät eivät sovellu pieniin uomiin ja koska paikalliset energiahäviöt voivat olla suuria. Uomaan sivuilta tuleva vesimäärä saattaa myös olla niin suuri, että se tulisi ottaa huomioon mallinnuksessa, mutta sen arvioiminen on hankalaa. Lisäksi pienet virtaamat ovat laskennallisesta näkökulmasta yleisiä pienissä uomissa, ja nämä saattavat aiheuttaa numeerisia hankaluuksia mallinnuksessa.

Kehitetyn mallin verifioimiseksi sitä käytettiin virtaaman mallintamiseen Nummelan Ridalinpurossa. Jotta voitaisiin varmistua, että malli toimii myös haarautuvassa uomaverkostossa ja muissa suunnitelluissa tilanteissa, lisää testejä tulisi suorittaa sopivaa dataa käyttäen. Nykyisellään malli tarjoaa työkalun, jota voidaan käyttää virtaaman mallintamiseen yksittäisessä avouomassa ja jota voidaan helposti muokata ja laajentaa vastaamaan tulevia tarpeita.

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LIST OF SYMBOLS

A	[m^2]	flow area in cross section
A_s	[m^2]	flow area in hydraulic structure
b	[m]	width of cross section water surface
C_c	[-]	coefficient for contraction loss in hydraulic structure
C_e	[-]	coefficient for expansion loss in hydraulic structure
C_f	[-]	flow coefficient for the discharge curve
C_{in}	[-]	input parameter for contraction loss coefficient
C_{out}	[-]	input parameter for expansion loss coefficient
D	[m]	hydraulic depth
e	[m]	iteration stopping criteria for the steady state computation
Fr	[-]	Froude number
f	[-]	Darcy-Weisbach friction coefficient
g	[m/s^2]	acceleration due to gravitation
H	[m]	total head
h	[m]	water depth in cross section
h_c	[m]	head loss in constriction
H_E	[m]	error in total head
h_f	[m]	head loss due to friction
j, n	[-]	indexes of computational points in space and time
jj, nn	[-]	number of computational points in space and time
		indexes of last computational points in space and time
K	[m^3/s]	conveyance of the channel
n	[m$^{-1/3}$s]	Manning coefficient
P	[m]	wetted perimeter of the cross section
Q	[m^3/s]	discharge
q	[m^2/s]	lateral inflow to cross section per unit length
R	[m]	hydraulic radius of the cross section
Re	[-]	Reynolds number
S_f	[-]	friction slope
S_c	[-]	slope due to energy losses in channel constriction
S_0	[-]	channel bottom slope
t	[s]	time

v	[m/s]	average flow velocity in cross section
v_s	[m/s]	average flow velocity in hydraulic structure
x	[m]	longitudinal space co-ordinate in horizontal plane
y	[m]	water surface elevation above the datum
y_b	[m]	channel bottom elevation above the datum
z	[m]	transverse space co-ordinate in horizontal plane
α	[°]	channel bottom slope
β	[-]	non-uniform velocity distribution coefficient
θ	[-]	weighting coefficient for space derivatives
φ	[°]	angle of the opening in Thomson weir
ν	[m²/s]	kinematic viscosity of water
Δh	[m]	change in water depth
Δt	[s]	time step
Δx	[m]	distance between points j and j+1

$A1$, $B1$, $C1$, $D1$, $E1$	coefficients for the discretized and linearized continuity equation
$A2$, $B2$, $C2$, $D2$, $E2$	coefficients for the discretized and linearized momentum equation
F_j, G_j, P_j, O_j, R_j	recurrence coefficients for the double sweep method

1 INTRODUCTION

An urban area can be defined as an area of concentrated human activity, which is characterized by extensive impervious areas. Urbanization affects components of hydrological cycle in many ways; in case of surface waters the resulting effects include e.g. an increase in runoff volume and flow in receiving streams, which can further lead to many problems, e.g. flooding. As urban runoff is typically highly polluted it is as such also a significant source of non-point source pollution. Thus urban runoff can cause both quality and quantity problems in receiving water bodies. Understanding flow characteristics in urban streams is important for many direct reasons, such as to estimate flood inundation and the possible damage caused by flooding, but also for indirect reasons as flow is the dominant mechanism for transporting pollutants. If flow is not understood adequately, neither do the water quality predictions reflect true behavior of the system.

Open channel flow and transport of substances are some of the most complex and least understood processes in nature (Wu, 2008). With the advances in computer science, computational model usage has increased to support different stream flow engineering problems from 1970s. Since then, many commercial and non-commercial models are developed to solve flow in open channels and especially for analysis of river flow. Although many readily available models exist, writing own mathematical model has some advantages over using already existing models. First of all, equations and the scheme used in the model are known exactly. Secondly, later modifications to the model are easy to make if needed. Consequently, the model can be constructed as computationally heavy or light as wanted and thus easily implemented to satisfy current needs.

This work was done as a part of a European Commission funded Seventh Framework Programme research project HYDROSYS. The project aims at providing a system infrastructure to support teams of users in the on site monitoring and management of environmental processes. HYDROSYS is an interdisciplinary project utilizing Wireless Sensor Networks (WSN), on site monitoring, environmental modeling and simulation, visualization, and event driven campaigns. The project will allow users to analyze on site real time data provided by the wireless sensors deployed on the field, and run simulation models that will use the dynamic data provided by the sensors, using mobile phones and handheld computers supported by advanced user interface technologies. The Nordic scenario in HYDROSYS is dealing with flow and material transport modeling in a small stream located in a highly urbanized environment in the catchment of Kylmäoja in Southern Finland. Especially interesting phenomena are the effect of de-icing chemical use in a nearby airport and the effect of urbanization to water quantity and quality in the stream.

To support the Nordic scenario, HYDROSYS has to employ a hydraulic model satisfying a set of requirements. In order to decouple data from the model, the baseline static data is stored in a spatial database, and thus the model has to accept input from this database. If the model is to be usable in the project, it has to be accessible to other HYDROSYS components in a generic fashion. Also, in order to run and visualize simulations while on site, the model has to be fast. From the practical point of view the model should also be computationally stable and easy-to-modify for the future needs of the project, for example to be easily adapted to be used as a basis for a transport model. The chosen study site sets also constraints for the flow model like relative flatness, high urbanization, and limited amount of available data.

Keeping in mind these requirements, the main objective of this thesis was to develop a hydraulic model to simulate open channel flow in an urban stream, which could later on be used to simulate flow in Kylmäoja as a part of the HYDROSYS project. The second objective was to develop an accompanying database to store data used by the hydraulic model. The third objective of the thesis was to study how modeling of flow in small urban streams differs from traditional flow modeling in bigger and less urbanized streams, and what challenges are related to modeling flow in such streams.

The study considered only one-dimensional flow in order to keep the developed model simple, powerful, and fast, and in this way facilitate future changes of the model. In addition, one-dimensional models require less input data than two- or three-dimensional models, and are also therefore more suitable to be used in the project. To further keep the model and its data requirements simple, actions resulting in division of either flow or cross section to multiple components were also left out of the scope of this work. This means e.g. excluding flow in channel floodplains and use of only one resistance coefficient per cross section.

In this thesis, a one-dimensional open channel flow model capable to simulate both unsteady and steady flow was developed in Fortran 95 programming language and was connected to PostgreSQL/PostGIS database using a simple middleware program written in Perl to ensure that the two components work together. To verify that the developed system works, it was applied to a small urbanizing stream Ridalinpuro in Nummela, Southern Finland, from where data of good quality was available. Application to Kylmäoja was not possible because enough baseline data for the model was not available at the time of conducting this study.

This thesis consists of three parts. First part corresponds to Chapter 2 and describes the theoretical part of modeling flow in open channels by means of equations describing one-dimensional steady and unsteady flow and algorithms for numerical solution to these equations. Also, estimation methods for flow resistance in open channels are presented as well as challenges related to flow modeling in small and urban streams. In the second part, corresponding to Chapter 3, the developed modeling system, consisting of a hydraulic model, a database, and a middleware program, is presented in detail. Third part corresponds to Chapter 4 and deals with running the model on Ridalinpuro stream, as well as consideration of applying the model to Kylmäoja stream as part of the HYDROSYS project. The study ends with Chapter 5 that contains discussion and conclusions in respect to the results of the work.

2 COMPUTING WATER FLOW IN OPEN CHANNEL

2.1 Flow classification

Based on its characteristics the open channel flow may be classified as follows (Chow, 1959):

- steady flow and unsteady flow
- uniform flow and non-uniform flow
- subcritical flow and supercritical flow
- laminar flow and turbulent flow

Flow in open channel is said to be steady or stationary if the discharge and other flow factors, such as flow depth or average flow velocity, do not change with time ($\partial f / \partial t = 0$). If the flow factors do change with time ($\partial f / \partial t \neq 0$), the flow is classified as unsteady. In natural channels the flow is nearly always unsteady. However, if the time averages of the flow factors are near constant the flow may be approximated as steady flow.

Open channel flow is considered to be uniform if the flow factors do not vary with distance along the channel ($\partial f / \partial x = 0$) and non-uniform or varied flow if the flow factors vary with distance ($\partial f / \partial x \neq 0$). Since unsteady uniform flow is nearly impossible in practice the term uniform flow usually implies that the flow is also steady and the term unsteady flow implies that the flow is also non-uniform (Chow, 1959). In natural channels the flow is always non-uniform. Non-uniform flow may be further classified as either gradually or rapidly varied. The flow is rapidly varied if the depth changes abruptly over a short distance, otherwise it is gradually varied (Chow, 1959). In nature the flow is usually gradually varied.

Depending on the ratio of inertial forces to gravitational forces, the flow is classified to be either subcritical or supercritical. This ratio is given by the Froude number, defined in open channel as:

$$\mathrm{Fr} = \frac{v}{\sqrt{gD}} \tag{2.1}$$

where v is the mean velocity of flow, g is the acceleration of gravity and D is hydraulic depth, defined as flow area A divided by the width of the water surface b, or:

$$D = \frac{A}{b} \tag{2.2}$$

If Fr < 1, the flow is subcritical and the gravitational forces are dominant. Subcritical flow is common in nature and is relatively deep and slow moving. On the other hand, if Fr > 1, the flow is supercritical and the inertial forces are dominant. Supercritical flow is less common and is relatively shallow and very fast moving. In case of Fr = 1, the flow is critical and the inertial and gravitational forces are in equilibrium. A distinguishing criterion between subcritical and supercritical flow is that in subcritical flow disturbances travel both upstream and downstream direction, but in supercritical flow disturbances travel only in downstream direction (Chow, 1959). Whether the flow is subcritical or supercritical has an effect on the direction of computing the flow in a channel. In subcritical flow, the downstream condition controls the depth of flow upstream, and as a result the calculation must be started from the downstream control point and worked back upstream (Hamill, 2001). In supercritical flow the situation is reversed.

Depending on the ratio of inertial forces to viscous forces the flow may be classified as laminar or turbulent. In laminar flow the viscous forces are so strong compared to inertial forces that viscosity plays significant part in determining flow behavior. In such a flow, the water particles appear to move in definite, smooth paths which do not intersect. In turbulent flow the viscous forces are weak compared to inertial forces and water particles move in chaotic, irregular paths which do intersect. Between laminar and turbulent flow is so called transitional flow. In nature the flow is usually turbulent. The classification between laminar, transitional and turbulent flows is based on dimensionless parameter called Reynolds number, defined in open channel as:

$$\mathrm{Re} = \frac{vR}{\nu} \tag{2.3}$$

where v is the mean velocity of the flow, ν is the kinematic viscosity of water and R is the hydraulic radius of the cross section defined as flow area A divided by the wetted perimeter P, or:

$$R = \frac{A}{P} \tag{2.4}$$

Open channel flow is laminar if Re ≤ 500, transitional if 500 ≤ Re ≤ 12 500 and turbulent if Re ≥ 12 500 (French, 1986).

Flow can be classified to be either one, two or three-dimensional. Although flow in natural channels is always three-dimensional, one-dimensionality is often assumed since it makes computation of the flow easier. If one is concerned only with studying the longitudinal profiles of the cross section averaged properties of flow, assumption of one-dimensionality and use of one-dimensional model is sufficient. On the contrary, if one is interested in studying also the lateral or vertical components of flow, such as transverse flows in channel or flow in floodplains, two or three-dimensional model is needed. Within this thesis only one-dimensional flow is considered.

2.2 Equations for unsteady open channel flow

One-dimensional open channel flow can be described with two dependent variables, for example by knowing water stage and discharge in every cross section (Karvonen, 1986). Instead of discharge, water velocity may be used as other dependent variable and water stage can be replaced by water depth or flow area in a cross section. Since two dependent variables are adequate to describe one-dimensional flow, only two equations representing physical laws are required. These equations are conservation of mass and momentum, also called the Saint Venant equations after Adhémar Jean Claude Barré de Saint Venant who first derived the laws as early as 1871.

The Saint Venant equations for unsteady flow are based upon the following series of assumptions (Cunge et al., 1980):

- The flow is one-dimensional, meaning that 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, and therefore the pressure is hydrostatic.

- The effects of boundary friction and turbulence can be accounted for using the same resistance laws as for steady state flow.

- The average channel bed slope is so small that the cosine of the angle it makes with the horizontal may be replaced by unity.

The Saint Venant equations can be expressed according to Cunge et al. (1980) as:

$$b\frac{\partial y}{\partial t}+\frac{\partial Q}{\partial x}=0 \tag{2.5}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right)+gA\frac{\partial y}{\partial x}+gAS_f=0 \tag{2.6}$$

where $b = b(y)$ is the width of the water surface at the cross section, y is the water surface elevation above the datum, t is time, Q is discharge, x is longitudinal space co-ordinate in horizontal plane, $A = A(y)$ is flow area, g is acceleration due to gravitation and S_f is the friction slope defined as $S_f = \partial h_f / \partial x$ where h_f is the head loss due to friction.

Equation (2.5) represents the conservation of mass and is usually referred to as continuity equation whereas equation (2.6) represents the conservation of momentum and is usually called the dynamic equation.

The friction slope S_f in equation (2.6) is calculated using:

$$S_f=\frac{Q^2}{K^2} \tag{2.7}$$

where $K = K(h)$ is called conveyance of the channel. When Manning's formula is used conveyance is computed as (French, 1986):

$$K=\frac{1}{n}AR^{2/3} \tag{2.8}$$

where n is called Manning's resistance coefficient.

In case an inflow or outflow is continuously distributed along the channel, a supplementary term is added to the continuity equation to account for this lateral inflow between two consecutive cross sections (Cunge et al., 1980). Consequently, the equation (2.5) becomes:

$$b\frac{\partial y}{\partial t}+\frac{\partial Q}{\partial x}-q=0 \tag{2.9}$$

where q is lateral inflow entering the channel from sides per unit length.

In many cases the cross sections are so irregular that the basic assumption of uniform velocity over cross section is not valid. A common example of such a situation is flow in cross sections which include overbank areas. Because of its high resistance to flow, the flow velocity in overbank area is close to zero and the area thus contributes only to storage. To take account for the non-uniform distribution of velocity, a correction coefficient β can be used in dynamic equation. As a result, the equation (2.6) becomes:

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\beta\frac{Q^2}{A}\right)+gA\frac{\partial y}{\partial x}+gAS_f=0 \tag{2.10}$$

Theoretically the value of β can be deduced from the following relation (Cunge et al., 1980):

$$\beta=\frac{\int_0^b v_z^{\,2}h_z dz}{v^2 A} \tag{2.11}$$

where subscripts z denote local values of depth-averaged velocity and depth at position z in the cross section.

In practice, the evaluation of β from equation (2.11) is hardly possible, and the value has to be computed for example on the base of steady flow resistance laws (Cunge et al., 1980). For uniform flow the value of β is equal to unity and in natural open channels it is usually around 1.1 – 1.2 (Karvonen, 1986).

