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formulas for irregular  
sections

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# Uniform flow formulas for irregular sections

E. Spada<sup>1</sup>, T. Tucciarelli<sup>1</sup>, M. Sinagra<sup>1</sup>, V. Sammartano<sup>2</sup>, and G. Corato<sup>3</sup>

<sup>1</sup>Dipartimento di Ingegneria Civile, Ambientale, Aerospaziale, dei Materiali (DICAM), Università degli studi di Palermo, Viale delle scienze, 90128, Palermo, Italy

<sup>2</sup>Dipartimento di Ingegneria Civile, dell'Energia, dell'Ambiente e dei Materiali (DICEAM), Università Mediterranea di Reggio Calabria, Via Graziella, 89122, Reggio Calabria, Italy

<sup>3</sup>Centre de Recherche Public “Gabriel Lippmann”, 41 rue du Brill, 4422 Belvaux, Luxembourg

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Correspondence to: E. Spada (eleonora.spada@unipa.it)

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## Abstract

Two new methods for uniform flow discharge computation are presented, validated and compared with other available formulas. The first method derives from the well-known Huthoff algorithm, which is first shown to be dependent on the way the river cross-section is discretized into several sub-sections. The second method assumes the vertically averaged longitudinal velocity to be a function only of the friction factor and of the so-called “local hydraulic radius”, computed as the ratio between the integral of the elementary areas around a given vertical and the integral of the elementary solid boundaries around the same vertical. Both integrals are weighted with a linear shape function, equal to zero at a distance from the integration variable which is proportional to the water depth according to an empirical coefficient  $\beta$ . Both formulas are validated against (1) laboratory experimental data, (2) discharge hydrographs measured in a real site, where the friction factor is estimated from an unsteady-state analysis of water levels recorded in two different river cross sections, (3) the 3-D solution obtained using the commercial ANSYS CFX code, computing the steady state uniform flow in a short reach of a prismatic channel, with known water level in the downstream section.

## 1 Introduction

Both dynamic and diffusive forms of the Saint Venant equation include a closure relationship linking maximum water depth inside the cross section, discharge and energy slope (or the piezometric gradient in its diffusive form). This closure relationship is assumed to be the same relationship holding in the case of uniform flow. For this reason, an accurate prediction of uniform flow in channels with compound or irregular section is a central issue for good shallow water modeling.

The uniform flow formula almost universally applied is still the Chezy equation (Herschel, 1897). The advantage of using the Chezy equation is that the associated Manning’s coefficient has been calibrated worldwide for several types of bed surface and

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a single value is ready to use for each application. However, it is well known that the Chezy equation was derived from laboratory measurements taken in channels with a regular, convex cross-sectional shape. When the section results from the union of different parts, each with a strongly different average water depth, one of two options is usually selected. The first option, called Single Channel Method (SCM) is simply to ignore the problem. This leads to strong underestimation of the discharge, because the Chezy formula assumes a homogeneous vertically averaged velocity and this homogeneous value provides strong energy dissipation in the parts of the section with lower water depths. The second option, called Divided Channel Method (DCM) is to compute the total discharge as the sum of the discharges flowing in each convex part of the section (called subsection), assuming a single water level for all parts (Chow, 1959; Shiono et al., 1999; Myers and Brennan, 1990). In this approach, the wet perimeter of each subsection is restricted to the component of the original one pertaining to the subsection, but the new components shared by each couple of subsections are neglected. This is equivalent to neglecting the shear stresses coming from the vortices with vertical axes (if subsections are divided by vertical lines) and considering additional resistance for higher velocities, which results in overestimation of discharge capacity (Lyness et al., 2001).

Knight and Hamed (1984) compared the accuracy of several subdivision methods for compound straight channels by including or excluding the vertical division line in the computation of the wetted perimeters of the main channel and the floodplains. However, their results show that conventional calculation methods result in larger errors. Wormleaton et al. (1982) and Wormleaton and Hadjipanios (1985) also discussed, in the case of compound sections, the horizontal division through the junction point between the main channel and the floodplains. Their studies show that these subdivision methods cannot well assess the discharge in compound channels.

The interaction phenomenon in compound channels has also extensively studied by many other researchers (e.g., Sellin, 1964; Knight and Demetriou, 1983; Stephenson and Kolovopoulos, 1990; Rhodes and Knight, 1994; Bousmar and Zech, 1999; van



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A simplified version of the EDM, called Interactive Divided Channel Method (IDCM), was proposed by Huthoff et al. (2008). In IDCM lateral momentum is considered negligible and turbulent stress at the interface is assumed to be proportional to the span wise kinetic energy gradient through a dimensionless empirical parameter  $\alpha$ . IDCM has the strong advantage of using only two parameters,  $\alpha$  and the friction factor,  $f$ . Nevertheless, as shown in the next section,  $\alpha$  depends on the way the original section is divided.

An alternative approach could be to simulate the flow structure in its complexity by using a three-dimensional code for computational fluid dynamics (CFD). In these codes flow is represented both in terms of transport motion (mean flow) and turbulence by solving the Reynolds Averaged Navier Stokes (RANS) equations (Wilcox, 2006) coupled with turbulence models. These models allow closure of the mathematical problem by adding a certain number of additional partial differential transport equations equal to the order of the model. In the field of the simulation of industrial and environmental laws second order models (e.g.  $k-\varepsilon$  and  $k-\omega$  models) are widely used. Nonetheless, CFD codes need a mesh fine enough to solve the boundary layer (Wilcox, 2006), resulting in a computational cost that can be prohibitive even for river of few km.

In this study two new methods, aimed to represent subsection interactions in a compound channel, are presented. Both methods estimate the discharge as an integral of the vertically averaged velocities. The first method, named “INtegrated Channel Method” (INCM), derives from the previous Huthoff formula, which is shown to give results depending on the way the river cross section is discretized in sub-sections. The same dynamic balance adopted by Huthoff is written in differential form, but its diffusive term is weighted according to a  $\xi$  coefficient proportional to the local water depth.

The second one, named “local hydraulic radius method” (LHRM), derives from the observation that, in the Manning formula, the mean velocity per unit energy gradient is proportional to a power of the hydraulic radius. It should then be possible to get the total discharge as an integral, along the span wise direction, of the elementary values computed around each vertical, using for each elementary value the Manning formula,

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but also changing the original hydraulic radius with a “local” one. This “local” hydraulic radius should take into account the effect of the surrounding section geometry, up to a maximum distance which is likely to be proportional to the local water depth, according to an empirical  $\beta$  coefficient. The method gives up the idea of solving the Reynolds equations, due to the uncertainty of its parameters, but relies on the solid grounds of the historical experience of the Manning equation.

The present paper is organized as follows: two of the most popular approaches adopted for discharge estimation are explained in details, along with the proposed INCM and LHRM methods. The  $\xi$  and  $\beta$  parameters of respectively the INCM and LHRM methods are then calibrated from available lab experimental data and a sensitivity analysis is carried out. The INCM and LHRM methods are finally validated according to three different criteria. The first criterion is comparison with other series of the previous laboratory data, not used for calibration. The second criterion is comparison with discharge data measured in one section of the Alzette river Basin (Luxembourg). Because the friction factor is not known a priori, INCM and LHRM formulas are applied in the context of the indirect discharge estimation method, which simultaneously estimates the friction factor and the discharge hydrograph from the unsteady state water level analysis of two water level hydrographs measured in two different river sections. The third validation criterion is comparison with results of a 3-D numerical solver, applied to a small reach of the Alzette river. Conclusions follow.

## 2 Divided Channel Method (DCM) and Interactive Divided Channel Method (IDCM)

In the DCM method the river section is divided into subsections with uniform velocities and roughness (Chow, 1959). Division is made by vertical lines and no interaction between adjacent subsections is considered. Discharge is obtained by summing the

contributions of each subsection, obtained applying of the Manning formula:

$$q = \sum_i q_i = \sum_i \frac{R_i^{2/3} A_i}{n_i} \sqrt{S_{f_i}}, \quad (1)$$

where  $q$  is the total discharge,  $A_i$ ,  $R_i$  and  $n_i$  are the area, the hydraulic radius and the Manning's roughness coefficient of each sub section  $i$  of a compound channel and  $S_f$  is the energy slope, assumed constant across the river section.

