# Flood routing of regulated flows in Medjerda River, Tunisia

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## ABSTRACT

Released flows from the Sidi Salem Dam Reservoir on the Medjerda River (Northern Tunisia) were routed downstream along the river lower water course using both hydrologic and hydraulic flood routing techniques. The hydrologic flood routing method used is that of Muskingum while the hydraulic flood routing procedure used a numerical model RUFICC (Routing Unsteady Flows In Compound Channels). The model is based on the complete numerical solution of St. Venant equations using a four-point implicit finite difference scheme. Compared to observed hydrographs at downstream sections, a better agreement was achieved using the hydraulic flood routing technique. Statistical parameters and scattergrams were used to test and confirm this agreement.

**Key words** | flood routing, hydraulic and hydrologic methods, Muskingum method, St. Venant equations (RUFICC)

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## NOTATION

The following symbols are used in this paper:

| | |
|---|---|
| $A$ | cross-sectional area of flow |
| $C_i$ | coefficient of the routing equation for the Muskingum method |
| $I$ | inflow rate |
| $K$ | time of travel of the flood waves |
| $O$ | outflow rate |
| $Q$ | flow discharge rate |
| $Rso$ | correlation coefficient |
| $S$ | storage |
| $S_0$ | channel bed slope |
| $S_f$ | friction slope |
| $SEE$ | standard error of estimates |
| $SD$ | standard deviation of the residuals |
| $x$ | distance along the channel |
| $y$ | flow depth |
| $\Delta t$ | time step |
| $\Delta x$ | incremental distance along the channel |

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## INTRODUCTION

Unsteady open channel flow modeling is important in flood routing and prediction, stream flow modeling, river regulation and in the analysis of estuarine flows. Flood routing is the activity of mathematically modeling the progress of a flood wave (or hydrograph) while it moves downstream. It is an integral component in any hydrologic model and is the most important activity in predicting flood stages and discharges as functions of time and space along a river reach. Flood routing is employed in practice for the solution of a wide variety of problems associated with water use. Some of these include:

- predicting flood hydrographs for given or assumed initial conditions;
- determining hydrographs modified by reservoir storage;
- evaluating past floods for which records are incomplete;
- studying the effects of water resources development on the downstream flow conditions.

Flood routing is used in predicting the characteristics of a flood wave and their change with time in the direction of flow. These characteristics include:

- maximum water surface elevation and its rate of rise or fall (considered to be an important factor in the planning and design of structures across or along streams and rivers),
- peak discharge, which is required in the design of spillways, culverts, bridges and channels sections, and
- total volume of water resulting from a design flood to assist in the design of storage facilities for flood control, irrigation and water supply.

In this context, released regulated flows out of the Sidi Salem Dam Reservoir on the Medjerda River (Tunisia) are routed downstream using an integrated water management tool to (i) examine the impact of hydraulic structures such as bridges, dams and pumping stations on flood hydrographs, (ii) calibrate and control gauging stations and (iii) estimate sediment transport and the morphology evolution of the Medjerda river bed downstream of Sidi Salem. To design, plan and run these systems, the conventional hydrologic routing method is generally adopted.

This study, however, considers both hydrologic and hydraulic routing methods and compares their respective simulated hydrographs to observed data using statistical test procedures.

## THEORY AND MODELING APPROACH

### Hydrologic routing method

The "Muskingum" flood routing method is considered among the traditional hydrologic approaches the most extensively used and known in the studies of flood routing in rivers and channels. This method is based on the following continuity equation:

$$\frac{dS}{dt} = I - O \tag{1}$$

where $S$ is the storage in a river or control volume, while $I$ and $O$ are the inflow and the outflow rates, respectively.

The volume of storage at a time step $j$ is related to the inflow and outflow as follows:

$$S_j = K[xI_j + (1-x)O_j] \tag{2}$$

where $K$ is a proportionality coefficient representing the wave travel time and $x$ is a weighting factor varying between

0 and 0.5 (Chow *et al.* 1988). $x$ is assumed to be 0.2, which is representative of natural river systems while $K$ is obtained through the following equation:

$$K = \frac{0.5[(I_{j+1} + I_j) - (O_{j+1} + O_j)]}{x(I_{j+1} - I_j) + (1-x)(O_{j+1} - O_j)} \tag{3}$$

According to this equation, $K$ is equal to the slope of the line resulting from the graph of computed values of the numerator and denominator which are plotted for each travel time.

