Mecánica Computacional Vol. XXI, pp. 27–39
S. R. Idelsohn, V. E. Sonzogni and A. Cardona (Eds.)
Santa Fe–Paraná, Argentina, October 2002

SOME PROBLEMS IN INTEGRATED HYDROLOGIC, HYDRODYNAMIC AND RESERVOIR OPERATION MODELS

Pablo M. Jacovkis*

* Departamento de Computación and Instituto de Cálculo
Facultad de Ciencias Exactas y Naturales, Universidad de Buenos Aires,
Ciudad Universitaria, 1428 Buenos Aires, Argentina
e-mail: jacovkis@dc.uba.ar, web page: http://www.dc.uba.ar

Key words: Rainfall-runoff model, Flood routing, Hydrodynamic model, Reservoir operation, Flood forecasting.

Abstract. *An integrated hydrologic, hydrodynamic and reservoir operation model is a global model of a fluvial basin where, given the rainfall, we can estimate the input runoff to the different watercourses of the basin, propagate the discharge downstream by means of flood routing and/or hydrodynamic methods and operate one or more reservoirs for different purposes (water supply, flood control, hydropower, water quality, recreation, navigation). Each phase of the model implementation has its own problems: numerical treatment of the rainfall-runoff equations, of the streamflow routing equations and of the hydrodynamic equations, methods for optimization or simulation of reservoir operation, calibration of parameters, treatment of pluviometric data series through Voronoi diagrams and similar approaches. Some modifications are necessary when such a model is used for forecasting.*

1 INTRODUCTION

In fluvial hydraulics, and in water resource systems planning, one of the most important tools to solve problems of practical interest is the mathematical modelling – and computer implementation - of behaviour of rivers or fluvial basins. If the reach of the river to model is not too wide, and its bed movement is sufficiently slow to be neglected, a one-dimensional hydrodynamic shallow water model with free surface and fixed bed, that solves the pair of Saint-Venant quasilinear hyperbolic partial differential equations of hydrodynamics, is perfectly adequate for this purpose, providing values for discharges and water elevations along the river for the simulated time. The theory, both of the quasilinear partial differential equations and of the numerical methods to use, is solidly established, and an impressive amount of numerical models is available for implementation in any computer whatsoever. If, instead of a reach of river, a set of reaches of several rivers, some tributaries of others as parts of a fluvial basin, or tributaries and effluents as parts of a fluvial delta, is to be modelled, compatibility equations in the junctions must be included, but the problem is also solved. The only problem with these models is essentially practical: how to get enough reliable data to feed the corresponding model. In developing countries, this problem is particularly grave.

There are four types of data for a one-dimensional hydrodynamic model: geometric data (geometry of cross-sections, topographic characteristics of the reach or basin), physical parameters (i.e., conveyances), initial conditions and boundary upstream and (for subcritical models) downstream conditions. Inner boundary conditions may also be used, if a lateral inflow of a non-modelled tributary is included. Each type must be studied individually. If the river bed is mobile, at least a third equation must be introduced, representing the conservation of the solid discharge; the system continues to be quasilinear hyperbolic, and can be written in conservation form; maybe also a fourth equation must be included, if we model the phenomenon of suspension, resuspension and settling of solid particles, or pollutants are transported.

Geometric data are usually available. They are easy to measure, both with primitive and sophisticated technology (of course, the precision of the measurement is not the same). Conveyances must be calibrated, and this is not necessarily simple: a small model may have more than thousand conveyances (they may vary spatially and depending on the water level), and in general instead of calibrating the model using mathematical tools, solving an inverse problem, experience and feeling of modelists and engineers are necessary. We usually do not have initial conditions for the runs we need: in general, this problem is solved “warming-up” the model during a certain simulation time, with appropiate boundary conditions. And, finally, there are two classes of boundary conditions: “natural” boundary conditions, that, hopefully, simulate historical or simulated series of discharges or water elevations, and “artificial” or “controlled” boundary conditions, that simulate, for example, (upstream or downstream) reservoir operations.

