Assessment of flow rate in a complex sewer pipe by means of a water depth sensor and modelling

Estimation du débit dans une canalisation complexe en utilisant une mesure de hauteur d'eau et la modélisation numérique

Gislain Lipeme Kouyi[1], Frédéric Besson[1], Marc-Antoine Bier[1], Bernard Chocat[1], Patrick Lucchinacci[2]

[1] : Université de Lyon, F-69000, Lyon, France
INSA-Lyon, LGCIE, F- 69621 Villeurbanne, France
{gislain.lipeme-kouyi, frederic.besson, marc-antoine.bier, bernard.chocat}@insa-lyon.fr
[2] : Grand Lyon - Direction de l'eau du Grand Lyon, Service Exploitation Métrologie, 64 rue André Bollier, 69007 Lyon, France ;
plucchinacci@grandlyon.org

RÉSUMÉ

La méthode de mesure de débit à partir des mesures de vitesse moyenne et de hauteur d'eau est largement répandue. Malheureusement, dans certaines conditions, le capteur de vitesse ne fournit pas de valeurs représentatives. Dans cet article nous présentons une méthodologie originale qui permet d'estimer le débit dans une canalisation d'assainissement complexe à partir d'une mesure de hauteur d'eau et de l'utilisation des outils de modélisation 1D et 3D. Cette méthodologie a été appliquée sur le site industriel de Vénissieux géré par le service métrologie de la direction de l'eau de la communauté urbaine de Lyon (France). La méthodologie est fondée sur le couplage sous CANOE d'un modèle hydrologique calé et vérifié grâce à des mesures de hauteur d'eau enregistrées en continu sur deux ans au pas de temps de 6 minutes et d'un modèle hydraulique basé sur la résolution des équations de barré de saint venant en 1D. L'exploitation des résultats obtenus sous Canoe a contribué à l'élaboration d'une relation numérique permettant de calculer le débit à partir de la hauteur d'eau avec des incertitudes de l'ordre de 10 %. Cette relation numérique a été vérifiée en utilisant les résultats issus de la modélisation 3D des écoulements turbulents à surface libre dans le canal complexe. Ce type d'approche peut donc permettre d'estimer les débits à partir uniquement des mesures de hauteur d'eau lorsque le capteur de vitesse est défaillant.

ABSTRACT

The measurement method of the discharge based on both the water depth and the mean velocity measurements is widely used by practitioners in order to manage combined sewer networks. Unfortunately, in certain conditions, the velocity sensor doesn't provide often representative values. This paper highlights a new methodology which enables to assess flow rate in complex sewer pipe by means of only measurements of the water levels and 1D/CFD modelling tools. This original approach was applied on Venissieux industrial catchment (close to Lyon, France) managed by the metrology team of the water management service of Lyon (France). The methodology is based on the linkage under French CANOE software package between the calibrated hydrological and Barré De Saint Venant –based hydraulic models. 2 years time series water level data recorded with 6 minutes time step is used for the calibration and validation phases. Numerical data deriving from the calibrated Canoe model have been used to elaborate the numerical relationship which allows computing the discharge with uncertainties around 10%. The discharges computed with the proposed Discharge/water level relationship compare well with them arising from CFD modelling of turbulent open-channel flows. Thus, this kind of approach may help to assess flow rates with only water level measurements when the velocity sensor is often clogged.

KEYWORDS

Calibration, canoe software, CFD modelling, flow rate, water depth measurement

1 INTRODUCTION

In the framework of both European and French regulations (French water law in 2006; EC, 2000), the estimation of water and pollutant fluxes becomes the main objective of the practitioners in charge of the water management in cities. These fluxes are known to degrade the receiving water bodies. To assess accurately the pollutant loads, it’s necessary to know before the discharge transported in combined sewer pipe during both dry and wet weather periods.

The measurement method of the discharge based on both measurements of the water depth and the mean velocity is widely used by practitioners and researchers. Water level is easy to measure; however it’s not the case of the mean velocity measurement with for example acoustic Doppler sensor which is commonly used for continuously measuring flow rates in sewers (Larrarte et al., 2008). Consequently, many investigations are carried out on small scale physical models and other configurations in order to further measure or assess this quantity in both sub and supercritical open channel flows with rough or smooth wall (Nezu and Rodi, 1986; Tominaga and Nezu, 1992; Kraus et al., 1994; Ferro and Baiamonte, 1994; Muste and Patel, 1997; Carollo et al., 2002; Maghrebi and Ball, 2006; Knight and al., 2007; Bardiaux et al., 2008; Bonakdari et al., 2008). Indeed, there are many disturbances related to the measurement of the mean velocity, and among other things we can mention: i) the disturbance due to the presence of the sensor in the flow with low water depth providing uncertainties more than 25 % at 5 cm from the transducers (Mueller et al, 2007) or ii) the clogging of the sensor especially when the measurement is performed in an industrial combined sewer pipe with oil and large hydrocarbon loads. In this case, the models which enable to assess flow rates by means of only values of the water level and without the presence of the sensor or the weir in the flow become relevant. Sanderson and Baginska (2007) used the approach based on the utilization of the water level measurements and a mechanist model to assess the flow rates and applied it to evaluate the freshwater inflow rates towards a lake.

