World Meteorological Organization

FLOOD FLOW FORECASTING

by

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SUMMARY

1. FLOODS

1.1 Main causes and impacts

Floods may be generated by changes in land use such as urbanisation and deforestation, and their effects are felt most severely where flood-plain population densities are high. F*lood plains* are a common feature of large and intermediate-sized rivers, and where urban development is unplanned, or where there is no regulation governing where urban development will occur, informal settlements spring up on flood plains after a period in which annual floods are low (Figure 1); the flat topography of flood plains makes it easy to construct shelters quickly and cheaply, on land that is often within easy access of city centers. However when extensive flooding occurs, damage to such settlements is severe, and municipal government is pressed to invest in flood protection for the area.

The main impacts on the population occur where there is a lack of: (i) knowledge regarding the occurrence of flood levels, and (ii) planning of settlement and urban development that takes account of flood risk.

F*loods due to urbanization* are those related to the increase of the impermeable areas and man- made drainage such as conduits and channels. The land surface in small urban basins usually consists of roofs, streets and others impervious material. Runoff flows generated by these surfaces enters storm-sewers, so that the overland flow is increased, infiltration is decreased, and the hydrological cycle is changed. Under these circumstances the peak discharge increases together with the flood frequency. In addition, the wash from urban surfaces during rainy days increases the pollution load in the urban environment and in downstream reaches of rivers.

Since the 1970s, the volume of water generated by urban areas in developed countries has been controlled at source by means of measures such as detention and retention ponds, permeable surfaces, and infiltration trenches. In developing countries, however, this type of control seldom exists, and the impact of flooding is commonly transferred to downstream reaches of the major drainage system. The costs of controlling and alleviating this impact are transferred from the individual to the public, since local government agencies must invest in hydraulic structures to reduce the effects of downstream flooding.

Reliable experience in flood control of many countries has now defined some main principles in urban drainage management, which are:

- Flood control evaluation should take account of the whole basin, and not only selected flow sections;
- Urban drainage control scenarios should take account of future city developments;
- Flood control measures should not transfer the flood impact to downstream reaches, and should give priority to measures that control runoff at source;
- Pollutants washed from urban surfaces by runoff, and arising from other sources within urban areas, should be reduced;

- More emphasis should be given to non-structural measures for flood plain control such as flood zoning, insurance and real time flood forecasting.
- Flood management and control must start with the implementation of Urban Drainage Master Plans for cities;
- Public participation in urban drainage management should be increased;
- The development of urban drainage should be based on recovering the cost of investments.

These principles have been developed from experience in developed countries but are not always fully applied even there. Urban drainage practices in most developing countries rarely fulfill these principles, because of the following reasons:

- Urban development in cities of developing countries cities occurs too rapidly and unpredictably. Usually the trend in such development is from downstream areas to upstream areas, thereby increasing the damage due to flooding (Dunne, 1986);
- Urbanization in periurban areas commonly takes no account of city regulations. This urbanization has the following characteristics:

  Unregulated developments: In the periurban areas of big cities, the price of land is relatively low. Regulation of such areas requires investments which are about equal to the price of the land. As a consequence, owners of private land develop urbanized areas without the necessary infra-structure, for sale to people with low incomes;
  Invasion of public areas (such as public green areas): it is common for public areas to be invaded which were to be set aside under Urban Master Plans for future parks, public construction and even streets. Homeless people build shelters there, and slow decision-making by public administration results in such informal settlements being consolidated by delivery of piped water and power supplies. In 1973, 59% of the city of Bogota consisted of illegal developments, most of them without proper sanitation;

- Periurban and risk areas (flood plains and hill slopes) are occupied by low income population without any infra-structure. Spontaneous housing development in risk areas in cities of the Humid Tropics occurs on land prone to flooding: Bangkok, Bombay, Guayaquil, Lagos, Monrovia, Port Moresby and Recife; hill sides prone to landslides: Caracas, Guatemala City, La Paz, Rio de Janeiro and Salvador (WHO, 1988);
- Municipality and population usually do not have sufficient funds to supply the basics of water, sanitation and drainage needs;
- Lack of appropriate refuse collection and disposal decreases water quality and waste material clogs the urban drainage network. Desbordes and Servat (1988) found that in some African countries there is no urban drainage and when system drainage exists it is filled with garbage and sediments. Tokun (1983) also mentioned this type of problem in Nigeria where the drainage systems serves for garbage collection;

- There is no programme for preventing settlements being established in areas of flood-risk, and when floods occur, non-returnable funds are given to the local administration to cope with the problem, without any requirement for future settlement to be prevented.
- Neither the population at large nor the administrative bodies of cities and states have sufficient knowledge to manage floods according to the above principles;

- Lack of institutional organization in urban drainage at a municipal level such as: regulation, capacity building and administration. In Asian cities there is a lack of (Ruiter, 1990): comprehensive project organization and clear allocation of responsibilities; adequate urban land-use planning and enforcement; capability to cover all phases and aspects of technical and non-structural planning;
- Where there is no system of waste collection, solid waste is dumped into storm-sewers. This may occur even in some developed countries;
- When there are unrealistic regulations for urban occupation related to social and economic conditions, owners of land find ways to avoid complying with them.

1.2 Climate conditions

Most developed countries experience temperate or cool climates, whilst many developing countries lie in tropical regions where the temperatures are higher and rainfall both more intense and more frequent. Tropical climates, and the humid tropics in particular, give rise to much greater difficulties for environmental management of cities, in which urban drainage is one of the main challenges.

The main climate aspects related to urban drainage in the humid tropics are:

- Rainfall intensity is about 25% greater than in some temperate climates (Figure 1) which requires higher investment for the same level of risk in drainage control, since peak discharges are higher in proportion relative to rainfall intensity;

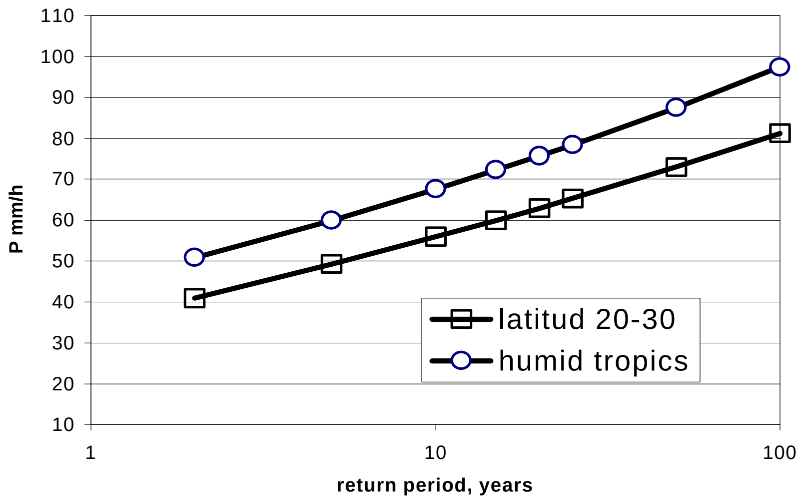


Figure 1 Comparison of mean maximum rainfall of 1 hour duration for gauges in Humid Tropics and gauges inside latitudes 20 and 30º S in Brazil (Tucci and Porto, 2001)

- Design conditions are not based only on the local rainfall intensity but also on the rainfall duration during the rainy season. Low-intensity, long-duration storms maintain a high water level in the major drains over long periods, thus creating adverse downstream boundary conditions (backwaters) for the drainage system. In this situation the volumes of water can be so large that streets are flooded every year. Examples can be seen during monsoon seasons in the cities of India and Bangladesh. The options for coping with such problems are: (a) to learn to live with floods, reducing their damage; (b) structural protection with dikes when economics conditions allow it; (c) to use a more appropriate return period when designing urban drainage systems for the humid tropics;
- High temperature conditions throughout the year allow the development of many related and water born diseases such as malaria, yellow fever, dengue (the host mosquito develops in stagnant waters under conditions of warm climate) and schistosomiasis (the host has been found in the urban lake of Pampulha in the city of Belo Horizonte).

1.3 Flood Control measures

Flood control measures can be classified as structural and non-strucutural. *Structural* measures are related to the physical control of basin drainage by means of constructions or devices such as dams, dikes, channel canalization, and planting forests. Non-structural measures are those in which floods are mitigated by procedures such as: insurance, flood zoning, and *flood forecasting*.

Structural solutions have higher costs and are feasible only when the cost of flood damage is higher than the cost of flood protection, or where there are intangible social aspects. Non-structural measures cost less, but there are some difficulties in their political implementation.

In the United States one of the main measure has been flood insurance, but it is feasible only in countries of large areas with great climate variation, since its risk can be distributed and the insurance cost can be redistributed among the population.

Flood zoning and forecasting are a combination of sound flood control measures. The main difficulties in the implementation of these measures are:

- Land owners will take action against regulation of their lands, and usually have influence in municipal politics;
- If floods occur, the affected municipalities may receive non-refundable compensation from State and Federal governments. Under this circumstances there is no incentive for programmes of flood prevention.
- Whilst structural measures may well not be economically feasible, they have much more political visibility. Non-structural measure require much effort from both the population and politicians. This is a difficult political task and politicians usually do not practice it.

1.4 Impacts due to flood-plain settlement

A common scenario is that unplanned settlements spring up on flood plains during a sequence of years during which annual floods are low, since the flat areas are favourable to settlement. When years with higher floods return, significant damage occurs and there are demands for government action to build control structures for flood protection and alleviation.

Examples of this scenario are as follows:

- Figure 2 shows floods levels on the River Iguaçu at União da Vitória. For a long time, floods remained below the 5-year return period. The floods after 1982 produced significant damage to the community (Table 1).

Table 1 Flood losses at União da Vitória and Porto União (JICA, 1995)

Year	Losses US$ millions
1982	10.365
1983	78.121
1992	54.582
1993	25.933

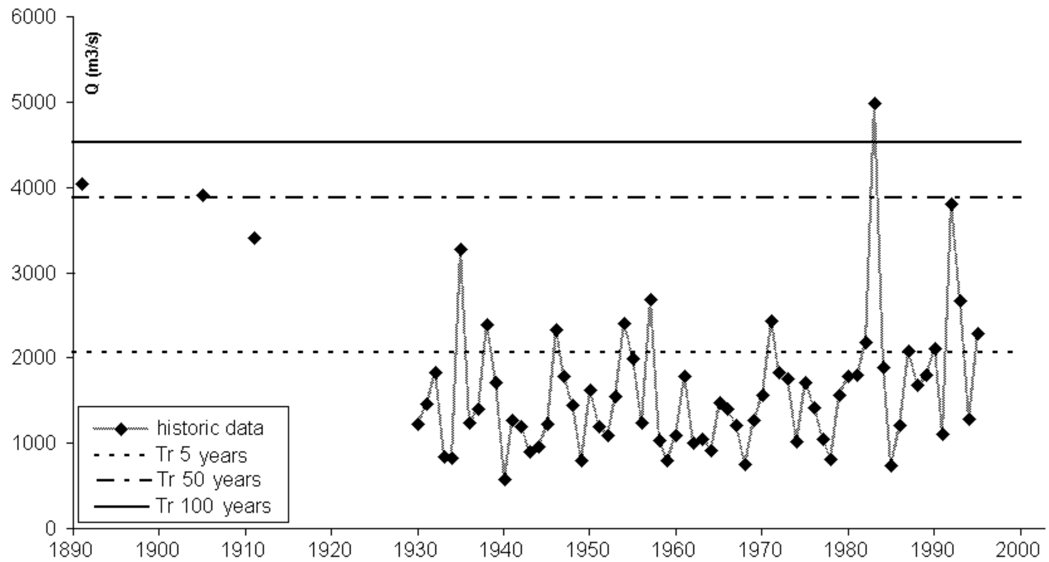


Figure 2 Maximum flood levels on the River Iguaçu at União da Vitória (a basin of approximately 25,000 km², Tucci and Villanueva, 1997)

- For the River Itajaí, a partial record of annual floods exists from 1852. Figure 3 shows the variability of these levels, and shows also that between 1911 and 1983 no levels were higher than 13.04 m (15-year return period, approximately), whereas before and after this period there were several levels of up to 17.19m. In both cases, the continuous record that began in 1930 gives a biased sample for risk assessment. Even so, the losses were significant; in 1983 (approximately 30-

year return period) they represented 8% of the GDP of the State of Santa Catarina at the time.

