PROCEEDINGS

ORAL PRESENTATIONS

TOPIC 1:
FLOOD MEASUREMENT TECHNIQUES

PAPERS:
COLLECTED AND ARRANGED BY JUDITH DOBMANN AND SIMONE HUNZIKER.
PLEASE NOTE THAT AUTHORS ARE RESPONSIBLE FOR CONSISTENCY IN SPELLING.

KEYNOTE SPEECH:
AN OVERVIEW OF FLOOD MEASUREMENT TECHNIQUES

Beat Sigrist

Federal Office for Water and Geology, 3003 Bern, Switzerland, beat.sigrist@bwg.admin.ch

SUMMARY

For the accurate or approximate measurement of flood flows there are many manual and automatic or continuously measuring methods available today.
For manual measurements we have the velocity area methods with, as the main representatives, the propeller type current meter, the acoustic doppler current profiler and the float. For turbulent water courses we can use the dilution method.
On the other hand, we have the automatic methods with the ultrasonic method, the horizontal acoustic doppler current profiler, the fixed weir and the dilution method.
For all these methods it is imperative to make sure that the site is suitable for the chosen method and all necessary safety precautions for undertaking such measurements during floods are in place.
To get more information about the uncertainty of such measurements and the stability of the site it is advisable to use different and independent techniques or to make a large number of discharge measurements during the time and over the whole range of water stages.

1 INTRODUCTION

When doing flood measurements there is considerable difference between a site which is upstream of any lake and in a catchment with steep slopes and therefore a rapidly flowing watercourse with a high amount of sediments and a rapidly changing stage, and a site which is downstream of a lake in a flood plane with rather moderate velocity and sediment amount.
During flood the river may be broad, deep, rapidly flowing, often turbulent, with big waves and highly sediment laden. The access to the gauging site may be difficult, even dangerous and all sort of hazards may occur during the measurement. On the other hand, the work has to be done quickly to a consistent high standard. So it is essential to choose the appropriate site and method in advance of the flood event. One has to be prepared for flood measurement and anything that can make it easier is welcome.

2 MANUAL METHODS

2.1 Current Meter

Propeller type current meters have been in use for centuries now and are still widely used and, in fact, state of the art for the velocity area method. A complete current meter measurement gives detailed information about the velocity distribution in the gauging section. The equipment can be used on a rod from a bridge, from a cableway attached to a heavy sinker or from a boat. But it is a time consuming method of measurement, two hours for such a discharge measurement is normal. And during flood it may be very difficult and dangerous to lower the equipment to the bed. So it may be more reasonable during flood gauging to execute only part of a discharge measurement with fewer verticals and points and to make additional velocity measurements at a lower and more stable water level. But with suitable equipment it should be possible to measure velocities up to 6 meters per second and more often the problem is not the high velocity itself but the surface waves which makes it difficult to place the current meter in the required position.

2.2 Acoustic Doppler Current Profiler

Current profilers are the newcomers of the last ten years. Originally used for measurements in the open sea, they were initially only useful in deep water courses of more than two meters. But with new developments it is now possible to use this equipment even in shallow water of less than half a meter. The ADCP may be mounted on a boat or attatched to a cableway and in one single traverse of the river collect all necessary data to compute the width, the profile and the velocity distribution. For these measurements the equipment may remain on the surface of the water. So a complete discharge measurement is done very quickly. A big advantage for rapidly changing water levels during a flood event.

2.3 Float Measurement

The use of floats is very old fashioned and low tech but nevertheless still in use and often a very suitable method to measure surface velocity. When the site and the staff are prepared well in advance, it is possible for almost anyone to be trained to throw a float and measure the time it needs to cover a certain distance. So it is possible to make use of local people for whom it is easier to be on site in time and execute this work. Once the surface velocity is known it is then possible to use another velocity area method at a lower water level, for instance a current meter measurement, to complete the vertical velocity profile by assuming similarities in the form of the velocity distribution and so complete the discharge measurement.

2.4 Radar Measurement

Radar is widely used today for the measurement of stage. It has the advantage that it can be located well above the highest flood, with a minimum risk of being damaged.
In recent years attempts have been made to develop a radar device suitable for velocity measurement. In simple terms, this method is very similar to float measurement, but on a higher technical level. The equipment is similar to that used by the police to measure velocity of cars. That means that the user has to stand on a bridge and focus on floating material coming towards or moving away from the observer to measure its velocity. The uncertainty of such measurements is rather high regarding the lateral traverse of the floating material and the fluctuations in velocity during flood.

2.5 Pitot Tube

The pitot tube is mostly used in laboratories where it offers very reliable measurements. There are some developments for use in open channel flow, even in combination with a sinker on a cableway. We made some testing of such equipment in our rating tank at the Federal Office for Water and Geology that was rather promising. However, the hazards of deployment are very similar for the pitot tube and the current meter. So, for manual measurements, staff prefer the current meter.

2.6 Dipping Bar

The dipping bar can measure the mean velocity of one vertical at a time. Today this equipment is used by hydrologists for the quick and easy measurement of low discharges in small, wadeable watercourses. There has been some research into the use of longer bars, which could be used from a bridge to measure mean velocities in verticals during flood. However, this resulted in a very bulky piece of equipment which is highly susceptible to wind effects. This adaptation is no longer being developed.

2.7 Electromagnetic Probe

Electromagnetic current meters may be used to advantage in smaller velocities where the propeller type current meter is unsuitable and in rivers with high weed growth. Here they are most useful because they have no movable parts. For velocities of more than 3 meters per second they are not the best choice.

2.8 Salt Dilution Method

The salt dilution method has been given new life with the introduction of data loggers and personal computers. Today, there is a big selection of very portable and easy to use equipment on the market which allow measurements in brooks and torrents to be made simply and quickly. Because of the low solubility of common salt in water, the maximum measurable discharge is only approximately five cubic meters per second. The most important requirements of the salt dilution method are a complete mixing of the dissolved salt in the water course within the mixing length and a stable value of background conductivity within the reach during the whole measurement.

2.9 Dilution Method with Fluorescent Dyes

Fluorescent dyes as sodium fluorescein offer the possibility to measure higher discharges, up to one thousand cubic meters per second. Here, it is no longer the solubility or the detectability of the tracer that is the main concern but that good mixing takes place in the water course within an acceptable reach.
With fluorescent dyes we can use the constant injection method. This method gives good information on the complete mixing within the reach, especially if samples of the diluted solution are taken and analyzed on both sides of the water course. It is more suitable for stable situations and therefore should not be used during rapidly changing floods. For flood flows the sudden injection method is more suitable. Here the analysis of the diluted concentration can be made by using a constant rate pump connected to a flow through spectrometer, or directly in the torrent using a so called field-fluorometer with a light probe. This detects the amount of tracer by emitting light into the water and measuring the amount of fluorescence produced by the tracer.

3 AUTOMATIC METHODS

3.1 Ultrasonic Method

For the ultrasonic method with fixed installations on both sides of a river it is absolutely necessary to have a regular profile over a long distance. Additionally no obstacles such as pylones in the reach upstream the site are permitted. These obstacles may cause the suction of air bubbles into the water at higher velocities. In addition the amount of suspended sediment has to be low because of the loss of signal due to reflection. It is possible to have ultrasonic devices at different stages for the same gauging section. This allows to determine the mean velocity in different depths and gives the possibility to use the ultrasonic method for complete discharge measurements over a whole cross section.

3.2 Horizontal Acoustic Doppler Current Profiler

The so called HADCP is a specially designed ADCP fixed on a river bank. It measures the horizontal velocity at one choosen depth. Additional measurements with for instance a current meter are then necessary to determine the vertical velocity profile. Compared to an ultrasonic measurement it has the advantage that this technology benefits from the suspended sediment in the water to measure the velocity of flow. This makes it suitable for the measurement of flood flows, where the amount of sediment may increase enormously, which would make the ultrasonic method unsuitable.

3.3 Pitot Tube

The fixed installation of a pitot tube on a river bank or on the bottom of a channel could provide a robust, accurate and cheap installation for the automatic measurement of the velocity in one point of a gauging section. The disadvantage is, that this measuring point has to be rather close to the bank or the bottom and therefore the use of this method is rather limited. However, it could be helpful in situations where it may be necessary to detect whether there is a supercritical flow situation during flood in a profile.

3.4 Electromagnetic Method

By laying a coil in the bed of a stream to create a magnetic field, a voltage is induced by the stream which is proportional to the flow. This makes it possible to determine the rate of flow continuously. However, this is a very expensive method, as it usually requires the temporary diversion of the whole river while the coil is being laid. Another problem is the leakage current which may reduce the quality of the measurement and which is very difficult to avoid. This method is used in rather slow flowing, well determined watercourses like canals, where it may be possible to take advantage of the construction of the canal to incorporate the coil.

3.5 Video Measurement

The video measurement by a Charged Coupled Device is a new development to take advantage of the newly available cameras and interpretation methods. More often in use in laboratory conditions until now, the technology looks promising. The method needs seeding of the water to get clearly identifiable floating particles on the surface. It also needs illumination of the site to allow measurements during bad light conditions and during the night. But if the problem of seeding is solved, or seeding is not necessary because of the high amount of naturally flowing particles on the surface, then it is an automatic method for surface velocity measurement from above. An additional difficulty could be the relatively small sample of the surface that is measured. This makes it difficult to take it to represent the mean velocity of the surface, especially in the presence of large surface waves that may ocurr during a flood.

3.6 Laser Doppler Method

Theoretically the laser doppler method could be very suitable for velocity measurement in an open channel. But because of the large dimensions of these channels and the capabilities of the equipment it is not yet very promising as a method for flood measurement in open channels. For the moment its use remains restricted more or less to the laboratory.

3.7 Dilution Method with Fluorescent Dyes

The sudden injection method with fluorescent dyes can be used for automatic but non continuous measurements of discharge during floods. At a certain stage, a water level measuring device can give a signal to a device containing predissolved tracer so that it injects an accurate amount of tracer into the torrent. Downstream, a lightprobe can be installed to measure the diluted concentration of tracer and so determine the actual discharge. The main difficulties for this method are to make sure that the whole amount of tracer is injected and dissolved immediately, that there is good mixing over the whole profile and to be certain of a reliable measurement from the lightprobe over the required interval.

3.8 Fixed Weir

A well calibrated fixed weir may be a very useful device for the measurement of flood discharge. The general equations for the discharge determination may be known from theoretical calculations, from laboratory measurements and from calibration measurements on site. It remains stable over a long time period, therefore only few discharge measurements are necessary. However, these days it is becoming more difficult to built such a weir in the face of restrictions imposed by other interests in the water business. More often it is much easier to chose a site where a weir already exists and is maintained. With some adaptions to produce a discharge measurement structure such a weir could be used to advantage for the precise determination of peak flow.

4 CONCLUSION

It is very rare that the highest flood at a gauging site is measured directly. As most hydrologists know: "The highest floods are likely to occur during the night and at weekends, preferably during Christmas". So, most often the staff are not on site, or not on site in time, or it is too dangerous to use the equipment because of lightning, high velocities, floating material or waves. Therefore the determination of peak flow has to be undertaken with the help of a stage discharge curve based on discharge measurements at lower stages.

Everyone establishing such rating curves knows how important each flow measurement is, even a partial measurement, and it is also preferable to choose a site where the parameters are stable and unlikely to change with every higher discharge in an unforseeable way.

On the other hand it is very useful to have some additional measurements as redundant measurements which allow us to determine the so highly appreciated peak flow using different approaches.

If someone says "One measurement is no measurement" my answer would be "No measurement at all is definitely much less"!

With the increasing possibility to determine the velocity-distribution and the discharge during the measurement with the use of a laptop, the staff on site has very usefull feedback on the quality and the usefulness of their efforts, which is both very motivating and tends to lead to an increase in quality at the same time.
Even at very well established and equipped gauging sites an extreme flood is a terrible phenomenon. Measurements that are easy to carry out in so called "normal" conditions become difficult, dangerous or impossible.
Automatic methods may help to have more measurements of floods in a shorter time and without exposing staff to the dangers of rapidly flowing and changing flood. But most often full advantage can only be gained if one has a good knowledge of what is happening during a flood, especially what is happening at the measuring section during the flood event. This means that manual methods and additional measurements on site remain important topics for a "wet" hydrologist.