If not enough geometric data are available, or calibration is an impossible task due to lack of historical records of discharge or water elevations or to ignorance of the river characteristics, a simplified flood routing model must be built, using only one of the two

Saint-Venant equations (and consequently only one unknown function, discharge or water elevation). This model, with which an arborescent structure can be easily modelled, has fewer parameters to calibrate, at most one or two per spacial interval of discretization, but it must assume a one-to-one relationship between discharge and water elevation, for the reach or for each discretization interval. For a flood routing model, besides, only upstream boundary conditions are necessary. It is very likely that, if a large basin is being modelled, some parts may be subject to a complete hydrodynamical approach, and for some other parts a flood routing model will suffice. In this case, the discharges (or water elevations) of the flood routing model obtained downstream, as function of time, will be upstream boundary conditions of the hydrodynamic model, so that both models may be integrated and coupled into a global one.

Perhaps the river becomes too wide for the one-dimensionality assumption to be valid, or the lake or estuary into which the river flows must also be modelled. Then a two-dimensional model, much more complex, must be included in the simulation: the river inflow into the lake or estuary is a delta-like pulse inflow, varying in time, that has to be considered as a boundary condition to the lake. In a sense it is in general easier to model a lake than an estuary, because in a lake, usually, except for the rivers inflows or outflows the boundary is closed, so that the boundary condition is simply that the normal velocity is nil, while an estuary has open boundaries whose treatment is much more complex.

Anyway, the problem remains of obtaining upstream boundary conditions, both for the hydrodynamical and for the flood routing model. Many times no gauges are available upstream, and, besides, a better knowledge of the hydrologic cycle of the basin is desired. It is very likely that, instead of water elevation gauges, raingauges are available. If we can transform the rainfall over a basin into discharges into a river, we will have the boundary conditions we are interested in. For that, we need a rainfall-runoff balance model. This kind of model is in general more empiric than the others: a hydrodynamic model is governed by well known and well studied equations, conceptually clear. A flood routing model is a simplification of a hydrodynamic model, conceptually clear also. But a rainfall-runoff balance model has many empirical equations, because the process by which the rainfall is transformed into discharge in rivers is more complex, and less completely known, than the others. Besides, in such a model the basin under study will be decomposed into sub-basins, and the raingauges are not necessarily located in places “representative” of the sub-basins. A new problem emerges: given the sub-basins and the raingauges, how to assign rainfall to sub-basins? This is a clustering problem, that engineers have solved with the use of Thiessen polygons; essentially, Thiessen polygons are what in computational geometry are called Voronoi diagrams, and several very interesting algorithms exist for treating them, that have also very rich theoretical consequences.

Finally, a hydrodynamic or flood routing model is often implemented for water resource planning purposes: a reservoir of a dam must be operated, and the operation must take into account hydraulic or water management constraints. Downstream boundary conditions may be simply the reservoir operation criterion; the reservoir operation usually will try to optimize some economic function (hydroelectric generation, for instance). A downstream boundary condition indicates the physical feasibility of a reservoir operation criterion; but, as upstream boundary condition for a hydrodynamic or flood routing model downstream the dam, it can indicate the consequences of this criterion, so that a downstream model may complete the global integrated model, that includes hydrologic balance, flood routing, hydrodynamics and reservoir operation. The reservoir operation model solves, sometimes in a very simple way, an optimization or control problem. The analysis of different alternatives leads to interesting complications in a reservoir operation model.

This paper is not a state-of-the-art article on all these models, that is an almost impossible task that should need a long special article for each of the models. Its object is to offer a glimpse of the main ideas in the analysis and construction of each of the models, of the practical difficulties, and of how to "paste" the models into a unified and useful tool. The bibliography is therefore very succinct, but has been chosen in order that the reader that consults it will find sufficient additional references; it includes also some classical references.