In order to model the interaction between adjacent subsections of a compound section, the Reynolds and the continuity equations can be coupled (Shiono and Knight, 1991), to get:

$$\rho \frac{\partial}{\partial y} (H \bar{U}_v \bar{V}_d) = \rho g H S_0 + \frac{\partial}{\partial y} (H \bar{\tau}_{xy}) - \tau_b \left( 1 + \frac{1}{s^2} \right)^{1/2}, \quad (2)$$

where  $\rho$  is the water density,  $g$  is the gravity acceleration,  $y$  is the abscissa according to the lateral direction,  $U$  and  $V$  are respectively the velocity components along the flow  $x$  direction and the lateral  $y$  direction,  $H$  is the water depth, the sub-index  $d$  marks the vertically averaged quantities and the bar the time average along the turbulence period,  $S_0$  is the bed slope,  $s$  is the section lateral slope, and  $\tau_b$  is the bed shear stress. The turbulent stress is given by the eddy viscosity equation, that is:

$$\bar{\tau}_{xy} = \rho \bar{\varepsilon}_{xy} \frac{\partial U_d}{\partial y}, \quad (3a)$$

$$\bar{\varepsilon}_{xy} = \lambda U_* H, \quad (3b)$$

where the friction velocity  $U_*$  is set equal to:

$$U_* = \left( \frac{f}{8} \right)^{1/2} U_d, \quad (4)$$

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and  $f$  is the friction factor, depending on the bed material. The analytical solution of Eqs. (2)–(4) can be found only if the left hand side of Eq. (2) is zero, which is equivalent to neglecting secondary flows. Other solutions can only be found by assuming a known  $\Gamma$  value for the lateral derivative. Moreover,  $\lambda$  is another experimental factor depending on the section geometry. The result is that the solution of Eq. (2) strongly depends on the choice of two coefficients,  $\lambda$  and  $\Gamma$ , which are additional unknowns with respect to the friction factor  $f$ .

In order to reduce to one the number of empirical parameters (in addition to  $f$ ) Huthoff et al. (2008) proposed the so-called Interactive Divided Channel Method (IDCM).

Integration of Eq. (2) over each  $i$ th subsection, neglecting the averaged flow lateral momentum, leads to:

$$\rho g A_i S_0 = \rho g f_i P_i U_i^2 + \tau_{i+1} H_{i+1} + \tau_i H_i, \quad (5)$$

where the left-hand side of Eq. (5) is the gravitational force per unit length, proportional to the density of water  $\rho$ , to the gravity acceleration  $g$ , to the cross-sectional area  $A_i$ , and to the stream wise channel slope  $S_0$ . The terms at the right-hand side are the friction forces, proportional to the friction factor  $f$  and to the wet solid boundary  $P_i$ , as well as the turbulent lateral momentum on the left and right sides, proportional to the turbulent stress  $\tau$  and to the water depth  $H$ .

Turbulent stresses are modelled quite simply as:

$$\tau_{i+1} = \frac{1}{2} \rho \alpha (U_{i+1}^2 - U_i^2), \quad (6)$$

where  $\alpha$  is a dimensionless interface coefficient,  $U_i^2$  is the square of the vertically averaged velocity and  $\tau_i$  is the turbulent stress along the plane between subsection  $i - 1$  and  $i$ . If subsection  $i$  is the first (or the last) one, velocity  $U_{i-1}$  (or  $U_{i+1}$ ) is set equal to zero.

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Following a wall-resistance approach (Chow, 1959), the friction factor  $f_i$  is computed as:

$$f_i = \frac{n_i^2}{R_i^{1/3}}, \quad (7)$$

where  $n_i$  is the Manning's roughness coefficient and  $R_i (= A_i/P_i)$  is the hydraulic radius of subsection  $i$ .

Equation (5) forms a system with an order equal to the number  $m$  of subsections, which is linear in the  $U_i^2$  unknowns. The results are affected by the choice of the  $\alpha$  coefficient, which is recommended by Huthoff et al. (2008), on the basis of lab experiments, equal to 0.02. Computation of the velocities  $U_i$  makes it easy to estimate discharge  $q$ .

IDCM has the main advantage of using only two parameters, the  $f$  and  $\alpha$  coefficients. On the other hand, it can be easily shown that  $\alpha$ , although it is dimensionless, depends on the way the original section is divided. The reason is that the continuous form of Eq. (5) is given by:

$$\rho g \left( HS_0 - \frac{fU^2}{\cos \theta} \right) = \frac{\partial}{\partial y} (\tau H), \quad (8)$$

where  $\theta$  is the bed slope in the lateral direction. Following the same approach as the IDCM, if we assume the turbulent stress  $\tau$  to be proportional to both the velocity gradient in the lateral direction and to the velocity itself, we can write the right-hand side of Eq. (8) in the form:

$$\frac{\partial}{\partial y} (\tau H) = \frac{\partial}{\partial y} \left( \frac{\alpha_H}{2} \rho U \frac{\partial U}{\partial y} H \right), \quad (9)$$

and Eq. (8) becomes:

$$\rho \left( gHS_0 - \frac{fU^2}{\cos \theta} \right) = \frac{\partial}{\partial y} \left( H \frac{\partial}{\partial y} (\alpha_H \rho U^2) \right). \quad (10)$$

In Eq. (10)  $\alpha_H$  is no longer dimensionless, but is a length. To get the same Huthoff formula from numerical discretization of Eq. (10), we should set:

$$\alpha_H = 0.02\Delta y, \quad (11)$$

where  $\Delta y$  is the subsection width, i.e. the integration step size. This implies that the solution of Eq. (10), according to the Huthoff formula, depends on the way the equation is discretized and the turbulence stress term on the r.h.s. vanishes along with the integration step size.

### 3 The new methods

#### 3.1 Integrated Channel Method (INCM)

INCM derives from the IDCM idea of evaluating the turbulent stresses as proportional to the gradient of the squared averaged velocities, leading to Eqs. (6) and (10). Observe that dimensionless coefficient  $\alpha$ , in the stress computation given by Eq. (6), can be written as the ratio between  $\alpha_H$  and the distance between verticals  $i$  and  $i + 1$ . For this reason, coefficient  $\alpha_H$  can be thought of as a sort of mixing length, related to the scale of the vortices with horizontal axes. INCM assumes the optimal  $\alpha_H$  to be proportional to the local water depth, because water depth is at least an upper limit for this scale, and the following relationship is applied:

$$\alpha_H = \xi H, \quad (12)$$

where  $\xi$  is an empirical coefficient to be further estimated.

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## 3.2 Local hydraulic radius method (LHRM)

LHRM derives from the observation that, in the Manning equation, the average velocity is set equal to:

$$V = \frac{R^{2/3}}{n} \sqrt{S_0}, \quad (13)$$

- 5 and has a one-to-one relationship with the hydraulic radius. In this context the meaning of a global parameter, measuring the interactions of the particles along all the section as the ratio between an area and a length. The inconvenience is that, according to Eq. (13), the vertically averaged velocities in points very far from each other remain linked anyway, because the infinitesimal area and the infinitesimal length around two
- 10 verticals are summed to the numerator and to the denominator of the hydraulic radius independently from the distance between the two verticals. To avoid this, LHRM computes the discharge as an integral of the vertically averaged velocities, in the following form:

$$q = \int_0^L h(y)U(y)dy, \quad (14)$$

- 15 where  $U$  is set equal to:

$$U = \frac{R_1^{2/3}}{n} \sqrt{S_0}, \quad (15)$$

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The FCF series were named F1, F2, F3, F6, F8 and F10; the Knight and Demetriou series were named K1, K2, K3 and K4. Series F1, F2, and F3 covered different floodplain widths, while series F2, F8, and F10 kept the floodplain widths constant, but covered different main channel side slopes. Series F2 and F6 provided a comparison between the symmetric case of two floodplains and the asymmetric case of a single floodplain. All the experiments of Knight and Demetriou (1983) were run with a vertical main channel wall, but with different  $B/b$  ratios. The series K1 has  $B/b = 1$  and its section is simply rectangular. The  $B/b$  ratio, for Knight's experimental apparatus, was varied by adding an adjustable side wall to each of the floodplains either in pairs or singly to obtain a symmetrical or asymmetrical cross section. The geometric and hydraulic parameters are shown in Table 1; all notations of the parameters can be found in Fig. 1 and  $S_0$  is the bed slope. The subscripts "mc" and "fp" of the side slope refer to the main channel and floodplain, respectively. Perspex was used for both main flume and floodplains in all tests. The related Manning roughness is  $0.01 \text{ m}^{-1/3} \text{ s}$ .