Intense channel constrictions, for example due to bridges or culverts, cause energy losses that are usually referred to as eddy, turbulence or form losses. The losses occur as water is forced into and out of constrictions, causing it to flow in rapidly varying speeds and directions, which generate large-scale turbulence that dissipates energy as heat. These losses cannot be accurately modeled using one-dimensional shallow water flow equations. A typical solution to take account the eddy losses caused by channel constrictions is either to replace momentum equation with special equations describing the flow in constriction or, as has been done here, to use additional energy loss term in momentum equation (Syme, 2001). By introducing a term S_c describing the slope due to energy losses in constriction, the momentum equation (2.10) becomes:

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\beta\frac{Q^2}{A}\right)+gA\frac{\partial y}{\partial x}+gAS_f+gAS_c=0$$

or:

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\beta\frac{Q^2}{A}\right)+gA\left(\frac{\partial y}{\partial x}+S_f+S_c\right)=0 \tag{2.12}$$

The slope due to energy losses in constriction is defined as:

$$S_c=\frac{\partial h_c}{\partial x} \tag{2.13}$$

where h_c is the head loss in constriction.

For hydraulic structures the head loss is generally expressed as a function of the velocity head as in equation (2.14) (Syme, 2001). C_c and C_e are the coefficients for contraction and expansion losses, respectively, and v_s is the average flow velocity in the hydraulic structure.

$$h_c=\left(C_c+C_e\right)\frac{v_s^2}{2g} \tag{2.14}$$

For subcritical flow the contraction and expansion loss coefficients are related to input parameters C_{in} and C_{out} and to changes in velocity and area:

$$C_c = C_{in}\left(1-\frac{A_s}{A_1}\right) \text{ or alternatively: } C_c = C_{in}\left(1-\frac{v_1}{v_s}\right) \tag{2.15}$$

$$C_e = C_{out}\left(1-\frac{A_s}{A_2}\right)^2 \text{ or alternatively: } C_e = C_{out}\left(1-\frac{v_2}{v_s}\right)^2 \tag{2.16}$$

where subscripts *s* denote values of area and velocity in hydraulic structure and subscripts *1* and *2* denote values in cross sections immediately upstream and downstream of the structure, respectively. Common values for C_{in} and C_{out} are 0.5 and 1.0, respectively (DHI, 2009).

Equation (2.16) for the expansion loss coefficient C_e, is derived from Borda-Carnot expression which traditionally has been used to describe the energy loss associated with sudden expansions for pressurized pipe flow but can also be used to describe energy losses in open channel hydraulic structures (Tullis & Robinson, 2008). Equation (2.15) for contraction loss coefficient is based on experimental results (Syme, 2001).

2.3 Equations for steady open channel flow

As stated before, the flow is said to be steady if the flow conditions do not vary in time. Therefore the partial derivative term with respect to time can be dropped from continuity equation. Assuming also no lateral inflow equation (2.9) becomes:

$$\frac{\partial Q}{\partial x} = 0 \tag{2.17}$$

According to Cunge et al. (1980) momentum equation can be presented also with flow velocity v and water depth h as unknown variables. Then from equation (2.12) one obtains:

$$\frac{\partial v}{\partial t} + \beta v \frac{\partial v}{\partial x} + g\frac{\partial h}{\partial x} + g\left(S_f + S_c - S_0\right) = 0 \tag{2.18}$$

where S_0 is the channel bottom slope $S_0 = -\partial y_b / \partial x = \tan\alpha$ (y_b being the bottom elevation above datum). S_0 is assumed to be small, so that tan α ≈ sin α.

The second term of equation (2.18) may also be rewritten as $\partial\left(\beta v^2/2\right)/\partial x$. When dividing (2.18) with g and taking into account that in steady state $\partial v/\partial t = 0$ one obtains:

$$\frac{\partial}{\partial x}\left(\beta\frac{v^2}{2g}\right)+\frac{\partial h}{\partial x}+S_f+S_c-S_0=0 \tag{2.19}$$

When $S_c = \partial h_c/\partial x$ and $S_f = \partial h_f/\partial x$, and replacing partial derivatives with differential quotients equation (2.19) becomes Bernoulli equation:

$$S_0\Delta x + h_1 + \beta_1\frac{v_1^2}{2g} = h_2 + \beta_2\frac{v_2^2}{2g} + h_f + h_c \tag{2.20}$$

where h_1 and h_2 are the depths of water and v_1 and v_2 the average flow velocities in cross sections 1 and 2 and Δx is the distance between cross sections 1 and 2 as shown in Figure 1.

Bernoulli equation describes conservation of energy between two consecutive cross sections in a reach. It states that the total energy head at the upstream section 1 should be equal to the total energy head at the downstream section 2 plus the losses of energy h_f and h_c between the two sections. It applies to steady flow.

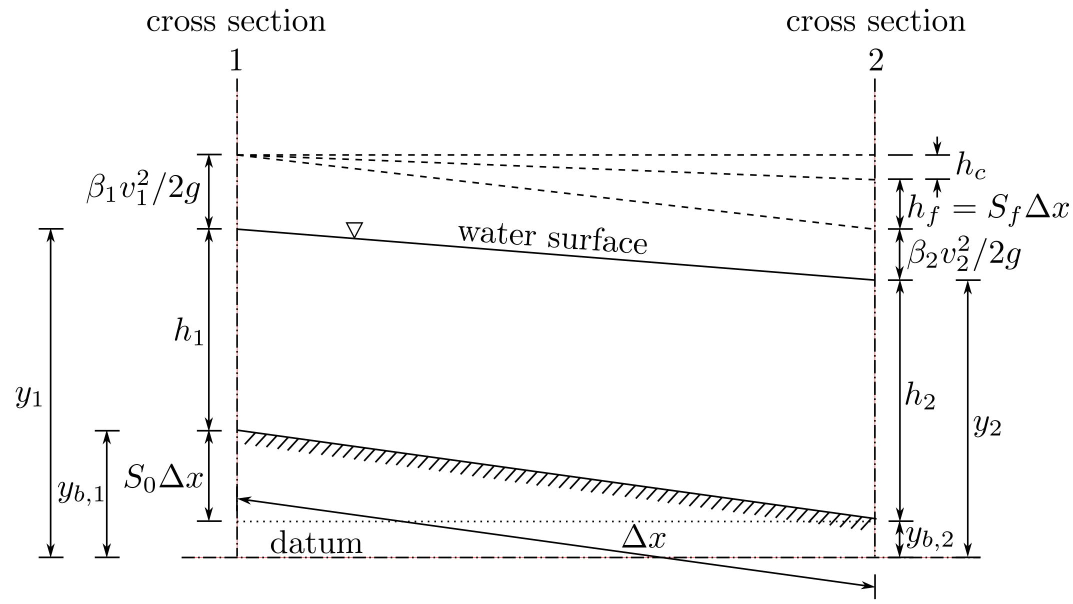

Figure 1. Principle of Bernoulli equation in non-uniform flow.

2.4 Numerical solution

2.4.1 Standard step method

Chow (1959) presents three groups of computation methods to evaluate flow profiles along a channel in case of non-uniform gradually varied flow: graphical-integration methods, direct-integration methods and step methods. The procedure in the graphical integration method is straightforward and easy to follow, but may become very laborious when applied to actual problems (Chow, 1959). The method of the direct integration on the other hand is not capable of predicting a depth of flow at specified longitudinal distance (French, 1986). From a great variety of step methods, the standard step method is used in steady flow computations of the developed hydraulic model.

The standard step method is applicable to both prismatic, and natural, non-prismatic channels. It is simple and straightforward, and also easy to convert to a simple and fast computer program. The method is called ”standard step” or ”step by step” method because the computation is carried on by steps from cross section to cross section where the hydraulic characteristics have been determined and the distance between the two cross sections is known. The procedure is usually carried out by trial and error (Chow, 1959).

The standard step method is based on solving the unknown water surface level from Bernoulli equation (2.20):

$$S_0 \Delta x + h_1 + \beta_1 \frac{v_1^2}{2g} = h_2 + \beta_2 \frac{v_2^2}{2g} + h_f + h_c \tag{2.20}$$

where the friction loss $h_f = S_f \Delta x$ and the head loss in hydraulic structure h_c is approximated with equation (2.14).

Following the descriptions given by e.g. Chow (1959) or French (1986) the standard step method may be explained as follows. According to Figure 1 (p. 18) the water surface elevations above the datum at the two ends of a channel section are:

$$y_1 = S_0 \Delta x + h_1 + y_{b,2} \tag{2.21}$$

$$y_2 = h_2 + y_{b,2} \tag{2.22}$$

The friction loss term h_f is approximated by:

$$h_f = \overline{S}_f \Delta x = \frac{S_{f1} + S_{f2}}{2} \Delta x \tag{2.23}$$

where the friction slope term $\overline{S}_f$ is taken as an arithmetic mean of the slopes at the ends of the channel section.

Substituting equations (2.21), (2.22) and (2.23) into Bernoulli equation the following may be written:

$$y_1 + \beta_1 \frac{v_1^2}{2g} = y_2 + \beta_2 \frac{v_2^2}{2g} + h_f + h_c \tag{2.24}$$

Now, the total heads, H_1 and H_2, at the two end sections are:

$$H_1 = y_1 + \beta_1 \frac{v_1^2}{2g} \tag{2.25}$$

$$H_2 = y_2 + \beta_2 \frac{v_2^2}{2g} \tag{2.26}$$

With these definitions, equation (2.24) becomes:

$$H_1 = H_2 + h_f + h_c \tag{2.27}$$

This is the basic equation that defines the procedure of the standard step method.

The trial and error solution of equation (2.27) depends on making the difference in H_1 determined by equations (2.25) and (2.27) zero. In computation procedure, if the difference between two computed values of total heads H_1 and H_1' is greater than predefined allowed error e, an error function is used to adjust the trial value of h_1.

By noting the difference between H_1 and H_1' with H_E, the change in H_E with respect to changes in h_1 can be measured with the derivative dH_E / dh_1. According to Henderson (1966) since z_1, H_1 and S_{f1} are constant:

$$\frac{dH_E}{dh_1} = \frac{d}{dh_1}\left(h_1 + \frac{v_1^2}{2g} - \frac{1}{2}\Delta x S_{f1} \right)$$
$$\Leftrightarrow \frac{dH_E}{dh_1} = 1 - \mathrm{Fr}_1^2 - \frac{1}{2}\Delta x \frac{dS_{f1}}{dh_1} \tag{2.28}$$

S_f varies approximately as the inverse cube of h so dS_{f1}/dh_1 can be written as (Henderson, 1966):

$$\frac{dS_{f1}}{dh_1} \approx -\frac{3S_{f1}}{h_1} \approx -\frac{3S_{f1}}{R_1} \tag{2.29}$$

and the previous equation becomes:

$$\frac{dH_E}{dh_1} = 1 - \mathrm{Fr}_1^2 + \frac{3S_{f1}\Delta x}{2R_1} \tag{2.30}$$

or:

$$\Delta h_1 = \frac{H_E}{1 - \mathrm{Fr}_1^2 + \dfrac{3S_{f1}\Delta x}{2R_1}} \tag{2.31}$$

where Δh_1 is the amount by which the water depth should be changed in order to make the error H_E disappear.

For subcritical flow the computation proceeds upstream since the flow is controlled by a downstream point. If the velocity head is small, the computation can be carried also in the wrong direction without generating serious errors, but this is not advisable (Chow, 1959). The computation procedure for standard step method between two cross sections can be presented as in Figure 2, if the discharge as well as the water depth in cross section 2 (see Figure 1, p. 18) are known.

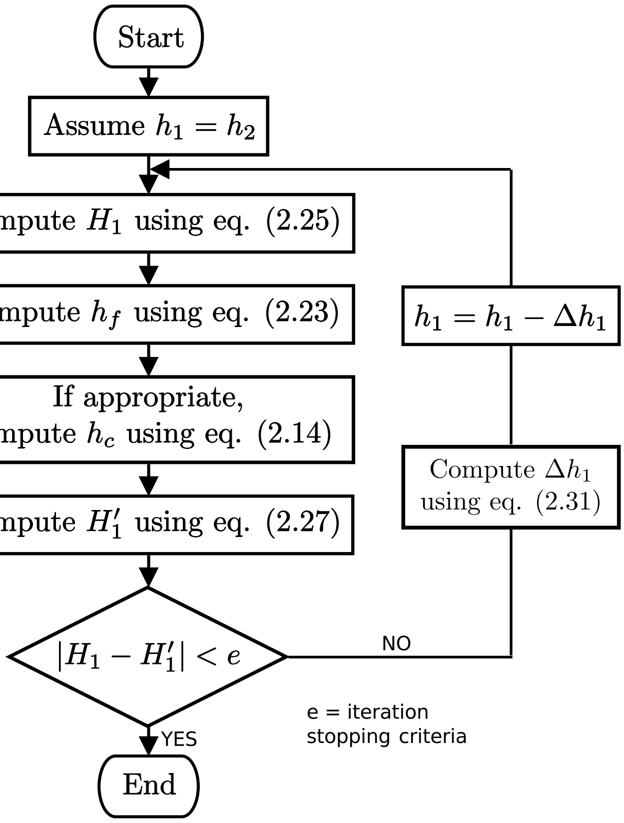

Figure 2. Procedure for standard step method for a single channel section as in Figure 1 (p. 18). Discharge and water depth at cross section 2 must be known.

For a channel with multiple cross sections the calculation procedure is the same as for a single section with minor modifications. The computation starts from the lowest cross section of the channel by defining the water level in that cross section as h_2. Then the water level is computed in the next cross section upstream as with a single section and as described in Figure 2. When the water level h_1 is computed for second cross section it is assigned as water level h_2 for the next channel section and computation is repeated in the same way for the rest of the sections in the channel until the water level is known in all cross sections.

In branched or tree-like channel networks there are confluences as presented in Figure 3. A defining feature of branched networks is that branches unite, not separate, so that from a given point in the network there is only one possible flow path to another point. The standard step method is applicable to this type of networks as long as a special computation order is used. The flow discharges at cross sections 1, 2 and 3 are denoted as Q_1, Q_2 and Q_3, respectively. The continuity equation at the confluence is:

$$Q_3 = Q_1 + Q_2 \tag{2.32}$$

When the flow is subcritical the water depth at cross section 3 is obtained first by using standard step calculation to channel 3 starting from lowest cross section of the channel and ending to cross section 3 while using Q_3 as the discharge. If the distances Δx_{13} and Δx_{23} are very small, the water levels or energy heads of the three cross sections at the confluence can be assumed to be identical (Wu, 2008). Thus, the computed water level at cross section 3 is specified as water level of cross sections 1 and 2 and the standard step method is then used to compute water depths in the rest of cross sections of channels 1 and 2. For this type of branched systems the water level has to be known at the lowest cross section of the lowest branch as well as the discharges in the upmost channels.

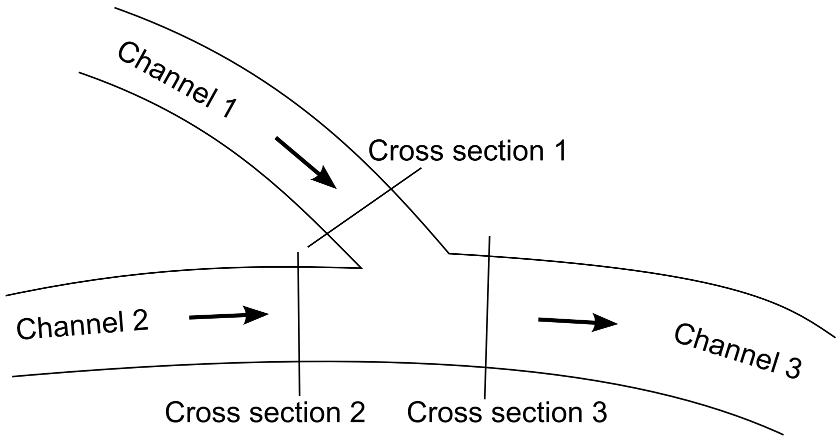

Figure 3. Configuration of a channel confluence in a branched or a tree-like channel network.