REFERENCES

Landeshydrologie (1982): Handbuch für die Abflussmengenmessung. Bern

International Standards Organization (var.): ISO/TC 113 Hydrometric Determinations. Standards on Measurement Technique and Equipment. Geneva

NON-INTRUSIVE DISCHARGE MEASUREMENTS DURING FLOODS

Olivier Baud, Willi H. Hager, Hans-Erwin Minor

Laboratory of Hydraulics, Hydrology and Glaciology (VAW), Swiss Federal Institute of Technology Zurich (ETHZ), ETH-Zentrum, CH-8092 Zurich, Switzerland
baud@vaw.baug.ethz.ch, hager@vaw.baug.ethz.ch, minor@vaw.baug.ethz.ch

SUMMARY

The actual discharge measurement methods in rivers are often manual, intrusive and require a long observational duration. They are known to be accurate for low and mean discharges, but during flood season sediment transport, floating particles, and high a and IGP of the Swiss Federal Institute of Technology Zurich ETHZ have developed a novel non-intrusive method to measure discharge during floods. The method proceeds in two steps. In a first step the water surface is recorded with three calibrated CCD cameras using identical frequency. In order to determine the geometry and the velocity field of the free surface, floating tracer particles are added to the flow. In a second step a finite element program is employed to compute the discharge based on recorded surface information. An iterative procedure using the commercial finite element program CFX allows to determine discharge. The current approach considers rivers portions with a strong local bed geometry variation such a drop structures. The river bed geometry is measured with conventional methods prior or after the flood event. Laboratory experimentation demonstrated an average error in discharge measurement of 2 - 3 %.

Keywords: non-intrusive experimentation, flood discharge measurement, CCD camera, tracer particles, finite element program, iterative procedure, drop structure

1 INTRODUCTION

Discharge measurement in rivers provide a basis for the definition of return period and flood intensity. Because the design discharge for infrastructures or river works are usually much higher than the usual discharge spectrum an extrapolation is required. Standard measuring methods involving current meters, electromagnetic flow sensors, or ultrasonic metering systems are manual, intrusive and require a considerable observational time. Because of sediment transport, floating particles, and high velocities no measurement during flood season is feasible and the depth-discharge curve has to be extrapolated. To improve the definition of the design discharge the Swiss Federal Institute of Technology Zurich ETHZ developed a novel method to measure discharge during flood season. Typically, a flood discharge with a one-year return period is considered in the following. Two research projects were conducted: one focused on the numerical approach at Institute of Hydromechanics and Water Resources Management IHW (Sulzer, 2001), whereas the second investigated an experimental approach at the Laboratory of Hydraulics, Hydrology and Glaciology VAW (Baud, 2002), to be described in the following.

2 PRINCIPLES OF NOVEL DISCHARGE MEASUREMENT METHOD

2.1 Purpose

The novel method was assigned to be non-intrusive in order to avoid problems during flood season. Only an automatic discharge assessment provides flood measurement because of access difficulties to the gauging station (Kölling, Valentin, 1995). To assess the flood peak discharge the developed method should be able to make recordings in a short time interval. The novel method proceeds in two stages: In a first stage the method was developed in a hydraulic laboratory to check the basic principles, a second stage to be worked out in the future aims to use laboratory findings in a natural river. The first stage was completed recently at the VAW, ETHZ (Baud, 2002).

2.2 Main steps

The novel method involves two main steps for laboratory discharge measurement: In a first step the water surface is recorded with three calibrated CCD cameras to determine its surface geometry and the free surface velocity field. In a second step a finite element program is employed to compute the discharge with an iterative procedure, thereby accounting for surface observations as a boundary condition. Floating tracer particles (Figure 2-1) were added to the flow to visualise the free surface. The three CCD cameras used the same frequency, such that three images were recorded at the same time step t_1. A photogrammetric program developed at the Institute of Geodesy and Photogrammetry IGP allowed to identify the coordinates of the tracer particles. The location of a specific particle during subsequent time steps was determined with a tracking algorithm developed at VAW. Accordingly, calculation of the flow surface and the corresponding velocity field for an averaged time step may be obtained.

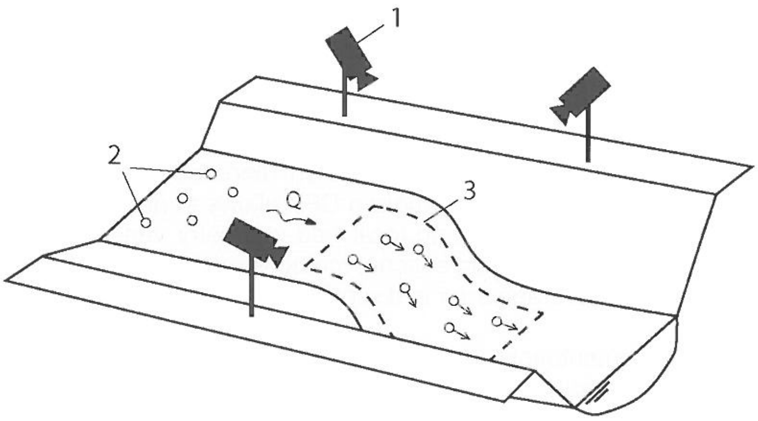

Figure 2-1: River section with (1) CCD cameras, (2) floating particles and (3) measured free surface characteristics at a drop structure.

To compute discharge accurately, drop structures in river courses should be selected. Then, roughness of the river bed has a small influence compared to its geometry. The current approach does not consider scour during flood, which seems unrealistic close to drop structures because of bed stabilisation with a sufficiently heavy riprap. The river geometry is measured with conventional methods prior or after a flood. A computational volume is defined between the averaged measured free surface and the river bed geometry (Figure 2-2). To determine the discharge an iterative procedure based on a Computational Fluid Dynamics CFD program was employed. For selected discharge Q the velocity field at the free surface was computed and compared with the measured velocities until a minimal difference resulted.

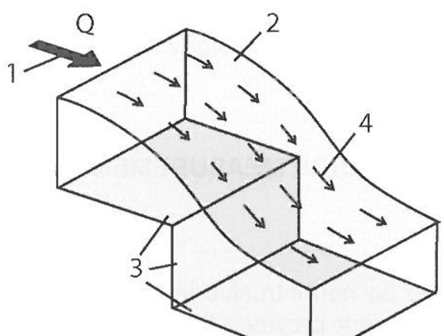

Figure 2-2: Computational volume with (1) selected discharge Q, (2) measured water surface, (3) river bed geometry and (4) computed velocity field at the free surface.

3 EXPERIMENTAL SETUP

A rectangular laboratory channel with an overall length of 6 m, a width of 0.40 m and a depth of 0.4 m was used for the present project. The side walls were of glass to allow for indirect illumination of the observational area. To avoid optical reflections, the channel bottom was coated black. The discharge in the closed circuit channel was regulated with a valve at the inlet pipe, measured with inductive discharge measurement IDM and checked with a standard gauging weir. The tracer particles consisted of white polyethylene having a density of 918 kg/m^3 and a diameter of 2 mm. Their colour was selected because of the sharp contrast with the black drop structure and their size was adjusted to suit a minimum of four pixels of the selected CCD camera. A shaking table allowed automatic feeding of the flow with tracer particles at a selected particle intensity.

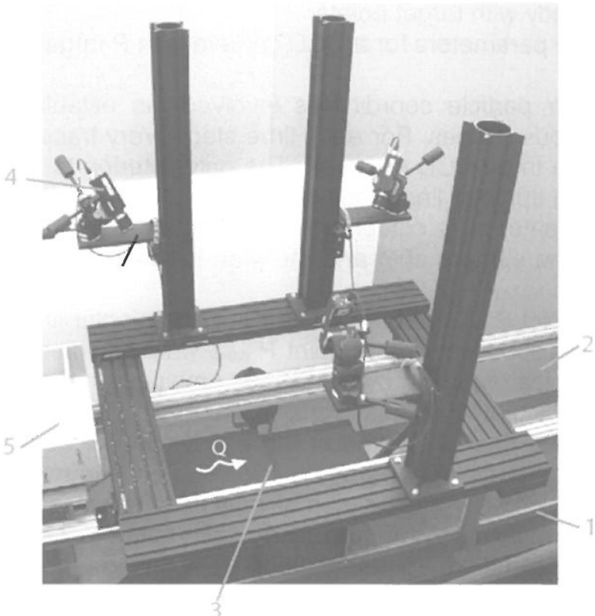

Figure 3-1: Experimental setup with (1) laboratory channel, (2) transparent side walls, (3) observational area including drop structure, (4) CCD calibrated camera and (5) shaking table for automatic flow feeding with tracer particles.

The monochrome CCD cameras (Pulnix TM-6702) had a progressive scanning interline transfer with 640 horizontal and 480 vertical lines resolution, allowing for a maximum scanning frequency of 60 Hz. An extern master clock imposed equal frequency for the three cameras. A maximum sequence of 128 time steps could be saved in the compute with a capacity of 128 RAM. The selected optical system had three manual standard lenses (Cosmicar B1214D-2) with a focal length of 12.5 mm.

4 WATER SURFACE MEASUREMENT

4.1 Spatial tracer particle coordinates

The calculation of the tracer particle coordinates was based on an eleven parameter optical model for each camera (Maas, 1992; Virant, 1996; Stüer, 1999). A calibration procedure defined these parameters. The black reference body with white target points illustrated in Figure 4-1 (a) was placed in the observational area and imaged by the three CCD cameras. Knowledge of the target point coordinates determined the parameters of the optical model. The redundant information on account of many target points require a resolution with the least squares adjustment. Among the six parameters used to describe the external orientation of a camera, three describe the lens centre C, and three related to the rotation angles α_C, β_C, γ_C (Figure 4-1 (b)). Three other parameters were used for the internal orientation with two for the camera chip centre C′ and one for the distance from the lens centre to the camera chip $d_{CC'}$. The last two parameters of the optical model described the lens distorsion by k_{radial} and $p_{tangential}$ (Brown, 1971).

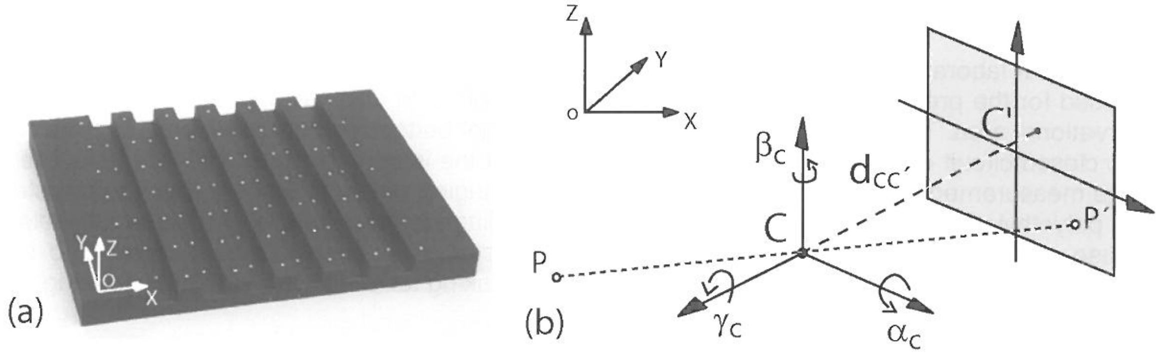

Figure 4-1: (a) Reference body with target points,
(b) optical model parameters for a CCD camera with P target point and P´ image point.

The calculation of the tracer particle coordinates involved the establishment of correspondences between the three simultaneous images. For each time step, every tracer particle had to be identified in the recorded images of the three CCD cameras. The only criterion to be applied for distinction of a certain tracer particle was the epipolar line procedure (Maas, 1990), given that a specific particle does not show any characteristic feature like colour or shape (Figure 4-2 (a)). The epipolar line procedure involves the intersection of the camera chip and the plan formed with the lens centres C of the two cameras and a target point P.

Proceeding from an image point P'_{cam1} of the first camera an epipolar line in the second camera was calculated, on which the corresponding image point P'_{cam2} was identified (Figure 4-2 (b)). Due to the large number of tracer particles, typically over 100, an ambiguity may occur because two or more candidates match with the calculated epipolar line. Therefore a third camera was used. The image point P'_{cam3} of the third camera thus is located at the intersection of both epipolar lines and allowed unambiguous definition of any particle in the array investigated (Figure 4-2 (c)). The tracer particle coordinates was determined by spatial intersection using a least squares adjustment. A second benefit of the three camera configuration as compared with a twin camera set was improved accuracy of tracer particles coordinates. For the experimental setup used, horizontal and vertical accuracies were, respectively, 0.15 mm and 0.3 mm.