The experiments were run with several channel configurations, differing mainly for floodplain geometry (widths and side slopes) and main channel side slopes (see Table 1). The K series were characterised by vertical main channel walls. More information concerning the experimental setup can be found in Table 1 (Knight and Demetriou, 1983; Knight and Sellin, 1987; Shiono and Knight, 1991).

Four series, named F1, F2, F3 and F6, were selected for calibration of the  $\beta$  coefficient, using the Nash–Sutcliffe (NS) index of the measured and the computed flow rates as a measure of the model's performance (Nash and Sutcliffe, 1970).

The remaining three series, named F2, F8 and F10, plus four series from Knight and Demetriou, named K1 K2, K3 and K4, were used for validation (no.) 1, as reported in

the next section. NS is given by:

$$NS = \left[ 1 - \frac{\sum_{j=1,2} \sum_{i=1,N_j} \sum_{K=1,M_{N_j}} \left( q_{i,j,k}^{\text{obs}} - q_{i,j,k}^{\text{sim}} \right)^2}{\sum_{j=1,2} \sum_{i=1,N_j} \sum_{K=1,M_{N_j}} \left( q_{i,j,k}^{\text{obs}} - \overline{q_{i,j,k}^{\text{obs}}} \right)^2} \right], \quad (18)$$

where  $N_j$  is the number of series,  $M_{N_j}$  is the number of tests for each series,  $q_{i,j,k}^{\text{sim}}$  and  $q_{i,j,k}^{\text{obs}}$  are respectively the computed and the observed discharge ( $j = 1$  for the FCF series and  $j = 2$  for the Knight series;  $i$  is the series index and  $K$  is the water depth index).  $\overline{q_{i,j,k}^{\text{obs}}}$  is the average value of the measured discharges.

Both  $\xi$  and  $\beta$  parameters were calibrated by maximizing the Nash–Sutcliffe (NS) index, computed using all the data of the four series used for calibration. See the NS vs.  $\xi$  and  $\beta$  curves in Fig. 2a and b.

Calibration provides optimal  $\xi$  and  $\beta$  coefficients respectively equal to 0.08 and 9. The authors will show in the next sensitivity analysis that even a one-digit approximation of the  $\xi$  and  $\beta$  coefficients provides a stable discharge estimation.

### 3.4 Sensitivity analysis

We carried out a discharge sensitivity analysis of both new methods using the computed  $\xi = 0.08$  and  $\beta = 9$  optimal values and the data of the F2 and K4 series. Sensitivities were normalized in the following form:

$$I_s = \frac{1}{q_{\text{INCM}}} \frac{\Delta q}{\Delta \xi}, \quad (19)$$

$$L_s = \frac{1}{q_{\text{LHRM}}} \frac{\Delta q}{\Delta \beta}. \quad (20)$$

The results of this analysis are shown in Table 2a for the F2 series, where  $H$  is the water depth and  $Q_{\text{meas}}$  the corresponding measured discharge.

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They show very low sensitivity of both the INCM and LHRM results, such that a one digit approximation of both model parameters ( $\xi$  and  $\beta$ ) should guarantee a computed discharge variability of less than 2 %.

The results of the sensitivity analysis, carried out for series K4 and shown in Table 2b, are similar to the previous ones computed for F2 series.

## 4 Validation criterion

### 4.1 Validation (no.) 1 – comparison with laboratory experimental data

A first validation of the two methods was carried out by using the calibrated parameter values, the same Nash–Sutcliffe performance measure and all the available experimental series. The results were also compared with results of DCM and IDCM methods, the latter applied using the suggested  $\alpha = 0.02$  value and five subsections, each one corresponding to a different bottom slope in the lateral  $y$  direction. The NS index for all data series is shown in Table 3.

The DCM results are always worse and are particularly bad for all the K series. The results of both the IDCM and INCM methods are very good for the two F series not used for calibration, but are both poor for the K series. The LHRM method is always the best and also performs very well in the K series. The reason is probably that the K series tests have very low discharges, and the constant  $\alpha = 0.02$ , the coefficient adopted in the IDCM method, does not fit the size of the subsections and Eq. (12) is not a good approximation of the mixing length  $\alpha_H$  in Eq. (11) for low values of the water depth. In Fig. 3a and b the NS curves obtained by using DCM, IDCM, INCM and LHRM, for series F2 and K4, are shown.

### 4.2 Validation (no.) 2 – comparison with field data

Although rating curves are available in different river sites around the world, field validation of the uniform flow formulas is not easy, for at least two reasons:

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1. The average friction factors  $f$  and related Manning's coefficient are not known as in the lab case and the results of all the formulas need to be scaled according to the Manning's coefficient;

2. River bed roughness does change, along with the Manning's coefficient, from one water stage to another (it usually increases along with the water level).

A possible way to circumvent the problem is to apply the compared methods in the context of a calibration problem, where both the average Manning's coefficient and the discharge hydrograph are computed from the known level hydrographs measured in two different river cross sections (Perumal et al., 2007; Aricò et al., 2009). The authors solved the diffusive wave simulation problem using one known level hydrograph as the upstream boundary condition and the second one as the benchmark downstream hydrograph for the Manning's coefficient calibration.

Although the accuracy of the results is restricted by several modeling assumptions, a positive indication about the robustness of the simulation model (and the embedded relationship between the water depth and the uniform flow discharge) is given by: (1) the match between the computed and the measured discharges in the upstream section, (2) the compatibility of the estimated average Manning's coefficient with the site environment.

The area of interest is located in the Alzette River basin (Gran-Duchy of Luxembourg) between the gauged sections of Pfaffenthal and Lintgen (Fig. 4). The river reach length is about 19 km, with a mean channel width of  $\sim 30$  m and an average depth of  $\sim 4$  m. The river meanders in a relatively large and flat plain about 300 m, with a mean slope of  $\sim 0.08\%$ .

The methodology was applied to a river reach 13 km long, between two instrumented sections, Pfaffenthal (upstream section) and Hunsdorf (downstream section), in order to have no significant lateral inflow between the two sections.

Events of January 2003, January 2007 and January 2011 were analysed. For these events, stage records and reliable rating curves are available at the two gauging sta-



tions of Pfaffenthal and Hunsdorf. The main hydraulic characteristics of these events, that is duration ( $\Delta t$ ), peak water depth ( $H_{\text{peak}}$ ) and peak discharge ( $q_{\text{peak}}$ ), are shown in Table 4.

In this area a topographical survey of 125 river cross sections was available. The hydrometric data were recorded every 15 min. The performances of the discharge estimation procedures were compared by means of the Nash–Sutcliffe criterion.

The results of the INCM and LHRM methods were also compared with those of the DCM and IDCM methods, the latter applied by using  $\alpha = 0.02$  and an average subsection width equal to 7 m. The computed average Manning's coefficients  $n_{\text{opt}}$ , reported in Table 5, are all consistent with the site environment, although they attain very large values, according to DCM and IDCM, in the 2011 event.

The estimated and observed dimensionless water stages in the Hunsdorf gauged site, for 2003, 2007 and 2011 events are shown in Figs. 5–7.

Only the steepest part of the rising limb, located inside the colored window of each figure, was used for calibration. The falling limb is not included, since it has a lower slope and is less sensitive to the Manning's coefficient value.

A good match between recorded and simulated discharge hydrographs can be observed (Figs. 8–10) in the upstream gauged site for each event.

For all investigated events the Nash–Sutcliffe efficiency  $NS_q$  is greater than 0.90, as shown in Table 6.

The error obtained between measured and computed discharges, with all methods of the same order as the discharge measurement error. Moreover, this measurement error is well known to be much larger around the peak flow, where the estimation error has a larger impact on the NS coefficient. The NS coefficients computed with the LHRM and INCM methods are anyway a little better than the other two.

### 4.3 Validation (no.) 3 – comparison with results of 3-D ANSYS CFX solver

The vertically averaged velocities computed using DCM, IDCM, INCM and LHRM were compared with the results of the well known ANSYS 3-D code, named CFX, applied

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to a prismatic reach with the irregular cross-section measured at the Hunsdorf gauged section of the Alzette river. The length of the reach is about four times the top width of the section.