2.4.2 Finite difference method

Equations (2.9) and (2.12) describing unsteady open channel flow are non-linear partial differential equations, because in addition to the two main unknowns (discharge Q and water level y) they have also terms that are dependent on water level (surface width b and flow area A). Therefore the equations are too complex to be solved using analytical methods. However, it is possible to solve the equations approximately at a finite number of discrete points in the time-space domain by using numerical methods. Numerical solution methods used in solving flow equations include finite difference method, finite element method, finite volume method (Wu, 2008) and method of characteristics (Cunge et al., 1980).

Method of characteristics was one of the first numerical methods used to solve Saint Venant equations numerically, but difficulties related to the method prevented its wide use (Ligget & Cunge, 1975). At present, the finite difference method dominates, especially amongst one-dimensional problems, and it is also used in the developed hydraulic model. However, for some specific flow problems, such as to model shock waves related to dam breaks, the finite volume method is recommended (Szymkiewicz, 2010). The finite element method on the other hand seems to be best suited for two and three-dimensional problems, although it can be modified to be an efficient tool also for one-dimensional problems (Szymkiewicz, 2010).

In the finite difference method the (x, t) -domain under investigation is covered with a computational grid and values of unknown variables are calculated only at the nodes of the grid. A computation grid for one-dimensional problem is shown in Figure 4. From the reach under investigation *jj* cross sections (along the *x*-axis) are chosen so that there are *jj – 1* space intervals $\Delta x_j = x_{j+1} - x_j$ of variable length. In the same way the discretization in time is performed by dividing the time period under investigation into *nn – 1* time intervals $\Delta t_n = t_{n+1} - t_n$.

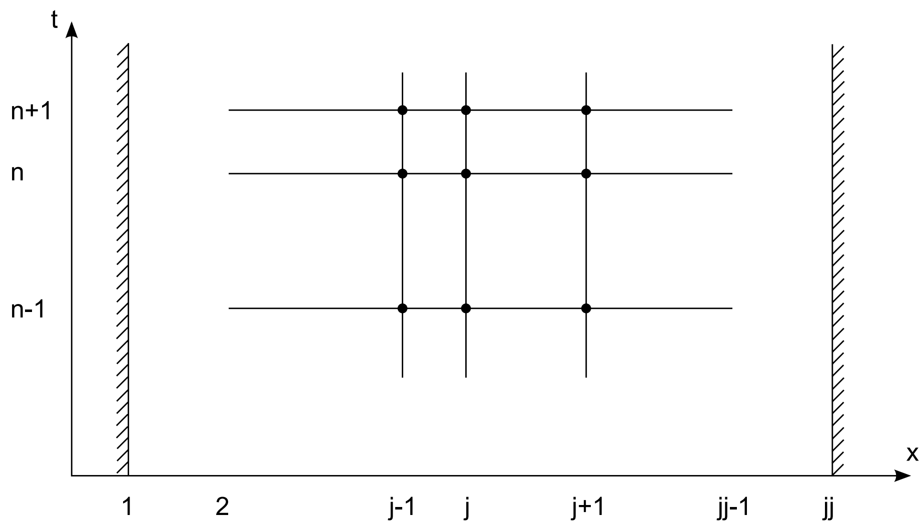

Figure 4. Computational grid for finite difference schemes. The reach is divided to jj cross sections along the x-axis, and the time is presented on t-axis.

During the discretization, the continuous functions describing the state of the flow are replaced by functions defined on a finite number of grid points within the considered domain and the derivatives are replaced by divided differences. Thus, the differential equations are replaced by algebraic finite difference equations. There are number of ways to perform the discretization, and different ways in which derivatives and integrals are expressed by discrete functions are called finite difference schemes.

Different schemes can be classified as either implicit or explicit. In explicit schemes the space derivatives are replaced using known values of discharge and water level from the time level t_n. This produces a large number of simple linear equations that can be solved directly for the unknown. In implicit schemes the space derivatives are replaced using both known values of discharge and water level from time level t_n and unknown values from the time level t_{n+1}. This results in a system of coupled equations that must be solved simultaneously. Even though explicit schemes are simpler than implicit ones, implicit schemes are usually used. The major advantage of implicit schemes is that they are stable regardless of the length of the time step Δt in contrast to explicit schemes where there is a restriction on the time step that may be employed (Karvonen, 1986). Therefore in implicit schemes the additional computational effort required to solve a system of equations is compensated for by a relaxation in the time step that can be used in the simulation (Zoppou, 2001).

2.4.3 Verwey's variant of the Preissmann scheme

Amongst the most widely used implicit finite difference schemes in engineering practice are the Preissmann and Abbot-Ionescu schemes. The schemes of Preissmann type have advantages over Abbot-Ionescu scheme because they allow grids with variable Δx and compute both the discharge and the water level at the same point (Chau, 1990). Thus, a variation of the Preissmann implicit finite difference scheme was chosen to be used in the hydraulic model. The Preissmann scheme uses values from nodes (j, n), (j+1, n), (j, n+1) and (j+1, n+1) to discretize the Saint Venant equations, as shown in Figure 5. In order to compute the unknown flow variables at every grid point at time level t_{n+1}, this scheme replaces the continuous function f and its time and space derivatives by (Cunge et al., 1980):

$$f(x,t) \approx \frac{\theta}{2}\left(f_{j+1}^{n+1} + f_j^{n+1}\right) + \frac{1-\theta}{2}\left(f_{j+1}^{n} + f_j^{n}\right) \tag{2.33}$$

$$\frac{\partial f}{\partial t} \approx \frac{\left(f_{j+1}^{n+1} + f_j^{n+1}\right) - \left(f_{j+1}^{n} + f_j^{n}\right)}{2\Delta t} \tag{2.34}$$

$$\frac{\partial f}{\partial x} \approx \frac{\theta\left(f_{j+1}^{n+1} - f_j^{n+1}\right) + (1-\theta)\left(f_{j+1}^{n} - f_j^{n}\right)}{\Delta x} \tag{2.35}$$

where $\theta \geq 1/2$ is the weighting coefficient for space derivatives. If $\theta = 1/2$ the solution is central approximation, usually referred as box scheme, and with $\theta = 1$ the solution is fully implicit. According to Fread (1974), the accuracy of the Preissmann scheme declines as θ departs from 1/2 and approaches 1.0. However using θ of 1/2 may result in numerical oscillations and therefore value of $0.55 \leq \theta \leq 0.6$ is often used (Islam et al., 2003).

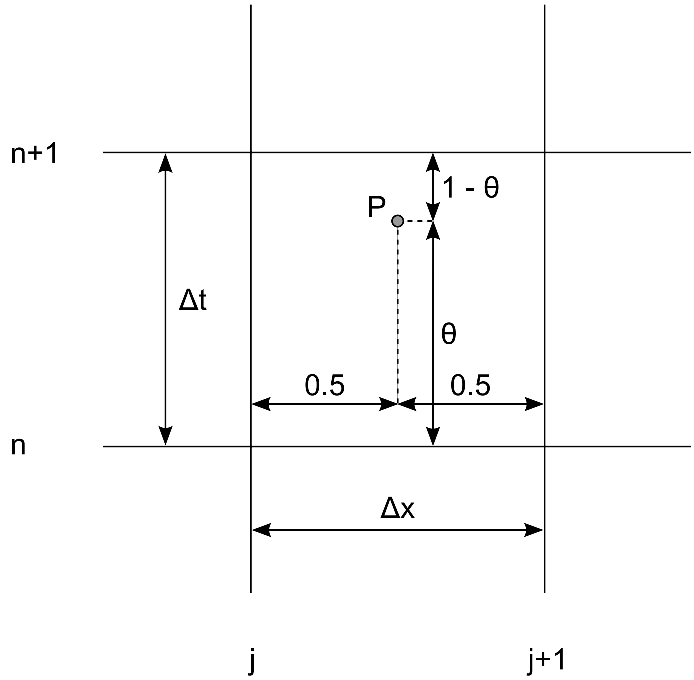

Figure 5. Computational grid for the Preissmann scheme. The scheme uses centralized discretization in time and weighted discretization in space to compute difference approximation for point P.

The most salient features of Preissmann-type schemes can be summarized as the following (Chau, 1990):

- Both unknown flow variables are computed at the same computational grid points.

- They are unconditionally stable as long as $\theta \geq 0.5$. Consequently the time step Δt can be chosen freely to give accurate enough results and to be comparable with physical phenomena under consideration.

- The space interval Δx may be variable, which enables a more flexible placement of cross sections.

In Finland e.g. Forsius (1984), Malve et al. (2003) and Harilainen (2007) have successfully used a variant of the Preissmann scheme derived by Verwey to simulate flow in open channels, and this variant was also used in the developed model. Here the non-linear terms of the Saint Venant equations are discretized differently than in the original Preissmann scheme (Karvonen, 1986):

$$\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) \approx \frac{1}{\Delta x}\left(\frac{Q_{j+1}^{n} Q_{j+1}^{n+1}}{A_{j+1}^{n+1/2}} - \frac{Q_j^n Q_j^{n+1}}{A_j^{n+1/2}}\right) \tag{2.36}$$

$$f(x,t) \approx \frac{f_{j+1/2}^{n+1/2} + f_j^{n+1/2}}{2} \tag{2.37}$$

Terms describing friction slope and losses due to channel constrictions, S_f and S_c, are discretized in the same way as in equation (2.36). Applying Verwey's variant of the Preissmann scheme to equations (2.9) and (2.12) leads to:

$$\begin{aligned} & b_{j+1/2}^{n+\theta}\left(\frac{y_j^{n+1} - y_j^n + y_{j+1}^{n+1} - y_{j+1}^n}{2\Delta t}\right) + \theta \frac{Q_{j+1}^{n+1} - Q_j^{n+1}}{\Delta x} \\ & + (1-\theta)\frac{Q_{j+1}^n - Q_j^n}{\Delta x} - q_{j+1/2}^{n+\theta} = 0 \end{aligned} \tag{2.38}$$

$$\begin{aligned} & \frac{Q_{j+1}^{n+1} - Q_{j+1}^n + Q_j^{n+1} - Q_j^n}{2\Delta t} \\ & + \frac{\beta}{\Delta x}\left(\frac{Q_{j+1}^n Q_{j+1}^{n+1}}{A_{j+1}^{n+1/2}} - \frac{Q_j^n Q_j^{n+1}}{A_j^{n+1/2}}\right) \\ & + gA_{j+1/2}^{n+\theta}\left(\theta \frac{y_{j+1}^{n+1} - y_j^{n+1}}{\Delta x} + (1-\theta)\frac{y_{j+1}^n - y_j^n}{\Delta x}\right) \\ & + \frac{1}{2} g A_{j+1/2}^{n+1/2}\left(\frac{\left|Q_j^n\right| Q_j^{n+1}}{\left(K^2\right)_j^{n+1/2}} + \frac{\left|Q_{j+1}^n\right| Q_{j+1}^{n+1}}{\left(K^2\right)_{j+1}^{n+1/2}}\right) \\ & + gA_{j+1/2}^{n+1/2} \frac{1}{2\Delta x} \frac{\left|Q_j^n\right| Q_j^{n+1}}{\left(A_s^2\right)_j^{n+1/2}} \left(C_c + C_e\right)_j^{n+1/2} = 0 \end{aligned} \tag{2.39}$$

where conveyance K is computed using equation (2.8).

In the last term of equation (2.39), describing the extra losses due to hydraulic structures, the coefficients with subscript j+1 are omitted because in the developed hydraulic model all the parameters of hydraulic structures are defined in cross section j.

Notation $f_{j+1/2}^{n+\theta}$, for an imaginary variable f, in equations (2.38) and (2.39) is an abbreviation for:

$$\begin{aligned} f_{j+1/2}^{n+\theta} &= \frac{1}{2}\left(f_j^{n+\theta} + f_{j+1}^{n+\theta}\right) \\ &= \frac{1}{2}\left[(1-\theta)\left(f_j^n + f_{j+1}^n\right) + \theta\left(f_j^{n+1} + f_{j+1}^{n+1}\right)\right] \end{aligned} \tag{2.40}$$

The rest of the terms involving either θ or 1/2 in super or subscripts are derived in the same manner.

Because the Preissmann scheme is implicit, the discretized equations (2.38) and (2.39) constitute a nonlinear system that needs to be solved iteratively. For the iteration process the non-linear equations are linearized using first-order Taylor series expansion (Wu, 2008). In practice, equations (2.38) and (2.39) are rearranged so that all the unknown terms (from time level n+1) are placed to left side of the equations and all the known terms (from time level n) are placed to right side of the equation (Karvonen, 1986). For a pair of points (j, j+1) equations (2.38) and (2.39) are written as:

$$A1_j Q_j^{n+1} + B1_j y_j^{n+1} + C1_j Q_{j+1}^{n+1} + D1_j y_{j+1}^{n+1} = E1_j \tag{2.41}$$

$$A2_j Q_j^{n+1} + B2_j y_j^{n+1} + C2_j Q_{j+1}^{n+1} + D2_j y_{j+1}^{n+1} = E2_j \tag{2.42}$$

The expressions for the coefficients $A1$, $B1$ … $D2$, $E2$ are obtained by collecting terms from equations (2.38) and (2.39). They are given in Appendix A.

Equations (2.41) and (2.42) alone are not sufficient to find the values of Q_j^{n+1}, y_j^{n+1}, Q_{j+1}^{n+1} and y_{j+1}^{n+1} because for these four unknowns there are only two equations available. The remaining two equations are obtained from upper and lower boundary conditions and thus this system of four equations may be solved for any time step Δt. In order to be able to compute discharge and water level at time levels $n = 2 \dots nn$ initial values of Q and y must be provided at every cross section at time level $n = 1$. These values are obtained from steady state calculation.

2.4.4 Double sweep method

When the boundary conditions are linearized in terms of Q and y, equations (2.41) and (2.42) may be solved for every computation point for every time step Δt using any standard matrix inversion method. However, this solution is the most time-consuming part of the computation and thus any time-saving procedure is preferred (Liggett & Cunge, 1975).

Widely accepted as the most efficient method for solving systems of equations is the double sweep method or Thomas algorithm (Islam et al., 2003). It uses the banded matrix structure of the linear system of equations (2.41) and (2.42) to compute the solution with number of operations proportional to number of cross sections N, and not to N^3 as required in direct matrix inversion techniques (Cunge et al., 1980).

A short description of the double sweep method is given here following description by Ligget & Cunge (1975).

Assuming that there exists a linear relationship of the type:

$$Q_j^{n+1} = F_j y_j^{n+1} + G_j \tag{2.43}$$

and substituting this into equation (2.41) one can express y_j^{n+1} as a function of Q_{j+1}^{n+1} and y_{j+1}^{n+1}:

$$y_j^{n+1} = P_j Q_{j+1}^{n+1} + O_j y_{j+1}^{n+1} + R_j \tag{2.44}$$

From equations (2.43) and (2.44) coefficients F_j, G_j, P_j, O_j and R_j are obtained by substituting (2.43) and (2.44) into equations (2.41) and (2.42) and arranging terms. They are given in Appendix A.

The computation starts from point $j = 1$ at the upstream end of the channel, where values for coefficients F_1 and G_1 are obtained from the boundary condition in the first gridpoint. Then, during the first sweep from point j = 2 to point $j = jj$ at the downstream end of the channel, the values of F_j and G_j are computed.

In the return sweep from point $j = jj$ to point j = 1 boundary conditions are applied to point $j = jj$, from which Q_{jj} and y_{jj} can be computed. Using equations (2.43) and (2.44) and the coefficients computed in the forward sweep, Q_j^{n+1} and y_j^{n+1} are then computed in the remaining points.