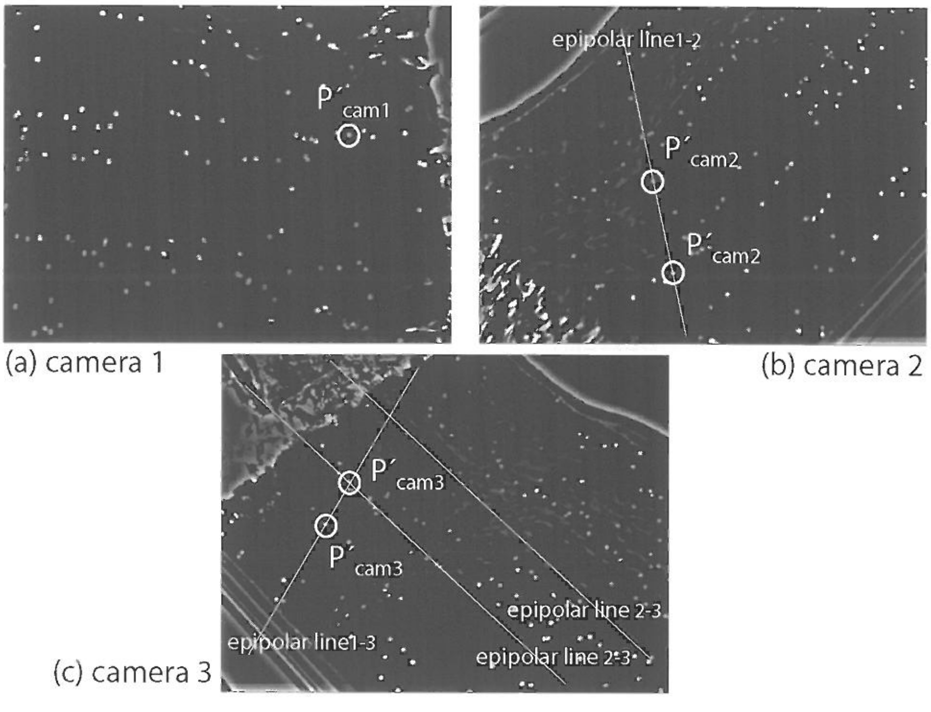

Figure 4-2: Establishment of correspondences between three simultaneous images using the epipolar line procedure.

4.2 Tracking Algorithm

Once the spatial tracer particle coordinates were defined for the several time steps, the particles had to be linked from one time step to the next. A program based on a tracking algorithm (Malik et al., 1993; Papantoniou, 1989) was developed at VAW. Knowledge of subsequent tracer particle positions allowed calculation of the velocity field. At the drop structure studied the illumination intensity at the free surface was non-uniform over the observational area. Because of the curved free surface light reflections made tracer particle detection locally difficult. Therefore, the developed tracking program considered only two consecutive time steps for calculation. To limit the number of candidates $P(t_2)$ originating from tracer particle $P(t_1)$, i.e. to find the particle trajectory within one time step $\Delta t = t_2 - t_1$, a search volume was defined by a minimum V_{min} and a maximum V_{max} probable velocity (Figure 4-3). The search volume was located between two concentric spherical surfaces with respective radii R_{min} and R_{max}.

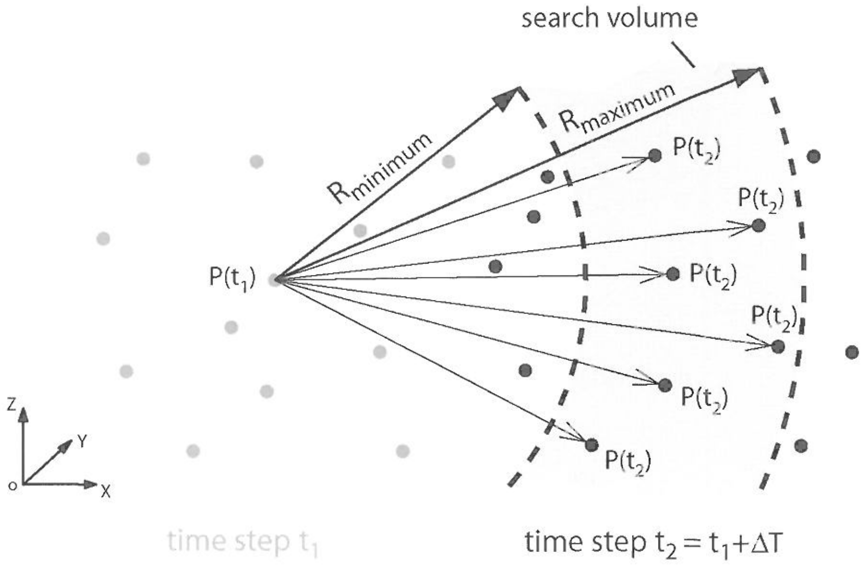

Figure 4-3: Search volume defined within two concentric spherical surfaces of radii $R_{minimum}$ and $R_{maximum}$.

The minimum of the function f defined as

$$
\begin{aligned}
f &= f_X \cdot [P_X(t_1) - P_X(t_2)]^2 + f_Y \cdot [P_Y(t_1) - P_Y(t_2)]^2 + f_Z \cdot [P_Z(t_1) - P_Z(t_2)]^2 \\
&\text{with } P(t_1) = (P_X(t_1), P_Y(t_1), P_Z(t_1)) \\
&\qquad P(t_2) = (P_X(t_2), P_Y(t_2), P_Z(t_2))
\end{aligned} \tag{1}
$$

calculated for each candidate $P(t_2)$ enabled to select the correct particle at time step t_2. The three directional weighting factors (f_X, f_Y, f_Z) allowed to consider the main flow direction. A program in Visual Basic language enabled an automatic computation of the velocity field. The maximum error in velocity calculated according to Adrian (1997) was $V_{error\_max} = \pm 0.02$ m/s.

4.3 Time-averaged free surface characteristics

Because of the short measurement time of less than 2 seconds, the flow conditions were supposed to be steady. The time-averaged water surface geometry was interpolated with tracer particle coordinates for all time steps. A horizontal scale allowed to calculate an average water surface position for every cell. The same grid was used to interpolate a time-averaged free surface velocity field.

5 EXPERIMENTATION

5.1 Observations

The novel non-intrusive discharge measurement method was tested in a laboratory channel for various bed geometries and flow conditions (Baud, 2002). The computed discharge Q was compared to the reference discharge measured with an IDM and checked with a standard gauging weir. Two drop structures with respective heights of 40 mm and 60 mm were considered. The measured discharges were 6 - 15 l/s for the small drop and 9 - 17 l/s for the larger drop. In the both cases a sequence of 50 time steps with a 50 Hz frequency was recorded. Figure 5-1 shows the measured time-averaged water surface characterics at the drop structure for a discharge Q =12 l/s.

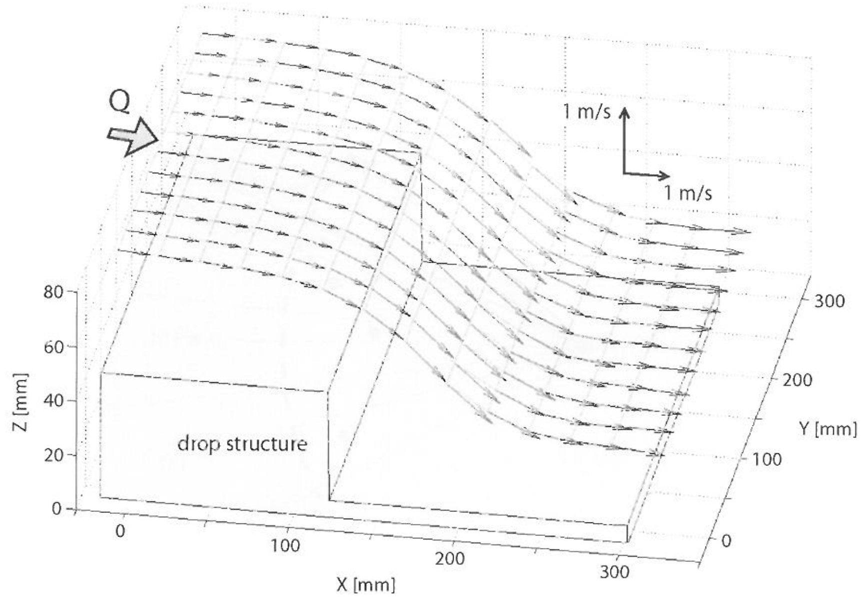

Figure 5-1: Measured time-averaged water surface at the drop structure for Q= 12 l/s.

The free surface characteristics do not reveal a significant transverse effect because observations close to the channel walls were impossible for the selected three camera set-up. The latter set-up arrangement was dictated by purely optical requirements regarding the accuracy of particle detection. The effect of the wall boundary layer on discharge prediction was accounted for by using the law of the wall (Graf, 1993).

5.2 Computation

Because knowledge of only the water surface geometry and free surface velocity field does not allow to compute discharge Q, an iterative procedure based on the commercial CFD program "CFX" was used. This program is based on the Navier-Stokes equations and a k-ε turbulence closure model. The flow conditions in the channel were similar in the transverse direction as mentioned previously such that a *two dimensional computational model* was applied. The area between the average measured free surface profile and the bed geometry was subdivided with a grid (Figure 5-2 (a)). Computations involved the following issues: (1) a grid with 78 horizontal and 15 vertical subdivisions, (2) kinematic condition at the free surface, i.e. no velocity component perpendicular to it and (3) law of the wall close to bottom geometry. The iterative procedure initiated with the computation of the velocity field for a selected discharge Q (Figure 5-2 (b)). The difference $E_{difference}$ between the computed $V_{compute}$ and the measured $V_{measure}$ free surface velocities (Figure 5-3) was used to compute

(2) $$E_{difference} = \sum \left(V_{compute} - V_{measure}\right)^2$$

The procedure continued for others selected discharges until a minimum for the difference $E_{difference}$ was reached. According to the principle of the inverse modelling, the minimum of the objective function resulted in the predicted discharge $Q_{predicted}$. The objective function was minimized by interpolating a curve between the calculated points (Figure 5-4).

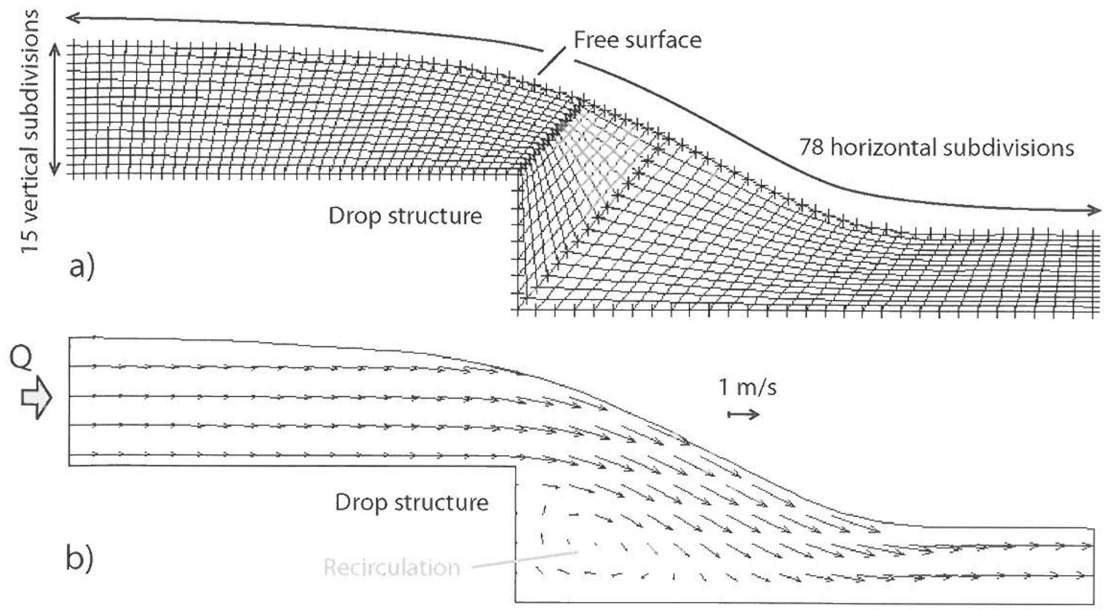

Figure 5-2: (a) Computational grid, (b) computed velocity field for Q=10 l/s.

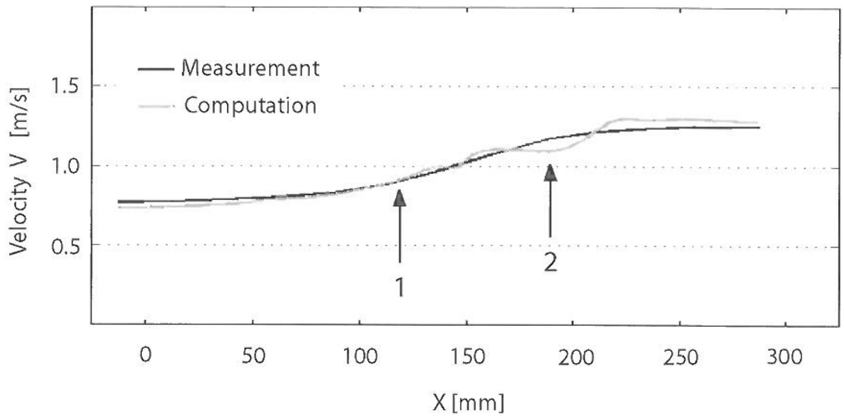

Figure 5-3: Comparison between measured $V_{measure}$ and computed $V_{compute}$ velocities for Q=12 l/s with (1) drop section and (2) end of recirculation zone.