- The Pantanal, one of the largest wetlands of the world, is located in the Upper Paraguay river. In this region the population has always lived without threat to the environment. Figure 4 shows the maximum flood levels at Ladário since the beginning of the century. Table 2 shows the values of the maximum mean level of flooding and of flooded áreas of the Pantanal, at three different periods. The great difference between the 1960s and the other periods is clear. During this period the flood valleys were occupied. This population was displaced during the following decades and, due to the changes in the bed caused by the variability of the Pantanal flows, they had to abandon their properties and are living in poverty on the outskirts of towns and cities in the region.

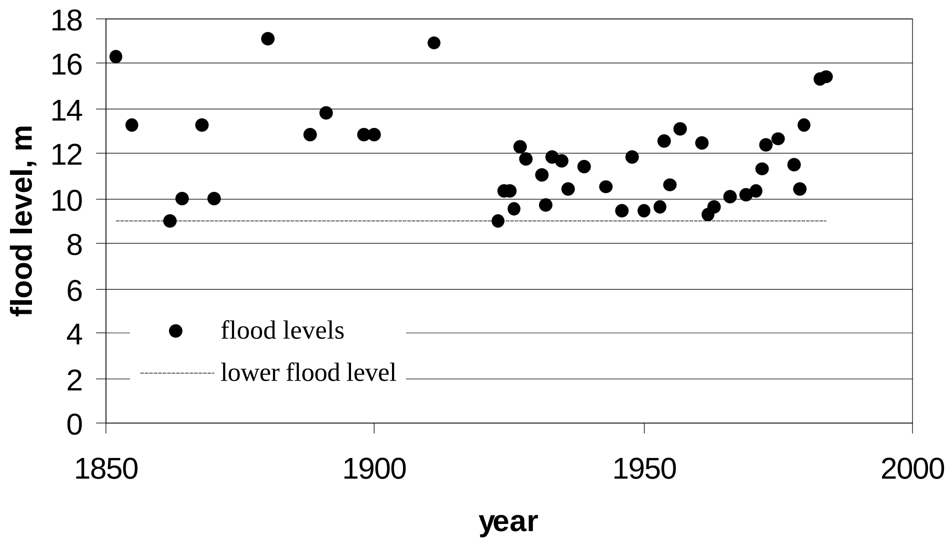


Figure 3 Flood Levels in Blumenau, S. Catarina, Brazil

Table 2 Estimated values for levels and flooded areas in the Pantanal

Period	Mean Maximum Level m	Mean flooded area in the Pantanal * 1000. km^2
1900-1959	4,16	35
1960 – 1972	2,21	15
1973-1992	5,49	50

(*) approximate values obtained from Hamilton et al (1995)

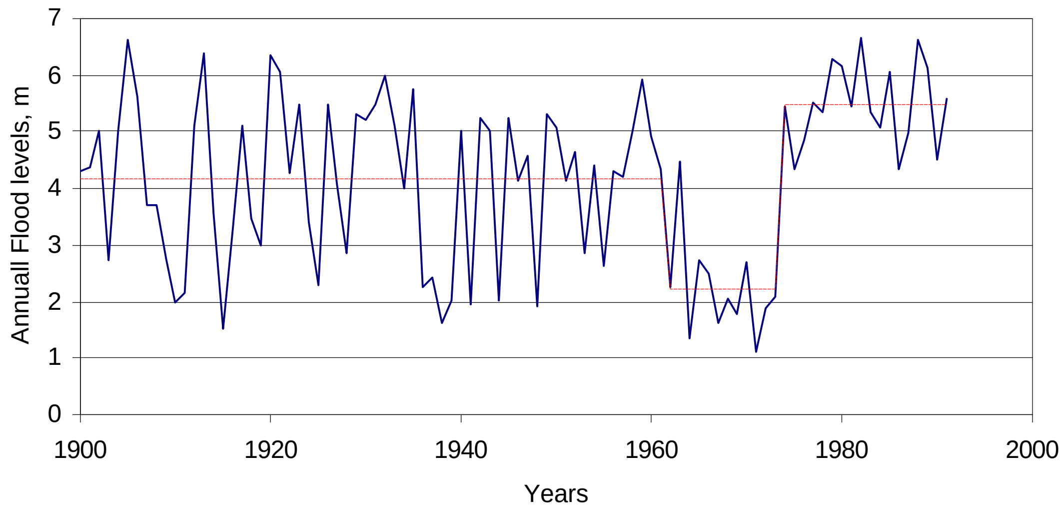


Figure 4 Annual flood water levels at Ladario in the Paraguay River and the mean of the periods: (i) 1900-1961;(ii)1961-1973; n(iii)1973-1991

2. FLOW FORECASTING AND PREDICTION

This chapter summarizes the basic concepts of flood forecasting, terminology, rainfall prediction, and hydrologic modeling; it also describes some case studies to illustrate the hydrologic components of flood forecasting. This paper seeks to use such case studies to present the hydrologic and water resource aspects of flow forecasting.

2.1 Definitions

Flow forecasting is related to flow evaluation in terms of a lead time. Prediction is the probability of the flow occurrence based on the historical flow records. *River Flow forecasting* is one of the measure used in water resources management in order to deal with the uncertainty of the climate. Management of water uses, such as hydropower, water supply irrigation, navigation, flood control and environment conservation are dependent on the amount of water in the river systems. Since the future climate is uncertain, all these water uses are planned based in the historical flow statistics which are assumed stationary. However, climate trends have been detected in a number of flow series around the world, and the possible effects of climate change on hydrologic regimes have also been noted (IPCC, 2001).

The main consequence from using non-stationary series in water resource engineering is the increased uncertainty for the water investments. River flow forecasting can be used to decrease the uncertainty and the risk of the water resources uses and conservation.

Forecasts of river flow can be made in the *short-term*, over periods of few hours or a few days of lead time: and in the *long-term*, up to nine months (Georgakakos and Krysztofowicz, 2001). Usually short-term flow forecasts are used for flood management, but there are many other contexts where short-term forecasts are useful, such as: navigation in rivers where the load transported is dependent of the flow depth in unregulated rivers; irrigation and water supply and integrated water uses such as floods and hydropower.

Long – term flow forecasting has been used to describe the methods used to forecast flow in seasonal systems (Villanueva et al, 1987; Druce, 2001), but after the use of climate models (Tucci et al 2002) or empirical and probabilistic relationship among climate variables and flow (Anderson et al, 2001) this forecast has been improved. Long-term forecasting can decrease the uncertainty of the economical evaluation of some commodities related to water resources such as: planning energy price in the system where hydropower has an important share of the production such as in countries as Brazil (~ 91%), Uruguay , Canada, Norway among others; agriculture production for non-irrigated areas; and management of water conflicts.

2.2 Short-term forecast

Short-term forecasting (called also real time) can be done continuously or only after some warning condition. The former is usually done when is required for operational purposes such hydropower and navigation. In hydropower systems usually the planning is done based in the flow statistics and adjusted on monthly, weekly and daily bases. When systems have multiple uses, such as flood control and power production, a waiting volume is used in the management.

Usually flood forecasting is done during flood season after a warning condition is reached in the basin which could be a specific river level, rainfall or climate condition. It can be classified based on the required lead time or basin time of response to rainfall. Some of these floods are: Flash floods, medium and large basin floods.

Flash floods are mainly combination of a meteorological event, usually convective storm-related with a particular hydrological situation such as small basin, steep slope and low infiltration capacity. The forecasting is strongly dependent of the quantitative precipitation quantitative forecast (QPF) since the time between the rainfall and the peak flow is very small for the warning and relief measures during the flood (Krystofowicz, 1995). Georgakakos and Hudlow (1984) mentioned that 25% of communities across US have a lag time of less than 4 h between rainfall and flow from the basin.

Usually Flash floods are related to rural basins but in large cities (São Paulo, Buenos Aires, Barcelona, etc) with the increase of impervious areas the basin time of concentration decreases and increases the peak flow. Managing the urban drainage system of conduits and control the traffic during heavy rainy days during the wet season requires a warning system based in a quick evaluation and forecast. In Brazil, the city of São Paulo uses radar and an empirical relationship between radar frequency and flood conditions of the city's main drainage channel to alert and organize the city traffic.

The main characteristics of this type of flood forecasting is the requirement of the rainfall evaluation for actual and future types steps which requires QPF (figure 5 a).

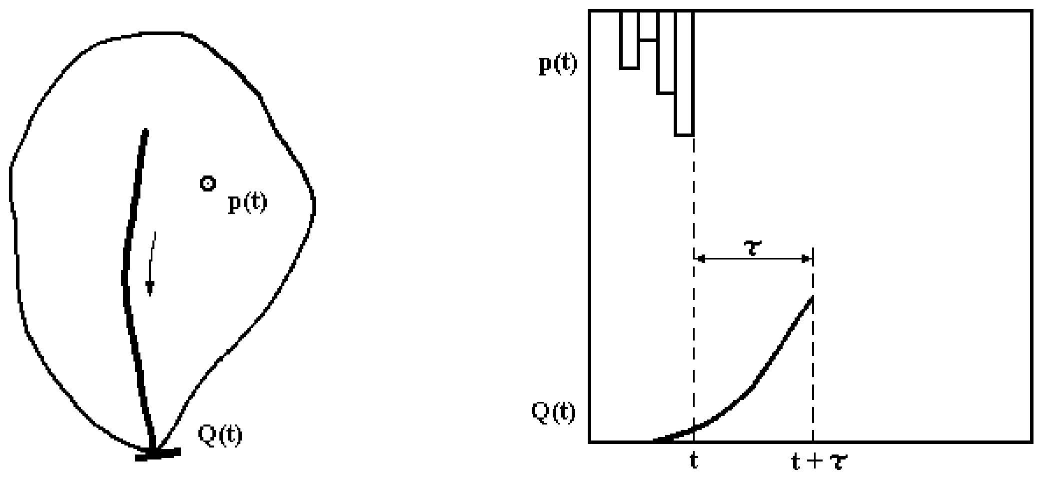
Medium basins flood forecast: This can be achieved through a combination of upstream observation of water level (Figure 5b) and rainfall evaluation in the intermediate basin together with the upstream level or discharge (Figure 1c). In flood forecast modeling there are two main modules: rainfall – runoff sub-basin simulation and river routing. In the scenario of Figure 5b, only the routing module is used and in the scenario of Figure 5c both modules are used.