2 A HYDRODYNAMIC MODEL

The one-dimensional quasilinear hyperbolic partial differential equations (Saint-Venant equations) that govern a gradually varied shallow water flow over fixed bed and free surface are

$$\partial (Q/S)/\partial t + \tfrac{1}{2}\,\partial (Q^2/S^2)/\partial x + g\partial Z/\partial x + gQ|Q|/D^2 = 0\,, \tag{1}$$

$$\partial S/\partial t + \partial Q/\partial x = 0\,, \tag{2}$$

where x represents the longitudinal axis of the river, t the time, $S= S(Z(x,t),x)$ the wetted cross-sectional area at point x and time t for water elevation $Z(x,t)$, measured from a fixed horizontal reference level, $Q(x,t)$ the discharge, $V=V(x,t)=Q(x,t)/S(x,t)$ the longitudinal velocity, $D=D(Z(x,t),x)$ the conveyance and g the acceleration of gravity. Equations (1) and (2) represent conservation of moment and of mass, respectively. The deduction of these equations may be consulted for instance in[1] or in the classical[2]. Usually, for known initial conditions of discharge and water elevation, and two boundary conditions, one upstream and the other downstream (for subcritical regime, i.e., for $V^2/(gS/B) < 1$, B being the width of the channel at the free surface) or both upstream (for supercritical regime, i.e., for $V^2/(gS/B) > 1$) one can obtain values of discharges and water elevations (and therefore velocities) using some numerical method, see for instance[3] and the references therein: except in exceptional cases, the Saint-Venant equations can not be analytically solved. The calibration parameters are the conveyances, and the number of values to calibrate is sufficiently large for rendering difficult the solution of the corresponding "inverse problem", in mathematical jargon. In fact, taking for instance a finite difference method with 100 discretization points and cross-sections discretized with 20 vertical discretization points, 2,000 values should be calibrated, and that is in general unfeasible. It is common then to use the professional engineers' experience, or when not enough data are available, to reduce the model to a simpler flood routing model.

As a matter of fact, the case with rectangular prismatic cross-section (the shallow water system) may be written in a simplified form (without loss of generality we take the value of the transversal width of the water elevation as one) as

$$\partial V/\partial t + ½\,\partial V^2/\partial x + g\partial (h+e)/\partial x + gV|V|h^2/D^2 = 0\,, \qquad (3)$$

$$\partial h/\partial t + \partial (hV)/\partial x = 0\,, \qquad (4)$$

where $h(x,t)$ is the height from the bed level and $e(x)$ is the distance from the bed level to a fixed reference level, so that $Z = h + e$. The simplified equations (3), (4) have the advantage of permitting an easier theoretical approach, using besides the analogy of equations (3), (4) with the equations of isentropic flow of polytropic gas dynamics. Equations (3), (4) are preferred when there is movement of bed particles that glide and roll along the bed, so that $e=e(x,t)$, because it is only necessary to add a third equation,

$$\partial e/\partial t + \partial Q_s/\partial x = 0\,, \qquad (5)$$

where Q_s is the bed-load (solid discharge per unit width), and the system continues to be quasilinear and hyperbolic, and admits a conservation law formulation. Formulae for bed-load transport are rather empirical, given the complexity of the erosion phenomenon; nevertheless, it can be said that bed-load depends on V and h, increasing monotonically with V and decreasing monotonically with h (see for instance[3] or[4]). If particles not only glide and roll but also are transported in suspension, or may settle or be resuspended, depending on the velocity, a source or sink must be added to equation (5), namely

$$\partial e/\partial t + \partial Q_s/\partial x = P\,, \qquad (6)$$

where P is the settled or eroded material per unit time, and an equation for particles that are being diffused must be included, such as

$$\partial C/\partial t + V\partial C/\partial x = \sigma\partial^2 C/\partial x^2 + S\,, \qquad (7)$$

where C is the concentration of the suspended material, σ the coefficient of diffusion, and S the sink or source term, conveniently related to P. Equations of state must be provided for P and S. A similar equation represents transport of pollutants.

Incidentally, when each cross-section is not approximately rectangular the mobile bed case is more involved: it is not easy to model changes of bed shape for irregular cross-sections; in particular, with an irregular bed, what value do we take for the bed elevation e? The solution is a "quasibidimensional model" with streamtubes (each with a different bed elevation), like in[5], or a general two-dimensional model. For instance, equations (6) and (7) hold in each streamtube and, of course, more than one type of material may be suspended, settled or eroded, and it is easy to modify (7) to include this.