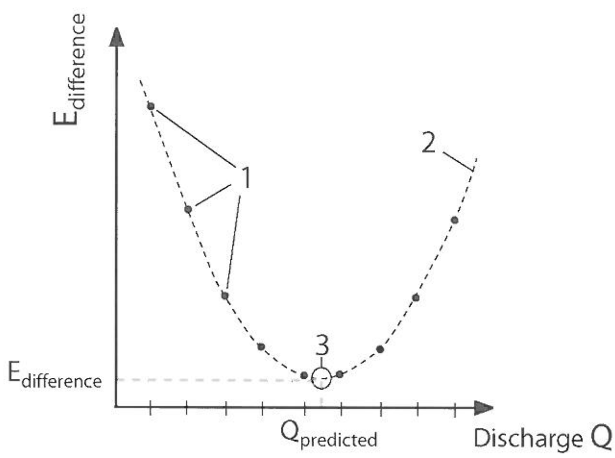

Figure 5-4: Objective function $E_{difference}(Q)$ with (1) computational points, (2) interpolated curve and (3) minimum of objective function identical with predicted discharge $Q_{predicted}$.

5.3 Results

Figure 5-5 (a) shows objective functions $E_{difference}(Q)$ for various discharges selected for the small drop structure. Note that all curves are continuous and have a well-defined minimum. According, for each curve, a prediction of discharge $Q_{predicted}$ results. The computed discharges $Q_{predicted}$ were compared to the references discharges. For the small drop structure resulted an average relative error $\overline{f_Q}$ = ±3.05 % with a standard deviation σ_{f_Q} = ± 0.8 % almost independent of discharge Figure 5-5 (b)). For the larger drop structure the relative average error was $\overline{f_Q}$ = ±2.3 % with a standard deviation σ_{f_Q} = ±0.9 %, again independent of discharge.

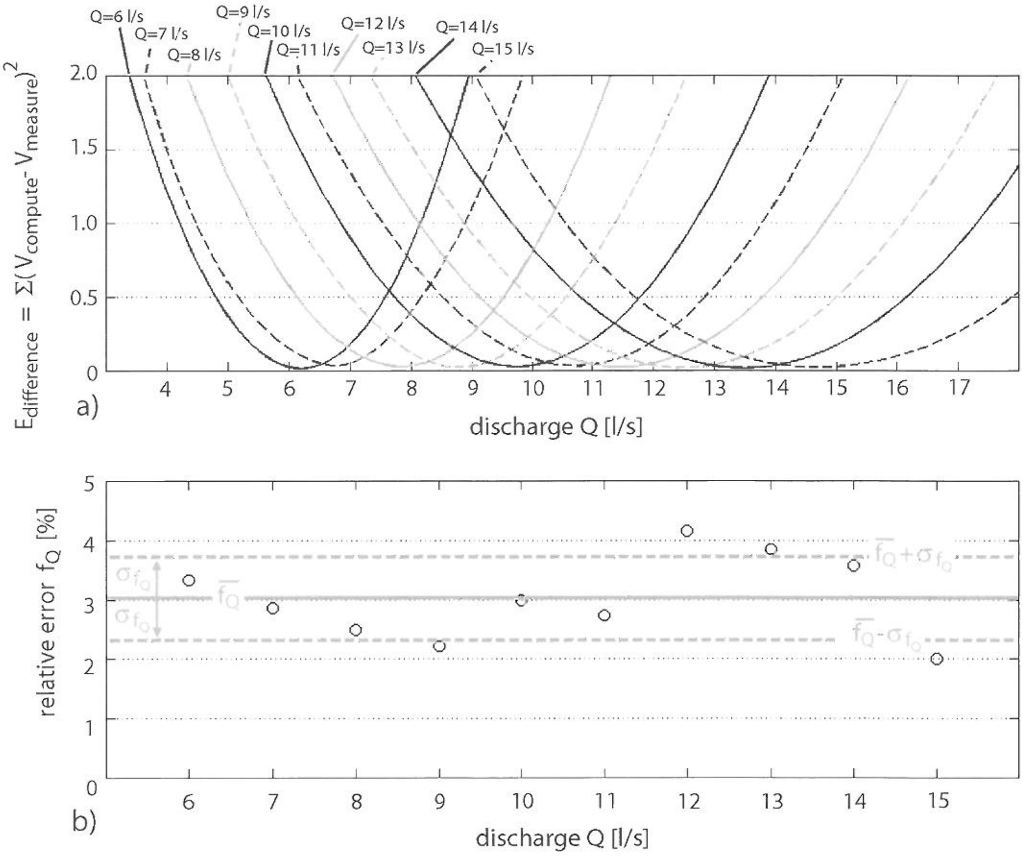

Figure 5-5: (a) Objective functions $E_{difference}(Q)$ for small drop structure, (b) relative errors as a function of discharge, with an average $\overline{f_Q}$ = ±3.05 % and a standard deviation σ_{f_Q} = ± 0.8 %.

6 CONCLUSIONS

A novel non-intrusive discharge measurement method was model-tested for various bed geometry and flow conditions. The method was based on inverse formulation of discharge determination using surface characteristics of the flow and a mathematical model by which these were iteratively solved. The experimental approach involved a tracking algorithm coupled with optical particle detection, whereas the mathematical model employed a two-dimensional formulation by which the axial flow features were investigated. To compute discharge accurately the flow over a drop structure was investigated, resulting in a relative error of 2 - 3 % with a standard deviation of 0.8 - 0.9 %. The observations indicated no reduction of relative accuracy for the larger discharges. In comparison with standard measuring methods such as current meters, the novel method may be considered an alternative in the future.

The next stage in the development of the method proposed aims to use laboratory findings in natural rivers. Some features of the optical system have to be adapted to particular site conditions. During flood season surface float could prevent the proper identification of tracer particles. The use of red tracer particles with RGB (Red-Green-Blue) CCD cameras or infrared cameras with warm tracer particles could solve the identification problem.

ACKNOWLEDGEMENTS

This research project was supported by "Doktorieren an der anderen ETH" of ETH-Rat, ETH-Zentrum, CH-8092 Zurich, Switzerland. The project focused on the numerical approach was initiated by Prof. Dr. W. Kinzelbach IHW, and supported in terms of photogrammetric methods by Prof. Dr. A. Grün IGP. We would also like to acknowledge support of Dr. F. Hermann VAW for computational aid.

REFERENCES

Adrian, R.J. (1997): Dynamic ranges of velocity and spatial resolution of particle image velocimetry. Measurement Science and Technology 8(12): 1393-1398. UK

Baud, O. (2002): Berührungslose Durchflussmessungen bei Hochwasser. Dissertation (in preparation), ETH: Zürich, Switzerland

Brown, D. (1971): Close-range camera calibration. Photogrammetric Engineering 37(8): 855-866. Bethesda (USA)

Graf, W. (1993): Hydraulique fluviale. Traité de Génie Civil, Volume 16, EPFL, Switzerland

Kölling, C., Valentin, F., (1995): SIMK-Abflussmessungen. Wasserwirtschaft 85(10). Stuttgart (Germany)

Maas, H.-G. (1990): Digital photogrammetry for determination of tracer particle coordinates in turbulent flows research. SPIE Proceeding Series Vol 1395, Part 1. Bellingham (USA)

Maas, H.-G. (1992): Digitale Photogrammetrie in der dreidimensionalen Strömungsmesstechnik. Dissertation 9665, ETH: Zürich, Switzerland

Malik, N., et al. (1993): Particle tracking velocimetry in three-dimensional flows, Part II: Particle tracking. Experiments in Fluids (15): 279-294. Berlin (Germany)

Papantoniou, D., Dracos, T., (1989): Analysing 3-D turbulent motions in open channel flow by use of stereoscopy and particle tracking. Advances in Turbulence (2): 278-285, Editor H. Fernholz, H. Heidelberg (Germany)

Stüer, H. (1999): Investigation of separation on a forward facing step. Dissertation 13132, ETH: Zürich, Switzerland

Sulzer, S. (2001): Flood discharge estimation for complex river geometries by inverse numerical modelling. Dissertation 14383, ETH: Zürich, Switzerland

Virant, M. (1996): Anwendung des dreidimensionalen PTV auf die Untersuchung von Dispersionvorgängen in Kanalströmungen. Dissertation 11678, ETH: Zürich, Switzerland

IMPROVING THE FLOOD HANDLING QUALITIES OF POWER STATIONS AND OTHER HYDRAULIC STRUCTURES

Christian Meier

Berater für Energieunternehmen, St. Niklausstrasse 17, CH-8103 Unterengstringen
christian.meier.lav@bluewin.ch

SUMMARY

Recent flood events have shown obviously that improvements in their management are always possible and necessary. This paper deals with advanced discharge measurement in river basins and the elimination of deficiencies in level control algorithms for dams and power stations.
Besides the tried limnigraphs, also the discharge measurement of power stations and regulation weirs can be used to improve decision making tools for flood handling. Therefore the formulae used for discharge determination for overflown and underflown regulation gates must be checked for accuracy also at extreme values. Neglecting the kinetic terms in the equations of continuity and energy conservation is not permissive and underestimates the discharges.
When establishing such additional measuring locations still leave white spots for discharge assessments during flood events, offline-simulation facilitates emergency procedure planning and on-line simulation permits the prognostic plotting of hydrographs for critical locations. Additionally, the retention volumina in the river basin can be evaluated for manual intervention. Such consideration of the retention volumina will be integrated in future flood handling procedures.
The discharge control in river power stations is primarily based on level control algorithms. These level control algorithms are unsuitable for the control of extreme flood events and lead to steeper discharge gradients and unnecessary high flood peaks.
For the elimination of these deficiencies, widely accepted quality assurance standards are required. The measurement and control equipment of weirs and power stations should be designed according these standards, assuring not only maximum energy production but also optimum flood handling qualities.

Keywords: weir gate discharge determination, level and discharge control, emergency procedures

1 INTRODUCTION

Two very impressive situations during the flood events in spring 1999 will introduce into the topic:

1.1 Events in Passau, Germany

Tuesday, 25th of May, 1999
At the Inn river, on the austrian side, the weir gates of the power station are opened after heavy rainfall to prevent flooding of Schärding. Consequently the level in the Donau river at Passau increases without early warning.
08:00: The level in the Donau at Passau reaches 930 cm, the city center is flooded, heavy complaints of the community of Passau towards the austrian power station operators (Pfeifer, 1999).

1.2 Observation in the Bremgarten-Zufikon power station:

Wednesday, 12th of May, 1999
Early in the morning, the level at the hydrometrical station "Mellingen" exceeds the measuring range and consequently no exact level and discharge values were available from this station. Later that morning, Bremgarten-Zufikon power station control room was frequently contacted by the desaster handling staff to assess the actual discharge in the Reuss river and to gather information about further increases of discharge. In close cooperation with this staff, early backwater reduction resulted in cutting off of the discharge peak by 60 m^3/s and prevented damages at the historical wood bridge and flooding downstreams of the power station.

1.3 3 Topics

The above mentioned experiences demonstrate, that power station personnel is often at the line of fire during flood events. There main task is debris removal. Additionally, they are contacted for discharge determination, and last but not least they are operating the hydraulic structures with highest possible impact on flood course.
To improve flood emergency procedures, the interface between water management authorities and power station operators must not be neglected. When a flood event announces, the personnel on duty often is confronted with the following questions about the peak discharge:
1. How can it be determined?
2. How can it be scheduled?
3. How can it be influenced and possibly diminuished?

Exactly those three questions are the main topics of this paper. It concentrates not on tools for flood prediction but rather on tools that can be used for emergency procedure planning and for decision making during the course of flood events.

2 DISCHARGE DETERMINATION

Most of the hydrometrical gauging stations are based on level measurement followed by the computation of the discharge using a look-up table. Recently installed gauging stations are also equipped with ultrasonic discharge measuring equipment. Additionally to these gauging stations, also hydraulic structures with suitable measurement equipment can be used for discharge determination, e.g. weirs, power stations, locks etc. (DVWK Guidelines 301/1990). Some problems arise at high discharge values, when the chosen simplifications in the calculation methods decrease the accuracy and end with a failure when the gates are fully opened.