2.2.1 Quantitative Precipitation Forecast (QPF)

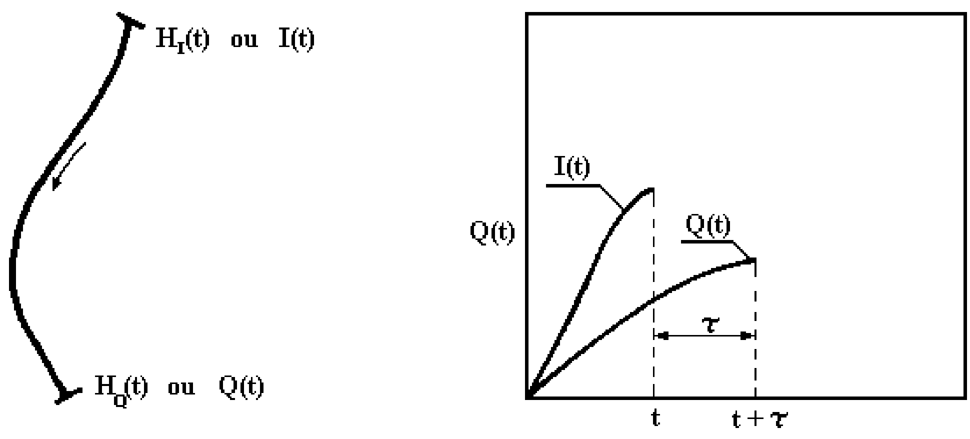
The rainfall used in combination with the hydrologic model in flow forecasting is the recorded rainfall until time step t (Figure 5a). This rainfall is recorded by raingauges and for the time interval between t and t+τ (lead time) the rainfall has to be forecasted. In terms of the forecast lead time it is *nowcasting* for 0 to 3 hours; short-term from 6 to 24 hours; long-term from 3 to 24 months lead time (Collier and Kzyzystofowicz, 2000).

Quantitative precipitation forecast has been developed based in statistical tools, radar measurement, satellite images and meteorological modeling.

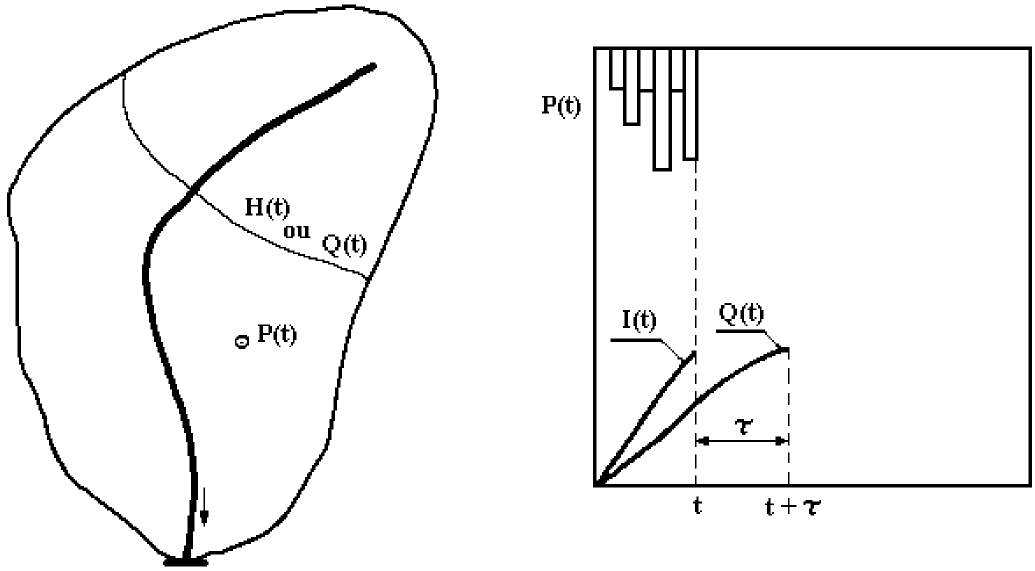
During many years the rainfall forecast was assumed equal to zero in hydrological flow forecast. For basin with a long time of concentration, this does not introduce much error for a small lead time, but for flash flood and for longer lead times, an estimate of the rainfall is an important requirement.



(a) Flow forecast based on the rainfall



(b) Flow forecast based on the upstream flowgage



(c) Flow forecast from upstream flowgage and rainfall on intermediate basin

Figure 5 Scenarios of hydrologic forecast

In hydrology stochastic models were used to forecast the rainfall in conjunction with hydrologic models (Bertoni et al,1992; Mine, 1998) but it did not bring much improvement in the forecast since the rainfall usually does not show significant time series correlation.

Usually the radar and telemetric rainfall allow the evaluation of the meteorological conditions and the storm space distribution and direction. The use of meso-scale weather models to forecast rainfall in a grid comparable to a distributed hydrologic model is one of the combined tools which can improve the estimates (Ibbitt et al 2000).

2.2.2 Hydrologic Models

The models used in the forecast can be classified in *empirical, conceptual or combination* of both. Empirical models do not take account of hydrologic concepts, using mathematical equations without relation to the system's physics. Conceptual models uses the hydrologic concepts in order to simulate the basin behavior.

Conceptual models usually have two main components: (a) *rainfall – runoff module* which transform rainfall in runoff through the water balance in the hydrologic components such as interception, upper soil zone, groundwater and overland flow ; (b) *routing module* which simulates the flow in the rivers and reservoirs. The former is used in the forecasting scenario of Figure 5 a, and the second in the scenario of Figure 5 b. Both are used in the scenario of Figure 5c.

Rainfall – runoff models can be *lumped or distributed.* Lumped models do not usually take into account the spatial variability of rainfall, state variables and model parameters. For small basins this type of model is very useful since it has a simple structure and can be easily updated in parameters or state variables. The distributed models can be distributed by sub-basin or by grids (Figure 6). The advantage of distributed models is that they can take into account the spatial variation of physical characteristics of the basin and rainfall conditions. This type of models has much more difficulty to update its parameters or state variables.

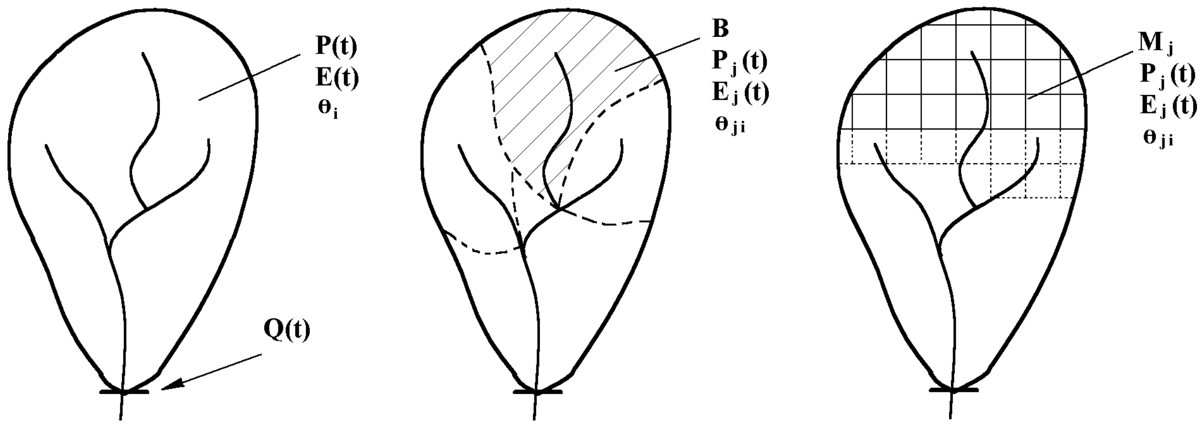
The flood forecasting simulation has two stages: fitting and verification of the model parameters (Figure 7) and flood forecasting which can be done with updating of parameters or/and state variables (Figure 8).

2.2.3 Routing Models

In large rivers where the flow velocity is slow and the intermediate volume is small compared to the main river volume, routing model usually allow small errors on the flood forecasting. Tucci et al (1987) compared different empirical models and they concluded that the model of flow differences presented the best resulted using updating parameters. The type of the equation is the following (figure 9)

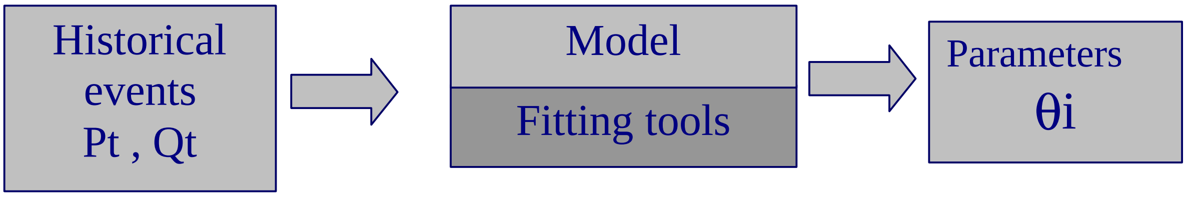
$$\Delta Q_{t+\tau} = a_1 \Delta I_t^1 + a_2 \Delta I_t^2 + ... + a_n \Delta I_t^n + b \Delta Q_t \qquad (1)$$

where $\Delta Q_{t+\tau} = Q_{t+\tau} - Q_t; \Delta I_t^i = I_t^i - I_{t-\tau}^i; \Delta Q_t = Q_t - Q_{t-\tau}$; a_1, a_2,...a_n and b are parameters fitted based in recorded data through minimum square root method. During the forecasting the parameters are updated after the recorded data is received, changing theses values with time t.