In equations (2.38) and (2.39), and furthermore, in coefficients $A1$, $B1$,…, $E2$ some terms are superscripted with time level $n+1$. Since these terms cannot be evaluated with the first pair of sweeps trough the reach at least two iterations for the solution at each time level have to be performed. During the first iteration the solution at time level $n+1$ is defined using the values from time level n. In the second iteration the solution is improved by using the average or weighted values from time level $n+1/2$ or $n+\theta$ defined with the solution of the first iteration. According to Forsius (1984) two iterations are needed and using three or more iterations does not improve the results anymore.

As discussed above the boundary conditions must be supplied both at the upstream (j = 1) and at the downstream ($j = jj$) end of each river reach. There are three types of boundary conditions that can be applied:

i. the discharge as tabulated function of time, $Q = Q(t)$

ii. the water level as tabulated function of time, $y = y(t)$

iii. the discharge given as a tabulated function of water level, $Q = f(y)$

The developed hydraulic model uses type (i) boundary condition at the upstream end of the reach. Clearly, from equation (2.43) can be seen that in this case the coefficients F_1 and G_1 should have values:

$$F_1 = 0$$
$$G_1 = Q_1^{n+1}$$

At the downstream end of the reach boundary condition of the type (ii) is used. Now, remembering that F_{jj} and G_{jj} are already known from the forward sweep when lower boundary condition is needed, it can immediately be seen from equation (2.43) that in order to be able to compute the value of Q_{jj}^{n+1} the boundary condition must be:

$$y_{jj} = y_{jj}^{n+1}$$

The double sweep method can also be applied to branched or tree-like channel system as long as certain computational order is respected. A simple network with three channels is presented in Figure 6, and is used to describe the procedure. Following the description given by Wu (2008) the computation can be performed as follows.

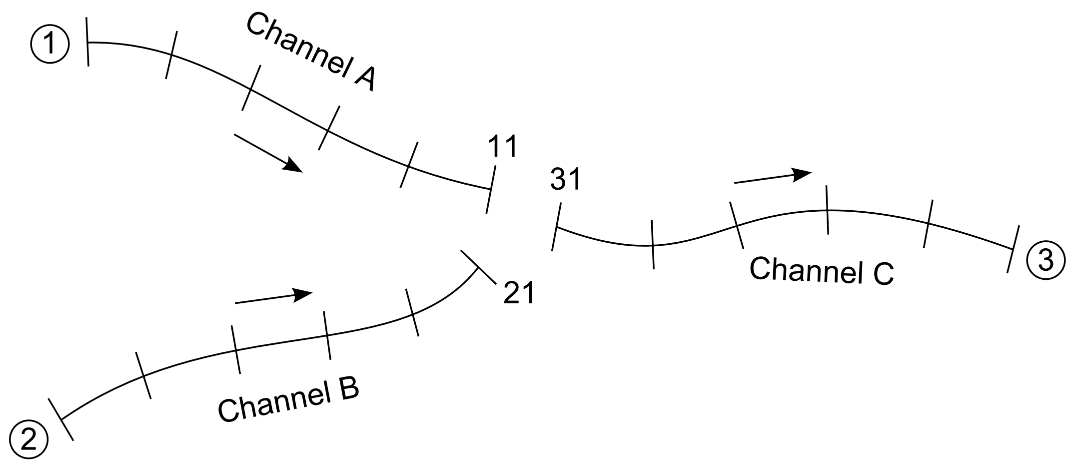

Figure 6. Double sweep algorithm for branched channel system. Boundary conditions must be supplied at points (1), (2) *and* (3).

The forward sweep to compute the recurrence coefficients for channel A starts from point (1), where boundary condition is given, and is carried on to point 11. Then another sweep is made along channel B from first to last point to compute the recurrence coefficients for this channel.

At the junction the three cross sections are located very close together and therefore it can be assumed that the water levels at the cross sections are equal. It is also assumed that the discharge at the downstream cross section is equal to the sum of those at the two upstream cross sections. Thus the compatibility equations for the junction are:

$$y_{11}^{n+1} = y_{21}^{n+1} = y_{31}^{n+1} \tag{2.45}$$

$$Q_{31}^{n+1} = Q_{11}^{n+1} + Q_{21}^{n+1} \tag{2.46}$$

Substituting equation (2.43) for the last points of channels A and B together with equation (2.45) to equation (2.46) one obtains following expressions for the first point of channel C:

$$F_{31} = F_{11} + F_{21} \tag{2.47}$$

$$G_{31} = G_{11} + G_{21} \tag{2.48}$$

The forward sweep can then be carried out from point 31 to point (3) in channel C and the recurrence coefficients computed along the channel.

The return sweep starts from the last point of channel C, point (3), where again boundary condition is applied. Values for discharge and water level are computed using equations (2.43) and (2.44) first along channel C back to the junction and further on along the two branches.

If flow velocities are high, so that velocity head becomes significant, the equal water level condition in equation (2.45) should be replaced with an equal energy level compatibility equation (Forsius & Huttula, 1982).

2.5 Resistance to flow

Flow resistance describes the effect of the forces resisting the flow in a channel caused by properties of the channel. In equations representing the flow, such as Bernoulli equation (2.20), resistance is described with an energy loss term h_f which itself may be described with many different equations and resistance coefficients. Along with a term resistance coefficient also terms friction coefficient and roughness coefficient are commonly used to describe resistance losses in a stream, although not all of these terms are synonym to each other (Järvelä, 1998). Resistance and friction coefficients describe the total energy losses of flow whereas roughness coefficient emphasizes the effect of bed roughness to energy losses.

Despite the fact that an extensive amount of literature exists on resistance to flow in river channels, resistance coefficients still remain as one of the least understood and most difficult to quantify hydraulic parameters in hydraulic modeling and fluvial hydraulics altogether (Reid, 2005). This is due to several causes for resistance (Table 1) and inability to identify and quantify each of these causes separately (Järvelä, 1998). Fundamental problem is also that factors affecting the flow resistance are not independent from each other. Most salient factors responsible for flow resistance are variations in the form of the channel shape, roughness and vegetation (Järvelä, 1998).

Table 1. Factors affecting flow resistance in natural open channels. (French, 1986; Järvelä, 1998)

- Roughness of channel bed and slopes
- Vegetation
- Irregularity and asymmetry of the channel
- Erosion and sediment transport
- Obstructions in the stream
- Cross section size and shape
- Water level and discharge
- Ice cover

The most common equations to evaluate energy loss term h_f are based on using either Manning coefficient n or Darcy-Weisbach coefficient f (Järvelä & Helmiö, 2003). With Manning coefficient energy losses may be estimated from:

$$h_f = n^2 \frac{Lv^2}{R^{4/3}} \tag{2.49}$$

and with Darcy-Weisbach coefficient from:

$$h_f = f \frac{L}{4R} \frac{v^2}{2g} \tag{2.50}$$

Manning coefficient is not dimensionless nor theoretically sound; therefore use of Darcy-Weisbach coefficient is recommendable. However, in practical engineering Manning coefficient is the most widely used and hence it is also used in this thesis.

The only exact way to evaluate Manning coefficient is to measure flow in a channel between two cross sections, and then compute energy losses from Bernoulli equation (2.20). Manning's n can then be solved from equation (2.49). This is however laborious and also often either impractical or even impossible. Therefore several studies have been conducted to determine an appropriate value for *n* using simple evaluation methods, such as guides or equations.

Simplest way to determine a proper value for *n* is to use values found in literature. There are many sources with either verbal descriptions or photographs of streams with known values of resistance coefficients, or alternatively tables with different values of resistance coefficients for different kinds of streams. For example Chow (1959) and Barnes (1967) have presented tables and photographs that can be used as a comparison material for estimating *n*. In Finland, Saari (1955) has presented tables and photographs of several small streams with measured resistance and Hosia (1980) has further used his results to present *n* as a function of Reynolds number. More recent studies of Manning's resistance coefficients in small Finnish streams have been presented by Helmiö (1997), Järvelä (1998) and Helmiö & Järvelä (2004). Even though looking proper values for *n* from literature is easy, the problem related to method of comparing photographs or verbal descriptions is its vagueness and subjectivity as Järvelä (1998) points out.

Another way to estimate Manning coefficient is to select basic *n* value for a uniform, straight and regular channel in a native material and then modify this value by adding correction values for different factors affecting flow resistance, such as those in Table 1. One of the most widely known methods using this approach is Cowan's (1956 ref. Chow, 1959) method. In this method value for *n* is estimated from equation:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \tag{2.51}$$

where n_0 is a basic *n* value for straight, uniform, smooth channel in the natural materials involved, n_1 is the correction value for irregularity of the channel bed, n_2 is a value for variations in shape and size of the channel cross section, n_3 is a value for obstructions, n_4 is a value for vegetation and flow conditions, and m_5 is a correction factor for the channel sinuosity. For example Chow (1959) has presented tables for proper values of n_0 to n_4 and m_5. As can be seen from equation (2.51) a problem with Cowan's method is that it does not take account flow conditions and assumes that resistance coefficient is only dependent on permanent properties of the channel. Since the values for correction factors in equation are chosen from tables or based on verbal descriptions these are also subjective opinions of users of the equation.

In recent years much research has been carried out on remote sensing technologies, such as light detection and ranging (LiDAR) or synthetic aperture radar (SAR), as tools to measure resistance coefficients. Research has mainly focused on floodplains (e.g. Mason et al., 2003; Straatsma & Baptist, 2008) although some studies have also been conducted on channel resistance (e.g. Schumann et al., 2007). The usual way to acquire resistance values is to segment and classify the land cover to main hydrodynamically relevant land cover types using high-resolution vegetation data from airborne altimetry, and determine a roughness coefficient for each land cover type (Schumann et al., 2009).

2.6 Challenges with small and urban streams

As has been described above, estimating resistance coefficients for natural channels is always challenging, but with small streams difficulties multiply. Helmiö (2004) noticed that with both small and large rivers the friction factor increases as river discharge (and Reynolds number) decreases. However, in small streams the discharge changes may be very rapid and the volume of flow might change several orders of magnitude in short period (Figure 7), and as a result the value of the resistance coefficient may change rapidly in large interval.

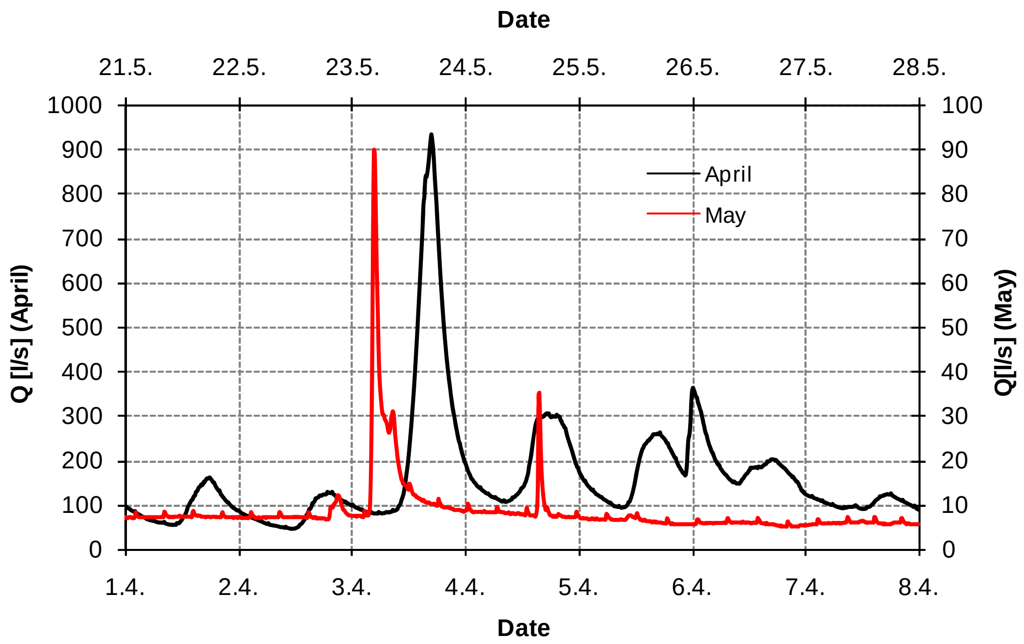

Figure 7. Measured discharge in small semi-urban stream Ridalinpuro in Nummela during snowmelt period (April) and in early summer (May) in 2009. Discharge variations are rapid and differ in several orders of magnitude. Notice the different scale in discharges between April and May.

The degree of urbanization in watershed affects also both the magnitude of flow and the rate of rise or fall of individual storm hydrographs. Hollis (1975) suggests that small floods may be increased by a factor of 10 or more depending on the degree of urbanization, and that the effect of urbanization decreases in relative terms as flood recurrence intervals increases. As a result of urbanization runoff travels more rapidly to streams resulting in faster rise of streamflow during storms and also faster receding after storms, often described as “flashiness” of streamflow (Figure 7).

Because of the rapid and large variations of discharge in small and urban streams, different resistance coefficients should be used for flows of different magnitude. Same Manning’s coefficient for the whole range from low to high flows will result in erroneous results, error being greater the further the coefficient is from its valid flow conditions. This poses a challenge to compute flows accurately for long time periods as the discharge and thus the resistance changes during computation.

The resistance coefficient values presented in literature are generally not suitable for low flows. Helmiö & Järvelä (2004) and Järvelä & Helmiö (2004) noticed that values presented in Chow (1959) and those computed with Cowan’s method underestimate the resistance for rivers Tuusulanjoki and Päntäneenjoki and brook Myllypuro during low flows. Same phenomenon had previously been noticed by Hosia (1980) for several small channels. The reason for this is that the values presented in literature are usually obtained for mean or high flow situations and often for bigger channels than the ones commonly studied in Finland.

In small streams spatial variations in the channel (e.g. location of vegetation or woody debris) together with temporal variations (e.g. vegetation) affect strongly to resistance factors (Järvelä, 1998). Järvelä and Helmiö (2004) point out that in small streams site-specific factors, such as individual logs or large rocks, may have significant impact to flow resistance. Therefore also the length of the reach analyzed may strongly affect the magnitude of resistance coefficient. Because of the errors caused by these singular roughness elements, or local energy losses, Järvelä & Helmiö (2003) state that unsteady flow modeling might be useless in small brooks and even steady flow modeling might lead to large errors.

If the local energy losses are caused by man-made elements in the stream, such as weirs or culverts in many urban streams, those may be estimated and taken account to some degree as has been done in the developed hydraulic model. On the other hand, if the reason for local energy loss is e.g. a log or bush in the channel as is common in small natural streams usually the only way to define its magnitude is by measuring and calculating it from the flow variables. Naturally, in this case the effect of individual elements cannot be distinguished from other resistance factors. Even with constructed elements the problem is to estimate the energy loss parameters in equations used to compute energy losses for particular elements, such as coefficients C_{in} and C_{out} in equations (2.15) and (2.16).

Although effect of lateral inflow may usually be neglected when modeling flow in rivers (Cunge et al., 1980), many small streams gain significant amount of water from lateral inflows (Runkel & Benkala, 1995) and as a result the discharge of the stream increases in the downstream direction. Especially in urban areas where storm waters are usually directed to streams via pipes or ditches lateral inflows might have strong effect to the flow. Estimating the amount and spatial distribution of lateral inflow is somewhat difficult and even more so in urban areas where storm waters may be channeled to the stream from some distance or even from different watershed. Thus it might not be enough to estimate the lateral inflow caused by surface runoff near the stream, but also in the areas where storm water pipes are collecting water.

Apart from problems concerning resistance coefficient estimation as described above, low flows cause also numerical difficulties, which in the worst case prohibit continuation of the computation if no special measures are taken. Especially two problems related to low flows are often encountered in hydraulic models: so called dry-bed situations and local supercritical flows in some isolated cross sections. These numerical difficulties may be emphasized in small streams, where low flows are often dominant from the computational point of view. Thorough description of both problems is given e.g. by Cunge et al. (1980) and here those are described only shortly.