This paper presents a new method enabling to assess flow rate by means of water level measurements and 1D/CFD modelling tools. We applied this method on an industrial site where the velocity sensor is often clogged.

2 MATERIAL AND METHODS

2.1 Experimental site

Monitoring of water depths and mean velocities are carried out in the combined sewer pipe located in a 334 ha Venissieux industrial catchment (close to Lyon, France). The discharge (by means of water depth and mean velocity) is recorded continuously with a time step of 6 minutes. Flow meters are located directly in the sewer channel. The measurement channel is a 79 cm diameter sewer pipe with a lateral sidewalk (Figure 1). Three rain gauges enable to measure the storm parameters on the catchment (Figure 1).

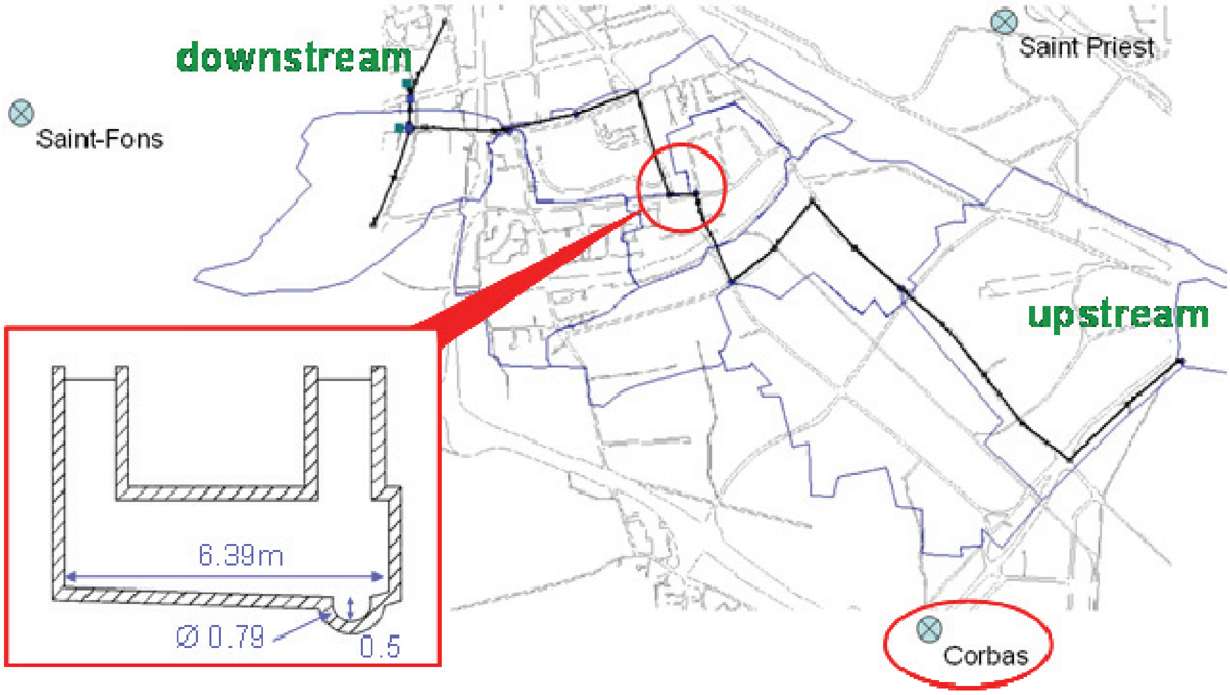

Figure 1. Vénissieux industrial catchement with the cross-section of the combined sewer pipe. Storm events measured with the rain gauge located in Corbas sub-catchment were used as inputs of the hydrological model.

Unfortunately, due to the clogging of the Doppler sensor there are many lacks in the data series. Indeed, it's an industrial catchment with oil and large loads of hydrocarbon in wastewater. Consequently, an approach based on the assessment of the discharge using only the implemented water level data seems relevant for this catchment. Figure 2 shows the longitudinal sketch of the measurement sewer pipe. The downstream combined sewer pipe has a negative slope.