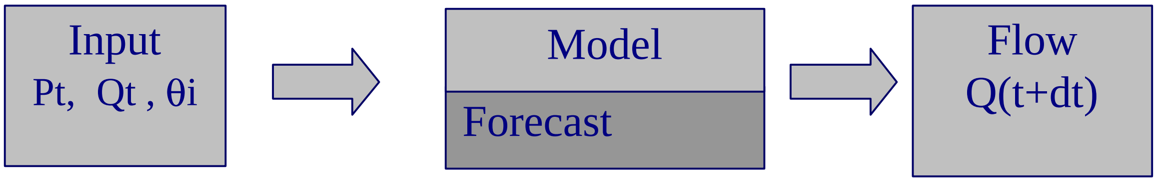


a- Lumped b-distributed by sub-basin c- distributed by grid

Figure 6 Models by discretization



a - Fitting off-line



b – Forecasting without parameter or state variables update

Figure 7 Flow forecasting without update

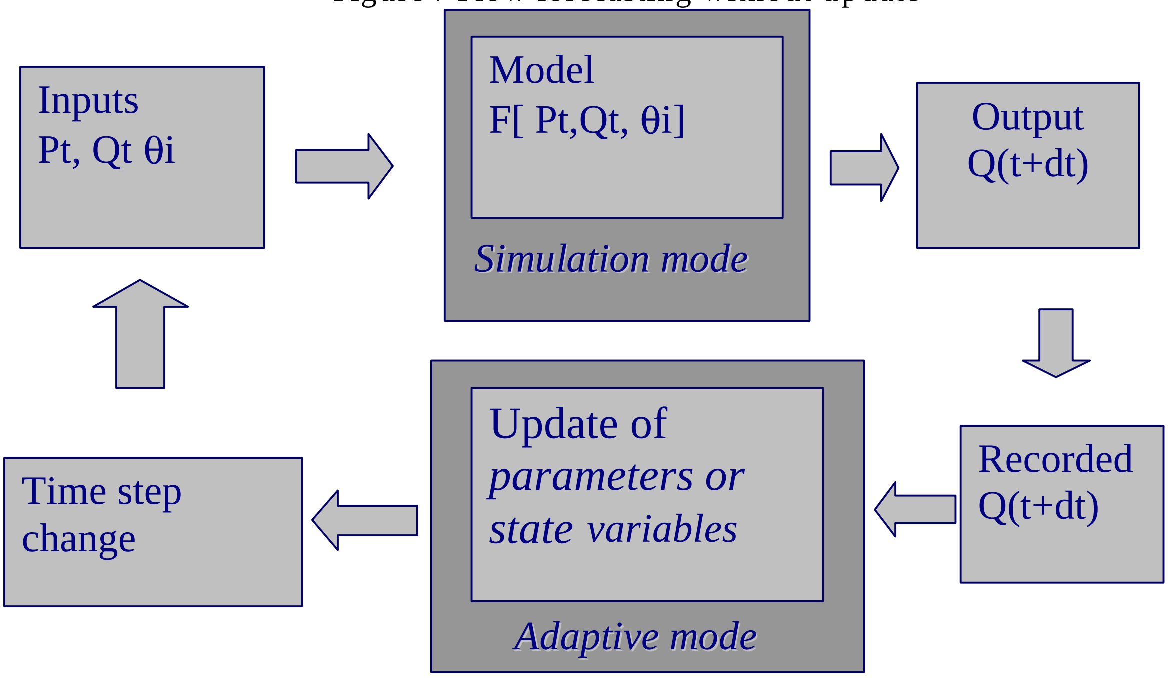


Figure 8 Flow forecast with parameters updating

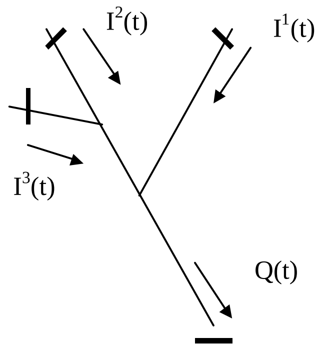


Figure 9 Forecast reaches

This model was used in a few reaches which are presented below:

São Francisco River (Brazil) between Porto Alegre and Manga gauging stations: The river reach extension is 240 km. The upstream basin has 178,500 km^2. Downstream at Manga the basin is 200,364 km^2. The intermediate basin does not contribute a significant volume of runoff since it is semi-arid. Figure 10 presents the forecast with 24 h lead time for about sixty days. The boundaries where calculated for 95% of probability, assuming a Normal distribution.

Rio Paraná at Corrientes : In this simulation there are two reaches upstream of Corrientes in Paraná river: in Paraguay the upstream flowgage is in Pilcomayo (near Assunción); in Paraná river the upstream flowgage is in El Dorado (downstream of Itaipu dam). The River Paraguay joins the Paraná upstream of Corrientes. The basin area in Corrientes is about 2 million km^2.

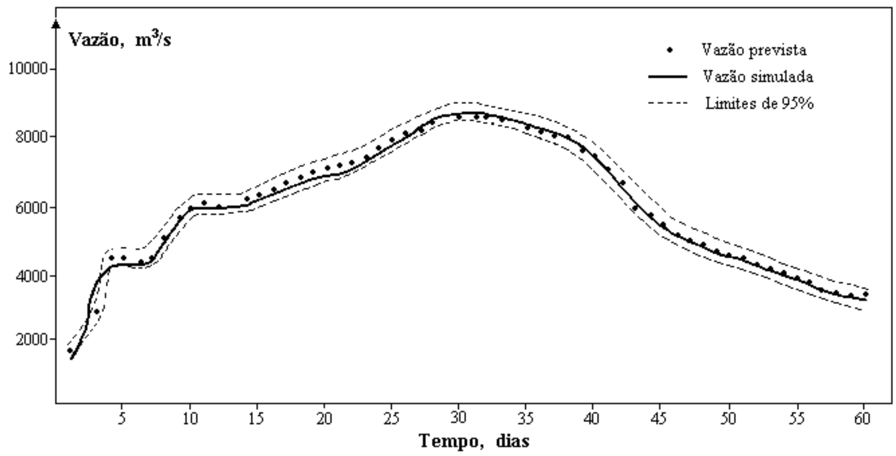


Figure 10 Forecast in São Francisco River at Manga with 24 hours lead time based on Porto Alegre and Manga flow gauge data.

The routing model was applied for some lead time. The model was fitted to two months of record from 1982 and the forecasts of flow were calculated for the flood period of six months in 1983. This was one of the greatest flood of the century. The parameters are updated in real time. Table 3 shows the statistics of the forecast. Three statistics were used in the evaluation (Table 3). R_D is a more realistic statistic since it compare the model result with the alternative of using the same recorded value of time t in the forecast and in R^2 the comparison with the model performance is done with the use of mean as a forecast.

Table 3 Statistics of forecasts for different lead times: River Paraná at Corrientes, using a routing model.

Lead Time	R^2	R_D	Standard error m
1	0,99	-0,187	0,056
2	0,97	0,135	0,086
3	0,95	0,284	0,112
4	0,94	0,461	0,124
5	0,94	0,638	0,122
6	0,92	0,620	0,145
7	0,81	0,328	0,216
8	0,57	-0,240	0,326

$$\mathbf{R^2} = \mathbf{1} - \frac{\sum (Qo_{t+1} - Qc_{t+1})^2}{\sum (Qo_{t+1} - Qm)^2}\ ; \ \mathbf{R_d} = \sum \frac{(Qo_{t+1} - Qc_{t+1})^2}{(Qo_{t+1} - Qo_t)^2}$$

The model of equation (1) is a lumped routing model. For some reaches where the velocity is high this type of model may give poor results. Tucci (1998) presented a non-linear version of a conceptual distributed model for this type of conditions and compared it with a linear version of equation (1) in a river reach of the River Jacuí in southern Brazil where the intermediate flow is negligible, 36 km of extension, slope of 1m/km and a basin of about 14,000 km^2 . The comparison of the results are presented in Table 5

Table 5 Comparison between distributed and concentrated models forecast in Jacuí River (Tucci, 1998)

Statistic	Linear concentrated	Non-linear distributed
Event 1(**)		
R^2	0,75	0,914
R_D	0,840	0,942
Standard error (m^3/s)	40,6	23,5
Event 2		
R^2	-0,14	0,933
R_D	0,698	0,969
Standard error (m^3/s)	32,1	11,1
Event 3		
R^2	0,68	0,920
R_D	0,881	0,958
Standard error (m^3/s)	46,3	25,2
Event 4		
R^2	0,754	0,917
R_D	0,644	0,891
Standard error (m^3/s)	49,4	27,0

* lumped model had parameter updated but the distributed did not
****** fitting event

2.2.4 Rainfall – Runoff Models

Forecasts based on rainfall- runoff modeling are highly dependent on the evaluation and forecast of the rainfall distribution in space and time. The lead time is related to time of concentration of the basin if there is not forecast in the rainfall. When the model is used for flood forecasting the parameter or state variables updating is a procedure used to correct the model for uncertainties related to the information and model capability. There are three main procedures: (i) state variables updating (Kitanidis and Bras, 1978) when the model variables are changed after recorded flow or level arrives; (ii) parameter optimization during real time updating (Tucci e Clarke, 1980; Bertoni et al 1992); (iii) optimization of a explicit objective function (Chander e Shanker, 1984) which requires some simplification of the model.

The rainfall – runoff model IPH II (Tucci et al 1981) has been used in hydrologic applications and its other versions with routing routines (Muskingum-Cunge or Hydrodynamics) where used in some flood forecasting studies which are described below. The model description can be found in Tucci and Porto (2001)

Flow forecasting for Ernestina Dam in Jacuí river: (Brun and Tucci, 2001) The basin in Ernestina dam has a basin area of 1,046 km^2. The reservoir started its operation in 1957 for hydropower. Its volume is 258,6 10^6. m^3 and flood area of 40 km^2. The simulation was done with time step of 6 hours. The inflow was estimated by a reservoir balance. The simulation was done in two steps: (i) fitting of the rainfall-runoff model parameters (IPH II) in a off-line procedure; (ii) flood forecasting with updating of soil moisture and state variable of the overland flow.

The forecasting was done for the wet semesters with lead time of 6, 12, 18, 24 and 30 h (1 to 5 time steps). Three scenarios were studied (a) without updated and without future rainfall (P=0);(b) with updating;(c) with future rainfall known (which is the scenario of the hydrologic error).

Figures 11 and 12 give the fitting of the peak discharge and volume for the same period studied. Figure 13 shows the simulation for 12 hours lead time. These results show the improvement of the forecast with updating procedure. Figure 14 shows the statistics of R^2 and volume balance for various lead times and scenarios of rainfall forecast. It shows that without the knowledge of the rainfall the solution degraded with time.

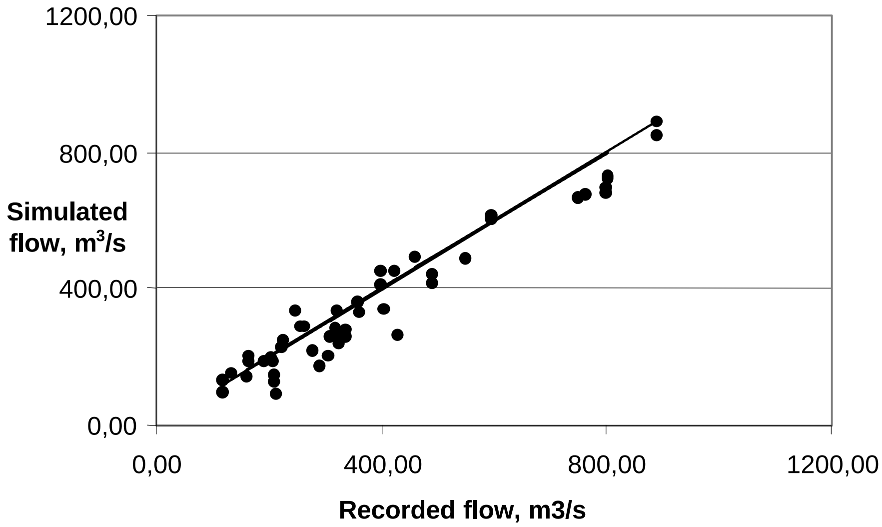


Figure 11 Simulated and Recorded peak flow (Brun and Tucci, 2001)

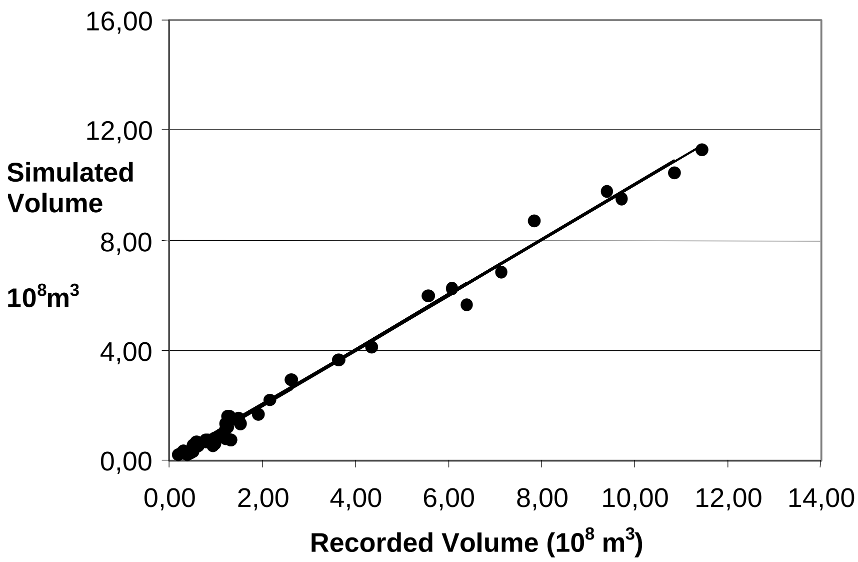


Figure 12 Simulated and Recorded volume (Brun and Tucci, 2001)