### Hydraulic routing method

Unlike the hydrologic routing method, which is based on the solution of the continuity of mass equation alone, the hydraulic routing approach is based on both continuity and momentum equations. The numerical model used for this exercise is RUFICC (Routing Unsteady Flows In Compound Channels), which is a one-dimensional model for routing floods in channels of composite sections (Abida 1992). RUFICC is based on a modified version of the St. Venant equations. It accounts for flood plain contributions to system conveyance and also for the lateral momentum transfer between adjacent deep and shallow zones of compound flow fields. The model was evaluated and validated through extensive use of large scale laboratory flow data corresponding to both "in-bank" and "over-bank" flow conditions (Abida & Townsend 1994). The model was also applied to an experimental reach of the River Main in Northern Ireland and yielded reasonably accurate simulation results (Abida 1992). For this particular application, however, the released hydrographs out of the Sidi Salem Dam Reservoir all correspond to "in-bank" flow conditions, reducing thereby the model equations to the conventional St. Venant equations given as follows:

The continuity equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{4}$$

The momentum equation:

$$\frac{1}{A}\frac{\partial Q}{\partial t} + \frac{1}{A}\frac{\partial (Q^2/A)}{\partial x} + g\frac{\partial y}{\partial x} = g(S_0 - S_f) \tag{5}$$

where $Q$ is the flow rate, $A$ is the cross-sectional area, $y$ is the water depth, $x$ is the distance along the channel, $S_0$ is the

channel bed slope, $S_f$ is the friction slope and $g$ is the acceleration due to the gravity.

These two unsteady flow equations represent a system of nonlinear hyperbolic partial differential equations, for which analytical solutions can only be obtained under certain linearizing assumptions in the case of simple channel geometries and boundary conditions. However, the objective of this study is to route floods in a natural channel with irregular geometry and varying boundary conditions, justifying therefore the use of numerical solutions.

A four-point implicit finite difference (FD) scheme (Amein & Fang 1970) was used for the numerical solution to overcome the limitations imposed on the size of the time step required for the numerical stability of explicit schemes. The adopted FD scheme is also well suited to applications involving natural channels, since it can handle varying channel geometry, even where changes from section to section and in the bottom slope are significant (Amein & Fang 1970).

Equations (4) and (5), written for all nodes of the continuous time–space region of the FD scheme, provide a system of nonlinear equations. The Newton–Raphson iteration method is first used to reduce the nonlinear system to successive linear equations. The method provides a means of correction of the supposed values with a minimization of the error corresponding to the degree of precision wanted in a completed number of iterations. The resulting linear equations are then solved using the double-sweep solution method.

The upstream and downstream boundary conditions are one of the following three possibilities:

- discharge hydrograph $Q = Q(t)$,
- stage or flow depth hydrograph $y = y(t)$,
- rating curve $Q = f(y)$.

In this study the upstream boundary condition consists of discharge $Q$, expressed as a function of time, and the downstream boundary condition is a rating curve.

## STUDY AREA AND DATA USED

The objective of this study is to route the released flows downstream of Sidi Salem Dam along the lower water course of the Medjerda River. The Medjerda River, located in Northern Tunisia, has its source in Algeria and discharges to the Mediterranean Sea. It is characterized by a reach length of 484 km and a watershed surface of $23,700\,\text{km}^2$, of which 32% is located in Algeria (Hbaeib 1992). The basin is characterized by a weak vegetative cover (30%) and irregular rainfall with a mean of 550 mm (Figure 1).

The data used in this study, which were provided by the Tunisian Ministry of Agriculture, Environment and Water Resources, consist of:

- rating curves at four stations: Slouguia, Medjez El Bab, El Herri and Borj Toumi,
- observed discharge and stage hydrographs,
- transverse cross sections and slopes for the different reaches.