A hydrodynamic model may be extended to a river basin (arborescent structure), see[6], or to a river delta (deltaic structure), see[7], maintaining the efficiency of the numerical code, introducing the compatibility equations at the nodes of confluence or bifurcation of river reaches that, in simplified form, are as follows (see[2]):

$$Q_i + Q_j = Q_k\,, \qquad (8)$$

$$Z_i = Z_j = Z_k \quad , \tag{9}$$

where i and j are the downstream points of tributaries flowing intto a reach where the upstream point is k; for a bifurcation the change of sign in (8) is obvious. When a fluvial network is being modelled, for a subcritical flow at each extreme point a boundary condition must be given, as shown in[8].

3 A TWO DIMENSIONAL MODEL

The two-dimensional extension of equations (1), (2) (with horizontal cartesian axes x and y) is described by equations (see[9])

$$\partial (hU)/\partial t + \partial (hU^2)/\partial x + \partial (hUV)/\partial y - fhV + gh\partial Z/\partial x + c_f U\sqrt{(U^2 + V^2)} = 0 \,, \tag{10}$$

$$\partial (hV)/\partial t + \partial (hUV)/\partial x + \partial (hV^2)/\partial y + fhU + gh\partial Z/\partial y + c_f V\sqrt{(U^2 + V^2)} = 0 \,, \tag{11}$$

$$\partial Z/\partial t + \partial (hU)/\partial x + \partial (hV)/\partial y = 0 \,, \tag{12}$$

where now U and V indicate the velocities in the x- and y-directions, and f is the Coriolis parameter. Equations (10), (11), (12) are in conservative form, and c_f is a friction coefficient, conveniently related to conveyance, that must be calibrated. From the numerical point of view, an enormous amount of methods have been studied and implemented in models, since the pioneer work of Leendertse[10]; many of them are described in[9]. We have in general three types of boundaries: closed boundary (the boundary condition is normal velocity nil), open boundary and influx of one-dimensional flows (from one-dimensional fluvial models), represented by a linear combination of Dirac deltas. Boundary conditions are more involved than for one-dimensional problems, because in an open boundary usually there exists the phenomenon of numerical reflection of waves, and it is not easy to get rid of it (see again[9]). It is of course much more difficult, because many more data are necessary, to obtain precise initial conditions, but, as before, “warming-up” the model during an initial simulation time usually works. The real problem is calibrating a two-dimensional model, because, if the friction coefficient c_f varies according to the spatial position and to the water level, the number of parameters to calibrate is subject to “the curse of dimensionality”; fortunately, in general we are satisfied with a calibration of fewer values of c_f, because (and this is an advantage over a one-dimensional fluvial model) in this case the geometry of each cross-section is very simple (rectangular) except near the closed boundary.

A mobile-bed two-dimensional model requires, if the particles only slide and roll, a fourth (conservation of solid mass) equation,

$$\partial e(x,y,t)/\partial t + \partial Q_{sx}(h,U,V)/\partial x + \partial Q_{sy}(h,U,V)/\partial y = 0 \,, \tag{13}$$

where now Q_{sx} and Q_{sy} are the bed-loads in the x- and y-direction, respectively. If particles are also transported in suspension, or if we want to model pollution, a two-dimensional generalization of equation (7) must also be included.

4 A FLOOD ROUTING MODEL

If instead of two unknown functions, say discharge and water elevation, we have only one, for instance discharge (for which a one-to-one relationship must be assumed between both), only one equation is necessary, namely, the classical scalar quasilinear partial differential equation, that can be written as a conservation law. In many underdeveloped countries, for many rivers and basins no possibility of modelling both equations exist, because of lack of actual data; but also in developed countries sometimes data are missing near the sources of a river, so that this kind of model may be very useful. Besides, such a model is easier to calibrate than a complete hydrodynamical one. Suppose for instance that at each point x of the reach under analysis a one-to-one relationship between the discharge and the water elevation is assumed, $Q = Q(x,t) = Q(Z(x,t),x)$. If we assume also that the discharge/water elevation function is the same for all the points of the reach, taking into account that $S = S(Z)$ we may write

$$\partial S/\partial t + \partial Q(S)/\partial x = 0\,, \tag{14}$$

that is a scalar conservation law for which a beautiful theory has been developed, see for instance[11]. In fact, for rivers with flood beds and rivers trapped in canyons it can be proved that the (generally nonlinear) flux function $Q(S)$ has some very distinctive features, which determine a very rich and peculiar phenomenology[12]. Usually equation (14) is written