Dry-bed situations occur because most of the solution algorithms for one-dimensional Saint Venant equations are based on a continuous transfer of information through a model network, with hydraulic equations linking together the computational points. These equations permit discharge to be continuously related to water level, but during low flows the computational algorithm may find itself ”disconnected” in certain locations where the depth becomes vanishingly small.

Appearance of supercritical flows in some cross sections is also a problem, because if the solution is based on implicit finite difference scheme it will develop unstable numerical oscillations whenever supercritical flow appears. This may be problematic especially during small discharges if the bottom slope is too high or if the selected time step for computation is too large (Cunge et al., 1980).

There are several solutions for the low flow problems. The easiest way to prevent low flows is to introduce a small base flow to channel that takes place if water levels drop below a certain limit, or simply prohibit model execution with too small discharge or water level as boundary conditions. These are of course highly artificial solutions and thus suitable at best to be used only temporarily. Other temporary solutions could be to modify bottom levels of cross sections so that no problems occur with low flows or to use only such cross sections where there are no problems, but again these are highly artificial and inadvisable. A slightly better solution for supercritical flows is to look for potential locations for supercritical flow to occur in the stream separately before running the model, and then use inner boundary conditions in such places when running the model. This solution is, however, laborious and difficult, as suitable boundary conditions must be found for each location and then the source code of the model must be modified to use each of them.

Best solution for both dry-bed and supercritical flows is to handle them mathematically in the model. Then user of the model does not have to consider handling low flow situations some way differently than normal flow as the model takes care of them. Advisably the model also notifies user when low flows have occurred, so that user can pay closer attention to results. For dry-bed problems for example Meselhe & Holly (1993) have presented some common solutions (e.g., inverse Preissmann slot, dynamic computation grid) and proposed their own approach. Widely used mathematical solution for supercritical flows is to reduce or totally drop the convective terms from momentum equation of Saint Venant equations (Meselhe & Holly, 1997); that is, to use diffusive wave approximation for supercritical flows. Transition between full Saint Venant equations and diffusive wave approximation could be done automatically by the model, for example by using full equations when Froude number is less than 0.9 and diffusive wave approximation for higher Froude numbers (Wu & Vieira, 2002).

Together with resistance parameter, representation of channel geometry is the most important factor affecting flow characteristics in hydraulic models (Merwade et al., 2008). Currently topography of the stream is frequently derived by using remote sensing technologies (Pappenberger et al., 2005) as are estimations for resistance coefficients, as mentioned in the previous chapter. Problems with small streams might occur because of the relatively small scale variations in stream environment compared to accuracy achieved with present remote sensing technologies. As a result, even if remote sensing technologies might work on estimating topography or roughness coefficients on medium and large rivers, they might not be accurate enough for small streams.

3 MODELING SYSTEM

3.1 Hydraulic model

3.1.1 General description

The developed hydraulic model computes discharge and water level as a function of time and space for a given stream network when topographic data and needed boundary conditions are given as input. It solves the Saint Venant equations using Verwey's variant of the Preissmann scheme and double sweep method for gradually varied, one-dimensional, subcritical unsteady flow as described in Chapter 2. Initial water levels needed by unsteady solution are obtained from steady state computation using Bernoulli equation which is solved for water levels with known discharge using standard step method as also described in Chapter 2. A flowchart of the model is presented in Appendix B, which explains the computational process in the model.

The model is implemented using Fortran 95 programming language; a very powerful language for numeric computation. The model can perform calculations in a stream network with a single junction, with cross sections that are of arbitrary shape and are located on irregular intervals. The friction losses are computed using Manning equation with single friction factor n for each cross section. Also simple estimation of the extra energy losses caused by hydraulic structures, namely culverts, in the stream is implemented based on equations described in Chapter 2. As a default the model returns the results of unsteady flow simulation, but with simple changes to the source code also the results of steady state computation can be chosen to be returned.

3.1.2 Input data

The data required by the hydraulic model can be grouped into three classes: topographic, hydraulic and parameter data. Topographic data describes the geometry of the simulated river system. It consists of geometries of cross sections, locations of cross sections in a particular branch, and relative locations of the branches in the stream network. Hydraulic data consists of discharge and water level data used as boundary conditions in the model, whereas parameter data describes the permanent variables used in the model.

Cross section geometries are represented as series of points specified by (m,z)-coordinates (Figure 8). The m-coordinate denotes the measure, or distance of the point along the cross section from one end. Distances can be measured from either one of the endpoints, but by default they are considered to be measured from left to right when looking downstream direction of the flow. The z-coordinate denotes the ground elevation above the datum, which can be chosen as any reference datum, but usually in Finland N60 reference system is used. To represent the geometry of the stream as accurately as possible, enough points should be used to create a cross section. Cross section should extend across the entire floodplain and should be perpendicular to the flow lines. For each cross section single value of Manning's friction coefficient *n* must also be provided.

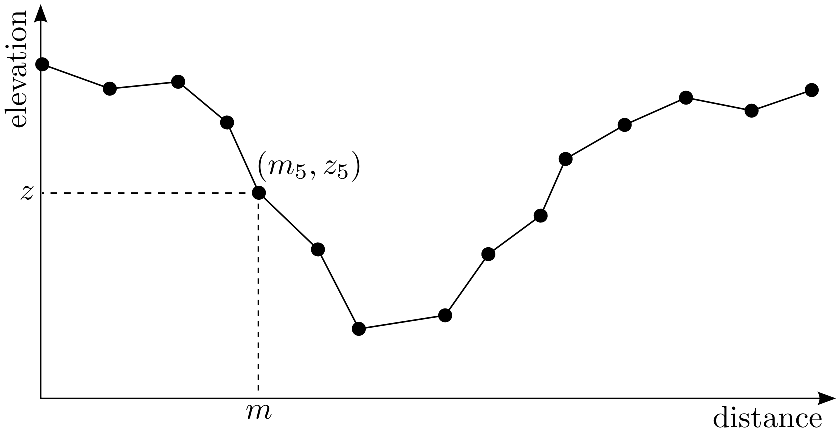

Figure 8. Cross section data needed by the hydraulic model. Each cross section is described as a series of (m,z)-coordinates, where m-value denotes the distance along the cross section, and z-value denotes the ground level above datum.

Cross sections in the stream should be located so that they result in the highest accuracy of computation for current hydraulic problem. For example Castellarin et al. (2009) have presented guidelines for optimal spacing of cross sections in one-dimensional hydraulic models using equations based on channel and flow properties. USACE (1993) on the other hand presents an extensive description of factors affecting placing of cross sections. According to recommendations by USACE cross sections should be located, e.g., at minimum and maximum cross sectional area, at points where roughness changes abruptly, closer together in expanding reaches and in bends, and closer together in reaches where the conveyance changes greatly as a result of changes in width, depth or roughness. In practice cross sections are usually spaced equidistantly based on either equations for optimal spacing or more commonly on previous experience from similar channels, and then additional cross sections are set in other necessary locations.

In the model the location of a cross section in a given reach is represented by its station value measured from the downstream end of the reach (Figure 9). Relative locations of the branches in a channel network are determined by naming the different branches around a junction as 1, 2 and 3 and defining branches 1 and 2 to discharge to branch 3 (Figure 9). Cross sections in the downstream ends of branches 1 and 2 are assumed to be very close to cross section in the upstream end of branch 3. If the network consists of only one channel it has to be named as 1.

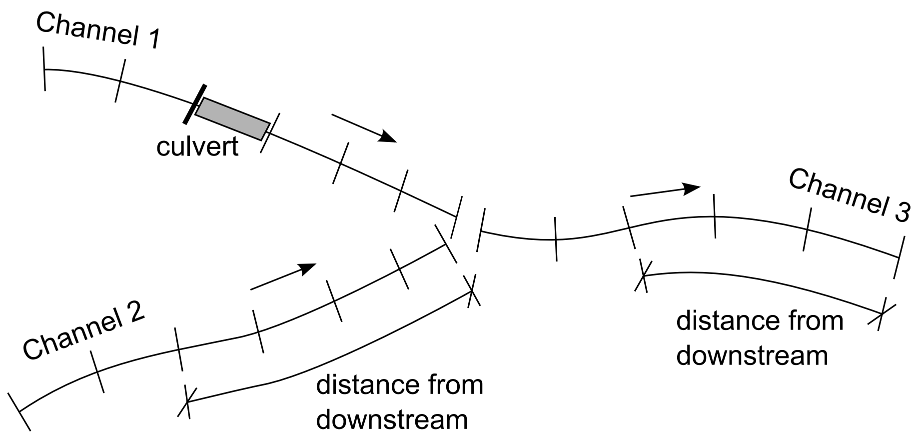

Figure 9. Relative locations of cross sections, branches and culverts in the model. Location of a cross section is its distance from downstream end of the channel. Branches 1 and 2 discharge to branch 3. Culvert is positioned to the nearest upstream cross section (bold in picture).

Hydraulic structures are defined in the model by giving as input data their locations in the stream and parameters describing their geometry. At the moment only two types of hydraulic structures are included in the model, namely circular and rectangular culverts. From circular culvert the model needs to know the inside radius of the culvert and from rectangular culvert the inside width and height. Also from any type of hydraulic structure the bottom level of the structure above the datum must be known. If any of these parameters varies with the length of the structure average values should be used. In the model the hydraulic structures are located in the nearest cross section upstream of the structure (Figure 9). The model computes the length of the structure as a distance between this cross section and the next one downstream, and therefore these cross sections should be placed as close as possible to the actual location of the hydraulic structure.

For the boundary conditions, the model uses known discharge in the upstream and known water level in the downstream end of the reach. In case of a branched system, discharge must be known in the upstream ends of the stream network and water level in the downstream end of the lowest branch. The boundary conditions must be supplied for every timestep $n = 1 \dots nn$ as tabulated functions of time. For steady state computation the model uses boundary conditions from time level $n = 1$ as constant values of discharge and water level in the end points of the stream network.

The parameter data needed by the model include values for θ, the weighting coefficient of space derivatives in the discretized equations describing the unsteady flow; β, the coefficient for non-uniform velocity distribution in cross section; Δt, the time step in the unsteady computation; and for the number of iterations performed in double sweep solution algorithm. As a default θ is set to value of 0.6, β to 1.0 and the number of iterations to 3, but these can and should be changed to fit the needs of modeling.

Selection of an optimal time step Δt is dependent on the simulated problem and the goal or scope of the simulation. Even though use of an implicit finite difference scheme allows arbitrary selection of the time step, the problem in hand might set some restrictions. For example, when simulating rapidly varying flow a small time step is required to adequately capture the behavior of the flow. At the present stage of the model the length of the time step is bound to boundary conditions, the time step being the time between two consecutive values of boundary conditions.

3.1.3 Structure of the hydraulic model

The developed hydraulic model is divided to two modules and it uses a total of five input files and one output file. Structure of the model is presented in Figure 10.

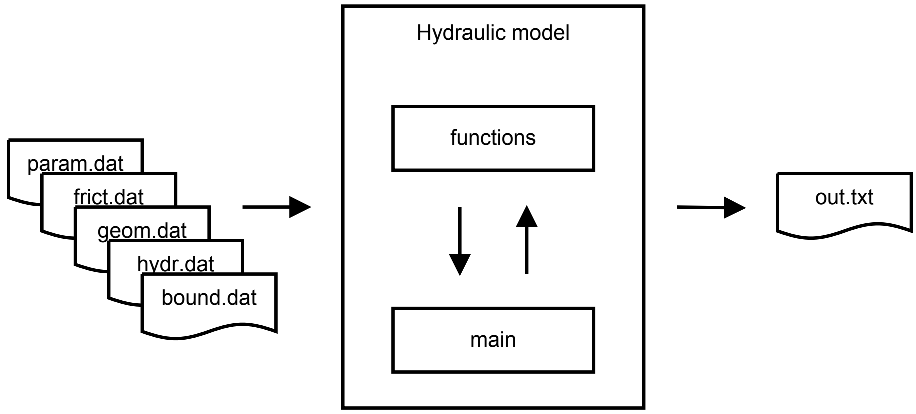

Figure 10. Structure of the hydraulic model. Rectangles with wavy bases represent files, rectangles modules of the model, and arrows data flow between different components.

Inside the hydraulic model the module "main" reads the input data from input files, calls for steady or unsteady solution and writes the results of computation to output file. The module "functions" stores all the functions needed in computation, including functions steady and unsteady which compute the discharges and water levels using methods described in Chapter 2. This procedural structure of the model allows high flexibility in terms of using individual components. It is possible for example to call the functions from another Fortran program or even from a program written in completely different programming language.

The five input data files store the input data needed by the model and described in Chapter 3.1.2. These files are text files where the data is stored in tabulated format. The functions steady and unsteady return an array consisting of discharge and water level data for every time level in every cross section of the stream network. This data is stored by main module to output file also in tabulated format.

3.1.4 Possible future extensions

Even though lateral inflow is presented in discretized continuity equation (2.38) and further still in coefficient $E1_j$ for double sweep method in Appendix A, it was omitted from the program code of the model. Including lateral inflow in the model would probably result in slower run time as number of operations would increase considerably, especially if it was provided as input variable for every cross section and every time step in case of continuous surface runoff to the stream. Estimation of lateral inflow is also a challenging task, and including it to model would increase the uncertainty related to results of the computations. If lateral inflow is to be added to the model, the source code of the model has to be modified so that a variable describing the lateral inflow is included in the coefficient $E1_j$ of the double sweep algorithm, and necessary changes are done to steady state computation as well. Also, lateral inflow as tabulated function of time and space has to be given as input data for the model in its own input file.

The model can handle a maximum of three channels in a branched stream network with one intersection. With some improvements to the source code and input files of the model, the model could handle more channels and junctions. The double sweep method used to solve the flow equations has to know the exact order of computation, and thus the order at which the branches are handled has to be specified to the model in the input files. For example, Nguyen and Kawano (1995) have proposed a node numbering scheme and double sweep solution algorithm applicable for large channel networks consisting of multiple branches and junctions with at most four branches per junction. Nguyen and Sugio (2001), on the other hand, presented an automatic node numbering scheme and double sweep solution that is applicable to complex branched networks regardless of the number of channels at junction.

At the moment, the model uses discharge as tabulated function of time as upstream and water level as tabulated function of time as downstream boundary condition. With some modifications, boundary conditions could be expanded to be also water level as function of time at the upstream and discharge as function of time or as function of water level at the downstream end of the channel.

The effect of hydraulic structures to flow is modeled in a very simplified manner. This is always the case in one-dimensional hydraulic models, but still treatment of hydraulic structures could be improved in the developed model. Simplest improvement would be to allow parameters C_{in} and C_{out} to be user-definable, instead of using default values of 0.5 and 1.0 as is the present state. Changing the values of these parameters is possible also in the developed model, but changes have to be done directly to the source code. This improvement would make it possible to use C_{in} and C_{out} as calibration parameters together with resistance coefficients. Other improvements would be to enable multiple hydraulic structures in one cross section instead of just one, to include different kinds of hydraulic structures instead of only circle and box culverts, and to allow differing sizes and bottom levels to upstream and downstream ends of the structure instead of using averages as is the present state. The first two would make the model more versatile, whereas last one would improve the computation of contraction and expansion losses.

At the present state of the model, the friction losses inside the structure are omitted. The reason for this is forcing the location of the structure to nearest upstream cross section, and computing the friction losses between this and next downstream cross section as in the case of any other cross section without hydraulic structure. If the friction losses were computed also inside the hydraulic structure they would be computed twice on the length of the structure. It also has to be noted, that the model does not compute water level profile inside hydraulic structures, but just estimates the effect of these structures to flow at particular locations. To enable water profile computation inside the structures major improvements to the source code should be made.

Computational problems related to low flows, i.e., dry-bed situations and occurrence of supercritical flows, could be handled mathematically in the model. To accomplish this, the source code of the model should be extended quite significantly.