Discharge and stage hydrographs correspond to the released flows for the years 1993, 1996 and 1997.

The river reach considered for the event RF0193 (Released Flows of Sidi Salem Dam of January 1993) is between the "Slouguia" and "Medjez El Bab" stations. Its total length is 19 km and its bed slope is 0.05%. Initially, the flow was uniform with a flow depth of 3.78 m. The average Manning's roughness coefficient representative of the entire reach, estimated on the basis of the steady uniform flow condition, was 0.04.

The data set RF0296 corresponds to regulated flows of February 1996 between the "Slouguia" and "Borj Toumi" stations. This reach is composed of three sub-reaches with lengths of 19, 21.6 and 12.2 km, respectively. The average bed slope was 0.05%, the initial depth was 2.87 m and the average Manning roughness coefficient was 0.04.

Finally the data set RF1297 consist of regulated flows of December 1997 between the "Slouguia" and "Medjez El Bab" stations.

## SIMULATIONS AND RESULTS

### Model calibration

Error generation is inherent in the mathematical and numerical description of physical systems and processes.
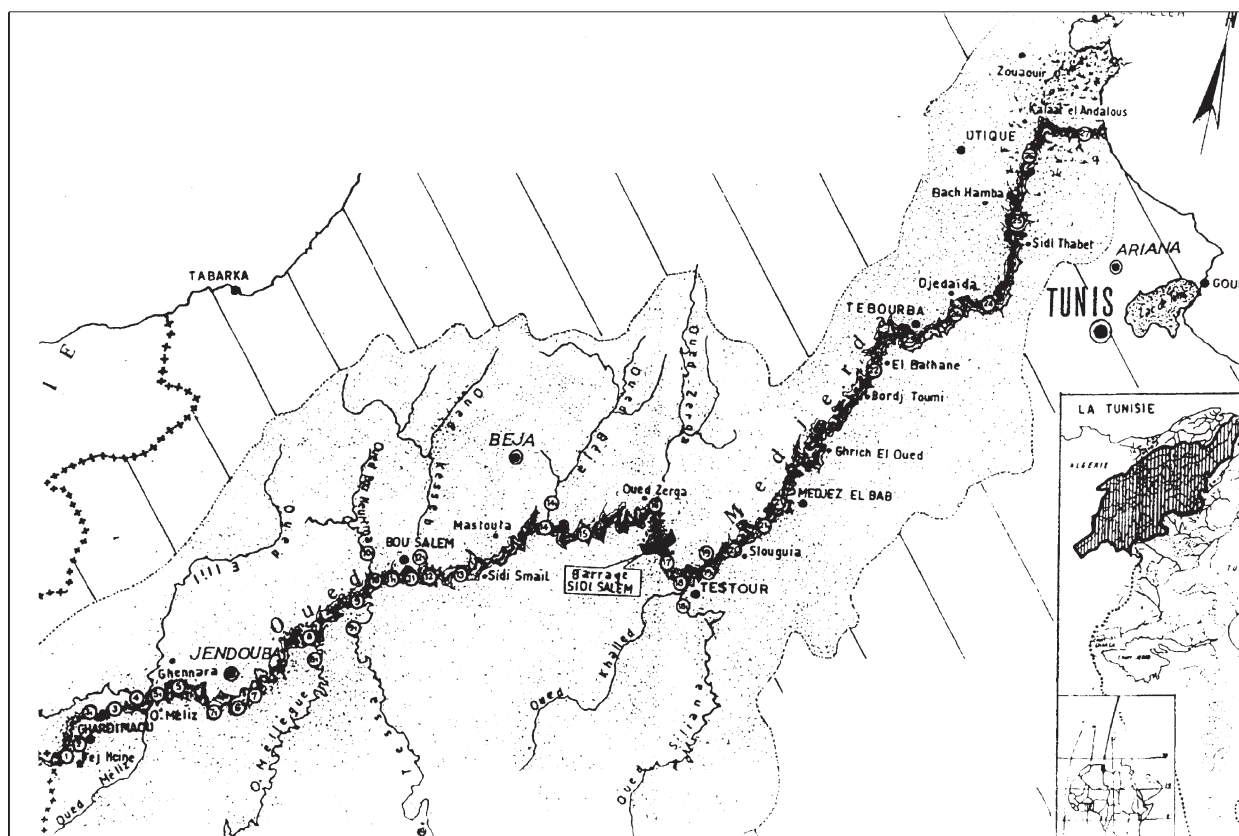
**Figure 1** | The Medjerda River.