In the homogeneous multiphase model adopted by CFX, water and air are assumed to share the same dynamic fields of pressure, velocity and turbulence and water is assumed to be incompressible. CFX solves the conservation of mass and momentum equations, coupled with the air pressure-density relationship and the global continuity equation in each node. Call  $\alpha_l$ ,  $\rho_l$ ,  $\mu_l$  and  $\mathbf{U}_l$  respectively the volume fraction, the density, the viscosity and the time averaged value of the velocity vector for phase  $l$  ( $l = w$  (water),  $a$  (air)), that is:

$$\rho = \sum_{l=w,a} \alpha_l \rho_l, \quad (21a)$$

$$\mu = \sum_{l=w,a} \alpha_l \mu_l, \quad (21b)$$

where  $\rho$  and  $\mu$  are the density and the viscosity of the “averaged” phase. The air density is assumed to be a function of the pressure  $p$ , according to the state equation:

$$\rho_a = \rho_{a,0} e^{\gamma(p-p_0)}, \quad (21c)$$

where the sub-index 0 marks the reference state values and  $\gamma$  is the air compressibility coefficient.

The governing equations are the following: (1) the mass conservation equation, (2) the Reynolds averaged continuity equation of each phase and (3) the Reynolds averaged momentum equations. Mass conservation implies:

$$\sum_{l=w,a} \alpha_l = 1. \quad (22)$$

The Reynolds averaged continuity equation of each phase  $l$  can be written as:

$$\frac{\partial \rho_l}{\partial t} + \nabla \cdot (\rho_l \mathbf{U}) = S_l, \quad (23)$$

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where  $S_I$  is an external source term. The momentum equation instead refers to the “averaged” phase and is written as:

$$\frac{\partial(\rho \mathbf{U})}{\partial t} + \nabla \cdot (\rho \mathbf{U} \otimes \mathbf{U}) - \nabla \cdot \left( \mu_{\text{eff}} \left( \nabla \mathbf{U} + (\nabla \mathbf{U})^T \right) \right) + \nabla p' = S_M, \quad (24)$$

where  $\otimes$  is the dyadic symbol,  $S_M$  is the momentum of the external source term  $S$ , and  $\mu_{\text{eff}}$  is the effective viscosity accounting for turbulence and defined as:

$$\mu_{\text{eff}} = \mu + \mu_t, \quad (25)$$

where  $\mu_t$  is the turbulence viscosity and  $p'$  is the modified pressure, equal to:

$$p' = p + \frac{2}{3} \rho k + \frac{2}{3} \mu_{\text{eff}} \nabla \cdot \mathbf{U}, \quad (26)$$

where  $k$  is the turbulence kinetic energy, defined as the variance of the velocity fluctuations and  $p$  is the pressure. Both phases share the same pressure  $p$  and the same velocity  $\mathbf{U}$ .

To close the set of six scalar equations (Eqs. 22, 23 (two) and 24 (three)), we finally apply the  $k$ - $\varepsilon$  turbulence model implemented in the CFX solver. The implemented turbulence model is a two equation model, including two extra transport equations to represent the turbulent properties of the flow.

Two-equation models account for history effects like convection and diffusion of turbulent energy. The first transported variable is turbulent kinetic energy,  $k$ ; the second transported variable is the turbulent dissipation,  $\varepsilon$ . The K-epsilon model has been shown (Jones, 1972; Launder, 1974) to be useful for free-shear layer flows with relatively small pressure gradients. Similarly, for wall-bounded and internal flows, the model gives good results, but only in cases where the mean pressure gradients are small.

The computational domain was divided using both tetrahedral and prismatic elements (Fig. 11). The prismatic elements were used to discretize the computational domain in the near-wall region over the river bottom and the boundary surfaces, where

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a boundary layer is present, while the tetrahedral elements were used to discretize the remaining domain. The number of elements and nodes, in the mesh used for the specific case are of the order respectively  $4 \times 10^6$  and  $20 \times 10^6$ .

A section of the mesh is shown in Fig. 12. The quality of the mesh was verified by using a pre-processing procedure by ANSYS® ICEM CFD™ (Ansys inc., 2006).

The six unknowns in each node are the pressure, the velocity components, and the volume fractions of the two phases. At each boundary node three of the first four unknowns have to be specified. In the inlet section a constant velocity, normal to the section, is applied, and the pressure is left unknown. In the outlet section the hydrostatic distribution is given, the velocity is assumed to be still normal to the section and its norm is left unknown. All boundary conditions are reported in Table 7.

The opening condition means that that velocity direction is set normal to the surface, but its norm is left unknown and a negative (entering) flux of both air and water is allowed. Along open boundaries the water volume fraction is set equal to zero. The solution of the problem converges towards two extremes: nodes with zero water fraction, above the water level, and nodes with zero air fraction below the water level.

On the bottom boundary, between the nodes with zero velocity and the turbulent flow a boundary layer exists that would require the modeling of micro scale irregularities. CFX allows the use, inside the boundary layer, of a velocity logarithmic law, according to an equivalent granular size. The relationship between the granular size and the Manning's coefficient, according to Yen (1994), is given by:

$$d_{50} = \left( \frac{n}{0.0474} \right)^6, \quad (27)$$

where  $d_{50}$  is the average granular size to be given as the input in the CFX code.

Observe that the assumption of known and constant velocity directions in the inlet and outlet section is a simplification of reality. For this reason, a better reconstruction of the velocity field can be found in an intermediate section, where secondary currents with velocity components normal to the mean flow direction can be easily detected (Peters and Goldberg, 1989; Richardson and Colin, 1996). See in Fig. 13 how the

intermediate section was divided to compute the vertically averaged velocities in each segment section and, in Fig. 14, the velocity components tangent to the cross section plane.

These 3-D numerical simulations confirm that the momentum  $\Gamma$ , proportional to the derivative of the average tangent velocities and equivalent to the left hand side of Eq. (2), cannot be set equal to zero, if a rigorous reconstruction of the velocity field is sought after.

To compute the uniform flow discharge, for a given outlet section, CFX code is run iteratively, each time with a different velocity norm in the inlet section, until the same water depth as in the outlet section is attained in the inlet section for steady state conditions.

See in Table 8 the comparison between the vertically averaged state velocities, computed through the DCM, IDCM, INCM, LHRM formulas ( $u_{\text{DCM}}$ ,  $u_{\text{IDCM}}$ ,  $u_{\text{INCM}}$ ,  $u_{\text{LHRM}}$ ) and through the CFX code ( $u_{\text{CFX}}$ ). Table 9 also shows the relative difference,  $\Delta u$ , evaluated as:

$$\Delta u = \frac{u - u_{\text{CFX}}}{u_{\text{CFX}}} \cdot 100, \quad (28)$$

As shown in Table 8, both INCM and LHRM perform very well in this validation test instead of DCM, which clearly overestimates averaged velocities. In the central area of the section the averaged velocities calculated by the INCM, LHRM and CFX code are quite close with a maximum difference  $\sim 7\%$ . By contrast, larger differences are evident close to the river bank, in segments 1 and 9, where INCM and LHRM underestimate the CFX values. These larger differences show the limit of using a 1-D code. Close to the bank the wall resistance is stronger and the velocity field is more sensitive to the turbulent exchange of energy with the central area of the section, where higher kinetic energy occurs. Thanks to the simulation of secondary flows (see Fig. 14) CFX allows this exchange and the related mixing. However, because the near-bank subsections are characterised by small velocities, their contribution to the global discharge is relatively small.

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## 5 Conclusions

Two new methods have been proposed for uniform flow discharge estimation. The first method, named INCM, develops from the original IDCM method and it is shown to perform better than the previous one, with the exception of lab tests with very small discharge values. The second one, named LHRM, has empirical bases, and gives up the ambition of estimating turbulent stresses, but has the following important advantages:

1. It relies on the use of only two parameters: the friction factor  $f$  (or the corresponding Manning's coefficient  $n$ ) and a second parameter  $\beta$  which on the basis of the available laboratory data was estimated to be equal to 9.
2. The  $\beta$  coefficient has a simple and clear physical meaning: the correlation distance, measured in water depth units, of the vertically averaged velocities between two different verticals of the river cross-section.
3. The sensitivity of the results with respect to the model  $\beta$  parameter was shown to be very low, and a one digit approximation is sufficient to get a discharge variability less than 2 %. A fully positive validation of the method was carried out using lab experimental data, as well as field discharge and roughness data obtained by using the unsteady-state level analysis proposed by Aricò et al. (2009) and applied to the Alzette river, in the grand Duchy of Luxembourg.
4. Comparison between the results of the CFX 3-D turbulence model and the LHRM model shows a very good match between the two computed total discharges, although the vertically averaged velocities computed by the two models are quite different near to the banks of the river.