3.2 Database

3.2.1 General description

A database is used to store the data needed by hydraulic model and the results of the model simulations. The hydraulic model itself does not need the database, and since it is implemented using Fortran, it cannot use the database directly but relies on flat files instead. However, database systems have many advantages over traditional file systems (Garcia-Molina et al., 2008) and therefore connection between the model and the database is enabled using middleware program written in Perl programming language. A benefit of databases is that they allow storage of very large amounts of data and still allow efficient access to single data items using a specialized query language. They also allow recovery of the database in case of failures, errors or misuse. In addition, database systems allow access to data for many users simultaneously, while isolating different users and their actions from each other and making sure that changes are synchronized and actions to data are never performed only partially. Especially the last feature is remarkable, as it allows access to the data stored in database server from any location over the internet and thus makes it possible to run the model or visualize the results of simulations without having the data in the first place.

The developed database uses PostgreSQL database management system with PostGIS geospatial extension. PostgreSQL is an object-relational database management system (ORDBMS) and according to its website is claimed to be ”the world’s most advanced open source database” (see www.postgresql.org). PostGIS adds support for geographic objects to PostgreSQL, thus allowing PostgreSQL server to be used as a backend spatial database for geographic information systems (GIS).

3.2.2 Conceptual design

Structure of the developed database is presented as a UML class diagram in Figure 11. Following describes the classes of the database in detail.

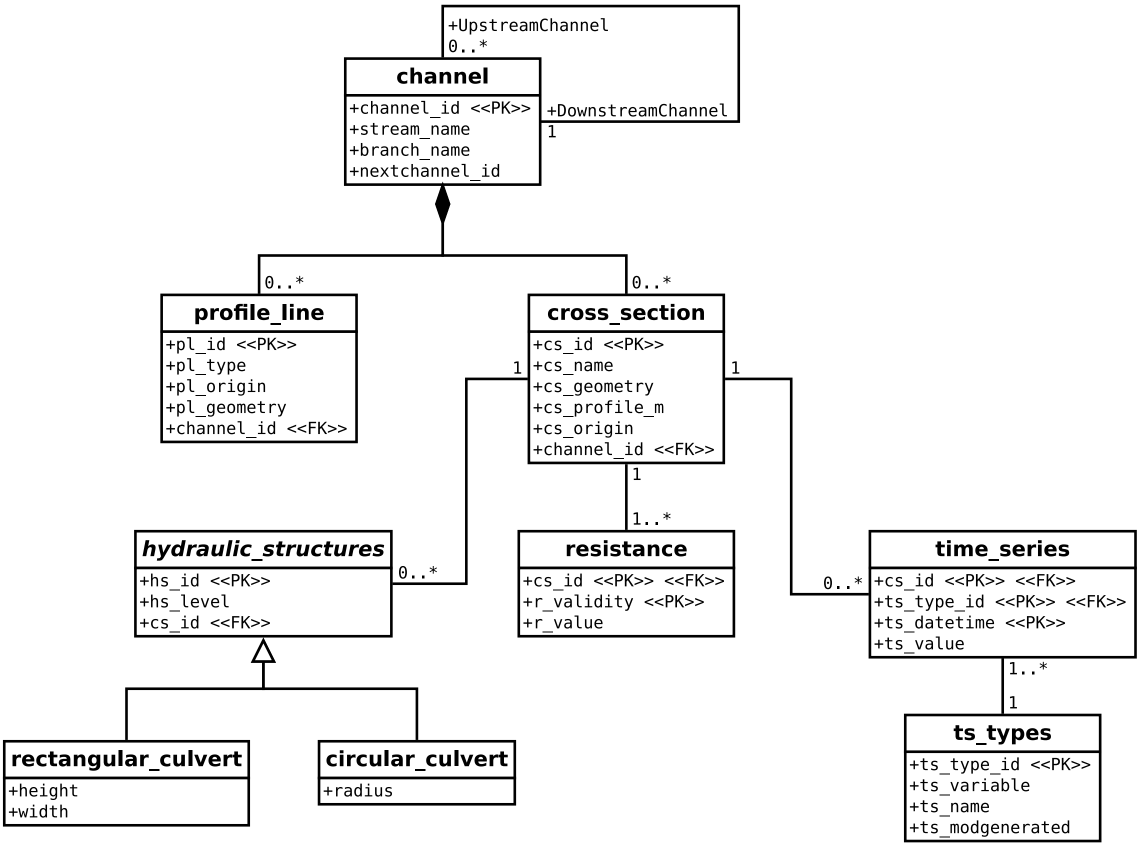

Figure 11. UML class diagram of the database developed for the hydraulic model.

The purpose of the class **channel** is to gather attributes that are common to all channel features. From each channel the name of the river or the stream is stored as well as the name of a particular reach or branch. Different channels are distinguished by a unique identifier for each channel, which is also used as a foreign key reference to a particular channel in other classes of the database. This identifier is also used to identify the order of different branches in a stream.

Class **profile_line** defines the longitudinal profiles of the channel parallel to the direction of the flow. The hydraulic model does not use data stored in this class but the data is valuable in visualization of channel properties, such as location of stream thalweg or the left and right bank of the channel. For each profile line unique identifier is given to distinguish them from each other. Other information stored for each profile line are the type or feature it represents, its geometry as xyz(m)-linestring and information about the origin or other distinguishing feature of the data.

Class **cross_section** defines the shape of the channel transverse to the direction of the flow. Once again unique identifier is used to distinguish cross sections from each other, and it is also used as a foreign key in other tables when referring to a particular cross section. Other data stored in this class includes the geometry of the cross section as xyzm-linestring, a name for each cross section, a station value of the cross section i.e. its distance from the downstream end of the channel, and origin or other distinguishing feature for a particular set of cross sections. The idea with the set of cross sections is that instead of referring to each cross section e.g. by its name, a group of cross sections can be chosen for example to represent a channel before and after restoration project.

Details of hydraulic structures in a stream are stored in an abstract class **hydraulic_structures**. This class defines properties shared by all structures, namely the unique identifier and the bottom level of the structure. Classes **rectangular_culvert** and **circular_culvert** inherit attributes of the class **hydraulic_structures,** and have their own specific attributes defining the geometry of these two possible types of structures.

Resistance coefficients for each cross section are defined in a class **resistance**. For each record a value is stored as Manning's resistance coefficient together with the classifier which identifies a certain set of resistance parameters in a same way as with cross sections. The idea here is to make it possible to use different coefficients for different flow situations, e.g., own sets of coefficients for low, normal and high flows.

Class **time_series** stores the time varying attributes of features in cross section; within this thesis it means the boundary conditions for the model and the results of simulations. Data stored in this class includes the value and the date and time of the attribute represented by the time series, as well as reference to a record in class **ts_types**, where different types of time series are defined. In class **ts_types**, unique identifier is given to each type of time series and used as a foreign key in table **time_series**. Also information about the variable represented by each time series is stored along with the name of particular time series and a Boolean value describing whether the time series represents model generated data or not.

3.2.3 Possible future extensions of the database

Since the developed hydraulic model gets its input data from the database and the results of the model simulation are entered back to the database, these two system components are strongly linked together. Thus if the data requirements of the hydraulic model are changed, care must be taken that the database is also updated to correspond to these changes. Vice versa, if the structure of the database is altered, it must be ensured that the data needed by the model can still be gathered from the database or the model source code is changed according to changes in the database. Possible extensions of the hydraulic model, e.g., related to hydraulic structure handling or lateral inflow, would thus also affect the structure of the database.

Since the database has been developed to serve the needs of the hydraulic model, it most likely does not need many major extensions or improvements of its own. The most notable exception is numbering of different branches in a channel network. Currently numbering is performed using a representation called adjacency list, where each item in the class **channel** of the database contains a pointer to the channel downstream of the current one, i.e., to the receiving stream. This numbering might be performed in a more suitable fashion using some other technique, such as nested sets, but in the end it is once again the needs of the hydraulic model that defines how order of branches has to be defined.

3.3 Connection between the hydraulic model and the database

As was already mentioned, the hydraulic model cannot connect directly to the developed database since it is programmed using Fortran 95. On the other hand Perl is well-known for its ability to provide easy interaction with many different databases, including PostgreSQL. Therefore a middleware program was written with Perl to provide a link between the developed hydraulic model and the developed database, with the original intention to make sure that the two work together. Since there might be a large number of data in the database and only some of it is used in each simulation, a simple user interface was also written with Perl, which asks questions from the user needed to acquire enough data from the database to perform the simulation (Figure 12).

```
********************************************************************************
********************************************************************************

Select stream:
(1) Ridalinpuro
Stream no.: 1
Selected: Ridalinpuro

Select channel from stream Ridalinpuro:
(1) Main stream
Channel no.: 1
Selected: Main stream

Select origin of cross sections for  channel "Main stream" in stream "Ridalinpuro":
(1) SYKE measurement 2007
Cross section origin no.: 1
Selected: SYKE measurement 2007

Creating geom.dat...
done.

Select resistance validity group for channel "Main stream" in stream "Ridalinpuro":
(1) computed high steady flow
(2) computed low steady flow
Resistance validity no.: 1
Selected: computed high steady flow

Creating frict.dat...
done.

Creating hydr.dat...
done.
```

Figure 12. Part of the user interface for the program connecting hydraulic model and database.

Idea of the modeling system built around Perl program is presented in Figure 13. For each input file needed by the model the program makes a query to the database and looks for available data. It then displays available datasets and lets user choose the correct one (Figure 12). Based on users choice the program makes another query to the database and writes fetched data to input file for the hydraulic model. When all input files are created, the program invokes the hydraulic model, which then runs the simulation and writes an output file. Finally the program reads the data from the output file and once more makes a query to the database to upload the data.

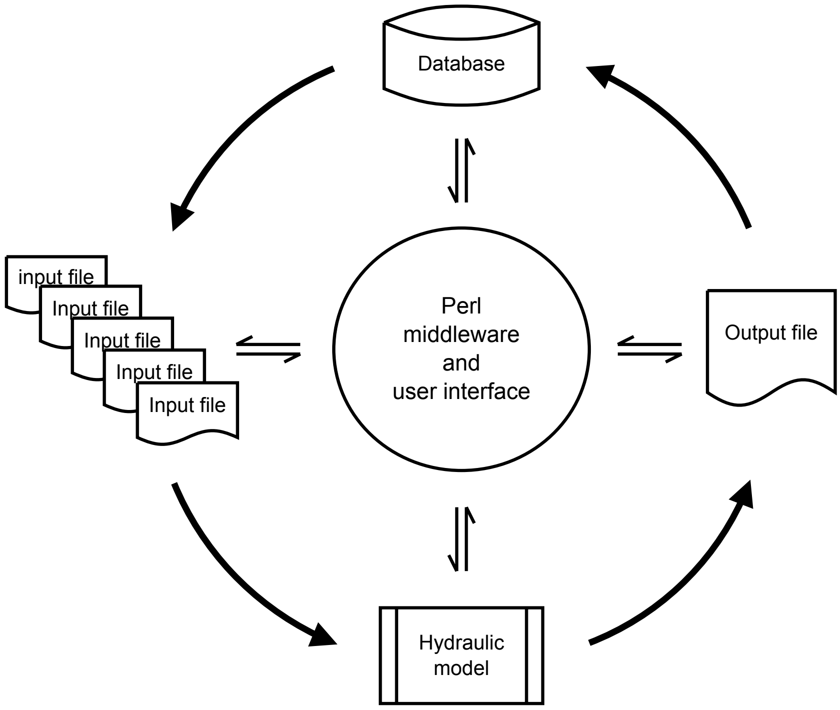

Figure 13. Structure of the modeling system. Bold arrows represent data flows between different components whereas thin arrows present the interaction of system components via the middleware program.

3.4 Modeling system in HYDROSYS

The developed hydraulic model and database are employed in HYDROSYS project in a interoperable fashion. A short description of the model and database implementation as parts of the HYDROSYS system is presented here based on the article of Ferencik et al. (2010) and personal discussions with Ferencik in 2010.

HYDROSYS is functionally composed of three distinct blocks: data acquisition, data processing and data visualization (Figure 14). The data acquisition block is supplying dynamic data from sensors installed in the site using Wireless Sensor Networks (WSN) to data processing block, whose design is gravitating around Global Sensor Network (GSN) middleware. GSN is a Java environment running on one or more computers aiming at dynamic integration of sensor networks and produced data streams. In the same time GSN acts like a proxy between the last block, the visualization block, and the hydraulic model in support of on site modeling (Figure 14). The interface between GSN and the visualization frontend is the HYDROSYS smart client, an application capable of consuming services provided by GSN, including running simulation models, in a spatial 2D or 3D context.

For interoperability and genericity purposes, the hydraulic model is published as a Web Processing Service (WPS), which enables the deployment of geospatial functionality on the web in a standardized way. Consequently, a Python extension module was created that exposes Fortran routines to the open source pyWPS software. Furthermore, the model wrapped inside pyWPS uses the developed PostGIS database as data repository, where the static baseline data like cross sections and roughness coefficients are stored. However, the model can now be executed over HTTP by supplying dynamic parameters (eg. boundary conditions) on the fly via HTTP stream. The model published as WPS treats and understands spatial data using OGR Simple Feature Library Python bindings. In addition, it writes the output as a spatial dataset, in Geographical Markup Language (GML), which is a complex data format supporting multiple layers. This fits perfectly the time dependent output produced by the hydraulic model, as a spatial GML layer is produced for every time step.

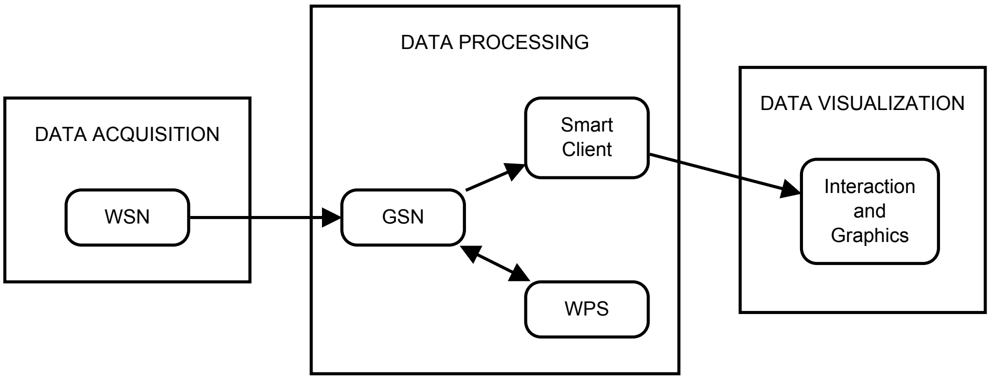

Figure 14. HYDROSYS system architecture. Rectangles represent system blocks, rectangles with rounded corners system components, and arrows data flow. WSN: Wireless Sensor Network, GSN: Global Sensor Network, WPS: Web Processing Service.

4 APPLICATIONS TO STREAMS RIDALINPURO AND KYLMÄOJA

4.1 Ridalinpuro

4.1.1 Watershed and stream

Ridalinpuro is a small creek located in the municipality of Vihti close to the urban centre of Nummela in Southern Finland (Figure 15). The catchment (5.4 km^2) of Ridalinpuro lies on clayey soils between gravel and moraine ridges and it has a semi urban character with approximately 45% of suburban area. The stream itself is a natural creek which has been heavily modified due to agriculture and urbanization during the last decades and centuries. It runs through a cultivated field with muddy soils that are prone to erosion and drains into a small (5 km^2) Enäjärvi lake, having negative impact on the lake (Salminen, 2010). Investigated part of the stream is approximately 200 m long reach from the middle parts of the creek which is limited upstream by a measurement weir and downstream by a constructed sedimentation pond.