2.1 Traditional Methods of Discharge Determination

2.1.1 Gate discharge without backwater effects

There is a free flow just downstream of the gate, where no water mattress lies above the downstream jet. The discharge is independent from the downstream water level.
Neglecting the head loss at the gate, the discharge can be calculated by the Torricelli formula:

(1) $$Q = B \cdot C_q (H_{UK} - H_S) \cdot \sqrt{2g((H_{OW} - H_S) - C_q(H_{UK} - H_S)}$$

where B is the width of the gate and C_q the jet contraction coefficient.

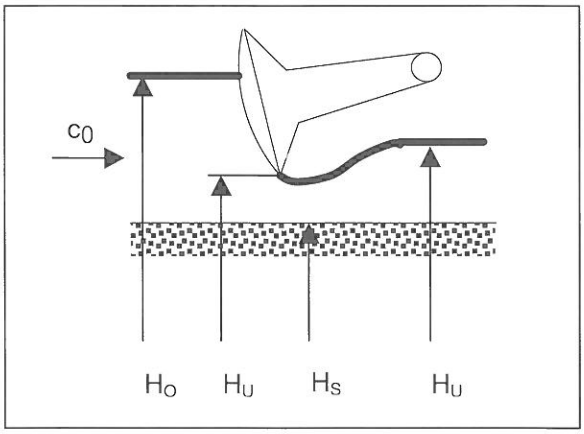

Figure 2-1: Symbols and definitions.

2.1.2 Gate discharge with backwater effects

The same equation may be used, taking into account the water mattress above the jump in the momentum balance. The equation simplifies to:

(2) $$Q = B \cdot C_q (H_{UK} - H_S) \cdot \sqrt{2g(H_{OW} - H_{UW})}$$

The accuracy for gate openings up to 50% is suitable, in some cases the computation with adaptive C_q leads to accurate results even with higher gate openings. The deficiency of this formula is, that the resulting discharge reaches saturation much earlier than in reality.
Application of this formula without adaptive contraction coefficient C_q for highest gate openings underestimates remarkably the discharges.

2.1.3 Gate discharge with relevant approach velocity

The coincidence with reality can be significantly improved by introducing the mean approach velocity c_0 in formula 2):

(3) $$Q = B \cdot C_q (H_{UK} - H_S) \cdot (c_0 + \sqrt{2g(H_{OW} - H_{UW})})$$

It must be considered, that the mean approach velocity c_0 is directly proportional to the resulting discharge, so a computation with iteration is necessary.

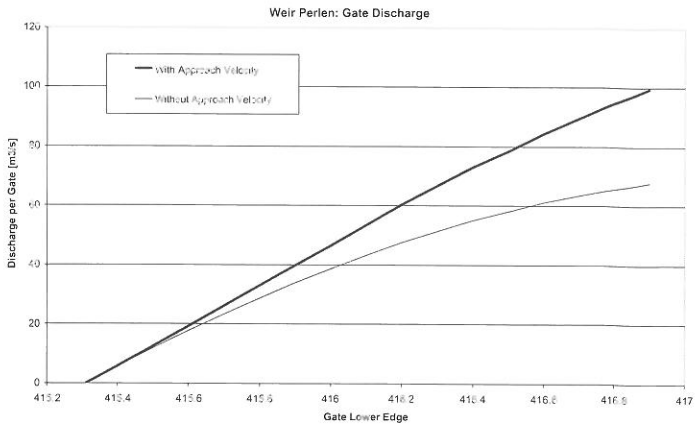

Figure 2-2: Influence of approach velocity.

Calibration performed at Perlen weir proofed the significantly increased accuracy. The chosen simplification using a mean velocity still has deficiencies in special cases, e.g. weirs downstreams of rivers bends, gates out of service.

2.2 Advanced Methods

Future improvements seam possible in the following fields:
- Transferring the results of hydraulic model tests into a form suitable for programmable logic controllers, taking into account also oblique and irregular velocity distribution after river bends and for the N-1 cases.
- Eventual coefficient adaption using higher algorithms or fuzzi logic.
- Formulae and coefficients for fully opened gates including the transition from and to supercritical flow surface

The aim is to obtain easy formulae with high accuracy up to the extreme flood values. Input values should be upstream level, downstream level and velocity distribution of the last computation step. The adaption of the coefficients should be in a transparent manner.

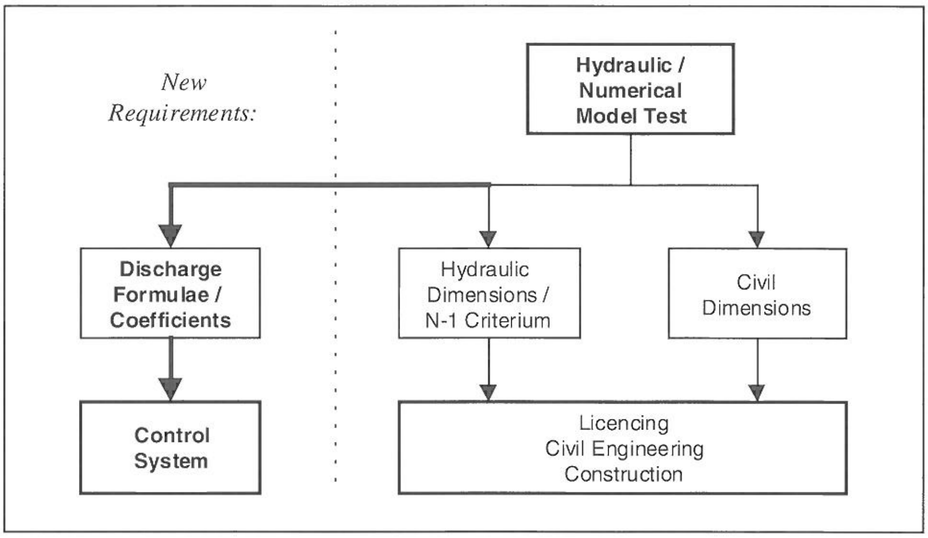

Figure 2-3: Hydraulic Model Test.

2.2.1 New requirements for hydraulic model tests

Advanced formulae have to be developed using model test results and finding admissible simplifications to suit the formulae to the use in a PLC system.

2.2.2 Free surface flow through gates with stilling basin

Discharge becomes independent from gate opening, when the gates are removed completely from the water surface.
Under certain circumstances, the formula for a broad crested weir can be used, but the kinetic term has to be integrated in a similar manner as in formula 3). Prerequisite is that the difference from the gate invert down to the stilling basin invert is bigger than the head difference from upstream level to downstream level.

2.2.3 Free surface flow through gates without stilling basin

More problematic becomes determination, when the invert is not lowered after the gate. The situation then is similar to a trash rack, where the gate piers are the analogon to the rack bars. When the pier shape is known, the loss coefficients can be estimated.
Nevertheless, determination with the trash rack formula (Kirschmer, 1926) is of poor accuracy due to the problematic measurement of head, but the advantage of this approach is the experience with irregular and oblique inflow, which can be incorporated (Zimmermann, 1969).

3 DISCHARGE SCHEDULING

3.1 Full scale test

Apart from few exceptions, full scale tests of flood events are out of reach for analysis of the dynamic river behaviour.

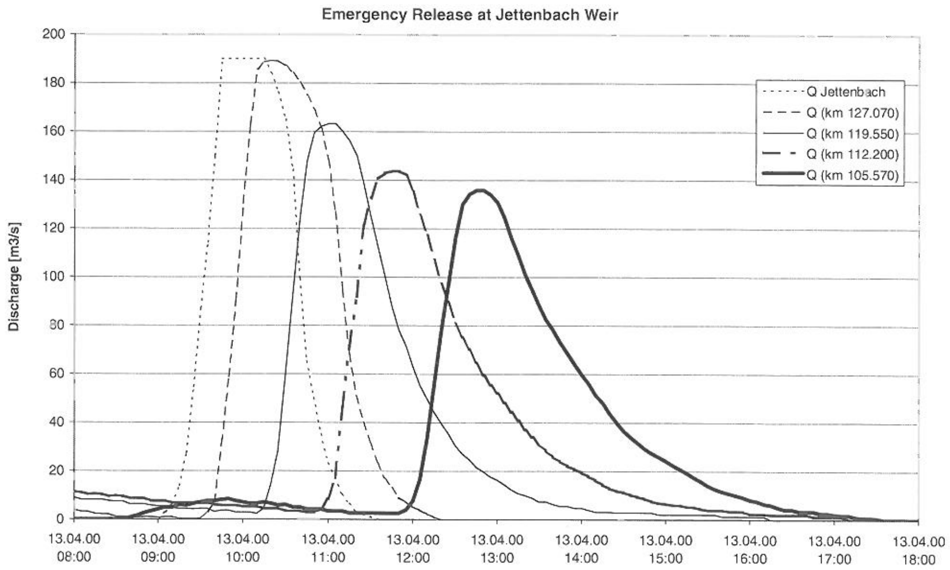

Figure 3-1: Full scale test "Jettenbach".

The analysis of the hydrographs at five different cross-sections during the course of a big wave propagation test at Jettenbach weir shows in principle the following main characteristics of natural river beds:

- Propagation times from cross-section to cross-section. This is the propagation of current and has to be clearly distinguished from the wave propagation (which is faster) and the particle propagation which is slower.
- Damping behaviour from cross-section to cross-section. It clearly can be seen that the damping in positive direction is remarkably lower than the damping in negative direction. This ambiguous behaviour has its origin in elements which show a dynamic characteristic depending on filling hight, e.g. barriers, hollows, aquatic vegetation. Such elements are completely missing in man-made canals with concrete linings. In case of such canals, the typical symmetrical damping characteristic is the direct consequence of the retention behaviour of free flow channels.

3.2 Retention behaviour determination with numerical simulation

The above mentioned ambiguous retention behaviour of natural rivers has to be modelled for good coincidence with reality, either by singularities or by variable friction coefficients. When this is taken into account, the numerical simulation shows the same results as the full scale test. For flood handling the retention volumina are of main interest for the setting of a suitable strategy to cut off the peak discharges.

Very important during flood routing simulation are the next topic's aspects which are level and discharge regulation and the so-called backwater lift-up, which have to fixed together with the power station operator and have to be modelled also.

4 HOW CAN THE PEAK DISCHARGE POSITIVELY BE INFLUENCED?

The aim of all flood prevention strategies is to avoid flooding as far as possible. Flooding occurs when discharges and levels increase above critical values. Discharges and levels are influenced not only by precipitation but also by the operational manoeuvres of the regulated hydraulic structures in the river stretches. These structures as dams, weirs and power stations are normally controlled by level control loops.

Level control loops in river systems with cascades of barrages are unsuitable due to there inherent tendency to a discharge overshoot in order to keep the upstream level constant. The resulting reaction of a cascade of such level controlled hydraulic structures is that the rate of change and also the peak discharge values will increase from barrage to barrage. This behaviour is exactly the contrary of the aim of flood handling procedures.

4.1 Discharge Control instead of Level Loop Control

Especially in rivers with cascades of barrages, so-called fully exploited rivers, additional feedforward control is the only possibility to avoid discharge overshoot and consequent aggravation of the flood situation. State-of-the-art feedforward control for level control loops use the inflow discharge value which is superimposed to the output of the first stage level control loop. That sum acts as input to the second stage discharge control loop (Meier, 2000).

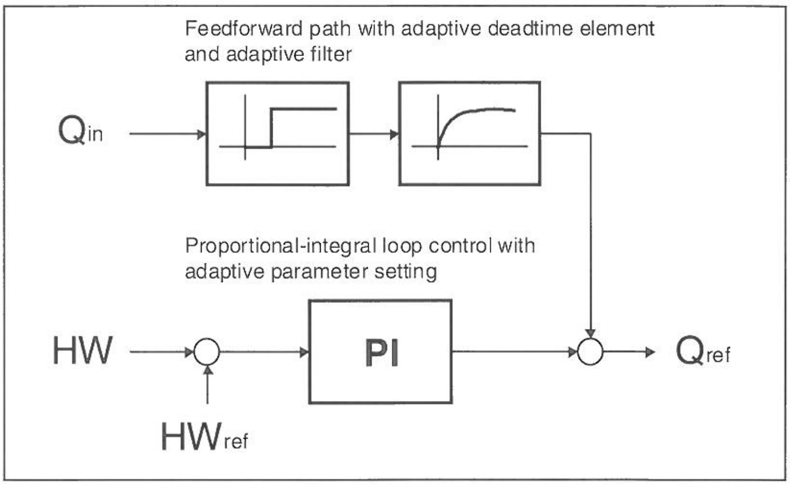

Figure 4-1: Feedforward Control.