In unsteady flow modeling, the major source of error is the fact that some of the parameters embedded in the model equations cannot be measured directly. Examples of such parameters are the Manning coefficient, the time step and the space variation. While channel properties such as bed slope, reach length and cross-sectional geometry can be directly measured, the other conceptual parameters mentioned earlier need to be estimated and then adjusted in a way to minimize errors between model output and the corresponding observed values of stage and discharge. This requires that the model be calibrated prior to its application.

Fread & Smith (1978) showed that the value of Manning's $n$ depends not only on discharge and flow depth but also on the particular schematization used to describe the continuous channel geometry by a series of discrete representations along the reach of channel being modeled. This leads to the conclusion that $n$ is best evaluated through calibration of the unsteady flow model, especially if reasonably accurate field data from past flood

events are available. The first discharge hydrograph of the release of January 1993 was used in the calibration process. The value of Manning's $n$ that resulted in the closest agreement between observed and simulated discharges was determined by trial and error to be 0.040.

Since the numerical solution is highly dependent upon the choice of the space increment $\Delta x$ and the time step $\Delta t$, these two variables have to be selected in a way to achieve its convergence and stability. Wormleaton & Karmegam (1984) showed that the following criterion between the time step and the space increments should be satisfied to minimize the finite difference error:

$$\frac{\Delta x}{\Delta t} \geq V_w$$

where $V_w$ is the flood velocity.

To test the validity of this latter criterion, 18 numerical experiments with different mesh sizes were performed for the first flood event (RF0193). Ratios of simulated to observed discharges and stages are displayed in Table 1.

**Table 1** | Numerical tests using different time steps and space increments (flood event RF0193)

| Test | $\Delta t$ (s) | $\Delta x$ (m) | $\frac{\Delta x}{\Delta t}$ | $\frac{Q_s}{Q_0}$ | $\frac{H_s}{H_0}$ |
|------|------|------|------|------|------|
| 1 | 720 | 50 | 0.07 | 1.034 | 1.007 |
| 2 | | 500 | 0.69 | 1.057 | 1.02 |
| 3 | | 750 | 1.04 | 0.6 | 0.85 |
| 4 | | 792 | 1.1 | ND | ND |
| 5 | | 1000 | 1.4 | ND | ND |
| 6 | 1440 | 50 | 0.035 | 1.019 | 1.007 |
| 7 | | 1191 | 0.83 | 1.005 | 1.007 |
| 8 | | 1584 | 1.1 | 1.004 | 1.007 |
| 9 | | 1650 | 1.145 | 1.008 | 1.007 |
| 10 | | 1680 | 1.167 | ND | ND |
| 11 | 2160 | 2376 | 1.1 | 0.98 | 0.99 |
| 12 | | 2808 | 1.3 | 1.01 | 0.99 |
| 13 | | 3024 | 1.4 | ND | ND |
| 14 | 3600 | 200 | 0.056 | 1.04 | 1.02 |
| 15 | | 500 | 0.14 | 1.036 | 1.007 |
| 16 | | 1000 | 0.28 | 1.032 | 1.007 |
| 17 | | 3960 | 1.1 | 1.019 | 1.007 |
| 18 | | 4464 | 1.24 | ND | ND |

ND: Numerical instability, program determination occurred.

Comparing the flood peak velocity determined to be 1.03 to $\Delta x/\Delta t$ ratios shown in Table 1, it can be seen that the error in peak discharge is maximum for runs 1–3, 6–7 and 13–15, which do not satisfy the criterion suggested by Wormleaton & Karmegam (1984). For the other numerical experiments (4, 5, 9, 12 and 18), even though the criterion is satisfied, instability problems caused the termination of the simulation and no solution was obtained. Therefore it can be concluded that the criterion suggested by Wormleaton and Karmegam yields accurate solutions but does not guarantee reaching one.