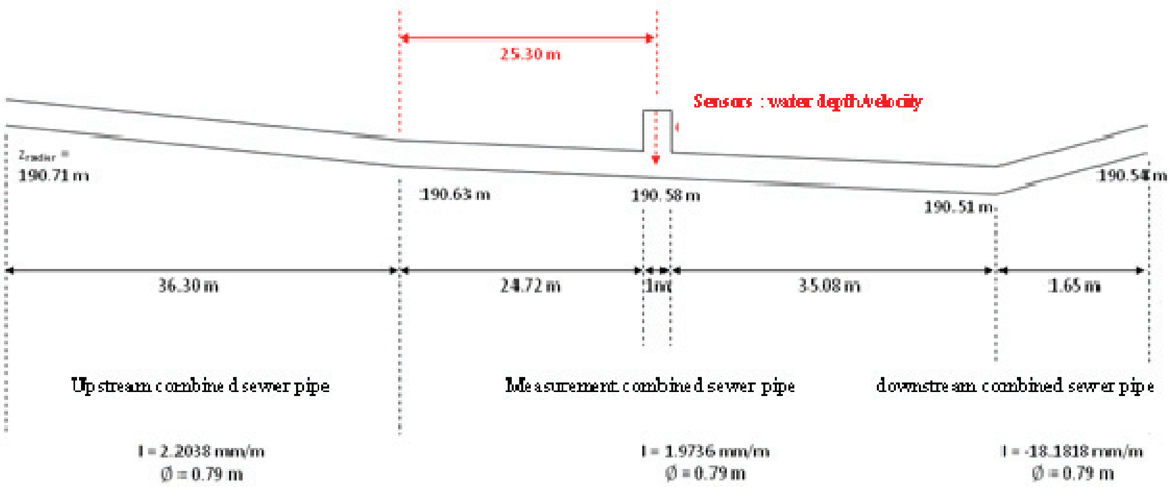

Figure 2. Longitudinal sketch of the combined sewer pipe – the mean upstream and measurement channel slope is around 0.2% and the downstream slope is around -2%

2.2 Methods

2.2.1 Description of the methodology

The methodology is based on seven main steps:

1) Analysis and validation of the available data.
2) Choice of rainfall events for the calibration and the validation of the hydrological model. The definition of the specific group of storm events is done by means of the diagram of the maximum rainfall intensity per 6 minutes versus the total rainfall level. Therefore some rainfall

events deriving from one group are chosen for the calibration and other storm events from another group are used for the validation phase.

3) Calibration of the hydrological Canoe model: the fixed runoff coefficient, initial losses, lag time and dry sewer flow parameters (number of equivalent inhabitants, mean day flow rate…) are used as parameters of the calibration phase. The Nash-Sutcliffe coefficient (Nash and Sutcliffe, 1970) computed as following is chosen to check the goodness of the hydrological model:

$$E = 1 - \frac{\sum_{i=1}^{n}\left(h_i - \hat{h}_i\right)^2}{\sum_{i=1}^{n}\left(h_i - \overline{h}_i\right)^2} \tag{1}$$

where h_i and $\hat{h}_i$ are respectively observed (deriving from the long time series of water depths recorded with 6 minutes time step) and simulated (deriving from Barré de Saint Venant – based hydraulic model with the input values of flow rates arising from the hydrological model) water levels; $\overline{h}_i$ is the mean value of observed water levels.

This step allows reproducing the dynamic of the flow related to: i) the runoff for several storm events and ii) the dry weather behaviour on the catchment.

4) Utilization of the calibrated hydrological model to provide the flow rates as input values for the 1D hydraulic simulations of flows in combined sewer pipe: for this step, only storm events which provided water depths less than 0.5 m were used as inputs of the hydrological model. Indeed, for the water levels higher than 0.5 m there is the effect of the sidewalk structure on the flow and 1D modelling is not relevant.
5) Elaboration of the numerical relationship between the hydrological model output flow rates and hydraulic model output water depths (at the position of the water depth sensor) in order to assess the discharge with one value of water level.
6) Comparison, for the same inlet flow rate, between water depths deriving from 1D hydraulic simulation and CFD modelling with Ansys Fluent v.12 commercial software. This step enables to check as well the location of the sensor thanks to the analysis of the 3D results of simulations (velocity field, shape of the free surface with or without hydraulic jump…).
7) Assessment of the uncertainties.

2.2.2 1D modelling of flows in the combined sewer pipe with Canoe Software

The Canoe software is based on Barré de Saint Venant equations which in x-direction are:

Continuity: $$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{2}$$

Momentum: $$\frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + g\frac{\partial h}{\partial x} = g(S_0 - S_f) + (\varepsilon - 1).q.\frac{U}{A} \tag{3}$$

where A is the cross-sectional area, Q is the discharge, q the lateral flow rate per unit of length, U is the mean velocity in the x-direction, g is the gravitational acceleration, h is the water depth, S_0 is the bottom slope, S_f is the energy loss and ε represents the lateral momentum transfer coefficient. In our study the lateral effects due to lateral flows was neglected.