$$\partial Q/\partial t + c\partial Q/\partial x = 0\,, \tag{15}$$

where c represents a wave velocity. We can see that the number of parameters to calibrate depends essentially on an estimate of c. For instance, for a finite-difference discretization and a formula $c = \alpha\, Q^{\beta}$, with α and β parameters to calibrate, the number of parameters will be twice the number of discretization intervals, a number much smaller than with a complete hydrodynamic model. Many numerical methods may be applied to solve this equation; the most successfull is probably the finite-difference Muskingum method. Interestingly enough, the method was empirically applied by the US Army Corps of Engineers in the '30, and only in 1969 Cunge[13] developed a theoretical justification. Equation (15) requires, besides the initial condition, an upstream boundary condition, and perhaps lateral boundaries, for which the right-hand side in (15) is replaced by a linear combination of Dirac deltas, so that it is very easy to model a basin where rivers have a tree structure (in the sense of graph theory): equation (8) is applied in the junctions; obviously, the "last" downstream point coincides with an upstream boundary (or a lateral boundary) of a hydrodynamic model, or is the inflow into a reservoir to be operated.

Experience indicates that in general it is easier to obtain data to run the model in the downstream region than in the upstream region, so that in many situations a hydrodynamic and a flood routing model are simultaneously implemented, and the hydrodynamic model receives as upstream (or lateral) boundary condition the downstream results of the flood routing model.

5 RAINFALL-RUNOFF BALANCE

The transformation of rainfall over a basin into runoff that feeds the rivers of the basin is a complex (and not totally known) process, in which not only hydrologists but also agronomists, geologists and meteorologists are interested. Models may be purely empirical (unit hydrograph, extreme frequency analysis, regression analysis), deterministic, based on complex physical theory, and conceptual, which are in an intermediate position between deterministic and empirical; see[14] for a good description of all types of rainfall-runoff models. Deterministic models require an impressive amount of data (spatially and densely distributed records in two-dimensional regions, for reasonably long time periods) and computational time; some of the data require expensive equipment for recording (telemetry networks, rainfall radar, satellite imagery, remote sensing), so that they are out of reach of many studies, particularly in developing countries. On the other hand, empirical models are often too coarse, so that, when not enough data are available, a conceptual model is a convenient trade-off between the other categories. In a conceptual "lumped" model, we take representative values for each sub-basin into which we divide the total basin. Once rainfall is assigned to a sub-basin for the time step being computed (time steps are not necessarily equal for all the models), it must be decomposed into surface flow, subsurface flow and baseflow, subtracting previously evapotranspiration. Formulae for computation of evapotranspiration can be consulted in[15]; according to the availability of equipment, evapotranspiration may be recorded, daily, say, or estimated monthly, for instance. More complex is the computing of the relative proportion of surface flow, subsurface flow and baseflow in the rainfall decomposition, after evapotranspiration has been subtracted; the often used Green-Ampt model of infiltration can be seen in[16].

Anyway, after rainfall has been transformed into some kinds of runoff, it is necessary to compute how and when these runoff values will feed a river, as boundary conditions. The basic conceptual approach considers some equation of the form $S = K\,Q^n$ with parameters K and n, where S is the storage and Q the outflow. Many modifications have been implemented; for instance in[17] we took

$$\mathrm{Q} = \alpha \mathrm{E} + \beta \mathrm{S}\,, \tag{16}$$

α and β being the propagation and attenuation coefficients (that must be calibrated) and E each type of input runoff. Coupling (16) with the equation of continuity

$$dS/dt = E - Q \tag{17}$$

we obtain an ordinary differential equation that will be numerically solved. It must be noted that a more involved model with interaction between the different types of flow has to be treated carefully because a stiffness problem may occur (time scales of the different flows may be too different).

For a very large basin (the Amazon basin), equation (17) has been very satisfactorily replaced[18] by

$$S = \{Q + \gamma\, dQ/dt + \kappa\, d^2Q/dt^2 - \alpha(E + \delta\, dE/dt + \eta\, d^2E/dt^2)\}/\beta \qquad (18)$$

with convenient coefficients (that it is necessary to calibrate); of course, the number of derivatives taken may vary depending on the basin.