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**Table 1.** Geometric and hydraulic laboratory parameters of the experiment series.

Series	$S_0$ [‰]	$h$ [m]	$B$ [m]	$b_4$ [m]	$b_1$ [m]	$b_3$ [m]	$s_{tp}$ [–]	$s_{mc}$ [–]
F1	1.027	0.15	1.8	1.5	4.1	4.100	0	1
F2					2.25	2.250	1	1
F3					0.75	0.750	1	1
F6					2.25	0	1	1
F8					2.25	2.250	1	0
F10					2.25	2.250	1	2
K1	0.966	0.08	0.15	0.152	0.229	0.229	0	0
K2					0.152	0.152		
K3					0.076	0.076		
K4					–	–		

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**Table 2a.** Sensitivities  $I_s$  and  $L_s$  computed in the F2 series for the optimal parameter values.

$H$ [m]	$Q_{\text{meas}}$ [m <sup>3</sup> s <sup>-1</sup> ]	$I_s$	$L_s$
0.156	0.212	0.2209	0.2402
0.169	0.248	0.1817	0.2194
0.178	0.282	0.1651	0.2044
0.187	0.324	0.1506	0.1777
0.198	0.383	0.1441	0.1584
0.214	0.480	0.1305	0.1336
0.249	0.763	0.1267	0.1320

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**Table 2b.** Sensitivities  $I_s$  and  $L_s$  computed in the K4 series for the optimal parameter values.

$H$ [m]	$Q_{\text{meas}}$ [m <sup>3</sup> s <sup>-1</sup> ]	$I_s$	$L_s$
0.085	0.005	0.3248	0.3282
0.096	0.008	0.2052	0.2250
0.102	0.009	0.1600	0.1709
0.114	0.014	0.1354	0.1372
0.127	0.018	0.1174	0.1208
0.154	0.029	0.0851	0.0866

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**Table 3.** Nash–Sutcliffe efficiency for all (calibration and validation) experimental series.

	Series	DCM	IDCM	INCM	LHRM
Calibration Set	F1	0.7428	0.9807	0.9847	0.9999
	F2	0.6182	0.9923	0.9955	0.9965
	F3	0.7219	0.9744	0.9261	0.9915
	F6	0.7366	0.9733	0.9888	0.9955
Validation Set	F8	−0.0786	0.9881	0.9885	0.9964
	F10	−0.0885	0.9965	0.9975	0.9978
	K1	−14.490	−0.7007	−8.2942	0.9968
	K2	−0.9801	0.3452	−1.8348	0.9619
	K3	0.1762	0.6479	−0.3944	0.9790
	K4	0.2878	0.888	0.3548	0.9958

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**Table 4.** Main characteristics of the flood events at the Pfaffenthal and Hunsdorf gauged sites.

Event	$\Delta t$ [h]	Pfaffenthal		Hunsdorf	
		$H_{\text{peak}}$ [m]	$q_{\text{peak}}$ [m <sup>3</sup> s <sup>-1</sup> ]	$H_{\text{peak}}$ [m]	$Q_{\text{peak}}$ [m <sup>3</sup> s <sup>-1</sup> ]
Jan 2003	380	3.42	70.98	4.52	67.80
Jan 2007	140	2.90	53.68	4.06	57.17
Jan 2011	336	3.81	84.85	4.84	75.10

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**Table 5.** Optimum roughness coefficient,  $n_{\text{opt}}$ , for the three flood events.

Event	DCM $n_{\text{opt}}$ [sm <sup>-1/3</sup> ]	IDCM $n_{\text{opt}}$ [sm <sup>-1/3</sup> ]	INCM $n_{\text{opt}}$ [sm <sup>-1/3</sup> ]	LHRM $n_{\text{opt}}$ [sm <sup>-1/3</sup> ]
Jan 2003	0.054	0.047	0.045	0.045
Jan 2007	0.051	0.047	0.046	0.045
Jan 2011	0.070	0.070	0.057	0.055

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**Table 6.** Nash–Sutcliffe efficiency of estimated discharge hydrographs for the analysed flood events.

Event	DCM $NS_q$ [–]	IDCM $NS_q$ [–]	INCM $NS_q$ [–]	LHRM $NS_q$ [–]
Jan 2003	0.977	0.987	0.991	0.989
Jan 2007	0.983	0.988	0.989	0.992
Jan 2011	0.898	0.899	0.927	0.930

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**Table 7.** Boundary conditions assigned in the CFX simulation.

Geometry Face	Boundary Condition
Inlet	All velocity components
Outlet	Velocity direction and hydrostatic pressure distribution
Side-Walls	Opening
Top	Opening
Bottom	No-slip wall condition, with roughness given by equivalent granular size $d_{50}$

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**Table 8.** Simulated mean velocities in each segment section using 1-D hydraulic models with DCM, IDCM, INCM, LHRM and *CFX*, and corresponding differences.

Sub-section	$u_{CFX}$ [ms <sup>-1</sup> ]	$u_{DCM}$ [ms <sup>-1</sup> ]	$u_{IDCM}$ [ms <sup>-1</sup> ]	$u_{INCM}$ [ms <sup>-1</sup> ]	$u_{LHRM}$ [ms <sup>-1</sup> ]	$\Delta u_{DCM}$ [%]	$\Delta u_{IDCM}$ [%]	$\Delta u_{INCM}$ [%]	$\Delta u_{LHRM}$ [%]
1	1.33	1.58	1.47	1.23	1.12	18.79	10.52	-7.52	-15.78
2	1.37	1.42	1.4	1.36	1.38	3.65	2.19	-0.73	0.73
3	1.38	1.53	1.48	1.38	1.4	10.87	7.25	0	1.45
4	1.47	1.64	1.6	1.56	1.57	11.56	8.84	6.13	6.80
5	1.53	1.94	1.8	1.59	1.61	26.79	17.65	3.92	5.23
6	1.57	2.01	1.81	1.6	1.68	28.02	15.29	1.91	7.00
7	1.46	1.66	1.65	1.49	1.5	13.69	13.01	2.05	2.74
8	1.42	1.48	1.46	1.44	1.43	4.22	2.82	1.40	0.70
9	0.88	0.91	0.90	0.70	0.69	3.40	2.27	-20.45	-21.59

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**Table A1.** Notations.

$A_i$	area of each subsection “ $i$ ” of a compound channel
$B$	top width of compound channel
$b$	main channel width at bottom
$f$	friction factor
$g$	gravity acceleration
$H$	total depth of a compound channel
$n_{mc}, n_{fp}$	Manning’s roughness coefficient for the main channel and floodplain, respectively
$P_i$	wetted perimeter of each subsection “ $i$ ” of a compound channel
$Q_{meas}$	measured discharge
$R_i$	hydraulic radius of each subsection “ $i$ ” of a compound channel
$S_0$	longitudinal channel bed slope
$S_f$	energy slope
$\tau$	turbulent stress
$\varepsilon$	turbulent dissipation
$\rho$	fluid density
$\mu$	fluid viscosity
$\alpha$	IDCM interface coefficient
$\beta$	LHRM coefficient
$\xi$	INCM coefficient

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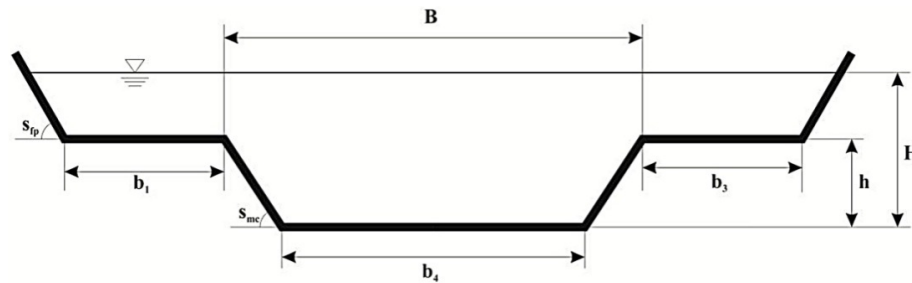


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**Figure 1.** Compound channel geometric parameters.