2.2.3 3D modelling of flows in the complex channel with Ansys Fluent

For large water levels (higher than 0.5 m), three dimensional flow occur in the combined sewer pipe near the measurement point because of the side-walk bank. So CFD approach becomes relevant. This approach is based on RANS equations (Reynolds averaged Navier-Stokes). Partial differential equations describing the flow (Reynolds equations) are written in a conservative form, to establish relations between the pressure, velocities and Reynolds stress (Versteeg and Malalasekera, 1995). The form of partial derivative equations for biphasic application is as follows:

- The continuity equation for each phase which is called q:

$$\begin{cases} \dfrac{\partial \alpha_q}{\partial t} + U_i \dfrac{\partial \alpha_q}{\partial x_i} = 0 \\ 0 \le \alpha_q \le 1 \\ \sum_{q=1}^{n} \alpha_q = 1 \end{cases} \tag{4}$$

where n is the number of phases, U_i the mean velocity components and α_q is the volume fraction of phase q. In each cell, the overall volume mass ρ and viscosity μ are computed using the volume fraction as follows:

$$\begin{cases} \rho = \sum_{q=1}^{n} \alpha_q \rho_q \\ \mu = \sum_{q=1}^{n} \alpha_q \mu_q \end{cases} \tag{5}$$

- The momentum equation :

$$\frac{\partial \rho U_i}{\partial t} + U_i \frac{\partial \rho U_i}{\partial x_i} = -\frac{\partial P}{\partial x_i} + \rho g_i + \mu \frac{\partial^2 U_i}{\partial x_j \partial x_j} - \frac{\partial \rho \left(\overline{u_i' u_j'}\right)}{\partial x_j} \tag{6}$$

where P is the pressure term and g is the gravitational acceleration. Equation (6) represents the Reynolds Averaged Navier-Stokes (RANS) equations system (for i and j equal to 1, 2 and 3). The terms $\rho \overline{u_i' u_j'}$ called Reynolds tensors can be estimated by means of closure equations such as RSM turbulence model.

This software uses the finite volume method for solving partial derivative equations presented above. So the computational meshes as volumes of control should be built.

Several kinds of boundary conditions are proposed in the CFD code, such as symmetry, pressure inlet and outlet, imposed velocity etc. Three of those conditions are used for our study: velocity-inlet, pressure-outlet and roughness for the assessment of the wall functions. The first boundary condition - velocity-inlet - is an imposed value of the velocity. The flow is thus injected through a wet section to obtain the expected inlet flow rate. In this case, the length of the inlet pipe must be sufficient to enable the velocity profile to be developed. The length required is 5 to 10 times the water depth at the inlet boundary. The second condition - pressure-outlet - is applied at the outlets or for the free surface modelling by setting the atmospheric pressure value. The roughness condition is used to account for the boundary layer near the wall.

The value of the water volume fraction is imposed equal to 1 in the water domain and 0 in the air domain. The computation of the turbulent intensity I and the hydraulic diameter D_h enables us to obtain the inlet boundary values for turbulence.

$I = 0.16 R_e^{-1/8}$ with $R_e = \dfrac{U D_h}{\nu}$ the Reynolds number (7)

2.2.4 Computational mesh

The greater the number of cells in the mesh grid, the more accurate will be the modelling results. However the computing time increases with mesh density (e.g for calculation of 25000 iterations with 500,000 cells, the current computation time can be up to 2 days with a PC running 64-bit Linux). Hence we tried to find a balance between quality of results and computation time, settling on 400 000 computational cells.

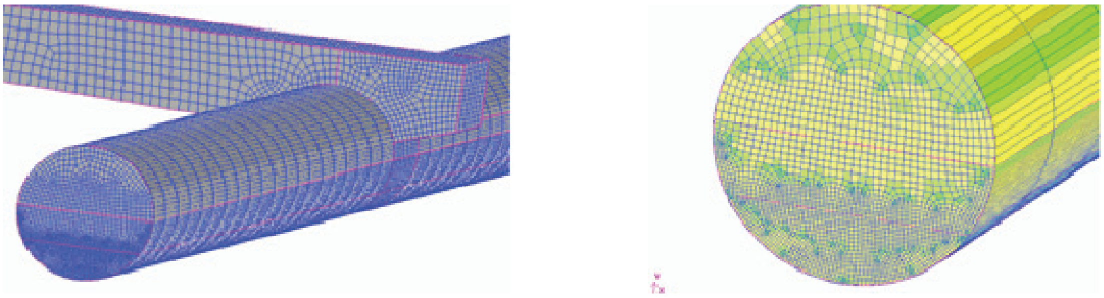

Figure 3. Computational hexahedral mesh of the conduit (with the sidewalk structure on the left part or the figure) for 3D simulations – 400 000 cells

For the 1D hydraulic simulations, the computational interval size of Δx = 2m and the time step of Δt =10s are used.

3 RESULTS AND DISCUSSION

3.1 1D modelling with Canoe software

After the first step of the methodology which is based on the validation of the available data (rainfall intensities and water level), we calibrated Canoe software. In order to evaluate the goodness of fit between measured and simulated data, the Nash-Sutcliffe coefficient (Nash and Sutcliffe, 1970) described by equation 1 has been chosen. The Figure 4a shows an example of the calibration phase result. The mean Nash-Sutcliffe coefficient for all storm events used for the calibration phase is around 0.85.