The reader must note that in the last years several very interesting rainfall-runoff models have been developed using modern mathematical and computational concepts, like neural networks and genetic algorithms (see for instance[19] and[20], respectively).

Finally, remark that this analysis has neglected the runoff due to snowmelt. For snowmelt to be modelled, snowpack is multiplied by a melt factor and by a degree-day temperature index; fortunately, techniques to compute (and to forecast) runoff from snowmelt are very well developed.

6 A COMPUTATIONAL GEOMETRY MODEL

Location of raingauges are generally decided for economic, not hydrologic, reasons, so that it is not easy to compute directly rainfall over all a sub-basin from raingauge data. The problem arises, then, of assigning rainfall to sub-basins when there are available data of certain raingauges (that may vary dynamically: for instance, the record of a raingauge may be interrupted during a week). There exists a straightforward solution, that is probably the simplest problem in planar computational geometry: each point of a basin is considered subject to the influence of the rainfall of the nearest raingauge. This method – in computational geometry jargon, of Voronoi diagrams - is equivalent to the time-honored method, used by hydrologists, of Thiessen poligons. For instance, let $\{P_1, P_2,..., P_m\} = R$ the set of raingauges with recorded rainfall for, say, day t. Let now G be one of these raingauges. Then the Voronoi region of G (where it is supposed that rainfall has been measured at G) is

$$V_G = \{\, x \in \mathbb{R}^2 : \|x-G\| \leq \|x-P\|,\ \forall P \in R \,\}, \qquad (19)$$

where $\|\ \|$ indicates Euclidean distance. For each sub-basin, its points belonging to V_G will be assigned (at day t) to raingauge G.

The problem is more involved if there are certain regions separated by a physical obstacle (for instance, mountains) so that one can not assign points to the nearest raingauge, or if, from meteorological studies, it is clear that rains are very local phenomena that can not be extrapolated to distant points. Then modifications have to be implemented: for instance, weights must be assigned to distances. A very good description of the general theory behind this procedure, and its generalizations, may be seen in[21].

7 RESERVOIR OPERATION

Anyway, one of the desired results of numerical experiments after implementing a flood routing or hydrodynamic model is related to reservoir operation. With a dam as downstream boundary condition, one can observe the physical feasibility of different reservoir operation criteria: the downstream boundary conditions is “artificially” imposed, and we analyze the incidence in the river stream of applying the different criteria. Conversely, with a dam as upstream boundary condition of a hydrodynamic or flood routing model, we can measure the results downstream the river of applying each criterion. In particular, when a forecast version of a global model is implemented, sound operation decisions may be taken almost "in real time" that optimize an objective function. Depending on the season, the number and characteristics of reservoirs, the water demands for different purposes, and many other conditions, there will be many possible constraints and objective functions. For instance, under the reasonable assumption that there exist monotonically increasing functions $S = S(Z)$ and $A = A(Z)$, where S is the reservoir storage, Z is the water level at the dam and A is the reservoir surface, and taking into consideration the equation of continuity

$$dS/dt = E - Q - E_v \, , \tag{20}$$

where E is the discharge flowing into the reservoir, Q is the water release and $E_v = E_v(A)$ is the reservoir evaporation, suppose the following constraints:

- There exists a maximum normal water level, that must not be surpassed;
- There is a maximum admissible water release, to avoid damages downstream;
- There is a minimum admissible water level, below which no energy can be generated;
- Depending on the water level, a certain number of turbines is working, with a certain efficiency;
- There is a minimum water release, below which the water supply for irrigation is insufficient;
- There is a bound to the daily increase or daily decrease of water release.

The decision makers search, under these constraints, to maximize the hydroelectric generation. Naturally, in case of unfeasibility, there must be a priority for violation of the constraints.

One can see that this is a very simple example of reservoir operation, and the policy may be (and usually is, in most reservoirs) much more complex, and may change according to the season. The criteria are the result, sometimes, of a mathematical approach (optimization under constraints), and mostly of simulating different policies.