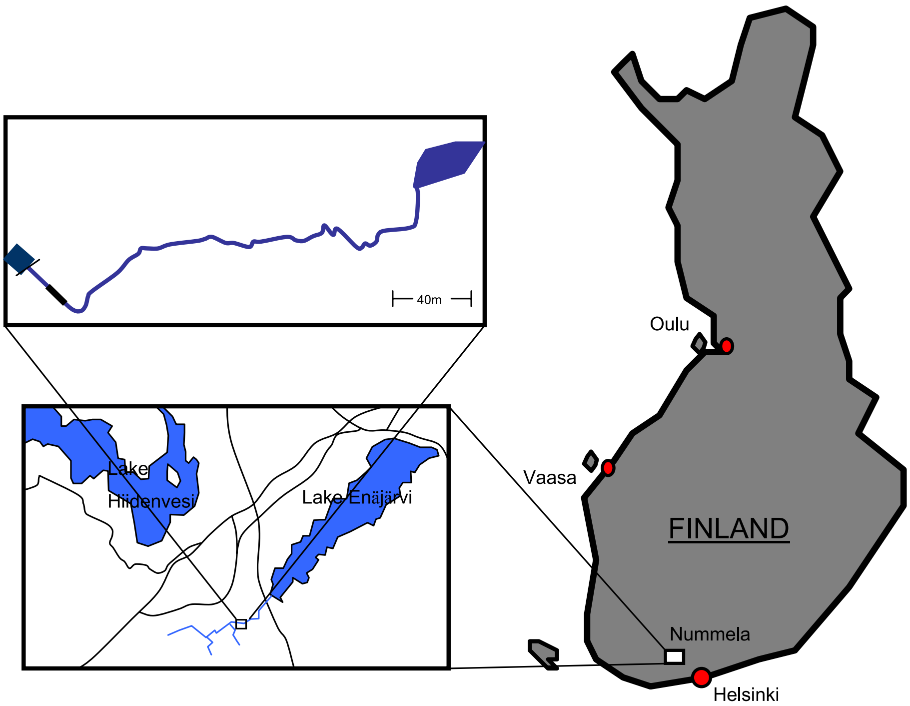

Figure 15. Location of Ridalinpuro in Southern Finland close to the urban centre of Nummela. Investigated part of the stream is shown in top left picture. (Blomfeldt, 2008)

4.1.2 Description of the data

All of the data used in here was gathered during the fall of 2007 either by Finland's environmental administration or by Water Resources Laboratory in Helsinki University of Technology. The data was originally used by Blomfeldt (2008) in his thesis and more detailed description of the data can be found in that paper.

A total of 15 cross sections were measured approximately 20 meters apart from each other by staff from Finland's environmental administration using GPS equipment. Along the stream six metal poles were also installed to provide the possibility of manual reading of water levels using vertical level markers. Poles I and II share locations with cross sections CS0 and CS20 and for poles III – IV cross sections were measured separately. Locations of the cross sections and measuring poles are presented in Figure 16.

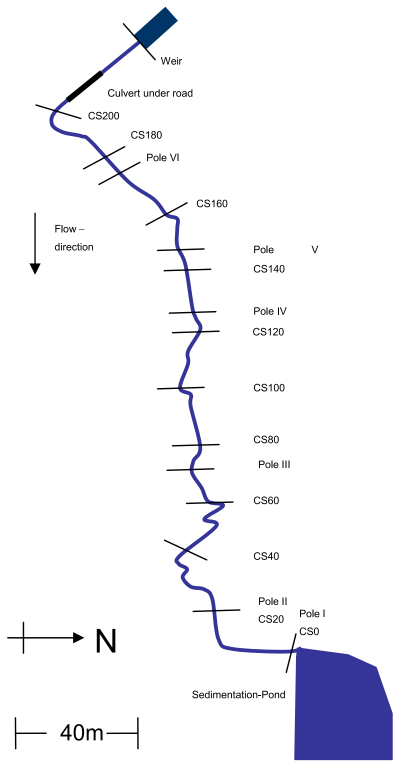

Figure 16. Locations of cross sections (CS) and measuring poles (Pole I - VI), each also having a corresponding cross section. (Blomfeldt, 2008)

A typical main channel width in the investigated part of the stream is approximately a little over 1 m and the depth of the main channel is a little less than 1 m. An example of a typical cross section is presented in Figure 17. Figures for rest of the cross sections as well as measurement data of the cross sections are presented in Blomfeldt (2008).

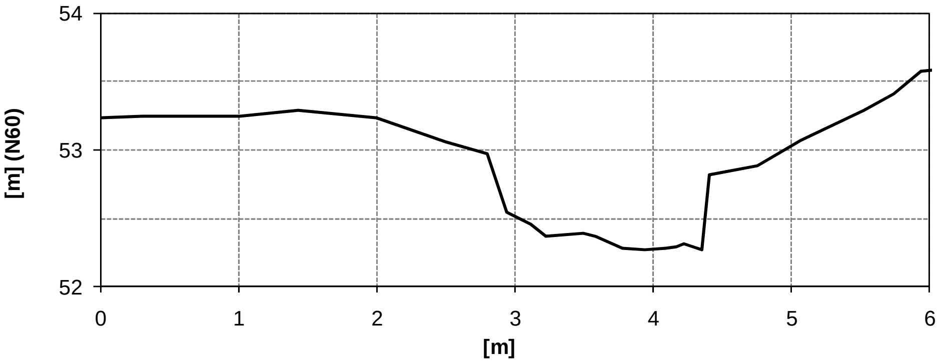

Figure 17. CS80 presented as an example of a typical cross section in investigated part of the stream Ridalinpuro.

Channel bottom profile is presented in Figure 18. The average bottom slope between first and last cross section is 0.65 %.

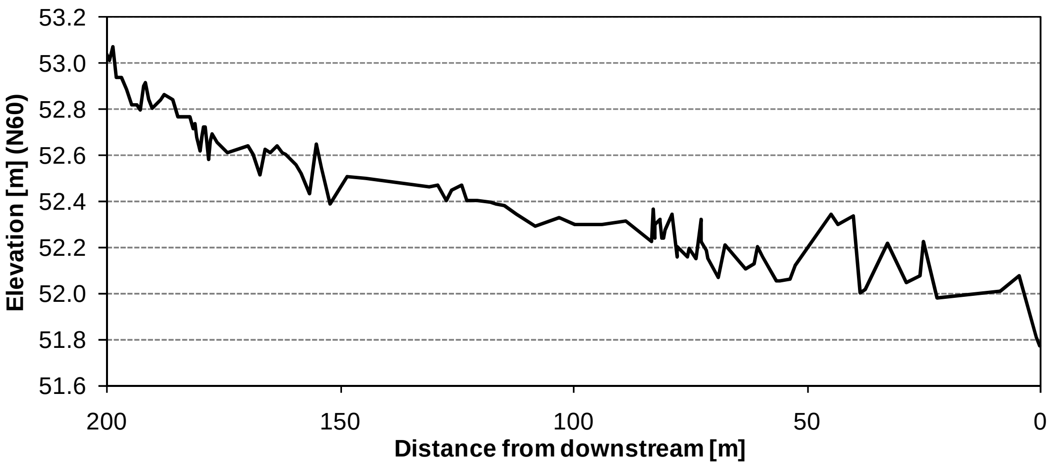

Figure 18. Ridalinpuro bottom elevation profile in investigated part of the stream.

Discharge was measured using Thomson overflow weir (Image 1) which was set up at the far most upstream part of the reach in August 2007. The flow depth in the weir was continuously measured with a pressure gauge and converted to discharge using equation (4.1) (Blomfeldt, 2008).

$$Q = 2227.5 C_f \tan\left(\frac{\varphi}{2}\right) h^{\frac{5}{2}} \tag{4.1}$$

where Q is discharge, C_f is the flow coefficient of 0.62, φ is the V-angle of 90° and h is the water depth over the V-shape. The maximum capacity of the weir within the V-shape is approximately 220 l/s after which the flow must be recalculated taking account the flow over upper rectangular section. (Blomfeldt, 2008)

Image 1. Discharge measurement weir upstream from investigated reach during low flow in July 2009.

Blomfeldt (2008) presents Manning resistance coefficients for the investigated stream during two flow scenarios, 114 l/s and 204 l/s. The coefficients are computed for reaches between measurement poles using equations of Bernoulli and Manning (equations (2.20) and (2.49) in this thesis) with known discharges, water depths and cross sections. Resistance coefficients for different reaches of the stream are presented for the two flow scenarios in Table 2. Cross sections where each of the resistance coefficients are applied in developed hydraulic model are also presented in the table. Resistance coefficients computed for the reach between poles V and VI are also used for the two cross sections upstream of the pole VI since there are no other resistance coefficients available for these cross sections.

Table 2. Manning's n for stream sections between measurement poles at discharges of 114 l/s and 204 l/s (Blomfeldt, 2008). The cross sections where each resistance coefficient is applied in the hydraulic model are shown as well.

Poles	Manning's n 114 l/s	Manning's n 204 l/s	Applied cross sections
I - II	0.104	0.086	CS0, CS20
II- III	0.245	0.197	CS40, CS60, Pole III
III - IV	0.132	0.141	CS80, CS100, CS120, Pole IV
IV - V	0.074	0.056	CS140, Pole V
V - VI	0.131	0.111	CS160, Pole VI, CS180, CS200

4.1.3 Modeling results

The simulation model provides water surface levels for each cross section, and the longitudinal water surface profiles based on the results of the two simulated discharges are presented in Figure 19. To be able to visually compare computed water levels to measured ones, also the measured water levels at each measuring pole are plotted in Figure 19.

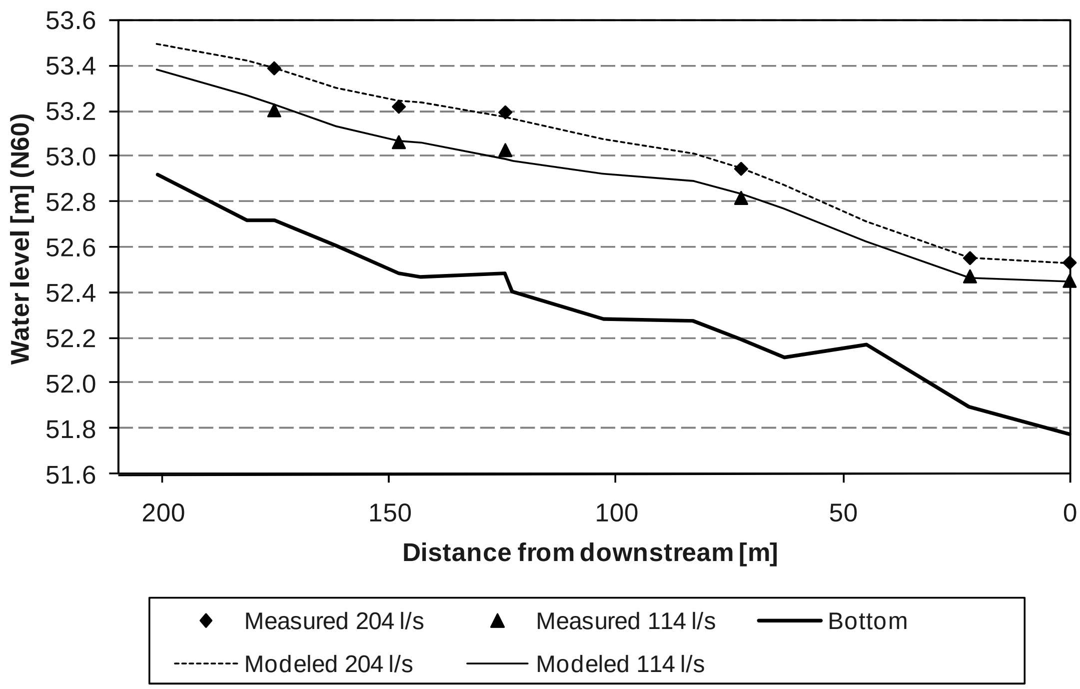

Figure 19. Simulated longitudinal water surface profiles and measured water level values for discharges of 114 l/s and 204 l/s.

In Figure 20 the accuracy of the simulations is presented by showing the error between measured and simulated water depths at each measuring pole.

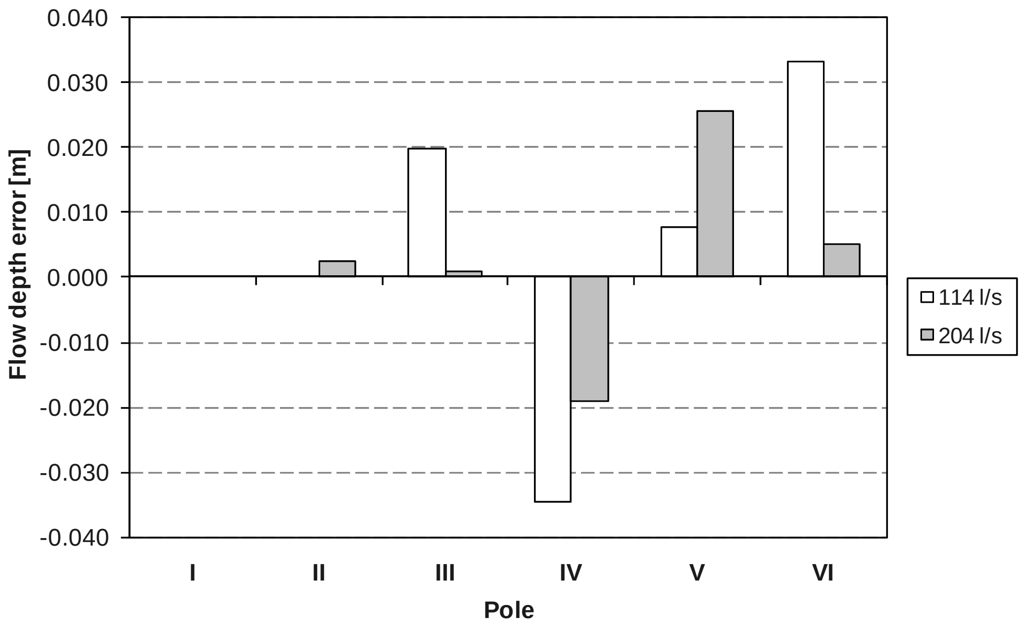

Figure 20. Flow depth error between modeled and measured water levels for discharges of 114 l/s and 204 l/s. A positive value represents overestimation of water levels.

Figures 19 and 20 show a small overestimation of water level for most of the poles, and a small underestimation of water level for pole IV. The highest error for the smaller flow is at Pole IV at a magnitude of 0.034 m, which corresponds to 6.3 % of the measured flow depth of 0.55 m. For the larger flow the highest error is at Pole V; 0.026 m or 3.5 % of the measured flow depth of 0.73 m. The average error is 0.02 m and 0.01 m for the discharges of 114 l/s and 204 l/s respectively. These errors correspond to 3.55 % and 1.49 % of flow depths in measuring poles. The averages are computed using the mean absolute error at poles II to VI, since the measured water level at Pole I acts as a lower boundary condition for the model and hence the simulated and measured water depths always agree.

4.1.4 Discussion about the results

The use of too large resistance coefficients might be the reason for overestimation of water depths in most of the poles. However, at the pole IV the water depth was too small, which suggests use of too small resistance coefficient. Because the resistance coefficients are computed only between measurement poles, they are average values for those reaches. They are however used for all the cross sections between the poles, and thus the local variations in resistance are not taken account. If resistance coefficients computed for shorter reaches were used instead of current values, the results might well be even better.

The model did somewhat poorer job on the smaller flow situation, which suggest that additional energy losses caused by low flow might have an effect on the results. During low flow relative effect of every single large rock or other obstruction in the stream emphasizes and thus the local variations in resistance are even larger than for bigger flows.