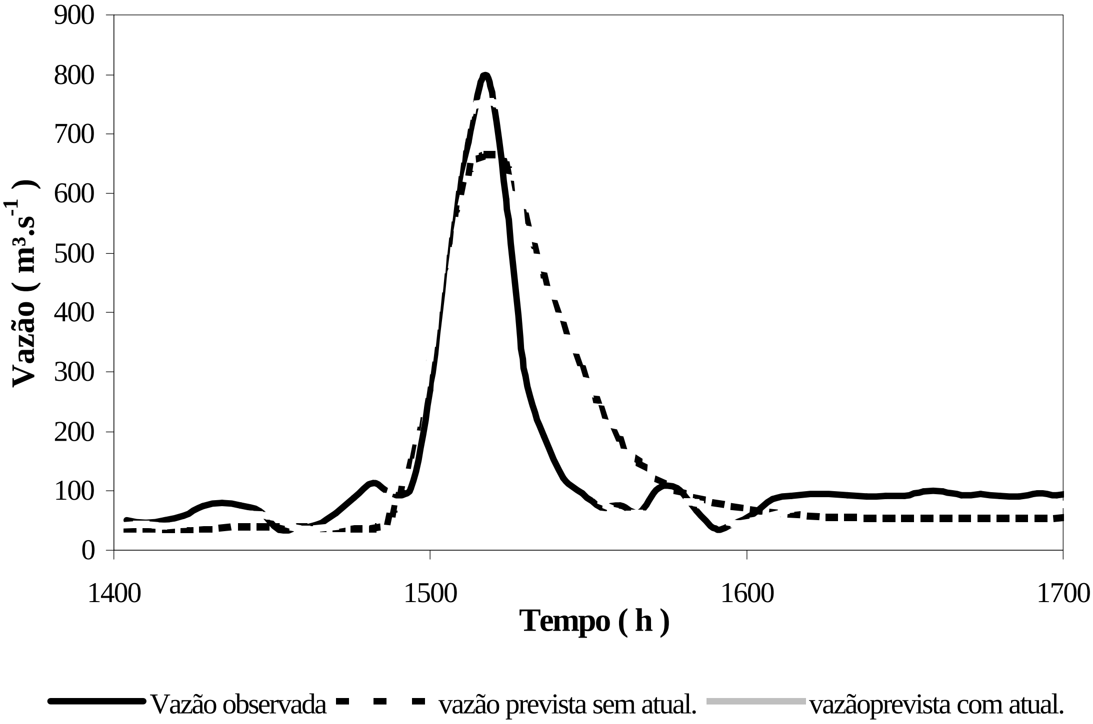


Figura 13 Forecast at Ernestina Dam with 12 h of lead time (Brun and Tucci, 2001)

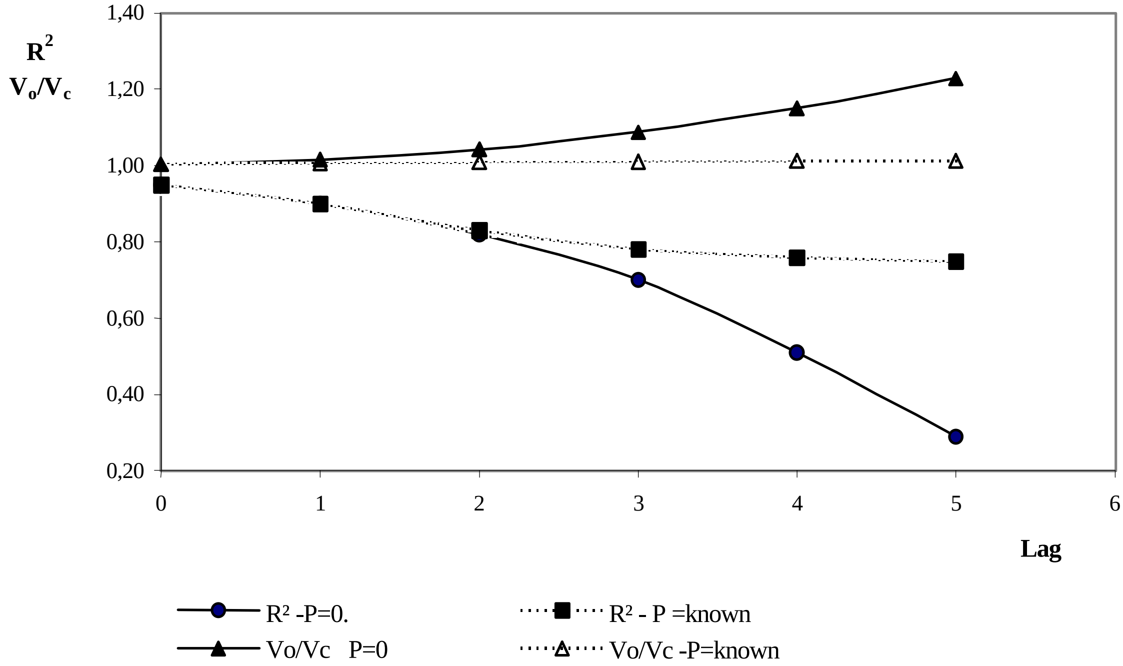


Figura 14 Statistic of Forecast at Ernestina Dam for some lead times (1= 6h) in the wet semester of 1983 with updating (Brun and Tucci, 2001).

2.3 Long – Term Forecast

Long term flow forecasting can be done through statistics tools on seasonal hydrologic behavior or correlation among variables such ocean temperature and rainfall or flow. For instance in a basin where the seasonal conditions is well explained, after the rainy period the flow can be forecasted by the recession curve of the hydrograph in the dry months (Villanueva et al,1987). In systems with long memory where the storage is large and the flow velocity is small the forecast of a few months of lead time is allowed. Tucci and Genz, (1996) showed that in Paraguay River in Pantanal region the hydrographs takes two months to travels from Cáceres (upstream port) to downstream in Porto Murtinho in Brazilian reach of the river.

However, in the basin with low memory (groundwater storage) the capacity of long term flow forecast depends on the climate variable forecast or other variables which has correlation with the climate such ENSO index or others.

Long term flow forecasting can be done through the following:

- prediction of local seasonal statistics : it always forecast the same values every year if the random term is not used;
- stochastic models taking into account the seasonal and temporal correlation : when the basin has a low memory this kind of models gives the same output as the above one;
- empirical models which relates ocean or climate variables with flow with some lead time: this kind of model usually explain only a small part of the flow variation.
- Deterministic climate and hydrologic models : these models are highly dependent on the capacity of the climate model to forecast the rainfall.

Tucci et al (2002) used the four models above to forecast the flow in Uruguay river basin. Uruguay river basin has low memory since its soil depth and the groundwater storage are small. There is no seasonal behavior and the mean flow along the years does not show great variation, but among years the standard deviation is great.

The empirical model was developed with temperature anomalies of Pacific and Atlantic Ocean and flow in Uruguay river basin at Iraí (62,300 km^2). The deterministic forecast was developed with a GCM (Global Climate Model) of CPTEC (Climate Center in Brazil) and a hydrologic model for large basins (Collischonn e Tucci, 2001). The climatic model discretization is presented at figure 16 and the hydrologic model basin representation (10 x 10km) is presented at figure 17. The hydrologic model was fitted based in five flow gages by a multi-objective optimization. The results where verified for other 12 flow gages in a period of tem years. Figure 18 shows the result for one flow gage.

In the rainfall forecasting the CPTEC model presented a bias results which could be corrected by a statistic distribution for each model grid and month. It was done for a period of 4 years (1995-1998) and verified for two years period (2000-2001).

The models tested are:

- <u>statistical model</u>: the model which has been used for forecast in hydropower management;

- rainfall known (PH): this model scenario is used to find the hydrologic error in the forecast;
- empirical model (Model EM: uses temperature anomalies of Pacific and Atlantic with flow;
- CPTEC model + hydrologic model (Model CH): both models without statistic correction for the climate model;
- CPTEC model with correction + hydrologic model (Model CHc): both deterministic models with climate model correction;

The forecast of the deterministic models where done based in five ensembles (out of 25 of the climate model). The results presented is the mean of theses ensembles.

The study was done for two periods:

- period 1: December of 1995 to December 1998 : for forecast and fitting statistics of climate model
- period 2: June of 1999 to October of 2001: used for verification of the models. In this period there where two events which were the filling of Itá and Machadinho Dams. These conditions distorted the recorded discharges of a few days.

Table 6 shows the standard error of forecast for monthly flow for both periods of simulation. The model used is the statistic of flow (also stochastic but with the same result), presented in column 3 of the table. The CH model shows a standard error greater than the statistic model. The CHc which uses correction on the climate model shows a reduction in the standard error. The model PH shows the error of the hydrologic module of the deterministic model.

The same results are presented for the three month mean in Table 7. In this scenario there also the forecast of the empirical model. The reduction of the standard error is greater for these time step mean and the model CHc presents the best improvement (54%). In figure 18 is presented the simulation and change in the reduction of the limits of the statistic model and the reduction of the limits of the ensemble simulated.

Tabela6: Summary of the comparison of the forecast in the standard error for monthly forecast in m^3/s) (Tucci et al, 2002)

Period	Characteristic	Statistic	CH	CH_c	PH
12/1995 to 12/1998		2069	2178	1331	558
6/1999 to 10/2001		1299	1661	1198	_
	Without Itá and Machadinho	1369	1818	1087	_
12/1995 to 10/2001		1785	1976	1276	_
	without Itá and Machadinho	1839	2051	1245	_

Table 7: Summary of the comparison of the standard error of three month mean forecast in m^3/s (Tucci el al 2002)

Period	Characteristics	Statistic	CH	CH_c	PH	EM
12/1995 to 12/1998		1570	1736	733	508	1153
6/1999 to 10/2001		808	1243	771	-	1154
	without Itá and Machadinho	900	1266	638	-	1086
12/1995 to 10/2001		1295	1541	750	-	1157
	Without Itá and Machadinho	1373	1587	701	-	1134

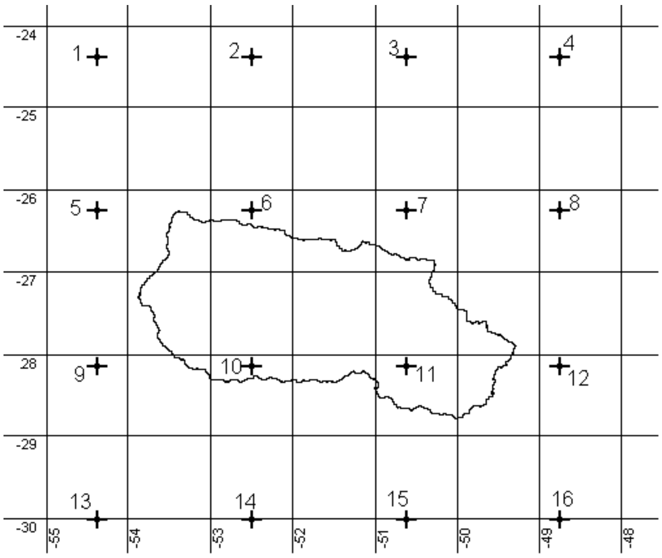


Figure 16 Climate model discretization (Tucci et al, 2002)