Of course, several chained models may be implemented, each one flowing into a reservoir, to obtain a global perspective of a water-resource system. With a forecast model, one takes into account, in order to choose operation criteria, not only the current situation (the "initial conditions") but also the future situation for the forecast horizon for all the reservoirs of the system, so that optimizing the objective function for each forecast, that is, choosing simultaneously criteria for all reservoirs, can be a very challenging problem. Each reservoir operation criterion determines a different mathematical formulation.

Here also some new mathematical tools have been implemented, for instance genetic algorithms[22] and neural networks[23]; but the research in optimal reservoir operation has a long history: interestingly, Morel-Seytoux research in optimization of reservoir operation (see for instance[24]) is based, according to him, in the work of one of the founding fathers of economic planning, Pierre Massé[25]. Anyway, there exists a gap between the academic research and the actual operation of reservoirs: many reservoirs continue to be operated without any formal operation policy. In that sense, it seems that, in the fields of hydrodynamics, flood routing and rainfall-runoff balance, the practical use of very updated and sophisticated mathematical models is more generalized than in the field of water resources planning. This situation will certainly change in the future.

8 FORECASTING

A forecasting model is an extremely useful tool for several purposes, especially for establishing day-to-day criteria for a reservoir operation. For a complex integrated model as proposed in this paper, it is very useful to construct and implement a forecasting version, taking into account that meteorological forecasting (and particularly rainfall forecasting) is more and more developed and accurate. The forecasting method is as follows: if we want today a three-day forecast, say, supposing that we have today good initial conditions, forecast rainfall for the next three days are boundary conditions, that, via the Voronoi procedure – or directly, with modern equipment for data acquisition - are transformed into rainfall over sub-basins, and then, via the rainfall-runoff model, in discharges into the flood routing and/or hydrodynamic models, that, in turn, will feed, if it is the case, the reservoir operation model; the planned discharges through the dam will be, if there exists a hydrodynamic or hydrologic model downstream the dam, its upstream boundary conditions. Then we shall have a complete forecast for the next three days. Suppose now that the forecasting model is operated once a day (it is very simple to adapt this idea to a different time step of operation): we need initial conditions for tomorrow, when we shall operate the forecasting model again. At several points of the model we have (tomorrow) streamgauges or water elevation records and, besides, we have the actual data of rainfall fallen in the last 24 hours – that will also contribute to the knowledge of the future initial conditions (care must be taken to be sure that the future initial conditions are accurate). The known future values, and the values computed in this run for time = 24 hours, or an interpolation of future known values, or a combination of both, will be the future initial conditions. Naturally, several different tentative forecasts may be tested, and a subroutine is easily implemented that updates the data, for the future run, only when we decide which run will be the “true” forecast, from which the future initial conditions will be extracted. This author has implemented several forecast models for differente clients.

9 CONCLUSIONS

A general description of how to construct a global water-resources model has been presented. One must be aware that almost all the subjects touched have been already investigated, sometimes very carefully, and of course a large amount of commercial software is available. One must also be aware that, besides the theoretical interest that global water-resource models have from the point of view of applied mathematicians and engineers, not all commercial software is equally good, and a knowledge of the different problems and difficulties allows saving a lot of time and money. We remark that a global model of the type just described needs a massive amount of data, and yields a massive amount of results. In particular, as soon as the model has been calibrated (for which probably each submodel should be calibrated independently, to facilitate the work) different alternatives must be simulated. But in many cases the time (or money) necessary for running in a computer all the alternatives exceeds the time (or money) available, so that it should be very useful to design carefully the experiments to be run. For that reason, it should be convenient to use techniques of design of experiments to obtain the maximum feasible information as soon as possible, and as inexpensively as possible; nowadays, the use of these techniques in computational modelling is much less developed than it should be. And this commentary holds not only for the models designed by a research group of engineers, applied mathematicians and computer scientists, but also for proprietary models bought.