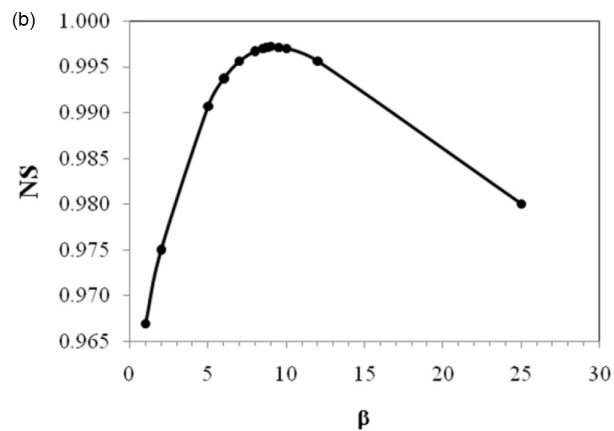
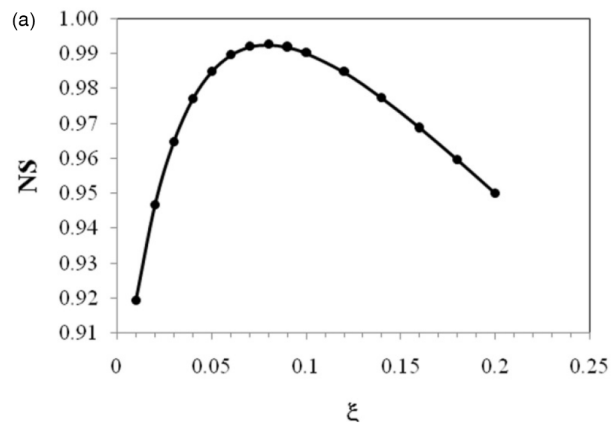
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**Figure 2.** NS vs.  $\xi$  and  $\beta$  curves respectively for INCM (a) and LHRM (b) methods.

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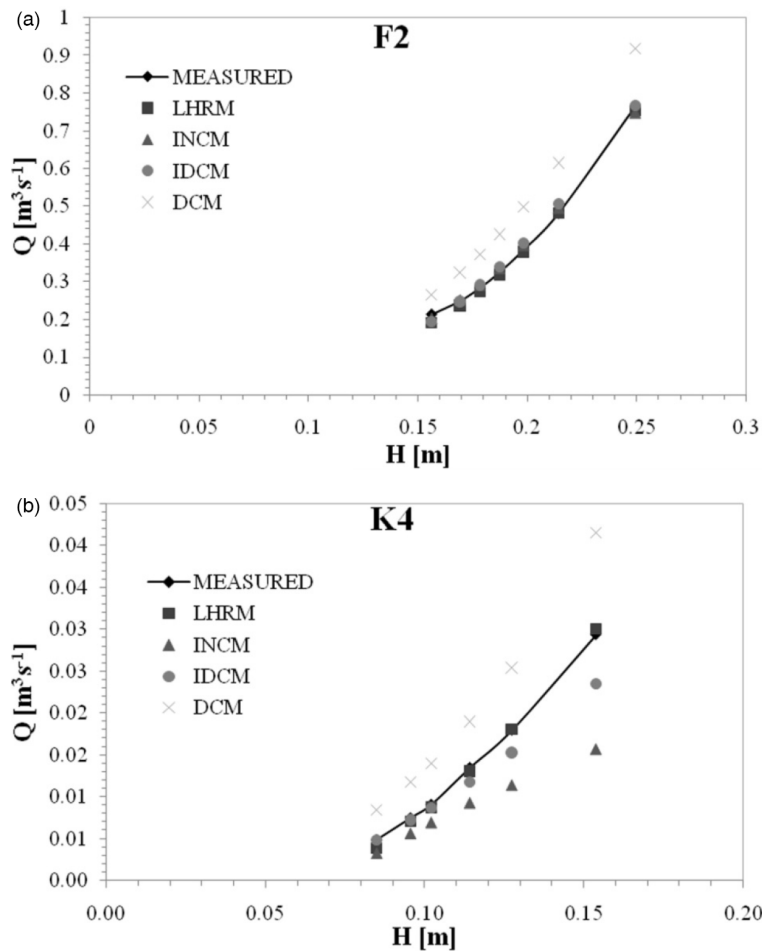
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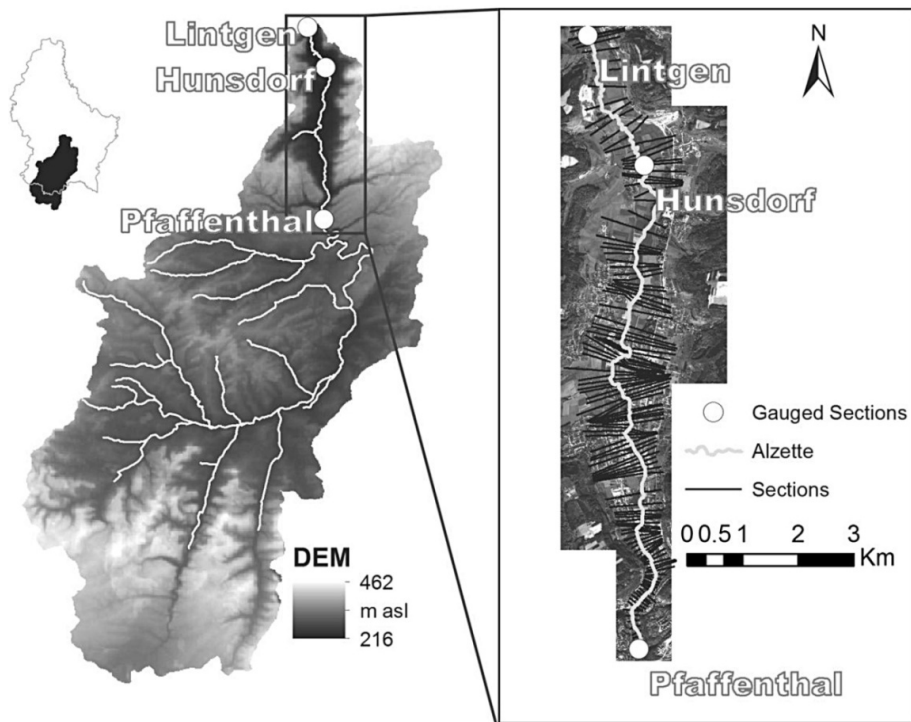
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**Figure 3.** Estimated discharge values against HR Wallingford FCF measures for F2 (a) and K4 (b) series.



**Figure 4.** The Alzette study area.

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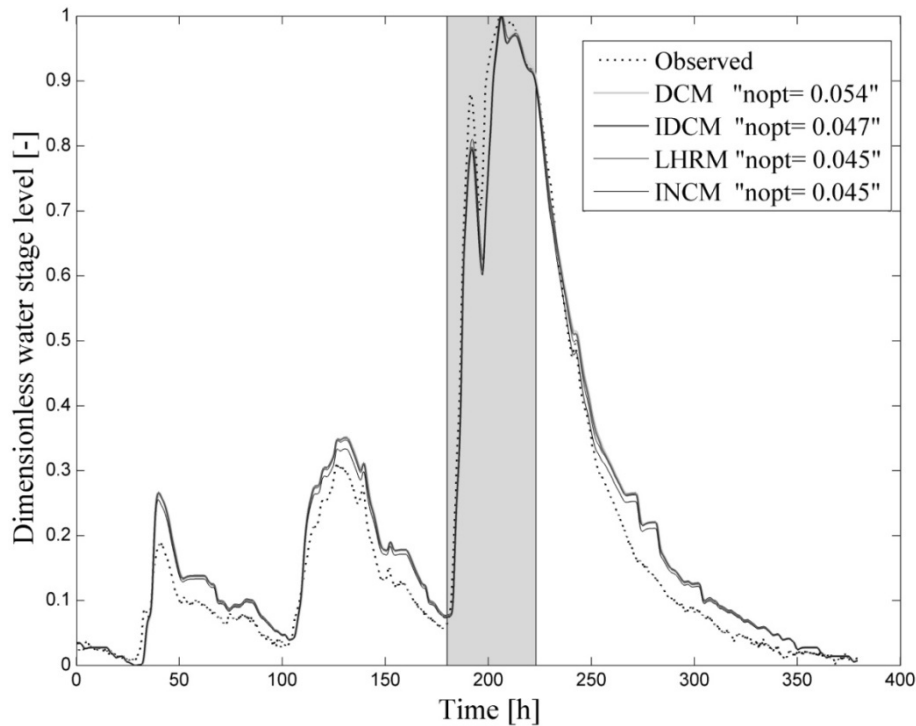
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**Figure 5.** Observed and simulated stage hydrographs at Hunsdorf gauged site in the event of January 2003.