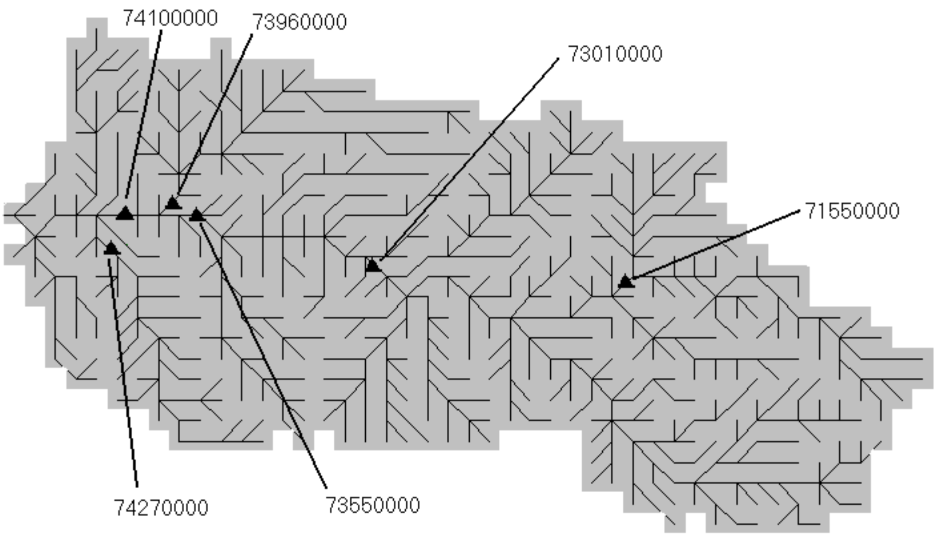


Figure 17 Hydrologic Model representation and the flowgages used in the fitting (Tucci et al, 2002)

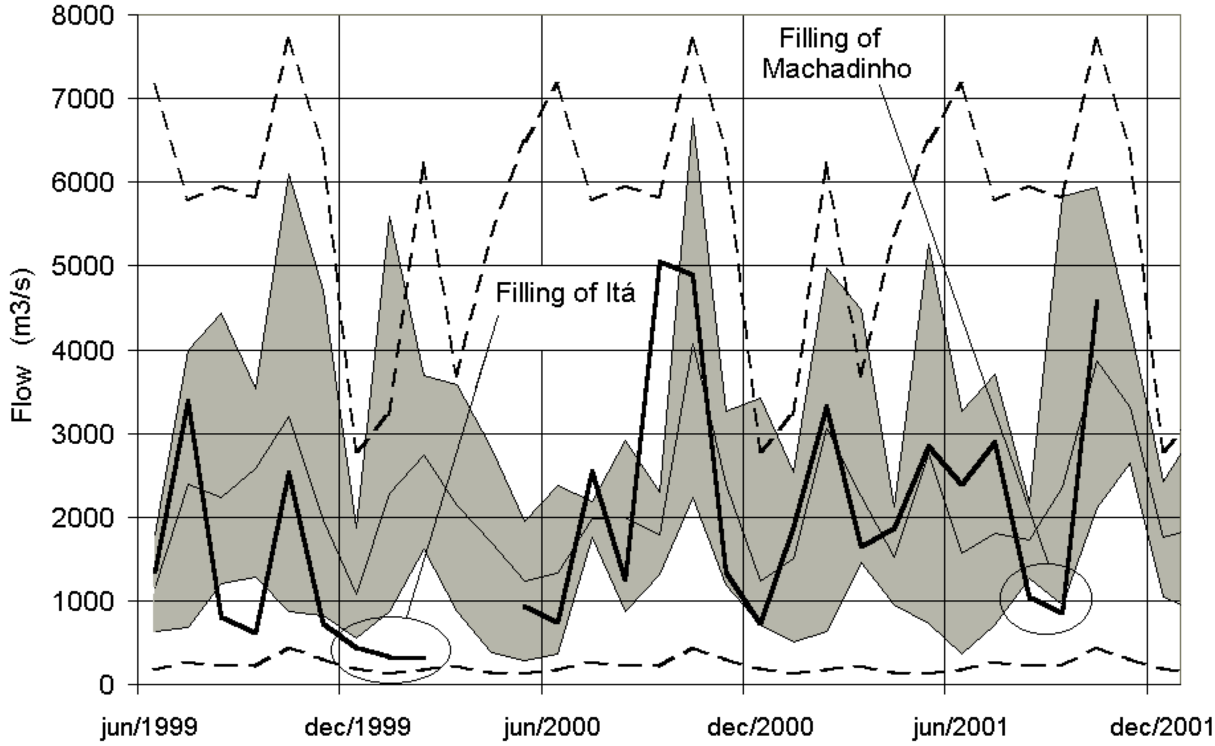


Figure 18 Range of predicted monthly flows (shaded band) compared with range of observed monthly flows in the historic record (maximum and minimum shown as broken lines). The heavy black line shows observed flow (Tucci et al, 2002)

3. FLOOD FORECASTING AND DAM MANAGEMENT

3.1 Use of dams

Dams are usually designed for one of the following purposes: power production, water supply, irrigation, navigation and flood control. Usually there are some conflict in the use of the dam for water supply, irrigation and energy with flood control.

Traditional flood control dams are designed based on a limit of outflow discharge at the outlet from the dam and on the useful volume Vu necessary to damp the flood. The project should have overflow outlets that will allow all the inflow to flow downstream of the dam, up to a discharge which is below a value which could flood the plains. After this limit the reservoir uses the volume to damp the flood. The operation of this system may become even more efficient if there is a forecasting and warning system that enables decisions regarding the use of the useful volume throughout the flood.

3.2 Flood control and hydropower

Assuming a reservoirs was built for hydropower production, there is a potential conflict with flood control, since the hydropower developments aim to keep the water level as high as possible during the rainy period (greater hydropower generation), with a consequently smaller empty volume. Under these circumstances, the dam may worsen flooding conditions, both upstream and downstream from the dam.

Most hydropower developments were not designed to contain floods. Since building reservoirs tends to regulate downstream discharge, the low risk floods no longer occur due to damping caused by the reservoir. Thus, a larger amount of risk areas is occupied, because of the structures and the development of flat areas. When the higher risk floods occurred, that the reservoirs were not prepared to dampen, large floods occurred, with conflicts between the population and the developments.

For instance, during the 1970s, several relevant facts occurred in the Brazilian electricity sectors which produced a change of attitude as regards hydropower dam operation. First two dams broke on Pardo river in the state of São Paulo. The rupture of the first due to operational and design failure led to the break in the second one due to the rupture wave. The second fact were the floods in São Francisco river, in 1979, which produced impacts downstream and strong public pressure. For this reason, criteria were established for these systems, so that, besides producing power, they could contain part of the floods by creating a waiting volume.

Figure 19 shows the stages and the definition of the waiting volume of a reservoir. This volume is kept free to receive the flood volumes and reduce streamflow downstream, trying to satisfy the upstream and downstream constraints.

There are several methodologies to estimate this volume, based on the statistics of the historical series of dam flows. The methods used in the Brazilian electricity sector have been the Volume x Duration Curve Method (adaptations of the methodology presented by Beard, 1974), or the method of critical pathways (Kelman et al. 1983). The former uses the historical series observed and the latter, the series of streamflows generated by a stochastic model. Both methods determine, statistically, the waiting volume that should be maintained by the reservoir during each day of the rainy period for a given risk of analysis.

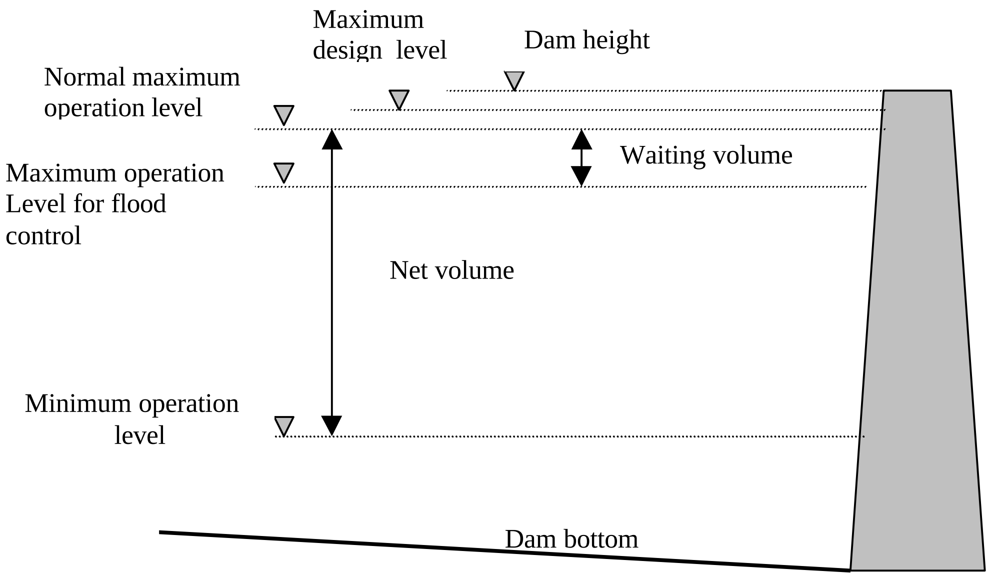


Figure 19 Operation levels in a Hydropower

These procedures do not take into account the information existing in the basin during the flood period. For basins where seasonality is not clearly defined, the model may underestimate or overestimate the waiting volume with significant losses. On the one hand losses due to flooding and, on the other, due to loss of power generated.

The use of information that exists in the basin involves the flow forecasting of inflows to the reservoirs. For a safe forecasting system the waiting volume could be reduced if the inflow can be predicted in advance, and therefore, the volume could be increased according to need, taking downstream constraints into account.

3.3 Flood forecasting and dam operation

Flood forecasting and dam operation has three main modules: (a) rainfall forecast; (b) flow forecast from rainfall; (c) dam operation optimization (figure 20). The rainfall forecast has been developed in meteorology and are important to leave more lead time for the dam operation. The flow forecast is developed in hydrology modeling as described in chapter 2. The dam operation has to take into account the upstream and downstream restrictions due potential flood impacts, water balance in the reservoir and the capacity of the spillway and turbines. The objective of the optimization is to minimizing the waiting volume and do not break the dam restrictions.

Silveira (1996) used a flood routing forecasting model together with optimization procedure in order to calculate the waiting volume for Sobradinho in São Francisco River in Brazil. Mine (1998) used a combination of rainfall forecasting, routing and rainfall-runoff model together with the optimization method of Silveira to forecast and operate the Foz de Areia Dam in Iguaçu River (see description of the case study below).