10 REFERENCES

[1] M. H. Chaudhry and M. E. Barber, Open channel flow, in: R. W. Johnson (ed.), *The handbook of fluid dynamics*, Chapter 45, CRC Press, Boca Raton, (1998)

[2] J. J. Stoker, *Water waves*, Interscience, New York, (1957)

[3] W. H. Graf, *Fluvial hydraulics*, Wiley, New York, (1998)

[4] M. H. Gradowczyk, Wave propagation and boundary instability in erodible-bed channels, *J. Fluid Mech*., **33**, 93-112 (1968)

[5] M. H. Gradowczyk and P. M. Jacovkis, Un modelo hidrodinámico impermanente de simulación de escurrimiento sobre fondo móvil, transporte de partículas en suspensión, decantación y resuspensión, *Anales del XI Congreso Latinoamericano de Hidráulica*, Vol. II, Buenos Aires, 465-476 (1984)

[6] P. M. Jacovkis, Modelos hidrodinámicos en cuencas fluviales, *Revista Internacional de Métodos Numéricos para Cálculo y Diseño en Ingeniería*, **5**, 295-320 (1989)

[7] P. M. Jacovkis, Modelos numéricos hidrodinámicos en redes fluviales complejas, *Revista Internacional de Métodos Numéricos para Cálculo y Diseño en Ingeniería*, **6**, 543-572 (1990)

[8] P. M. Jacovkis, One-dimensional hydrodynamic flow in complex networks and some generalizations, *SIAM J. Appl. Math*., **51**, 948-966 (1991)

[9] C. B. Vreugdenhil, *Numerical methods for shallow-water flow*, Kluwer, Dordrecht, (1994)

[10] J. A. Leendertse, Aspects of a computational model for long-period water-wave propagation, *The Rand Corporation*, Santa Monica, California (1967)

[11] J.Smoller, *Shock waves and reaction-diffusion equations*, Springer, Berlin, (1983)

[12] P. M. Jacovkis and E. G. Tabak, A kinematic wave model for rivers with flood plains and other irregular geometries, *Mathl. Comput. Modelling*, **24**, 1-21 (1996)

[13] J.-A. Cunge, Au sujet d'une méthode de calcul de propagation de crues (Méthode Muskingum), *Journal of Hydraulic Research*, **7**, 205-230 (1969)

[14] M. G. Anderson and T. P. Burst, Modelling strategies, in: M. G. Anderson and T. P. Burst (eds.), *Hydrological forecasting*, Wiley, New York, 1-13 (1985)

[15] R. G. Allen, L. S. Pereira, D. Raes and M. Smith, *Crop evapotranspiration. Guidelines for computing crop water requirements*, FAO (Food and Agriculture Organization of the United Nations) Irrigation and Drainage Paper No. 56, Rome (1998)

[16] D. L. Baker, http://www.aquarien.com/sptutor/pdeb3, (2001)

[17] M. H. Gradowczyk, P. M. Jacovkis, A. Tamusch and F. M. Díaz, A hydrological forecasting model of the Uruguay River basin system, in: *Hydrological Forecasting (Proceedings of the Oxford Symposium)*, IAHS Publ. 129, 517-524 (1980)

[18] A. Tamusch, P. M. Jacovkis, J.-M. Roussel and F. M. Díaz, Un modelo hidrológico conceptual apto para modelización de grandes cuencas hídricas, *Actas del X Congreso Latinoamericano de Hidráulica*, México, 452-460 (1982)

[19] J. C. Mason, R. K. Price and A. Tem'me, A neural network model of rainfall-runoff using radial basis functions, *Journal of Hydraulic Research*, **34**, 537-548 (1996)

[20] V. Babovic and M. B. Abbott, The evolution of equations from hydraulic data, *Journal of Hydraulic Research*, **35**, Part I: Theory, 397-410, Part II: Applications, 411-430 (1997)

[21] H. Edelsbrunner, Triangulations and meshes in computational geometry, *Acta Numerica*, **9**, 133-213 (2000)

[22] R. Wardlaw and M. Sharif, Evaluation of genetic algorithms for optimal reservoir system operation, *Journal of Water Resources Planning and Management*, **125**, 25-33 (1999)

[23] T. R. Neelakantan and N. V. Pundarikanthan, Neural network-based simulation-operation model for reservoir operation, *Journal of Water Resources Planning and Management*, **126**, 57-64 (2000)

[24] H. J. Morel Seytoux, Optimal deterministic reservoir operations in continuous time, *Journal of Water Research Planning and Management*, **125**, 126-134 (1999)

[25] P. Massé, *Les réserves et la régulation de l'avenir dans la vie économique I: avenir determiné*, Hermann & Cie., Paris, (1946)