Regardless of the small differences between simulated and measured water levels, the results for both flow situations are clearly acceptable. There are always uncertainties related to measuring cross sections and discharge that will result as errors in simulations. These errors can be minimized with right selection of cross section locations and careful measurements, but their effect cannot be completely undone. In addition, the many assumptions made in deriving Saint Venant equations (see Chapter 2.2) are not necessarily true for natural stream and thus may cause small errors to results. The effect of separate false assumption is however hard to predict, and with errors this small it in any case appears to be like the model predicts flows in a single reach of open channel correctly if the input data is of good quality. To verify that the model works also in a channel with hydraulic structures or in a branched channel network it should be tested with appropriate data.

4.2 Kylmäoja

4.2.1 Watershed and stream

Kylmäoja is an urban stream located in the eastern parts of the city of Vantaa with its headwaters within the northern neighboring municipality of Tuusula in Southern Finland (Figure 21). The stream is a tributary of River Keravanjoki, which drains trough river Vantaanjoki into the Gulf of Finland in the middle of the Helsinki Metropolitan Area.

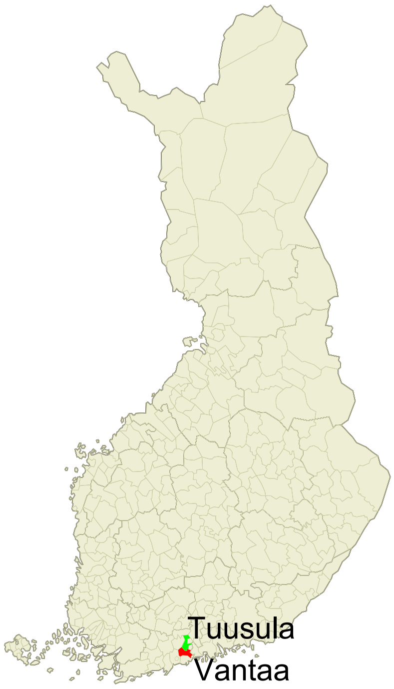

Figure 21. Municipalities of Tuusula (green) and Vantaa (red) in Southern Finland where Kylmäoja watershed is located.

Kylmäoja catchment (20.8 km^2) is highly urbanized although land-use coverage presents high variability. Krebs (2009) estimated that the impervious areas covered 19 % of the whole catchment in 2007 and predicted that the imperviousness will rise to 26 % of the catchment area by 2030. The stream consists of three main branches, the eastern, the central and the western branch which merge into a main stream in Ristipuro approximately in the middle of the catchment area (Figure 22). Especially the western branch of the stream is interesting since it collects water from Helsinki-Vantaa airport and has suffered for a long time from de-icing chemical contamination caused by runoff from the airport. According to Tiensuu (2008) the ecological status of the whole stream is bad and physicochemical status in different branches is either moderate or poor. The western branch is organically polluted, most likely due to the contaminant load from the airport (Tiensuu, 2008).

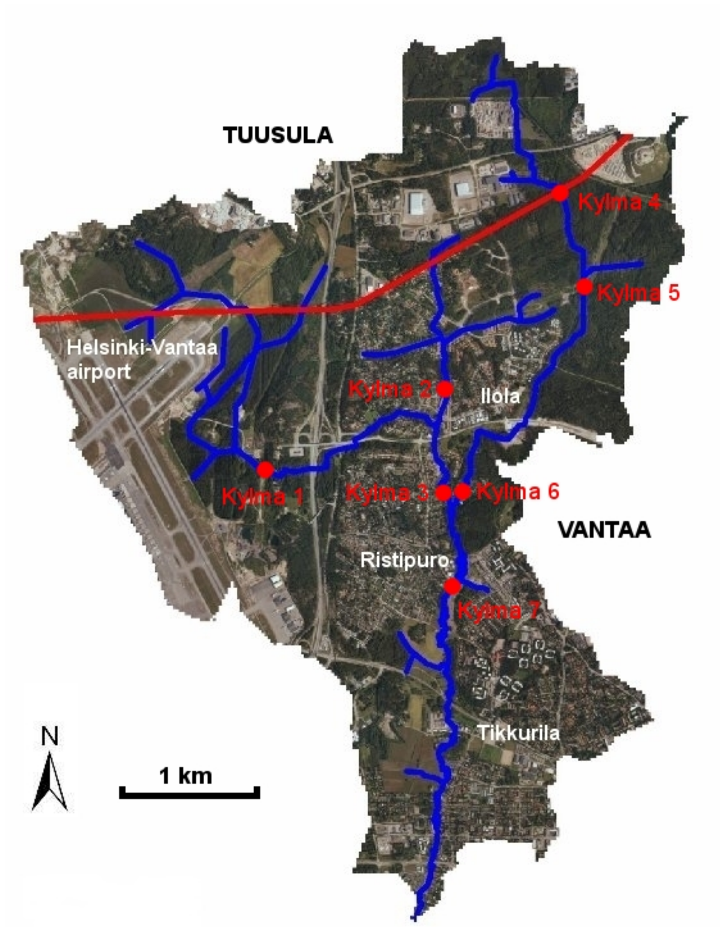

Figure 22. Kylmäoja stream and catchment shown on orthophoto from the area. Locations of the sensorstations are presented with red dots. Areas above the red line belong to municipality of Tuusula and areas below the line to municipality of Vantaa. (Modified from Krebs, 2009; original orthophoto from City of Vantaa, 2007)

4.2.2 Application of the hydraulic model to stream Kylmäoja

As a part of the HYDROSYS project the developed hydraulic model is to be used in future to simulate flows in the stream Kylmäoja. There are also plans to implement simple physical and chemical models on top of the hydraulic model to simulate material transport and chemical processes in the stream. The processes that especially are of interest are sediment transport caused by construction and other human activities as a part of the urbanization in the catchment, and degradation of the stream because of the use of de-icing chemicals in the airport.

The input data for the hydraulic model is collected from several sources. The cross sections are obtained from digital elevation model (DEM) based on LiDAR-data. Kylmäoja is a small stream, its width varies approximately from half a meter to couple of meters, and thus the cross sections derived from for example a standard 1 meter resolution DEM are not likely to be accurate enough but the resolution of the DEM must be increased. With the available LiDAR-data from the area this is a difficult task. Extra challenge is posed by the urban environment, which requires manual correction of the DEM near bridges and some other urban structures.

The discharge data is obtained from water level measurements which are converted to discharge values with the aid of stage-discharge curves. During fall 2009 and spring 2010 measurement campaigns took place in Kylmäoja, when flow velocity measurements were taken in seven locations in stream Kylmäoja (Figure 22, p. 61), and stage-discharge relations were established for these locations. The flow velocities were measured using propeller-type current meter and reduced point method. The discharge was further derived from the velocity measurements using mean-section method described for example in Herschy (1999). In future only water level measurements are to be done in Kylmäoja. The measured water level can then be used directly as a lower boundary condition in the hydraulic model, or it can be converted to discharge using stage-discharge curves and used as upper boundary condition instead.

Resistance coefficients for the hydraulic model must be acquired most likely from the literature since no discharge or water level measurements are taken in the stream except in seven measurement station locations. These stations are too far apart to reasonably compute energy losses in the stream and further to compute resistance coefficients from the losses. Some estimates of the resistance coefficients for some parts of the stream could possibly be obtained from available LiDAR data, although it might well not be accurate enough to be useful for this purpose.

There exist several culverts in Kylmäoja which should be taken account in simulations. Locations of these culverts can be obtained from available maps or from the LiDAR data, but sizes and elevations of culverts must be investigated on site. Culverts are also likely to gather all kinds of obstructions inside and on the inlet side of them, and during on site visits those can be removed and thus the prediction capacity of the model improved.

Since there was no cross section data available from Kylmäoja at the time of writing this thesis, the model could not be tested in this site. Although the model worked very well when applied to Ridalinpuro, the results from Kylmäoja are not expected to be as good as the former. There are several reasons for this, which are identified as follows.

First of all, there are many uncertainties related to the input data of the model. As discussed previously, the cross sections obtained from LiDAR data might not be accurate enough for a small stream like Kylmäoja. The discharge data is obtained from stage-discharge curves and is thus only an estimate of the true discharge. Furthermore, there are uncertainties related to all stages of acquiring data to create a stage-discharge relation. Unsuitability of resistance values presented in literature for small streams was previously discussed in Chapter 2.6. The effect of culverts to the flow in Kylmäoja is unknown since there is no measurement data available. Without verification it is also hard to predict how well the effect of culverts is simulated in the model. The amount of water coming to the stream as lateral inflow is also unknown, and whether or not it should be taken account in modeling.

Another major problem related to flow simulations in Kylmäoja is the lack of calibration data. Without measured water levels along the stream results of the simulations can not be evaluated but for very roughly. Water levels acquired from the LiDAR data can possibly provide a solution to roughly calibrate the model, if somehow the discharge from the time of LiDAR measurements can be estimated. This is however unlikely.

With all these difficulties it is going to be a challenging, yet unquestionably interesting, task to apply the model to Kylmäoja and see whether any reasonable results are obtained. To avoid the numerical problems related to low flows, it might be necessary to either modify the cross sections or place them so that these problems are less likely to occur. It is also usually wise to begin with as large flow as possible and little by little move towards smaller flows.

5 DISCUSSION AND CONCLUSIONS

The main objective of the study was to develop a hydraulic model to meet the needs of a project HYDROSYS, where intention is to simulate flow in a small urban stream of Kylmäoja suffering e.g. from pollution generated at the nearby airport and transported to the stream with runoff. The baseline static data needed by the model resides in a spatial database, and another goal of this thesis was to create a database for the input and output data of the model. As a result of this thesis, these two components were built, and were then connected with a middleware program written in Perl in order to ensure that they work together and to provide a simple user interface for the model-database-system.

Since the stream Kylmäoja consists of a main branch and its tributaries the model was built in way that it can handle flow in a channel network consisting of three branches. As is often the case with urban streams, also in Kylmäoja many culverts are installed into the stream. To better take account the effect of these hydraulic structures to the flow, a simple solution using additional energy loss parameter for culverts was implemented in the model.

The constructed system was applied to a small urban stream Ridalinpuro in Nummela to verify that it works. Results of the simulation were compared to measured water levels at five cross sections in the stream. Simulated water levels agreed well with measured levels, the average errors being 3.55 % and 1.49 % of the measured level for the two model scenarios. The errors are acceptable and thus confirm that the model works when applied to single reach of open channel that does not have any hydraulic structures.

Application to stream Kylmäoja was not possible at the time of writing this study because of the lack of baseline data needed by the model. It also prevented verification of the model with real data to confirm that it computes water flow correctly also in a channel network consisting of more than one branch, or in a stream with culverts. In future these tests have to be carried out to make sure that the developed system functions in all situations it is intended to.

Modeling of flow in small and urban streams is challenging, and a minor goal of the study was to investigate what challenges there exists. Resistance coefficient estimation is always one of the most difficult tasks in flow modeling and especially in the case of small streams. Problems may occur because of rapid and large variations in the flow, which affect to actual resistance in the stream and further to results of simulation as usually only one value for resistance is used in the model for the whole flow range. Also, estimation of a resistance coefficient even for a single flow situation in a small stream is difficult, since the methods commonly used are more suitable for bigger streams. Another challenge is the effect of natural and man-made obstructions or structures in the stream, such as logs, large rocks or culverts, which cause local energy losses and thus alter the flow. In modeling, the effect of man-made structures can be taken account to some level, but local energy losses caused by natural elements are more difficult, often even impossible to estimate. Topography of the channel has to be known accurately, and thus remote sensing methods suitable for cross section derivation for bigger streams, e.g. LiDAR, might not work as well with small streams. Lateral inflows can usually be neglected when modeling flow in rivers, but in small streams they might have a significant impact to flow and thus should be included. The amount of lateral inflow is however difficult to estimate, since it is caused both by surface runoff directly to the stream and water flowing to the stream via storm water systems which both may be challenging to quantify. Lastly, low flows can cause numerical difficulties which in the worst case prohibit continuation of the computation.

Decision to omit lateral inflow from the developed model causes errors to the simulation in the case that there is remarkable amount of flow coming to the stream between the cross sections. In Kylmäoja, notable amounts of lateral inflow are expected during snow melt period as direct runoff to the stream, and on the other hand during intense rains from storm water drainage systems. Estimating the volume of lateral inflow is however difficult and including it in the computation would increase the uncertainty of the results. Furthermore, it is expected that the time of computation would increase if lateral inflow was taken into account, especially if it was given as an input parameter for each cross section and each time step.

In its present state the model, and furthermore the built model-database-system, provides a platform that can be used as such to simulate flow in single reaches of open channel. Chosen implicit solution scheme with double sweep algorithm make the model computationally fast, and the use of modular structure enables exploiting only the computational part of the model as has been done in the HYDROSYS project. The model is easily modified or extended to suit different needs, either in this project or for some other use, and is thus applicable also outside the scope of this project. However, the most important improvements considering the case of Kylmäoja would be handling of lateral inflow and handling of a stream network consisting of more than three branches. Also mathematical handling of problems caused by low flows is recommended to be included in the model, since they are expected to occur when applying model to Kylmäoja.

In future it would be interesting to compare simulation results obtained using accurate, field measured cross sections against results obtained using cross sections derived from remote sensing data in a small stream. In fact there are plans to do this kind of comparison for Ridalinpuro stream, from where there exists both on-site measured cross sections and LiDAR data that can be used to derive cross sections for the same reach of the stream. Also the effect of using only one resistance coefficient for different flow scenarios could be investigated, since there is measured water level and discharge data for several flow situations for Ridalinpuro stream.

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APPENDIX A. COEFFICIENTS FOR DISCRETIZED AND LINEARIZED SAINT VENANT EQUATIONS AND FOR THE RECURRENCE RELATIONSHIPS.

$$A1_j = -\theta$$

$$B1_j = \frac{1}{2} b_{j+1/2}^{n+\theta} \frac{\Delta x}{\Delta t}$$

$$C1_j = \theta$$

$$D1_j = B1_j$$

$$E1_j = -(1-\theta)\left(Q_{j+1}^n - Q_j^n\right) + B1_j\left(y_j^n + y_{j+1}^n\right) + q_{j+1/2}^{n+\theta}\Delta x$$

$$A2_j = \frac{1}{2}\left(\frac{\Delta x}{\Delta t} + g\Delta x \frac{\left|Q_j^n\right| A_{j+1/2}^{n+1/2}}{\left(K^2\right)_j^{n+1/2}}\right) - \beta \frac{Q_j^n}{A_j^{n+1/2}} + \frac{1}{2}\left(C_c + C_e\right)_j^{n+1/2} \frac{\left|Q_j^n\right| A_{j+1/2}^{n+1/2}}{\left(A_s^2\right)_j^{n+1/2}}$$

$$B2_j = -\theta g A_{j+1/2}^{n+\theta}$$

$$C2_j = \frac{1}{2}\left(\frac{\Delta x}{\Delta t} + g\Delta x \frac{\left|Q_{j+1}^n\right| A_{j+1/2}^{n+1/2}}{\left(K^2\right)_{j+1}^{n+1/2}}\right) + \beta \frac{Q_{j+1}^n}{A_{j+1}^{n+1/2}}$$

$$D2_j = -B2_j$$

$$E2_j = \frac{1}{2}\frac{\Delta x}{\Delta t}\left(Q_j^n + Q_{j+1}^n\right) - gA_{j+1/2}^{n+\theta}(1-\theta)\left(y_{j+1}^n - y_j^n\right)$$

$$P_j = -\frac{C1_j}{A1_j F_j + B1_j}$$

$$O_j = -\frac{D1_j}{A1_j F_j + B1_j}$$

$$R_j = \frac{E1_j - A1_j G_j}{A1_j F_j + B1_j}$$

$$F_{j+1} = -\frac{eO_j + D2_j}{eP_j + C2_j}$$

$$G_{j+1} = \frac{E2_j - A2_j G_j - eR_j}{eP_j + C2_j}$$

$$e = A2_j F_j + B2_j$$

APPENDIX B. FLOWCHART OF THE UNSTEADY COMPUTATION.