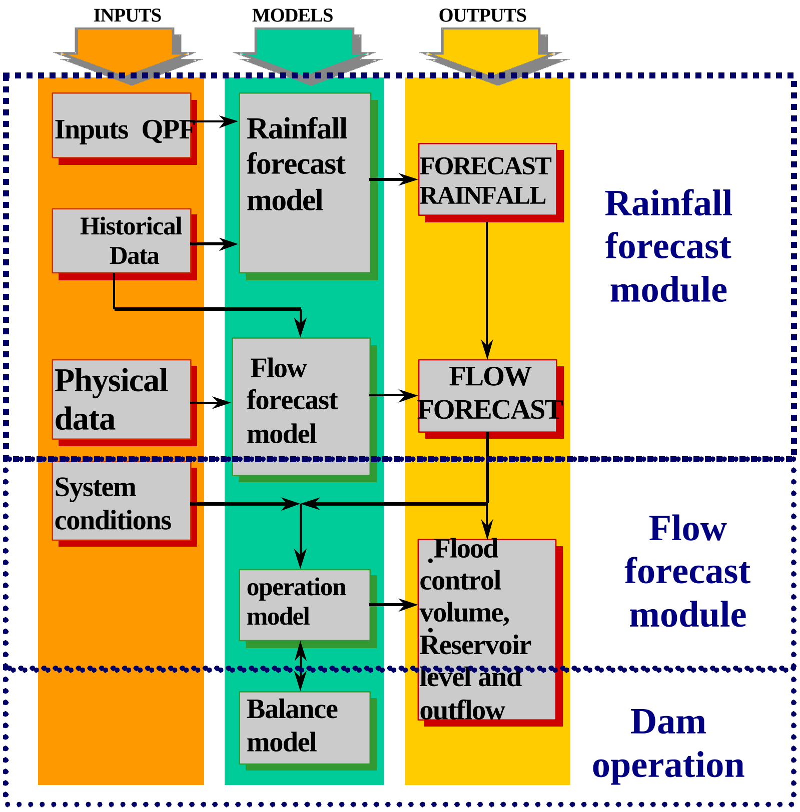


Figure 20 Integration of Rainfall forecast, flow forecast and Dam operation (Mine, 1998)

3.3 União da Vitória and Porto União flood conflict

In recent years (1983 and 1992), União da Vitoria and Porto União in Brazil suffered severe floods, after a long period (50 years) of normal to low floods. The economic losses to some industries, businesses and homes created a depression and psychological impact on most of the population. The population believed that the main cause of the floods was the impact of Foz do Areia Dam, a Hydropower Plant located downstream from the city (figure 21). It created a major conflict lasting more than ten years between the population and the State Power Company.

Cities and flood conditions

The cities of *União da Vitória* and *Porto União* (Figure 21) are located on the banks of the Iguaçu River, State of Parana, Brazil, where, for about 50 years (1930 to 1982, see figure 2), only low or medium floods occurred. During the early 80's the *Foz do Areia* Dam was built, about 100 km downstream.
The backwater curve for the dam design flood may reach the cities, depending on its operation water level. While designing the dam, two alternatives were studied to cope with this influence, protection levees or operational rules to prevent the rise of flood levels in the cities. The first operational level chosen was 744 m. Studies carried out by Parana State Power Company (owner and operator of the dam) in 1982 and 1985, showed that 744 m was a high level, and recommended 742 m at first, and later 741.5 m. In 1983 an extreme flood occurred, which caused severe economic losses (table 2). The

flood level was the highest in 107 years (estimated return period of 170 years, and a duration of 62 days), and the estimated losses were U$S 78,1 millions.
At that time the population blamed the Power Co. for the Dam operation and high flood levels. The flood recurrence was estimated as being about 1000 years, using continuous records (1930-1983). This calculation, however, did not take historical marks (figure 2) into account, and this led to an overestimate of the return period. When historical marks (found after some research done by local personal) were included in the statistical analysis, the return period decreased to 170 years. In 1992 there was another flood, smaller than the last, but of similar magnitude and impact (return period of 30 years, duration of 65 days and US$ 54,6 millions in losses). As the people had been told that the risk was very small, and, in less than ten years, the cities were flooded again, their reaction was very strong, and distrust towards previous studies and official statements was widespread.

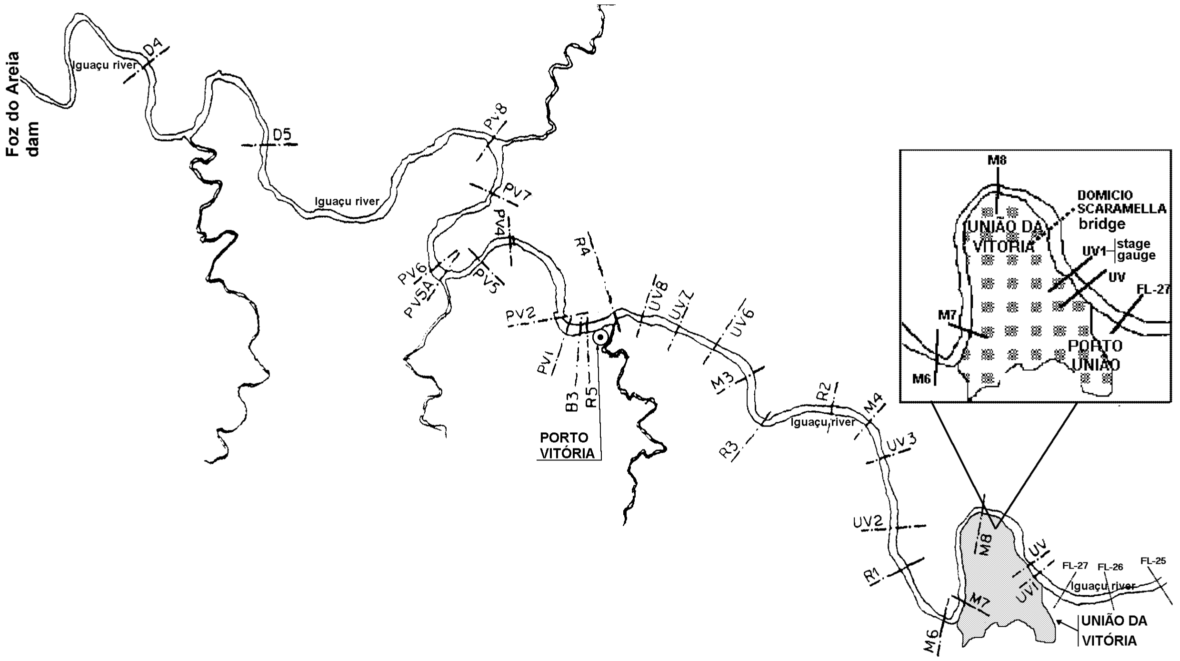


Figure 21 Iguaçu River at Foz de Areia and União da Vitória (Tucci e Vilanueva, 1997)

A NGO (Non Governmental Organization) called *CORPRERI* (Regional Permanent Commission for Flood Prevention of Iguaçu River) was created by the population to deal with the flood problem. This organization has been the representative of the cities in the discussion with the Parana State Eletric Power Company (COPEL). Tucci and Vilanueva (1997) presented the alternative study described here.

Simulation of Iguaçu River with Dam operation

In order to answer most of the questions asked by the population, a hydrodynamic model was applied (Tucci, 1978), taking into account the characteristics of the flood valley and main channel in the reach between the cities and the dam. Usually the flood plain is represented only by a storage function assuming an infinite roughness. Since depths in the flood plains can reach more than 5 m, there are flow dynamic effects in this part of the river system. The model used to describe the dynamic effects was the Lateral Distribution Method described by Wark et al (1991) and Villanueva, (1997) to

compute the conveyance and velocity distribution coefficient for each section. The boundary conditions used for the simulations (fitting and scenarios analysis) were the discharges in the cities (upstream boundary) and the level at the dam (downstream).
Model fitting was performed comparing simulated and recorded levels at União da Vitória (section M8, figure 22) and discharge and level at Porto Vitória (figure 21) for the 1983 and 1992 events. The discharges in those floods varied between 240 m^3s^{-1} to 5,000 $m^3.s^{-1}$ (figure 23).

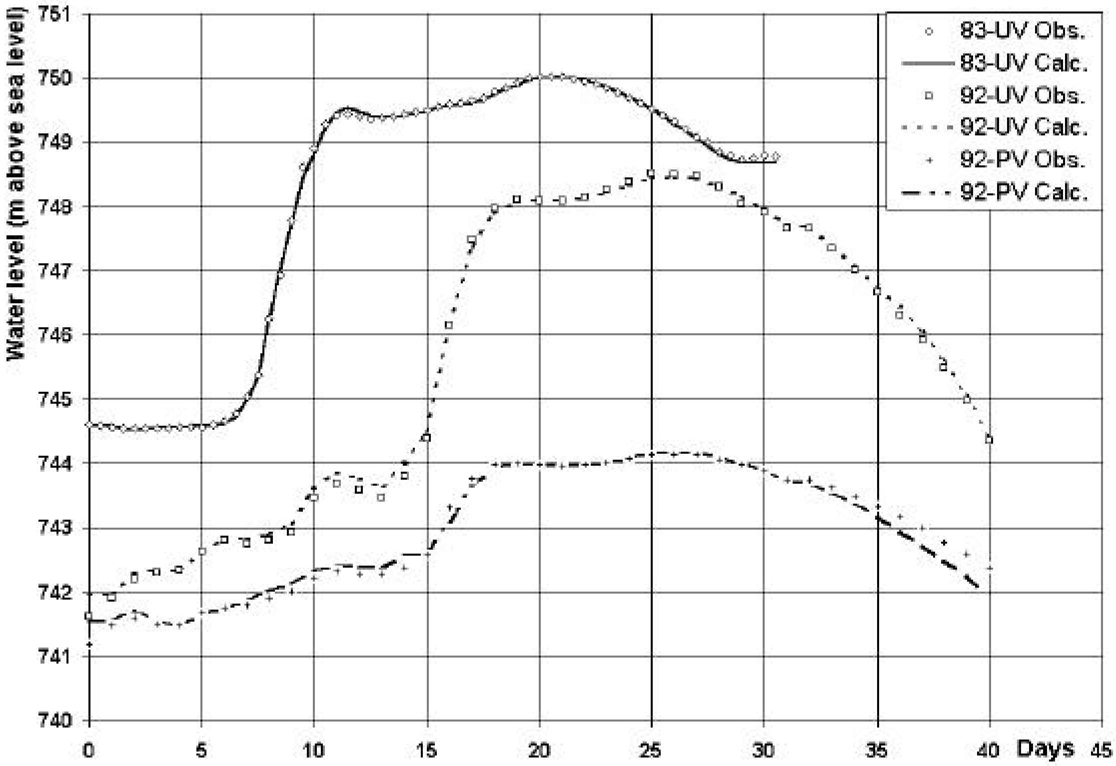


Figure 22 Simulation of the floods at União da Vitória (Tucci e Villanueva, 1997)

Flood impacts

The first question to be answer was the influence of the dam operation in the cities' floods and the causes of the high levels. Some other questions were raised by the population related to other solutions for flood alleviation such as improvement of channel conveyance and bridges impacts.
Using the two major floods, 1983 and 1992, two different downstream boundary conditions were used in the model: (i) the actual operational levels at the time of the flood; (ii) safe operational level at the dam, established at 741.50 m by previous studies.
The results showed that the operational procedure in 1983 increased the flood levels at the cities by 14 cm during the first part of the flood but did not show any influence during the flood peak. During the 1992 flood the operational levels at the Dam did not influence the flood levels at the cities.

Figure 23 União da Vitória Flood Map (Tucci e Vilanueva, 1997)

Flood Control Measures

Non-structural measures: The non-structural measures developed were: (i) Flood zoning and (ii) flood forecasting.