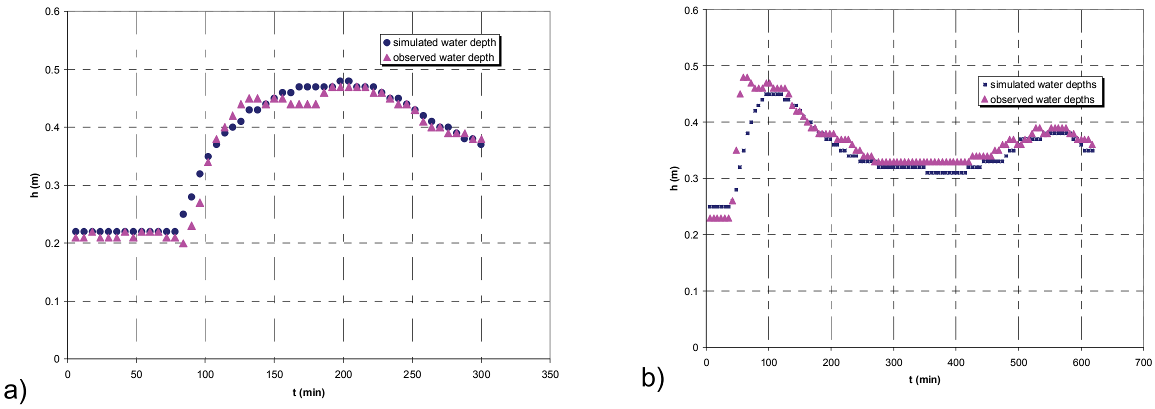

Figure 4. Results of the comparison between observed and simulated water depths: a) calibration phase: comparison between observed and simulated (with the hydraulic model after calibration step) water levels for the storm event of 11/01/2008 ; b) validation phase: comparison between observed and simulated water levels for the storm event of 06/08/2007

The Figures 4b and 5 show two examples of the validation results. These results highlight the goodness of the fit between water levels deriving from hydraulic model and those observed in situ. Hence, Canoe model enables to represent satisfactorily the dynamic related to the runoff and the flow in the combined sewer pipe.

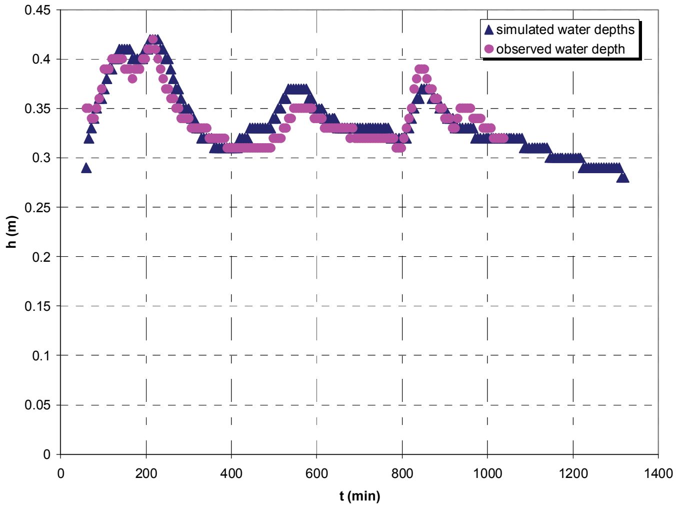

Figure 5. Result of the validation phase: comparison between observed and simulated water levels for the storm event of 17/05/2008

All results of hydraulic simulation are gathered in order to elaborate the numerical relationship which enables to assess the flow rate versus the values of water level between 0.2 and 0.5 m, particularly for rainfall intensities higher than 40 mm/h. The regression method was used and uncertainties were also evaluated (figure 6).