The other prerequisites to aim improved flood and low water control characteristics are:
Suitable inflow signal which represents the real inflow situation.
For correct setting of the feedforward control, the signal deadtime must depend from the actual discharge, short deadtime for high discharges and vice versa.
For correct reaction of the combined loop/feedforward control in all situations, the retention behaviour must be modelled, preferably with adaptive filters for the inflow signal.
For correct setting of the loop control, gain and integral action time must depend from the actual discharge which take into account the rapidly increasing system damping constants for increasing discharges and vice versa.

4.2 Backwater Lift-up and its Timing

A further relaxation of the flood situation is even possible, when the hydrographs can even be flattened. This can cut the peak discharges in case of short events, or at least gain some time to introduce protection measures in case of long lasting events. Therefor the upstream retention volume has to be maximized. This means that the upstream level has to be lowered down to the unrestrained level, the so-called backwater lift-up. Backwater lift-up leads to the complete opening of all available gates, and to the removal of backwater effects induced by structures farther downstreams, their backwater lift-up also. When upstream retention volume is maximized by the backwater lift-up, maximum flattening of the flood hydrograph (see Figure 3-1) occurs. Additionally, so-called detention volume is made available for cutting off the peak of the flood wave manually.

4.2.1 Difficult decision making for backwater lift-up

Power station operators are confronted with the solution of the following timing problems:

- Backwater lift-up means reducing the head to such a degree, that turbines have to be shut down. The operator therefor has to renounce to energy production. Consequently, it will be initiated at the latest possible moment.
- Backwater lift-up means increasing release discharge above the inflow discharge. This procedure has to be coordinated with the downstream power station operators. Most favourable seams the simultaneous backwater lift-up on the whole river stretch. All operators have to be convinced of the correct timing of this measure.
- The backwater lift-up has to be completed before the peak discharge is expected. So it has to be initiated early enough, based on a reliable forecast of the course of the flood event.

4.3 Efficient Implementation

4.3.1 Planning

The feasability of improved flood handling procedures for river streches can easily be checked with a rough volume estimation. If improvements seam possible, a numerical simulation settles the limitations and necessary measures. The results of the numerical simulations allow all involved parties to negociate and fix an adequate flood handling strategy. This includes a catalogue of measures to been executed below the desaster level, with clear responsabilities and also clear allocation of measures to each actual or forecasted discharge. This includes manually interventions as well as all automatic functions in a manner, that control systems can be designed accordingly.
As a precaution, also back-up strategies for the most probable failure scenarios should be defined for the clear separation of responsabilities in case of failures.

4.3.2 Quality Assurance

For the implementation of the planned flood handling procedures and their functional assessment, widely accepted quality assurance standards are required:

- Especially the fully automatic functions implemented in control systems of weirs and power stations should not only be designed according these standards, but also commissioned and maintained.
- The calibration and parameter setting must include also extreme situations like low water and flood discharges up to the design flood.
- Operating personnel must be trained not only for the daily business jobs but also in emergency procedures. Simulators similar to the ones used for planning can generate realistic training situations.

Due to the high regulation impact of the inflow discharge value to the correct functionning of fully automatic flood regulation, the following measures are highly recommended:

- Measuring range should cover also extreme flood dischages.
- Signal back-up (e.g. tailwater level of the upstream power station) for plausibility check.
- Signal supervision and suitable, fully automatic control system reactions in case of signal failures.

4.3.3 Responsabilities

Water management authorities release manuals and guidelines for flood handling planning and all necessary procedures (Bundesamt für Wasser und Geologie, 2001), but below the desaster level a series of measures, as outlined herein, have to be settled additionally. A clear separation of the responsabilities between water management authorities and power station operators has been drawn.

5 CONCLUSIONS

When the aims outlined in this paper can be reached and all measures implemented, the operators of power stations can face flood events more comfortably and headlines like "the city center is flooded, heavy complaints of the community towards the power station operators" will be avoided in the future. Nevertheless, clear limits of liability have to be settled.
Scientists, hydraulic laboratories and water management authorities are summoned to contribute also to reach these goals.

REFERENCES

Pfeifer, H. (1999): Das Pfingsthochwasser 1999 - Eine Chronik, Bayerischer Rundfunk, München

DVWK Guidelines 301/1990 (1990): Manual for Water Level Gauging and Discharge Measurements, Verlag Paul Parey, Hamburg

Kirschmer, O. (1926): Untersuchung über den Gefällsverlust an Rechen, Mitteilungen des Hydraulischen Instituts der Technischen Hochschule München, München

Zimmermann, J. (1969): Widerstand schräg angeströmter Rechengitter, Mitteilungen des Theodor-Rehbock-Flussbaulaboratoriums, Technische Hochschule Karlsruhe, Karlsruhe

Meier, C. (2000): Wasserhaushaltsregelungen und neue Anforderungen an die Führung von Flussstauketten", Wasser, Energie, Luft, Baden

Bundesamt für Wasser und Geologie (2001): Hochwasserschutz an Fliessgewässern - Wegleitung 2000, EDMZ, Bern

INVERSE MODELLING TO ESTIMATE FLOOD DISCHARGE IN RIVERS

Sabine Sulzer[1], Wolfgang Kinzelbach[1], Peter Rutschmann[2]

[1] Institute of Hydromechanics and Water Resources Management (IHW), ETH Zurich, sulzer@ihw.baug.ethz.ch, kinzelbach@ihw.baug.ethz.ch

[2] Laboratory of Hydraulics, Hydrology and Glaciology (VAW), ETH Zurich, rutschmann@vaw.baug.ethz.ch

SUMMARY

A new approach to estimate flood discharges in complex river geometries is presented. Discharges are determined through the combination of non-intrusive measurements of surface velocities and water levels with a Navier-Stokes solver and an inverse optimization algorithm. The inverse model is based on the Levenberg-Marquardt minimizing algorithm and can be combined with an arbitrary numerical model. In this work the commercial CFD-program FLUENT is used, allowing for 3D computations of turbulent flows with a free water surface. In order to rule out uncertainties from the numerical model and to strictly quantify the effect of measuring errors, measurements are generated synthetically through forward computations. The methodology is illustrated for a laboratory flume representing a 90°-bend of a natural graded river section at a flood situation with an inflow Froude number of 0.89. For perfect measurements the discharge can in principle be estimated to an accuracy of ≈2%, independently of the number of measurements. Measurements should be taken at locations with high sensitivity to discharge changes in order to arrive at a more reliable discharge estimation. Random error, e.g., due to non-considered turbulent fluctuations in the numerical model, can be minimized by increasing the number of measurement points and choosing appropriate measurement positions.

Keywords: Flow simulation, numerical models, optimization, discharge measurement, gauging stations, free surfaces

1 INTRODUCTION

Flood discharge measurements provide the basis for many hydrological tasks, such as the calculation of the magnitude and return period of a design flood. The discharge often cannot be measured directly and is usually deduced from a stage-discharge curve, derived from gauging station records. This rating curve represents the discharge as a function of the reference water depth, which is often only based on calibration measurements in the low and intermediate flow regime. For large flood events the curve is extrapolated, leading to major uncertainties in the estimation of the corresponding flood discharge and any further hydrologic calculations based on it.

In this research an inverse numerical model was developed to estimate flood discharges using water level and surface velocity measurement data. These can be obtained using Particle Tracking Velocimetry (PTV), an image processing method based on stereo video recordings. PTV is non-intrusive and can therefore be utilised during flood conditions. This measuring technique has already been investigated by Skripalle (1996) and Siedschlag (1998). Kölling (Kölling, 1994; Kölling, Valentin, 1996) has developed a numerical model to estimate the discharge in sewers and open channels based on velocity measurements. This method considers a 2D cross section and is therefore limited to flows with negligible velocity gradients in flow direction. The method also does not use an inverse formulation. Our 3D model allows computations of an arbitrary flow situation. During the inverse modelling cycle the unknown parameters, in this case the discharge, are optimized until a minimal deviation between computed quantities and measured data is obtained. The combination of non-intrusive measurements with the inverse method can lead to a more accurate determination of extreme flood discharges, as a large number of velocity values are used in a regressive way and the influence of local turbulent variations will be damped out.

2 MODEL ASSUMPTIONS

2.1 Numerical Model

3D flow was modelled solving the incompressible and steady-state Reynolds-averaged Navier-Stokes equations for turbulent flow and the continuity equation. Turbulence was taken into account with the k-ε turbulence model (Rodi, 1980). The free surface was computed with the Volume-of-Fluid (VOF) algorithm (Hirt, Nicols, 1981) using a standard second-order finite-difference interpolation scheme near the surface.
The viscous sublayer at the solid boundary was not resolved, having the centroids of the near-wall cells at a distance from the wall where the law-of-the-wall is valid. The normal components of the velocity in those cells were set to zero, whereas the tangential components were calculated with the law-of-the-wall. In the forward simulation mode, the velocity distribution at the inflow boundary was calculated from the discharge Q, assuming a logarithmic profile in height and width,

(1) $$U_{h,b} = Q\frac{ln\left(\frac{Z_i}{\delta_{w,H}}\right)+p_H}{H\cdot ln\left(\frac{H}{\delta_{w,H}}\right)+p_H-1}+\frac{ln\left(\frac{B/2-|Y|_b}{\delta_{w,B}}\right)+p_B}{B/2\cdot ln\left(\frac{B/2}{\delta_{w,B}}\right)+p_B-1}$$

with $U_{h,b}$ the velocity at height Z_h, lateral coordinate Y_b and discharge Q. The water depth is H, the channel width is B, the thickness of the viscous sublayer is δ_w, where the tangential velocity is assumed to be zero, and $p_H = p_B = 3.4$ is a form factor. Uniform values were taken for k and ε□ at the inflow boundary assuming fully-developed turbulent flow.
A locally refined hexahedral grid was generated using the finite volume method for discretization. A second-order upwind scheme and the SIMPLE (Semi-Implicit Method for Pressure-Linked Equations) algorithm for the velocity-pressure coupling in the Navier-Stokes equations was used. The turbulence parameters k and ε were determined iteratively in a decoupled manner. The flow field was computed with the CFD-code FLUENT (2000).

2.2 Inverse Model

The purpose of the inverse model was to optimize the discharge Q, set as the inflow boundary condition, such that measured water surface elevations and surface velocities in a channel section were reproduced best. The inverse modelling technique is widely utilised in geophysics and hydrology, where Yeh (1986) gave a review of typical inverse solution techniques. A common classification divides the inverse solution techniques in direct and indirect methods (Sun, 1994). The direct method requires a dense distribution of observations. Nodal equations have to be rearranged such that the parameters are then considered as unknown variables while the state variables are substituted by their observations. In this work a standard indirect method is used, which solves the inverse problem indirectly through the repeated solution of the forward problem (Carrera, Neuman, 1986a,b,c). The deviations between measured and computed water surface elevations and velocities are minimized, using a least-squares criterion as the objective function according to a maximum likelihood estimation,

(2) $$\zeta(Q) = \sum_{i=1}^{N_h}\frac{[h_i^* - h_i(Q)]^2}{\sigma_i^2} + \sum_{i=1}^{N_{|u|}}\frac{[|U|_i^* - |U(Q)|_i]^2}{\sigma_i^2} = \sum_{i=1}^{N} f_i^2$$

where h_i^* are the measured water levels and $|U|_i^*$ the measured surface velocities. The corresponding calculated values at the measurement locations for a certain discharge Q are $h_i(Q)$ and $|U(Q)|_i = \sqrt{(U_x(Q)^2+U_y(Q)^2+U_z(Q)^2)}$. σ_i are the standard deviations for the measured values, which allow for the weighting of different variables and locations according to their measurement accuracy. The total number of measured values is $N = N_h+N_{|u|}$ with N_h the number of measured water levels and $N_{|u|}$ the number of measured surface velocities. $\zeta(Q)$ is the objective function, which is minimal for the exact discharge.