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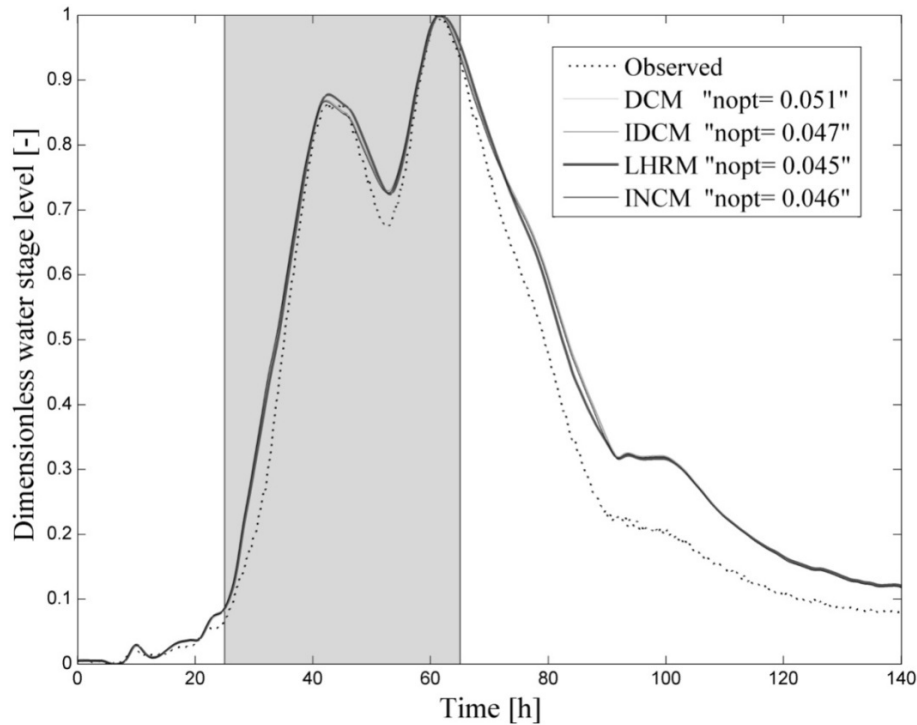
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**Figure 6.** Observed and simulated stage hydrographs at Hunsdorf gauged site in the event of January 2007.

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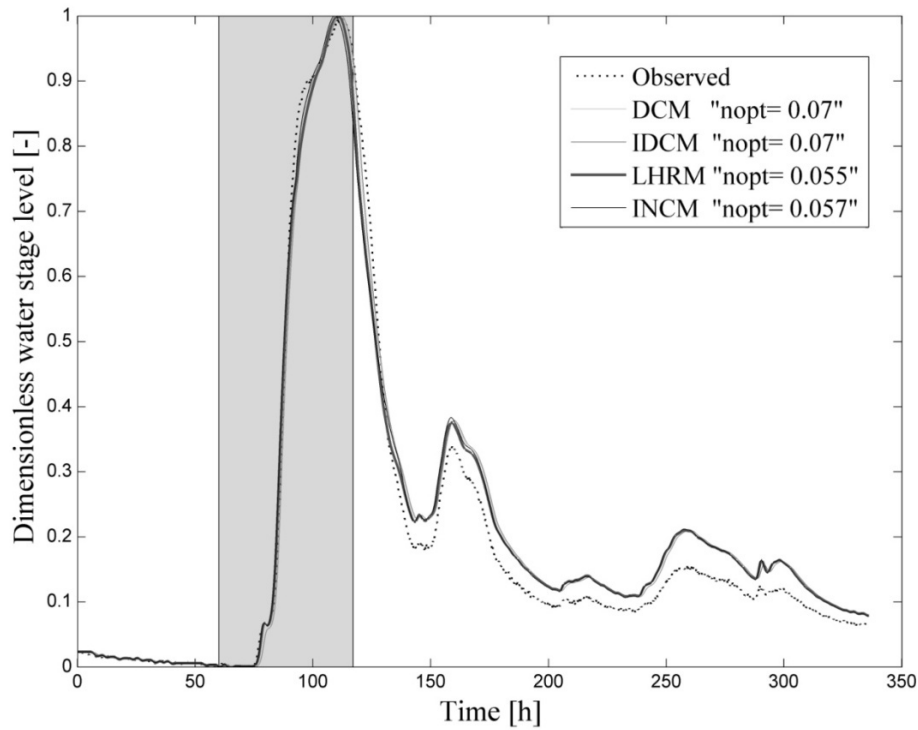
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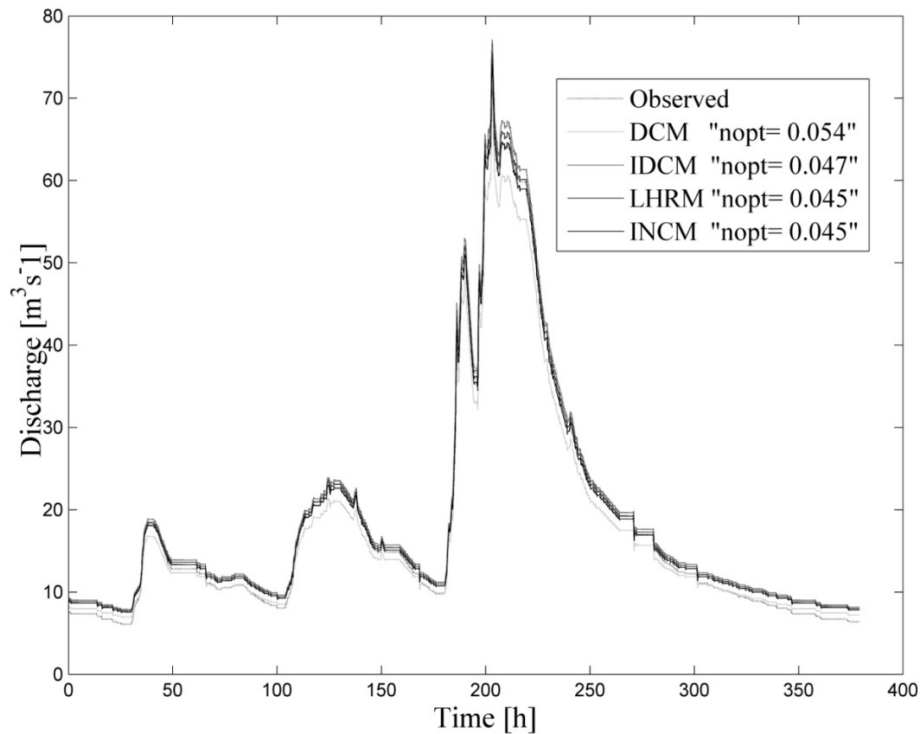
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**Figure 7.** Observed and simulated stage hydrographs at Hunsdorf gauged site in the event of January 2011.