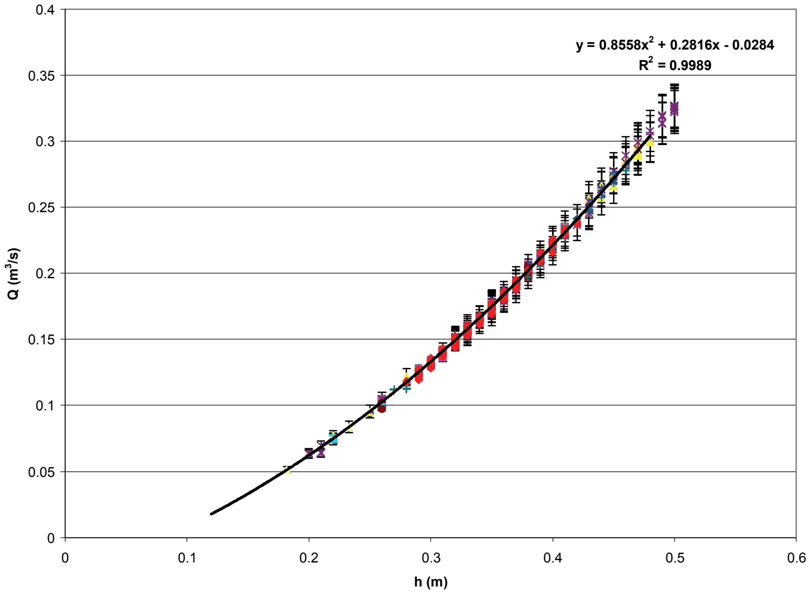

Figure 6. Numerical relationship between the flow rates deriving from the hydrological model and the water depths performed by means of 1D Canoe hydraulic model – numerical relationship is reliable with relative uncertainties of +/- 10 % for values of the water level between 0.2 and 0.5 m

Almost all water levels used for the calibration and the validation steps are larger than 0.2 m due to the daily industrial wastewater discharge storage (for low velocities) in the pipe because of the negative slope at the downstream. So the numerical relationship seems appropriate to assess flow rates on Venissieux catchment. The relative uncertainties have been assessed around 10 %.

3.2 3D modelling of flows accounting for sidewalk effect

For the water levels larger than 0.5 m, 3D modelling has been carried out with successive stationary flow consideration. Figure 7 shows the effect of the sidewalk structure on the main flow in the combined sewer pipe. We can note side vortices at the free surface. VOF approach was used to compute the free surface. This method is widely validated by Lipeme Kouyi et al (2003) and others (Mueller et al, 2007; Guo et al, 2008)

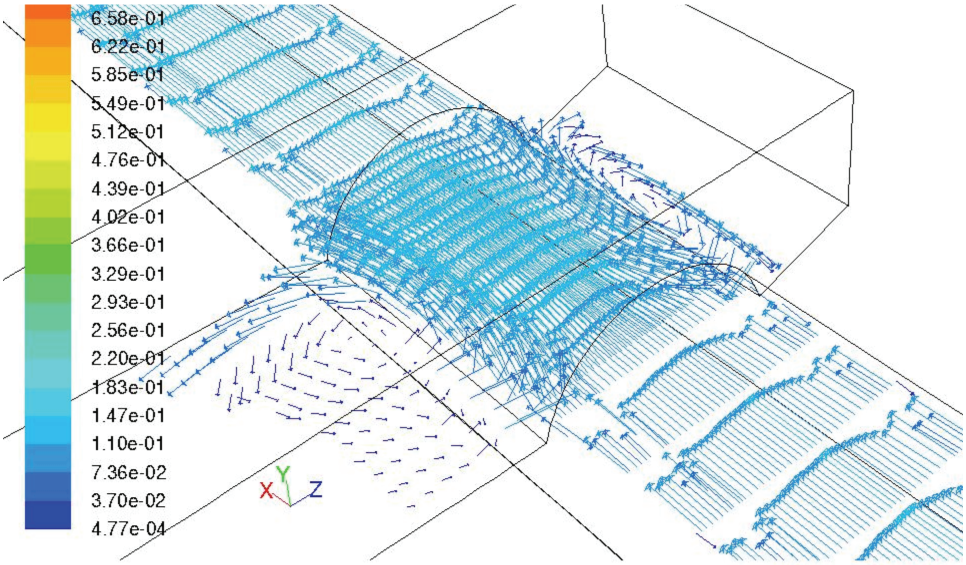

Figure 7. Free surface velocity field computed with the CFD Ansys Fluent software – vortices occurs at the free surface due to the sidewalk structure effect.

The supercritical flows occur towards the downstream with hydraulic jump (Figure 8) due to the change of the slope from 0.2% to -2%. So there is no backwater effect in the measurement pipe and the slope of the energy line seems almost constant (regarding the results of 3D simulations) between the location of the sensor and the upstream of the hydraulic jump (figure 8). So, in our case one value of water depth may provide one value of discharge. Indeed, the loop of the stage-discharge relationship in the unsteady flow depends upon the slope of the energy line (Cunge, 1975). In the other hand, Tominaga and Nezu (1992) demonstrated that in supercritical flows, the integral constants in the log law for the mean velocity assessment changes with the bed slope. So when the bed slope is constant, the log law which allows estimating the mean velocity of the flow could be performed – and for one water level, only one value of discharge is computed. However, when supercritical flow occurs in a channel, there is instability of the free surface and the discharge increases and changes more quickly than the increasing of the water level. The utilization of the Rhodamine WT tracer on this site will enable us to further validate the proposed numerical relationship.