This nonlinear objective function was minimized by means of the Levenberg-Marquardt algorithm (Fletcher, 1987)

(3) $$(\boldsymbol{J}_k^{\mathrm{T}}\boldsymbol{J}_k + \mu_k \boldsymbol{I})\Delta Q_k = -\boldsymbol{J}_k^{\mathrm{T}}\boldsymbol{f}_k$$

where ΔQ_k is the correction of the discharge Q_k in iteration step k, $\mu_k > 0$ is the Levenberg parameter, and $\boldsymbol{I}$ is the identity matrix. The vector $\boldsymbol{f}_k$ contains the deviations between the measured and the calculated values at the measurement locations, scaled with the respective standard deviations (see equation 2), and $\boldsymbol{J}_k$ is the Jacobian matrix at iteration step k, which contains the partial derivatives of the vector $\boldsymbol{f}_k$ with respect to the parameters to be optimized. The dimension of the Jacobian matrix is NxM with M the number of parameters to be optimized. In this work the Jacobian matrix has the dimensions $Nx1$:

(4) $$\boldsymbol{J} = \begin{bmatrix} \frac{\partial f_1}{\partial Q} \\ \dots \\ \frac{\partial f_N}{\partial Q} \end{bmatrix}$$

The sensitivity coefficients ($|U(Q)|_i' = \frac{\partial |U(Q)|_i}{\partial Q}$, $h_i(Q)' = \frac{\partial h_i(Q)}{\partial Q}$) in the Jacobian matrix can be obtained by several approaches, e.g., the sensitivity equation method, the finite difference approach or the adjoint state method (Sun, 1994). In this work the finite difference approach is used. The sensitivity equation method cannot be applied, because the partial derivatives of the water depth can not be obtained with the sensitivity equation, the free surface being calculated with a separate algorithm, and the adjoint state method has no advantage here, as only one parameter is optimized.
In the finite difference approach, the flow field and the position of the free water surface were determined for the discharges $Q \pm \Delta Q$ and the finite differences were computed by subtracting the surface velocities and water levels at discharges $Q+\Delta Q$ and $Q-\Delta Q$ and dividing the difference by $2\Delta Q$.
One inverse modelling cycle k includes one run of the forward model to determine $\boldsymbol{f}_k$. If $\boldsymbol{f}_k$ is smaller than $\boldsymbol{f}_{k-1}$, two more runs of the forward numerical model with discharges $Q \pm \Delta Q$ are needed to determine the Jacobian matrix. The discharge is then updated with equation (3) and the convergence criteria are checked. When no convergence is achieved and the maximum number of inverse iteration steps is not yet reached a new inverse iteration step is performed.

3 EXAMPLE

Hersberger (2000) examined a 90° bended river section with a naturally graded river bed (average bed roughness $d_{90(avg)}$ = 14.8 mm). Measurements were available for the position of the river bed as well as the water surface at 1404 points in the bended section for a discharge Q = 0.15 m^3s^{-1}, which corresponds to an inflow Froude number of 0.89.

3.1 Numerical model and synthetic data set

The numerical algorithm was validated independently of the inverse modelling part. The computed water levels and velocity magnitude were compared with laboratory measurements for a given discharge (Figure 3-1). The deviations in water depth are largest around the separation zone at 90°. The amplitude is between minus and plus 3 cm. For more than 90% of the river section, the difference between computed and measured water level is less than 20%. The largest relative differences occur at the inner side of the bend, where the water depth is very small. The absolute differences in water depth are more equally distributed over the entire section. The average relative deviation, calculated from the deviations at all measurement locations, is 6.2%.

.

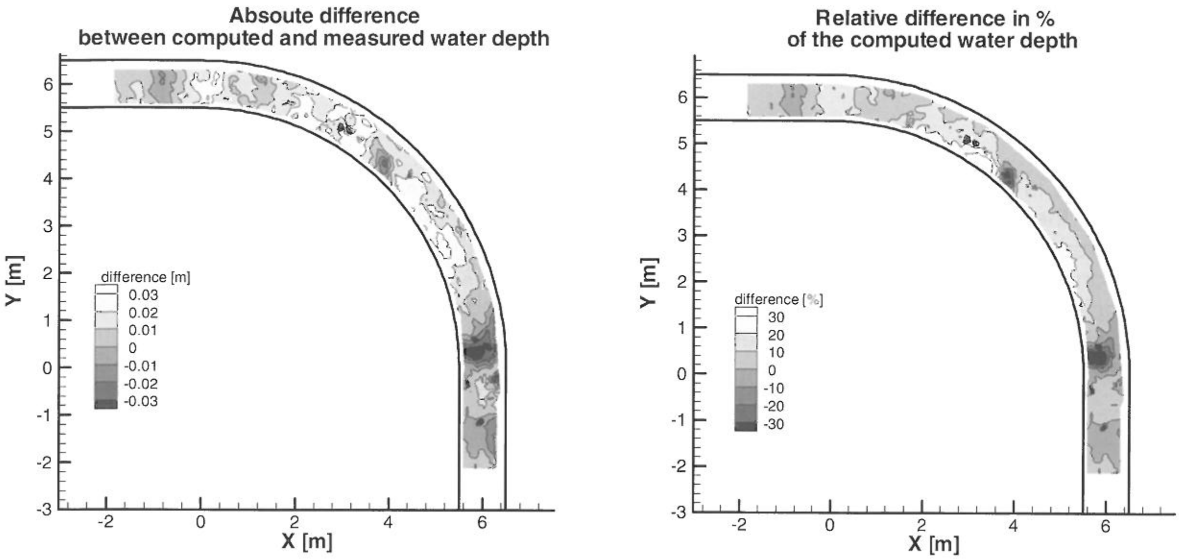

Figure 3-1: Comparison between computations and measurements of the laboratory experiment.

The deviations in velocity magnitude are largest at the inner bend, where the computed velocity magnitude is generally higher than the measured. The relative deviations are less than 5% at the outer bend where the main stream is.

The applicability of the inverse method was investigated by using synthetically generated series of measurements from these forward computations at a given discharge *Q*. The concept of synthetically generated data is the preferred approach at this stage as an identical numerical tool is used for both data generation and inverse optimization. Thus, the problem of identification can be analysed in a consistent fashion separately from the issue how well the model is calibrated to represent the real situation. Measurements were obtained by sampling the computational results at a number of locations, resulting in a series of surface velocities and water level data.

The surface velocity and the water level can have errors because of measurement uncertainty or different assumptions made in measurement and numerical model. Measurement errors can occur through the measurement set-up itself. The accuracy of the PTV-measuring technique is primarily a function of the pixel size in the image as well as the particle size on the water surface. A more significant error source can occur in turbulent flows because of the turbulent fluctuations of water surface and local velocities. PTV-measurements give an instantaneous image, whereas the numerical model is based on the time-averaged Reynolds equations, where the turbulent fluctuations are ruled out. The amplitude of the fluctuation can be calculated in a conservative way by considering the turbulent kinetic energy k and neglecting the dissipation rate ε,

(5) $$v' \cong \pm\sqrt{k}, \qquad h' \cong \frac{v'^2}{g} \cong \pm\frac{k}{g},$$

with v' the fluctuation of the velocity and h' the fluctuation of the water depth. This is a very simplistic approach, but it is sufficient to roughly estimate the maximal expected amplitudes, which are about 7 to 16% of the average surface velocity and about 1 to 3% of the inflow water depth for this example. This estimation was made with the turbulent kinetic energy k at the water surface obtained from the numerical simulation, which is between 0.005 and 0.025 m^2s^{-2}.

3.2 Discharge estimation

Besides the discharge itself, the stability and uniqueness of the discharge estimation is of interest, which depends on the number and position of measurements, measurement error and the objective function. Other sources of uncertainty such as bed roughness, physical model and numerical discretization were indirectly taken into account by verifying the forward computations with measurements, and so were not directly considered in this section. But in the final evaluation of the discharge estimation introduced here, all uncertainties have to be taken into account. This section only tests whether the inverse modelling technique chosen as well as the measurement set up is suitable for the discharge estimation.

If the unperturbed synthetic measurements were considered, the discharge could be estimated with a high accuracy of less than 2.1% deviation from the expected discharge. More accurate results were obtained with more measurements.

Stability and uniqueness is guaranteed for convex objective functions, which steeply descend to the only minimum. Figure 3-2 shows the objective function for the example chosen here with 5 and 25 measurements respectively. Both objective functions show only one minimum near the expected discharge. The minimum obtained with 25 measurements agrees better with the expected minimum, probably because the flow field is represented better with more measurements and outliers are less dominant.

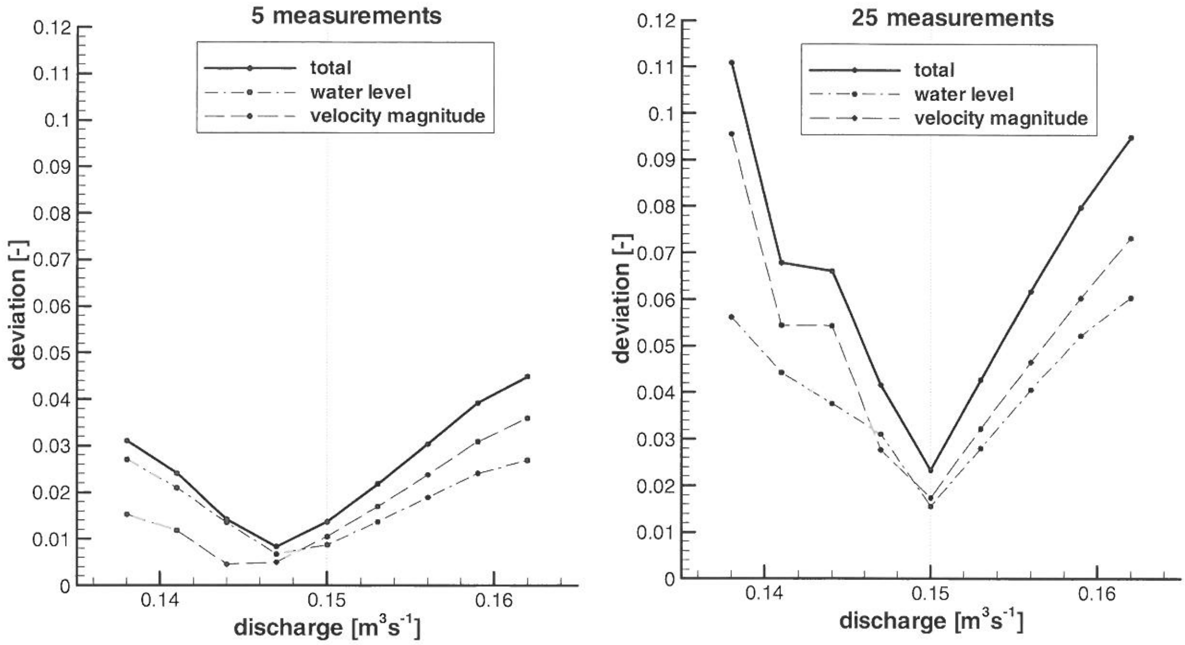

Figure 3-2: Objective function for 5 and 25 measurements with the contributions of surface velocities and water levels.

The sensitivity of the surface velocity and water level field to discharge changes, given by the Jacobian matrix, was examined for the whole river section to study the suitability of each variable for estimating the discharge in relation to its position. Figure 3-3 displays the sensitivities of the water level (left) and the surface velocities (right) for a discharge $Q = 0.15\ m^3s^{-1}$ in the bent section, while Figure 3-4 corresponds to a discharge $Q = 0.17\ m^3s^{-1}$.

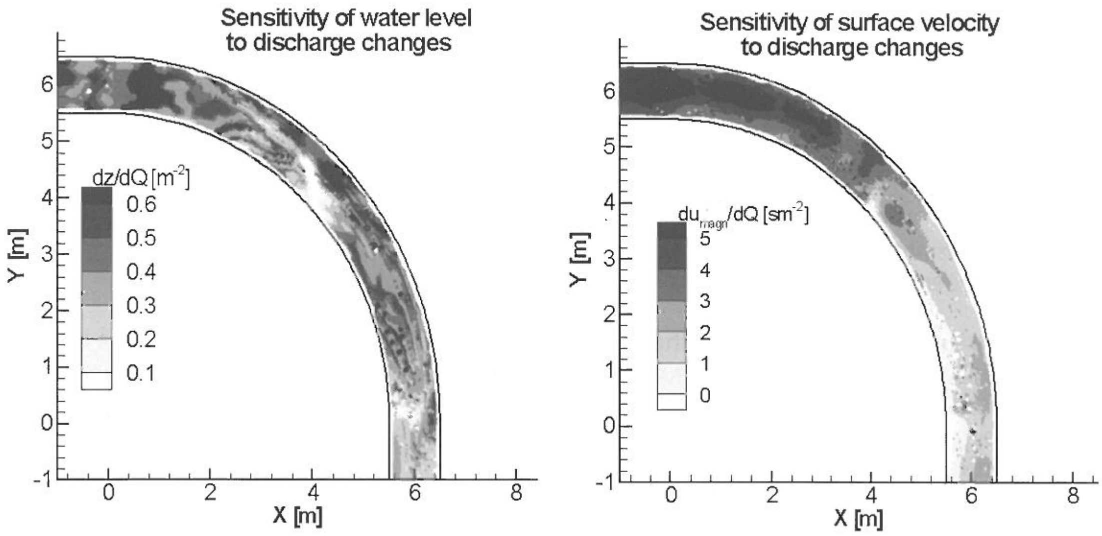

Figure 3-3: Sensitivity of water level and velocity magnitude to discharge changes for a mean discharge $Q = 0.15\ m^3s^{-1}$.