## Simulation and discussion

The quality of a model is measured by the results of its simulation, validated by a comparison with observed data. In fact, the output of an unsteady flow model that consists of stage and discharge hydrographs needs to be compared to observed data to check the model's performance. This validation can range from being completely subjective, by relying strongly on visual impressions of the correspondence between the observed and simulated hydrographs, to a detailed statistical analysis that tests the agreement between the respective time series.

Results of the hydraulic routing method (numerical solution of St. Venant equations) were compared with those obtained using the more conventional hydrologic routing method of Muskingum and the provided rating curves. Figures 2–5 show the results of unsteady flow simulation.

Figure 2 shows discharge and stage hydrographs observed at "Medjez El Bab" station as well as the corresponding simulated hydrographs using both the numerical solution of the St. Venant equations and the Muskingum method. It is important to note at this stage that simulated stage hydrographs by the Muskingum method were obtained from rating curves as this method provides only discharge hydrographs. St. Venant simulated hydrographs clearly show a better agreement to observed data than those obtained by the Muskingum method (Figure 2). Peak values correspond to relative errors of 8.1% and 20%, respectively.

Figure 3 presents hydrographs corresponding to released flows of February 1996, where three sub-reaches were considered in the simulation process. The figure shows two successive hydrographs separated by a period of low flows extending over a relatively long period (16–28 February).
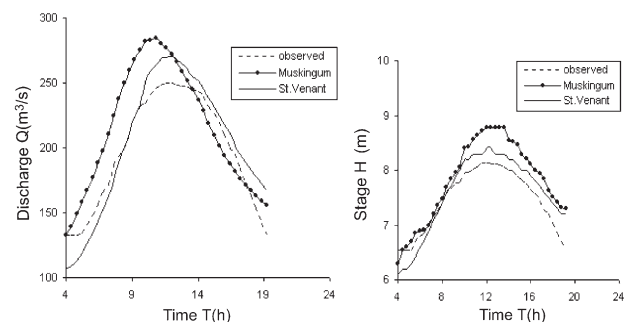


**Figure 2** | Observed and simulated discharge and stage hydrographs (RF0193) at Medjez El Bab station.
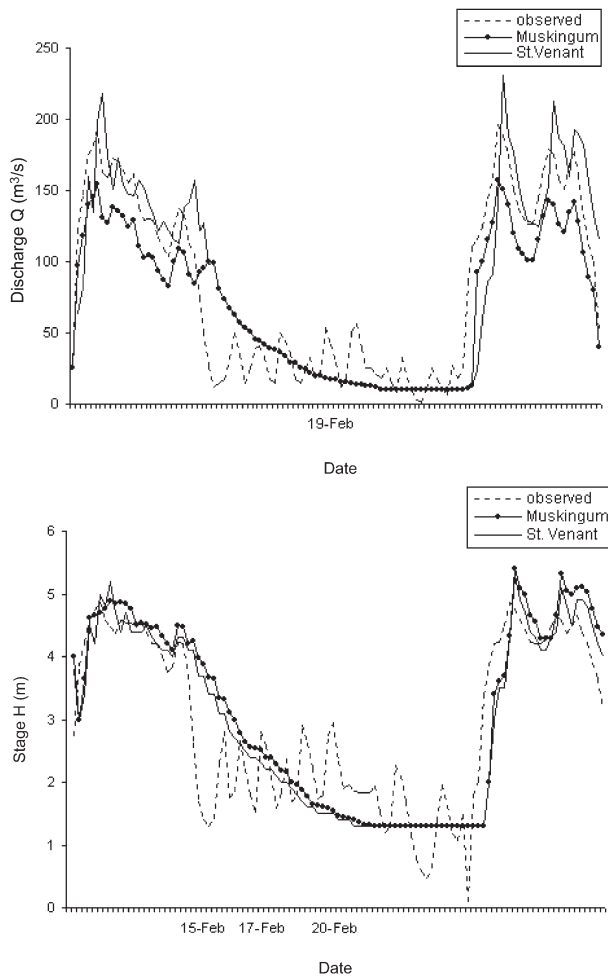
**Figure 3** | Observed and simulated discharge and stage hydrographs (RF0296) at Borj Toumi station.