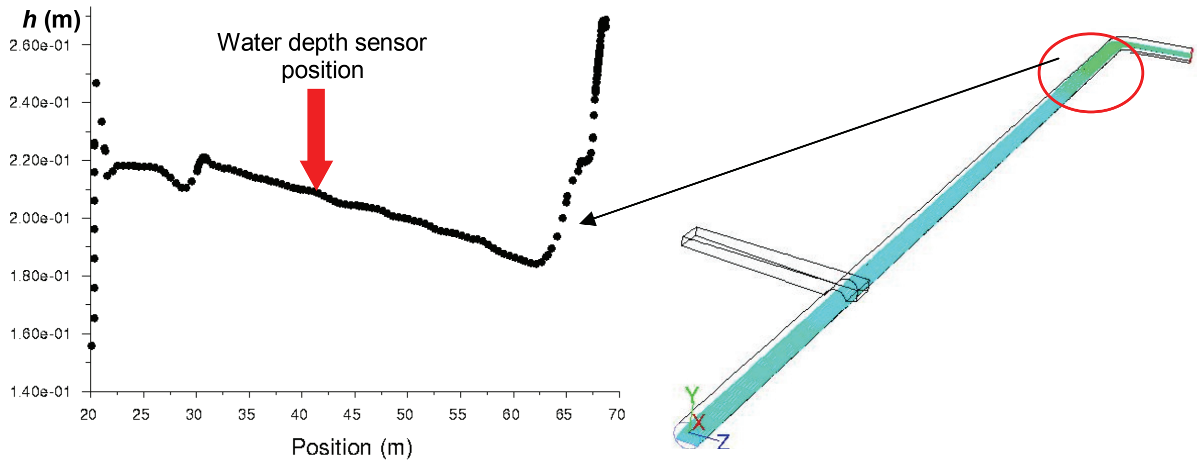

Figure 8. Hydraulic jump towards the downstream of the sewer pipe for inlet flow rate of 94 l/s

3.3 Comparison of the results deriving from 1D and 3D modelling

Figure 9 shows the results of comparison between the discharges deriving from 1D hydraulic model and 3D CFD modelling for the same water level. The discharges deriving from CFD modelling are higher than them computed with Canoe for water level up to 0.2 m. The opposite is noted for the water depth less than 0.2 m. Of course, further CFD simulations must be carried out in order to better test the proposed numerical relationship between water depth and discharge. Despite that, differences between 1D and CFD discharges are less than 12%. The numerical relationship arising from Canoe model has a good agreement compared to the first CFD results.

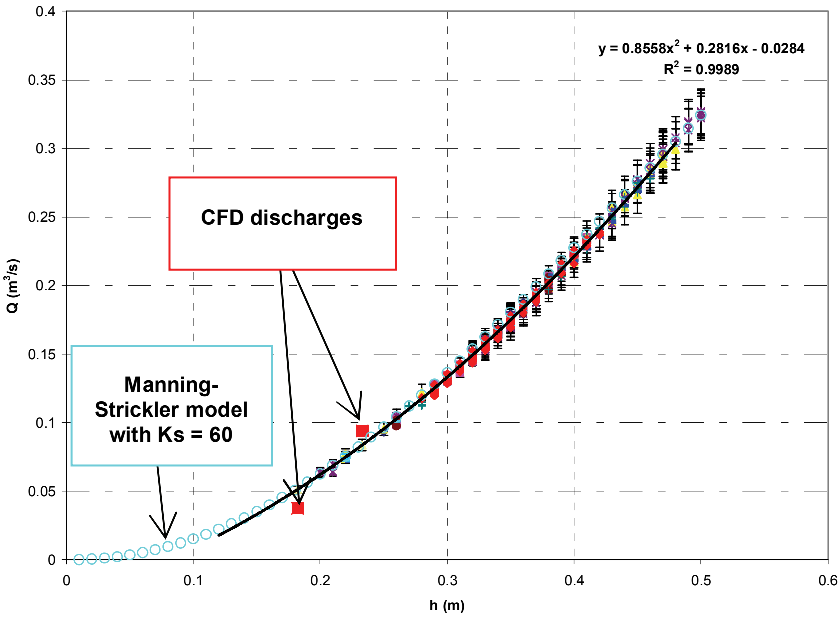

Figure 9. Comparison between 3D and 1D simulated water levels

We can also note that the uniform-flow Manning equation with Strickler coefficient of 60 $m^{1/3}/s$ enables to assess discharges (for the water level higher than 1 cm) with the same order of magnitude as them deriving from the proposed numerical relationship and CFD modelling. It's due to the shape of the free surface line which is almost constant so that the stationary model such as Manning approach may be performed (figure 9). This Strickler coefficient seems realistic due to the presence of industrial wastewater which degrades the combined sewer pipe wall and increases its roughness.