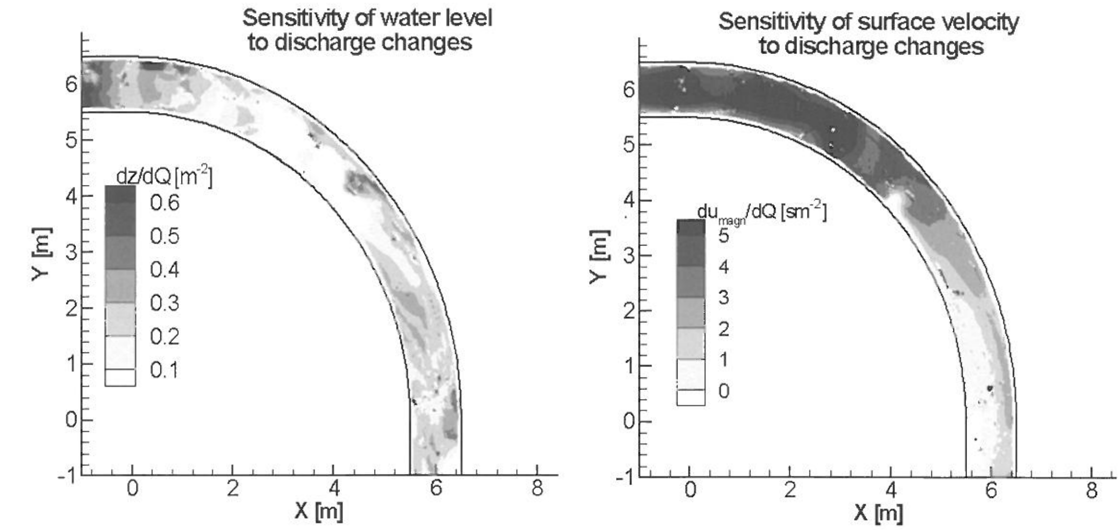

Figure 3-4: Sensitivity of water level and velocity magnitude to discharge changes for a mean discharge $Q = 0.17\ m^3s^{-1}$.

The sensitivity of the water level to discharge changes decreased with increasing discharge. This can be clearly seen on the left of the two Figures 3-3 and 3-4. The sensitivity of the surface velocity to discharge changes increased for an increasing discharge at the outside of the bend, whereas it decreased at the inside of the bend. This decreasing sensitivity indicates a stronger developed separation zone at the inside. Generally, a reduced sensitivity was determined in the separation zones on the inside of the bend for both variables.
The influence of the number of perturbed measurements on the estimated discharge was examined for the three different cases using 5, 10 and 25 measurements. Ten data sets with the same number of measurements were generated adding different white noise components of ±10% to the correct surface velocity. Figure 3-5 shows the results including a surface velocity error.

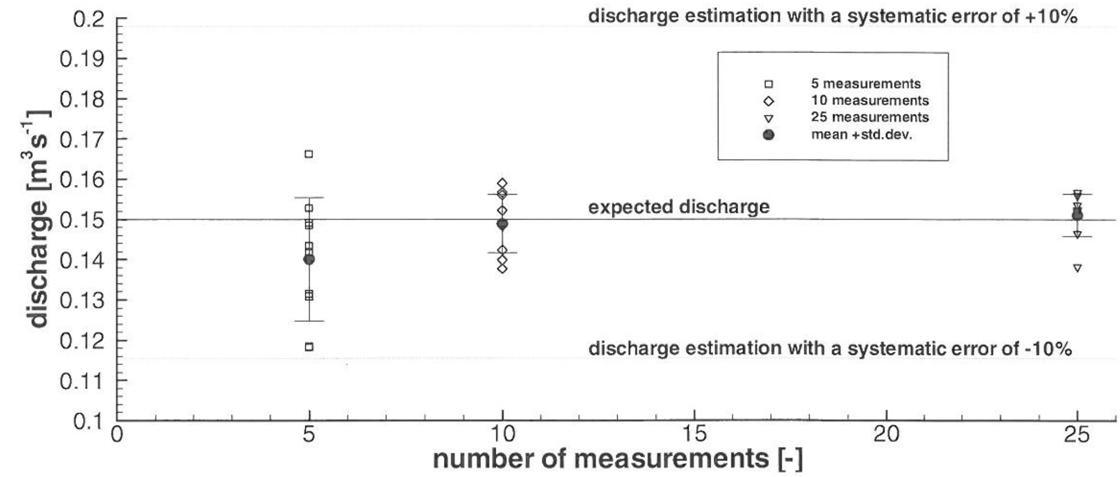

Figure 3-5: Estimated discharges with a white noise component of ±10% at the velocity measurements. Data sets with 5, 10 and 25 measurements.

The calculated standard deviations of the estimated discharges as well as the intervals between the largest and the smallest estimated discharge are largest for the case with 5 measurements, and become smaller with the number of measurements added. The deviations of all estimated discharges from the expected discharge are overall significantly smaller than the deviations obtained for data with a systematic error.
The influence of the location of the perturbed measurements on the estimated discharge was examined using only one measurement point for the optimization. Two different data cases were analyzed using either a measurement halfway through the bend, at 45° where the sensitivity of the

velocity on discharge changes is high ($\frac{\partial |U(Q)|_i}{\partial Q} \approx 3.7$) or one at the outside of the bend at 90° with a smaller sensitivity ($\frac{\partial |U(Q)|_i}{\partial Q} \approx 2.7$, see marks at right side of Figure 3-3). A surface velocity error of 10% for the more sensitive measurement (45°) led to a discharge error of about 15%. However, a surface velocity error at the less sensitive measurement had a much larger influence on the estimated discharge. The relative error of the estimated discharge was about 25% with a 10% error of the measured surface velocity.

3.3 Accuracy of the Discharge

The accuracy of the estimated discharge depends on the accuracy of the measurements, the assumptions in the CFD-model and the inverse formulation. Measurement errors and errors resulting from assumptions in the CFD-modelling lead to water level and surface velocity errors in the objective function (see equation 2). A measurement error leads to deviations in the measured quantities h_i^*, $|U|_i^*$ and a model error results in deviations of $h_i(Q)$, $|U(Q)|_i$. Both effects may shift the minimum of the objective function on the parameter axis leading to a discharge error. This discharge error is a combination of the two effects, which are discussed separately in the following.
The characteristics of a measurement error is mainly random through the resolution of the image by the CCD-sensor and the size, resp. shape of the particles. This allows to minimize the influence of a measurement error on the estimated discharge by increasing the number of measurements. The presented case-study showed that a randomly distributed 10%-error in surface velocity resulted in a discharge error of up to 20% when considering 5 measurements but was no larger than 8% when considering 10 measurements. In river sections, a measurement error of 5% for surface velocities is realistic (Siedschlag, 2001), resulting in a discharge error of 4% when considering 10 measurements.
An error in the computed quantities can lead to a significant discharge error, because these errors are mainly systematic through imprecise model description. The velocities could be computed with deviations less than 5% in the main stream. A relative velocity error of 5% would lead to a relative discharge error of 7.5% with an error propagation factor of 1.5 as was evaluated for optimal measurement positions.
The inverse formulation can be responsible for a part of the discharge error too, because it may not describe and/or find the exact minimum of the objective function. The results with correct synthetic measurements showed that the estimated discharge was within 2.1% of the exact discharge. The error associated with the inverse procedure is definitely smaller than that originating from the other two sources of error.
The accuracy of the discharge is affected by the 3 error sources discussed above. Assuming a discharge error of 4% from PTV-measurements, 7.5% from CFD-modelling, and 2% from the inverse problem, the expected error is the geometric sum and amounts to 9%.

4 CONCLUSIONS

The 3D Navier-Stokes inverse model is a powerful tool for identifying the discharge and its reliability of the prediction from surface measurements. Water level and surface velocity data can be obtained through non-intrusive methods (e.g., Particle Tracking Velocimetry, PTV), which can be used even for extreme floods. Surface velocity data are important for the estimation of flood discharge, because the suitability of water level data drops for larger discharges due to the decreasing sensitivity of water levels to discharge changes. The 3D model used can be applied to complex river bed geometries with a naturally eroded river bed and also for flow fields with separation zones.
Few measurements are needed to estimate the discharge. One measured surface velocity is enough to predict the discharge with a high accuracy (i.e., within 2.1%), when no measurement error is assumed and a model error through uncertainties in geometry, bed roughness or empirical assumptions, as well as through the discretization is neglected.
Considering a surface velocity error, an increasing number of measurements can decrease the error in the estimated discharge, if the velocity error is random. More reliable discharge estimates were obtained with an increasing number of measurements for a small number of measurements, but the improvement of the reliability is small for more than 10 measurements in the present case. Consequently, a larger number of measurements can help to minimize the influence of, e.g., outliers and turbulent fluctuations. The number of measurements showed, however, no effect on the estimated discharge, when a systematic error was considered.

The influence of an error in surface velocity measurements on discharge estimates becomes larger for measurements with smaller sensitivities to discharge changes. An a priori estimation of these sensitivities for an expected discharge range, based on computations only, will therefore allow to optimize the selection of locations for velocity measurements.
The error of the estimated discharge was calculated to 9%, but a larger deviation must be expected in natural rivers. While the PTV-measurement error was assumed under consideration of prototype dimensions, the error resulting from CFD-modelling may be larger due to uncertainties in river bed geometry and roughness. An evaluation of the proposed method in comparison to the conventional discharge estimation is not possible for flood events, because no generally valid estimation of accuracy is available for conventionally predicted discharges. However, the proposed method still seems attractive even if a discharge error of up to 20% may be expected.

ACKNOWLEDGEMENTS

This work has been supported financially by the Swiss National Science Foundation Grant SNF 2100-052559.97/1.

REFERENCES

Carrera, J., Neuman, S.P. (1986a): Estimation of aquifer parameters under transient and steady state conditions: 1.Maximum Likelihood Method incorporating prior information. J. Water Res. Research, AGU, 22(2): 199-210.

Carrera, J., Neuman, S.P. (1986b): Estimation of aquifer parameters under transient and steady state conditions: 2. Uniqueness, stability, and solution algorithms. J. Water Res. Research, AGU, 22(2): 211-227.

Carrera, J., Neuman, S.P. (1986c): Estimation of aquifer parameters under transient and steady state conditions: 3. Application to synthetic and field data. J. Water Res. Research, AGU, 22(2): 228-242.

Fletcher, R. (1987): Practical methods of optimization. John Wiley & Sons, Second Edition, London.

FLUENT Inc. (2000): Version 5.5. http://www.fluent.com. Lebanon, New Hampshire USA.

Hersberger D. (2000): Experimental data. Laboratory of Hydraulic Constructions, EPF Lausanne (personal communication).

Hirt, C.W., Nicols, B.D. (1981): Volume of Fluid (VOF) for dynamics od free boundaries. J. Comput. Phys., 39, 201-221.

Kölling, Ch. (1994): Finite-Element-Simulation der Geschwindigkeitsverteilung in Kanälen und teilgefüllten Rohren. Mitteilung 60, Institut für Hydraulik und Gewässerkunde, TU München, pp. 267 (in German).

Kölling, Ch., Valentin, F. (1995): SIMK-Durchflussmessungen. Wasserwirtschaft 85 (1995), 10, 494-499 (in German).

Rodi, W. (1980): Turbulence models and their applications in hydraulics. State-of-the-art- paper, IAHR, 1980.

Schlichting, H. (1965): Turbulente Rohrströmung. Grenzschicht-Theorie, Verlag G. Braun, Karlsruhe (in German).

Siedschlag, S. (1998): Durchflussermittlung mit Hilfe digitaler Bildverarbeitung und SIMK- Kalibration. Wasserwirtschaft 88 (1998), 6, 278-282 (in German).

Siedschlag, S. (2001): Private communication. Quantum Hydrometrie, Berlin.

Skripalle, J. (1996): Messung der Oberflächengeschwindigkeit mit Hilfe digitaler Bildverarbeitung. Landeshydrologie und -geologie Bern, Hydrologische Mitteilungen Nr.24. (in German).

Sulzer, S. (2001): Flood Discharge Estimation for Complex River Geometries by Inverse Numerical Modelling, Dissertation, Swiss Federal Institute of Technology Zurich.

Sulzer, S., Rutschmann, P., Kinzelbach, W. (2002): Flood Discharge Prediction Using Two-Dimensional Inverse Modeling. Journal of Hydraulic Engineering, ASCE, Vol. 128/1:pp 46-54.

Sun, N.-Z. (1994): Inverse Problems in Groundwater Modeling. Kluwer Academic Publishers, Netherlands.

Yeh, W.W.-G. (1986): Review of parameter identification procedures in groundwater hydrology: The inverse problem. J. Water Res. Research, AGU, 22(2):95-108.