**Figure 4** | Observed and simulated discharge and stage hydrographs (RF0296) at Medjez El Bab station.

While stage hydrographs simulated by both models are comparable, St. Venant simulated discharge hydrographs clearly show a better agreement to observed data for both consecutive events (Figure 3). The period of low flows was not properly simulated by either model as fluctuations in the observed data were not captured in the simulation. This might be explained by the backwater effects of Laroussia Dam, located downstream of the study reach, believed to be responsible for low flows between the two consecutive released hydrographs. Furthermore, the downstream reach is characterized by important sediment transport loads, which might affect stage measurements (Ghorbel 1996).

Simulations were repeated for this particular event and only the first sub-reach was considered. The corresponding
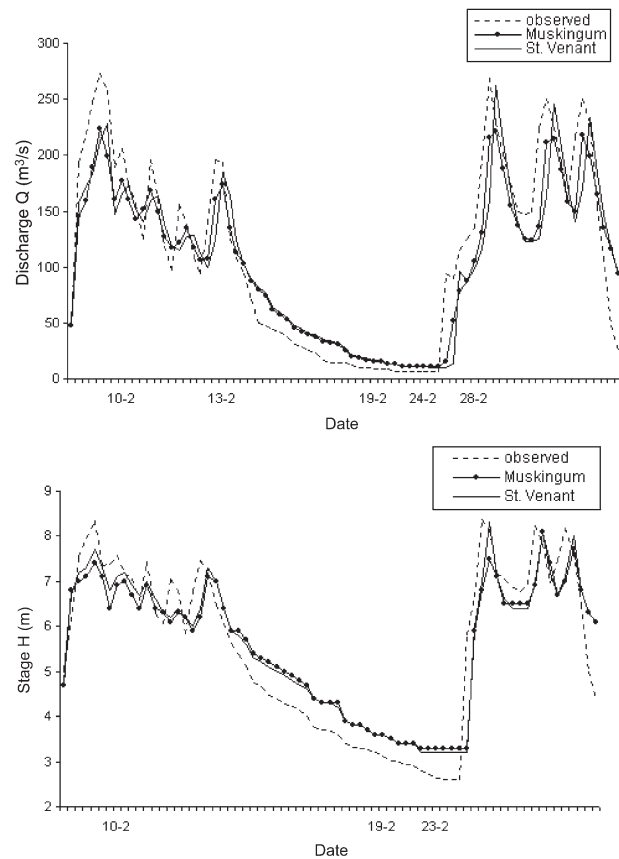
results are presented in Figure 4, which shows a reasonably good agreement between simulated and observed stages and discharge hydrographs, especially for the hydraulic routing method.

Figure 5 shows the results of the unsteady flow simulation for the event of 12 February 1997. For this
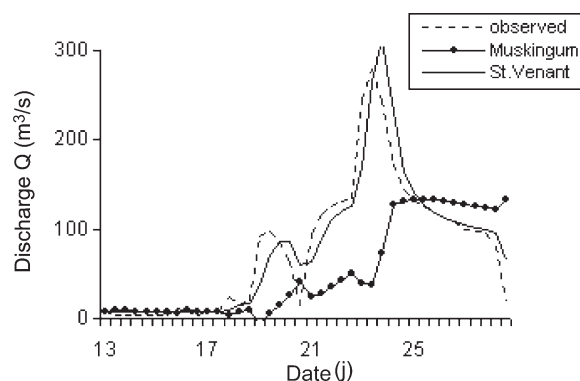


**Figure 5** | Observed and simulated discharge hydrographs (RF1297) at Medjez El Bab station.

**Table 2** │ Statistical results

| | RF0193 | | |
| Event | SEE | SD | Rso |
| --- | --- | --- | --- |
| St. Venant | 0.038 | 0.1 | 0.9984 |
| Muskingum | 0.066 | 1.47 | 0.799 |

particular event, the Muskingum method did not provide any results due to instability problems. On the other hand, the hydraulic method overestimated peak discharge with a relative error of approximately 12.4%.