4 CONCLUSIONS

This paper highlighted a new methodology which enables to assess flow rate in complex combined sewer pipe by means of only water level measurements and 1D/CFD modelling tools. This novel approach was applied on Venissieux industrial catchment (close to Lyon, France) managed by the metrology team of the water management service of Lyon (France). The methodology is based on the linkage under French CANOE software package between the calibrated hydrological and Barré de Saint Venant –based hydraulic models. Numerical data deriving from the calibrated hydrological model have been used to elaborate the numerical relationship which allows computing the discharge by means of water depth measurements with relative uncertainties around 10 %. The discharges computed with the proposed numerical relationship compare well with them deriving from CFD modelling (with Ansys Fluent software) of flows in the complex channel accounting for the effect of the sidewalk structure when large flow rates occur in the pipe. A Manning-Strickler model was performed with the Strickler coefficient *Ks* equal to 60 $m^{1/3}/s$. This Strickler coefficient seems realistic due to the presence of industrial wastewater which degrades the combined sewer pipe wall and increases its roughness. Hence, in order to assess the discharge in complex combined sewer pipe, an approach based on the use of only water level measurements and modelling tools seems appropriate when the mean velocity measurement sensor is often clogged.

Further simulations with others storm events are currently done with both 1D and CFD tools in order to continue the validation phase of the approach as well as the comparison between 1D and CFD water levels for the same inlet flow rates. Also, Rhodamine WT tracer will be used on this site to improve the models and assess more accurately the discharge in this combined sewer pipe. This approach is easily applicable on another site if the reliable geometric, hydraulic and rainfall data are available.

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LIST OF REFERENCES

Bardiaux J.-B., Vazquez J., Mosé R. (2008). *Assessment of velocity fields through open-channel flows with an empiric law*. Water Science and Technology 57 (11), 1763-1768.

Bonakdari H., Larrarte F., Lassabatere L., Joannis C. (2008). *Turbulent velocity profile in fully-developed open channel flows*. Environ. Fluid. Mech., Vol. 8, 1-17.

Carollo F.G., Ferro V., Termini D. (2002). *Flow velocity measurements in vegetated channels*. Journal of hydraulic Engineering, Vol. 128 (7), 664-673.

Cunge J.A. (1975). *Applied mathematical modelling of open channel flow*. Chapiter 10 of “Unsteady flow in open channels”, Volume 1, Edited by K. Mahmood and V. Yevjevich, 1975.

EC (2000). *Directive 2000/60/EC of the European parliament and of the council of October 23 establishing a framework for community action in the field of Water policy.* Official Journal of the European Communities, L327/1-L327/72. 22. 12. 2000.

Ferro V., Baiamonte G. (1994). *Flow velocity profiles in gravel-bed rivers*. Journal of Hydraulic Engineering, Vol. 120 (1), 60-80.

Guo Y., Zhang L., Shen Y., Zhang J. (2008). *Modeling study of free overfall in a rectangular channel with streap roughness*. Journal of Hydraulic Engineering, 134 (5), 664-667.

Knight D.W, Omran M., Tang X. (2007). *Modeling depth-averaged velocity and boundary shear in trapezoidal channels with secondary flows*. Journal of hydraulic Engineering, Vol. 133 (1), 39-47.

Kraus N.C., Lohrmann A., Cabrera R. (1994). *New acoustic metter for measuring 3D laboratory flows*. Journal of hydraulic Engineering, Vol. 120 (3), 406-412.

Larrarte F., Bardiaux J-B., Battaglia P., Joannis C. (2008). *Acoustic Doppler flow-meters: a proposal to characterize their technical parameters*. Flow Measurement and Instrumentation, Vol. 19, 261-267.

Lipeme Kouyi G., Vazquez J., and Poulet J-B. (2003). *3D free surface measurement and numerical modelling of flows in storm overflows.* Flow Measurement and Instrumentation, Vol. 14, No. 3, pp. 79-87.

Maghrebi M., Ball J.E. (2006). New method for estimation of discharge. Journal of hydraulic Engineering, Vol. 132 (10), 1044-1051.

Mueller D.S., Abad J.D, Garcia C.M., Gartner J.W., Garcia M.H., Oberg K.A. (2007). *Erros in acoustic Doppler profiler velocity measurements caused by flow disturbance*. Journal of hydraulic Engineering, Vol. 133 (12), 1411-1420.

Muste M., Patel V.C. (1997). *Velocity profiles for particles and liquid in open-channel flow with suspended sediment.* Journal of hydraulic Engineering, Vol. 123(9), 742-751.

Nash J.E., Sutcliffe J.V. (1970). *River flow forecasting through conceptual models, part 1 – a discussion of principles*. Journal of hydrology, 10 (3), 282-290.

Nezu I., Rodi W. (1986). *Open-channel flow measurements with a laser Doppler anemometer*. Journal of hydraulic Engineering, 112 (5), 335-355.

Sanderson B.G., Baginska B. (2007). *Calculating flow into coastal lakes from water level measurements*. Environmental Modelling & Software, Vol. 22, 774-786.

Tominaga A., Nezu I. (1992). *Velocity profiles in steep open-channel flows*. Journal of hydraulic Engineering, Vol. 118 (1), 73-90.

Versteeg H., Malalasekera, W. (1995). *An introduction to computational fluid dynamics, the finite volum method*. Prentice Hall, London, UK, 257 p.