**Figure 8.** Observed and simulated discharge hydrographs at Hunsdorf gauged site in the event of January 2003.

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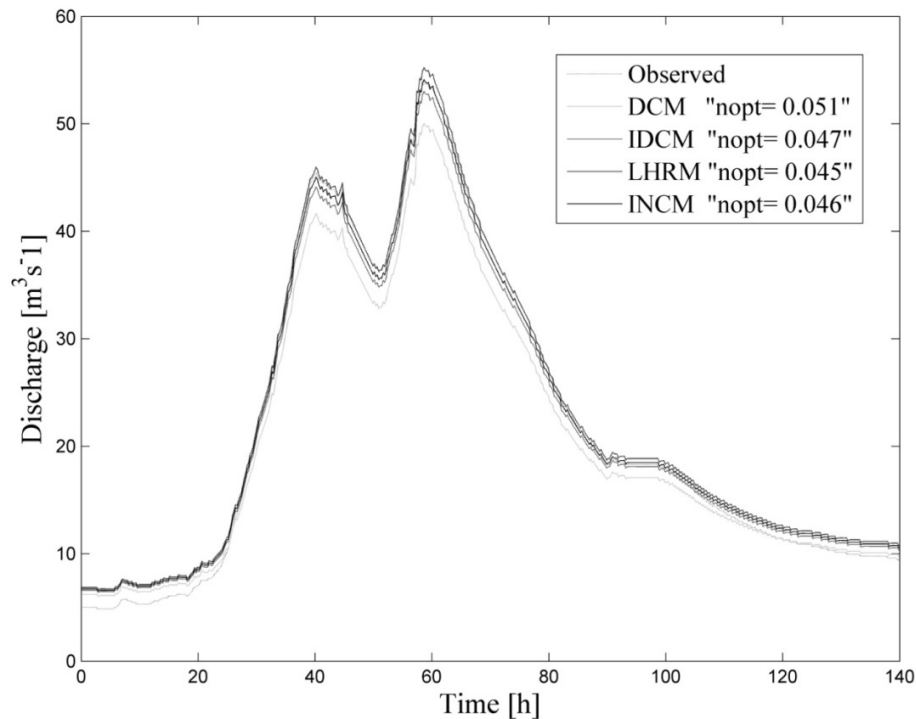
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**Figure 9.** Observed and simulated discharge hydrographs at Hunsdorf gauged site in the event of January 2007.

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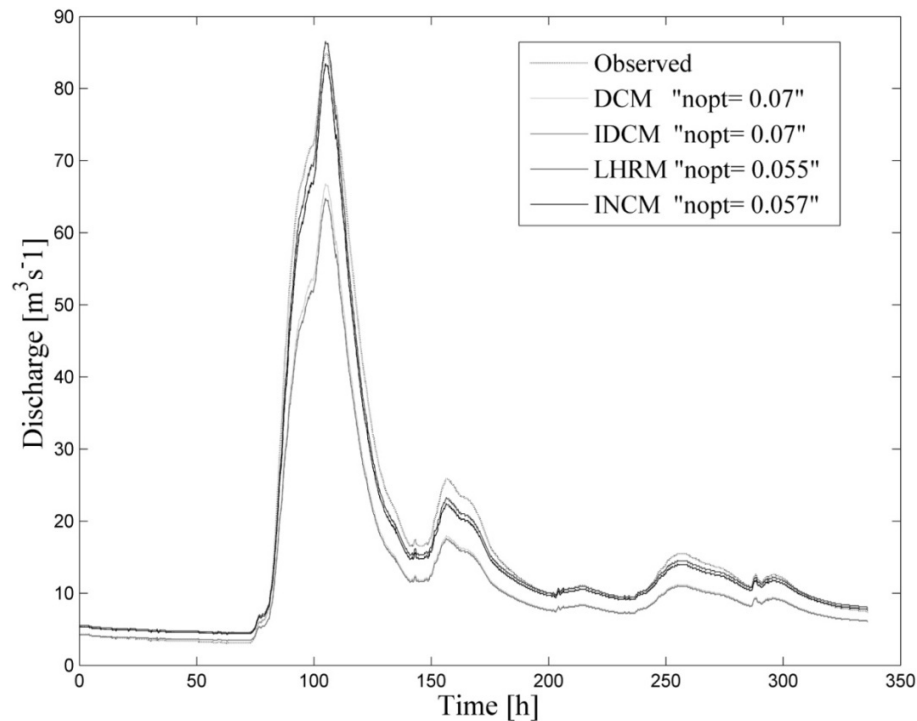
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**Figure 10.** Observed and simulated discharge hydrographs at Hunsdorf gauged site in the event of January 2011.

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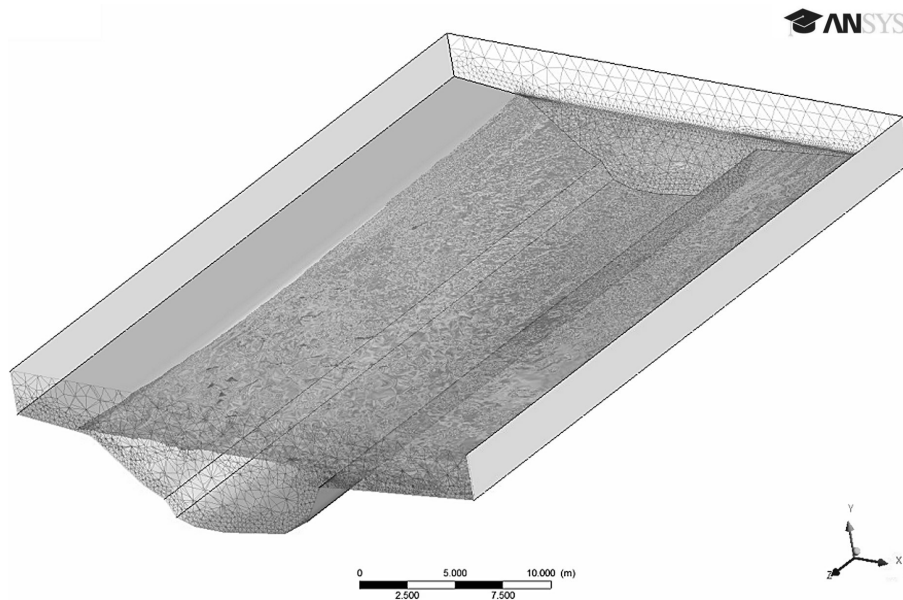
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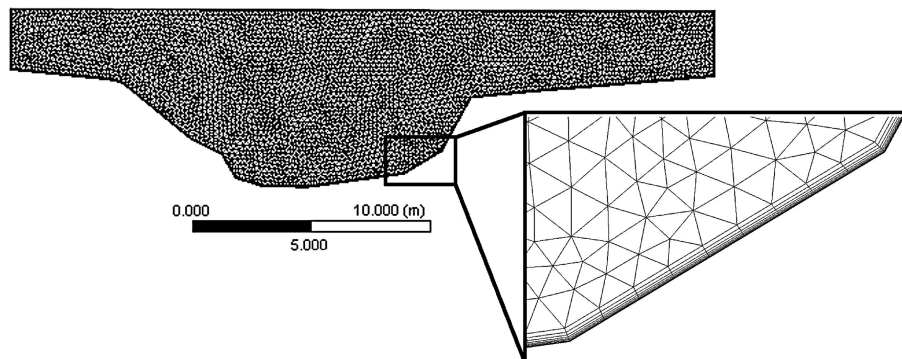
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**Figure 11.** Computational domain of the reach of the Alzette river.



**Figure 12.** A mesh section along the inlet surface.

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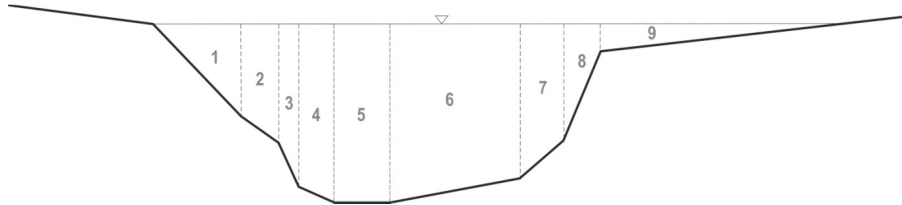


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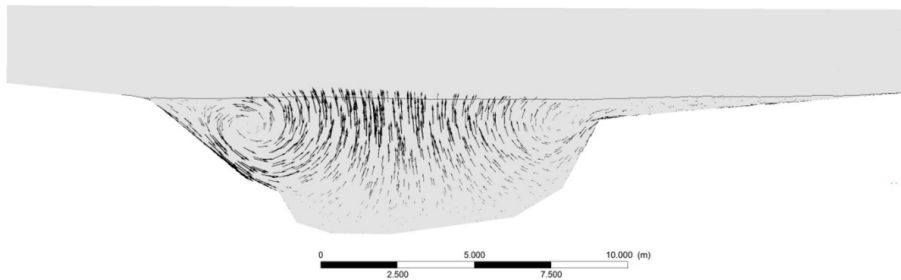
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**Figure 13.** Hunsdorf river cross-section: subsections used to compute the vertically averaged velocities.





**Figure 14.** Secondary flow inside the intermediate cross section.

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