Instead of relying exclusively on visual impressions of the correspondence between observed and simulated series, statistical measures were considered to test the agreement between measured and computed stage and discharge hydrographs. The statistics used, which were also adopted by Kolovopoulos (1990) among others, are: (i) standard error of estimate (SEE), (ii) correlation coefficient (Rso) and (iii) the standard deviation of relative errors between the residuals SD (observed and simulated values).

Scattergrams of observed data plotted against simulated values (stages or discharges) and those of their associated errors were used to test both the performance of the simulation models and the agreement between the observed and simulated time series.

The regulated flood event of January 1993 was selected as the sample application. Values obtained are presented in Table 2. According to this table, St. Venant model values of most of the coefficients were very close to unity and the correlation is excellent, which shows again the reasonably good agreement between simulated and observed data.

The scattergrams of the observed and simulated stages and discharges (Figures 6 and 7) indicate that the model reasonably simulated both high and low flows. Residuals in the simulated stage and discharge are also observed to be fairly low.

Compared to the conventional hydrologic routing method, better results were generally obtained using the hydraulic routing method and the complete numerical solution of the St. Venant equations. While the former is straightforward and easy to use, it does not describe closely the physical system. It just uses discharge inflow and outflow data to yield two conceptual parameters ($K$ and $x$) obtained through calibration. On the other hand, the hydraulic routing model RUFICC uses real measurable data of the study reach length, slope and cross-section geometry as well as initial and boundary flow conditions. The only parameter obtained through calibration is Manning's roughness coefficient, which is a non-physically measurable parameter. It depends not only on discharge and flow depth but also on the schematization used for channel modeling (Fread & Smith 1978).

## CONCLUSION

The proposed model for routing unsteady flows in natural channels (RUFICC), which is based on the numerical solution of the St. Venant equations, yielded good estimates of stage and discharge for a wide range of flows, with a relative error of 12%. Compared to the conventional hydrologic method of Muskingum, visual impressions, statistical and graphical tests all showed a better agreement between RUFICC simulated results and observed data.
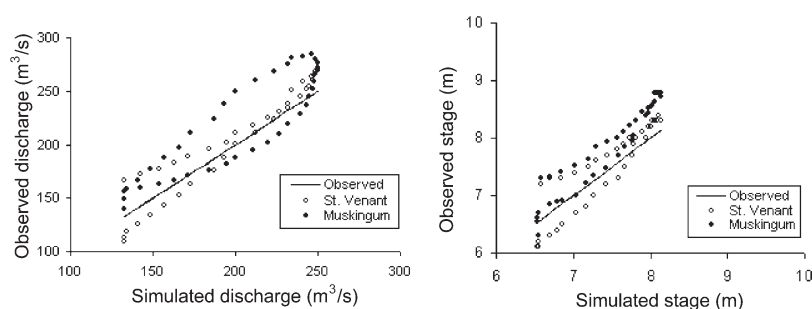


**Figure 6** │ Scattergram of observed and simulated discharges and stages for flood event (RF0193).
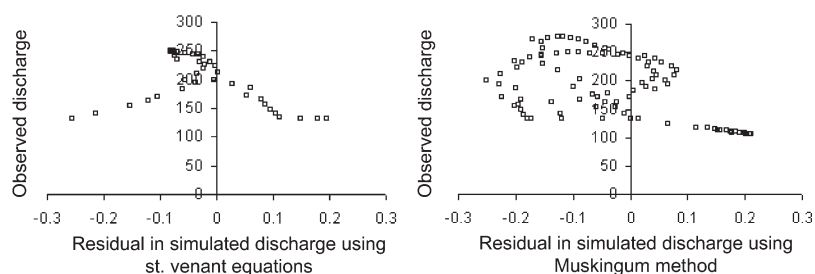
**Figure 7** │ Scattergram of observed and simulated discharges for flood event (RF0193).

In fact, the hydraulic routing approach better describes the physical processes as it is based on both continuity and momentum equations and mostly uses real measurable data in the simulation exercise.

## ACKNOWLEDGEMENTS

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