Flood zoning: In order to define the flood levels for the land use map, levels were calculated for each of the available cross sections, for several return periods, using the mathematical model. A 1:2,000 map was used. Flood level lines were established for 5, 10, 50 and 100 year return periods (considering historical marks). Figure 9 shows a view of the flood map.
Three zones were established for the flood control areas: (i) *Preservation area:* below 744.50, reserved for environmental protection and parks. This area was purchased by the Power Company that owned the dam. A park was constructed in the urban part of the land after an agreement between the communities and the Company; (ii*) Water resources protection area:* between the former area and the 10-year levels. Remove public

building such as school and hospital from this area; progressive taxes to be imposed for any use but the recommended ones: parks, sport fields, agriculture, and others. A tax deduction was recommended for these uses; (iii*) Low density area*: less restrictive than the former, but needing special care and protection against flood damage in the buildings.

***Flood forecasting**:* The flood zoning must also have a real time flood forecasting system, working together with the Civil Defense Authority. The proposed actions were the following: (i) definition of an alert system: mathematical model, forecasting range and alert steps; (ii) County Civil Defense Authority must be created, so as to be prepared to act, with well pre-established plans, during the floods; (iii) emergency plans must be prepared for the different parts of the city.

Three forecasting conditions have been recommended: (i) *watch condition*: from that level on, the behavior of the river must be accompanied carefully. Real time forecasting begins at this condition; (ii) *alert condition:* when the 744.0 m level is to be reached within 12 hours; (iii) *emergency condition:* when the 745.5 m level is to be reached within 12 hours;

Mine (1998) developed a model to forecast the flow and operate the dam taking into account the upstream and downstream restriction. The flow forecasting for União da Vitória (25.000 km^2) was done by an empirical model, the flow from the basin between União da Vitória and Foz do Areia (5.000 km^2) was simulated by the rainfall runoff model IPH II (Tucci et al 1981) and the flow in the river reach was simulated by the hydrodynamic model used in the other simulations. In the real time flood forecasting there were the following options: (i) zero rainfall for the lead time; (ii) rainfall forecasted by empirical model; (iii) known rainfall.

This flood forecasting model was integrated to a operation model based on the upstream and downstream flood restrictions. The forecasts were done with a lead time of 24 hours and updated every 4 hours. The Dam level has to be below 742,0 m due to upstream restrictions. In figure 24 are shown the results for the 1983 flood. It can be seen that the actual operation used during this event was above the limit for a few hours due to the amount of water from the tributaries and operation brought the level down very fast. Using the operational system with well-known rainfall, the operation is more efficient because it stays below the restriction and increases the dam level after the risk, improving energy production.

Structural measures: The structural measures studied were: (i) changing river characteristics and; (ii) levee protection along the city.

Some of the potential alternatives to decrease the flood levels were to modify the characteristics of the river at some critical reaches downstream from União da Vitória. These river modifications are: straightening and enlarging some bends, duplication of the channel and by-pass of the curve immediately downstream from the cities, and even deepening the channel stretch between the Porto Vitoria rapids and the cities (about 50 km long).

As a general conclusion of the analysis, it can be said that, even though the critical points contribute to increasing the levels, none of them is alone responsible for the high levels. Not even their joint effect increases the floods critically. The main problem is the lack of river flow conveyance for the floods in the sections along the cities and some contractions. Other factors such, as river bends and contractions far downstream have little influence.

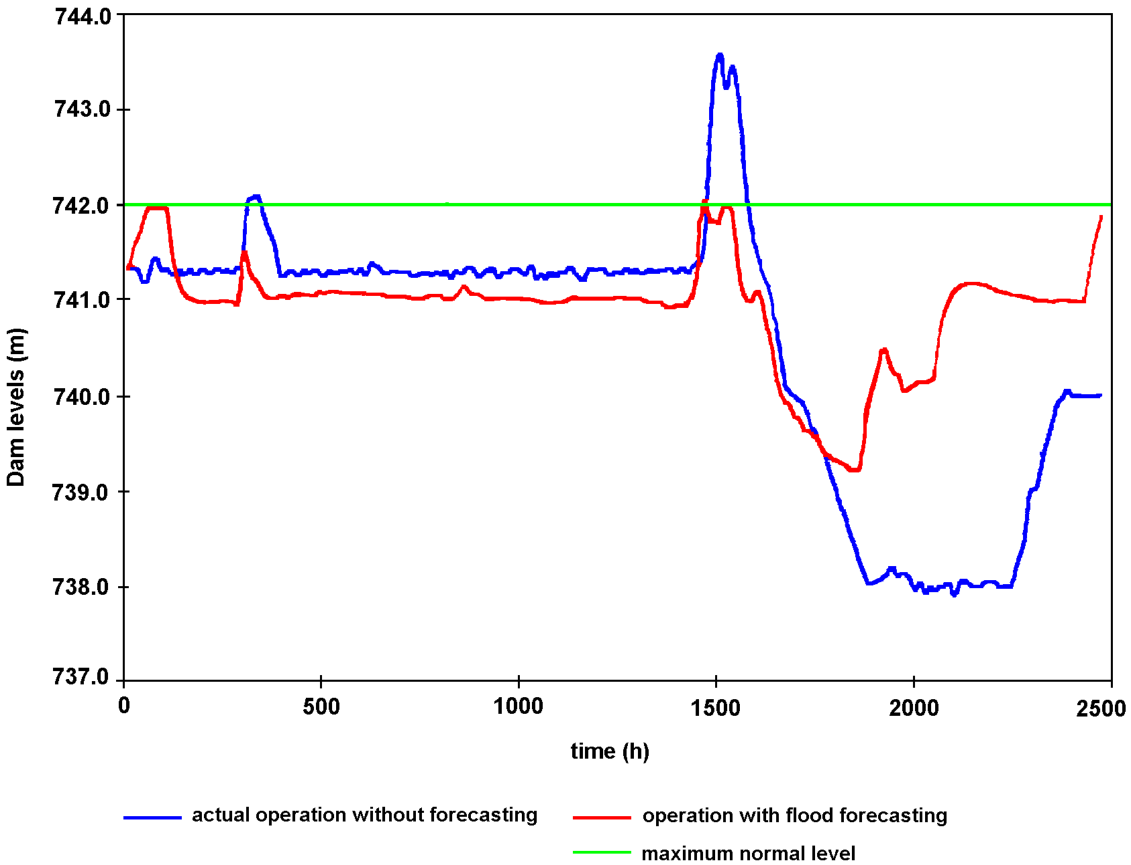


Figure 24 Levels at Foz do Areia actual operation and with flood forecasting and operation management model (Mine, 1998)

In fact, discharges for both floods considered (1983 and 1992) were very high, and the 1983 flood was the historical maximum. The historical records, however, show that flood levels of a similar magnitude had already occurred. In a previous study, JICA (1995) considered the levee alternative as a control measure and recommended a feasibility study for this solution.

Most of the structural solutions studied, relating to the river conveyance, were not feasible due to the high intervention cost. A feasibility study was recommended for this system and concluded that this was the acceptable combination of protection and cost. The levee system would be constructed protecting the areas above a 10-year return period.

This engineering system will change the cities characteristics and protecting it from low risk floods. The analysis done in this study led to some questions that the population had to answer before it decided in favor of this system:

- a system of that kind must be under the jurisdiction of the city authorities. This implies a cost that needs to be supported by taxes. The investment maintenance costs are high which may create a major impact on the city economy.

- the protection will benefit mainly the areas between the 10-year flood level and the 1983 flood level. Will the whole population be willing to pay for the benefit to only part of them?
- the impact of the levees on the flow conditions upstream and downstream of them must be carefully studied;
- not only the technical and economical aspects of the levees must be considered, landscape and urban environment issues also need to be included. Questions such as this have to be answered: Would you like to live with a 6 m wall, all along the city, which will protect levels above 10 year flood or create a new urban development toward more safe areas and use zoning measures?

Until now the decision has been to use non-structural measures in order to cope with flood conditions. But for political reasons there are pressures to show some structural measure that in this case represents high costs.

The analysis of the problem leads to some interesting conclusions regarding its origin and development. A long period without severe floods induced a false feeling of security in people, who began settling in the flood valley. Also as a result of this false security, no flood protection measures or planning were adopted. When the 1983 flood showed that there was an actual risk, it was disregarded, based on seemingly dependable (50 years data) statistical analysis. This analysis, however, did not take into account existing and very valuable information (the historical flood marks). When another severe flood came along in 1992, the population was upset, and distrusted the technical studies. This problem was aggravated because of communications problems with the responsible institutions. It must be stated that, except for the statistical analysis, the existing technical reports were basically right in their diagnosis and conclusions.

The lack of flood protection planning and preventive measures has caused losses evaluated at about U$S 150 millions, and the solution will have a similar cost, not to mention the indirect impacts in both cases.

Several decades without severe floods are not an unusual situation, it is logical that floods with high return periods seldom occur. The consequences of long lags between this kind of floods are also common: occupation of the river valley and disregard for protection measures.

4. CONCLUSION

This paper presents some experience on hydrologic components of flood forecasting and water resources management. Flow forecasting is of increasing economic importance in a society where this information can be used for reducing flood damage, for decreasing the costs of navigation, for forecasting agricultural production and in particular for determining energy prices. One could say that flow forecast is an important commodity for decision makers in the water resource developments.
The main benefits of the forecast are:

- decrease of the uncertainty of record hydrological series in water resources management;
- improve the operational capacity of the dams which increases the economical benefits and its security;
- decrease the flood damage as one the of the main non-structural measure for flood control;
- allow sazonal planning and management of the water uses and conservation based in the long term forecast.

The development of the science for practical use in this field requires a great deal of integration of knowledge among different scientists and professional. The great potential of some tools are only used when in its integration for a problem solution. For instance in Brazil energy market depends highly on hydropower but it has some diversification with thermal plants, among others, but usually are more expensive. This climate dependency requires models which integrate climate, hydrologic, system optimization for energy and price formation models. It requires different kind of professional which have various background and nomenclature. The challenge is the problem and all these scientist and professional working together requires an important technical and scientific coordination and many different opportunities of scientific development out of the specific of each field. Flow or Flood forecast is one part of the problem but is the input of various water resources development and conservation.

REFERENCES

ANDERSON, M.L.; KARVAS, M.L.; MIERZWA, M.D., 2001. Probabilistic/ensemble forecasting: a case study using hydrologic response distribution associeated with El Nino/Southern Oscillation (ENSO). Journal of Hydrology. V249 p134-146.

BEARD, L.R. 1974. *Flood Frequency Techniques*. Austin: Center of Resources University of Texas, Austin.

BERTONI , J.C.; TUCCI, C.E.M.; CLARKE, R.T., 1992. Rainfall-based real-time flood forecasting. Journal of Hydrology, 131 (1992) 313-339

BRUN, G.W. ; TUCCI, C.E.M. 2001. Previsão em Tempo Real do Volume Afluente ao reservatório de Ernestina. RBRH. V6 n.2 Abr/Jun.p73-80

CHANDER, S.; SHANKER, H., 1984. Unit hydrograph based forecast model. Hydrol. Sci. J., 20: 279-291;

COLLIER, C. G.; KZYZYSZTOFOWICZ, R., 2000. Quantitative precipitation forecasting. Journal of Hydrology 239 (2000) 1-2

COLLISCHONN, W.; TUCCI, C. E. M. 2001b Hydrologic simulation of large basins (in Portuguese) *Revista Brasileira de Recursos Hídricos*. Vol. 6 No. 1.

DESBORDES, M; SERVAT,E. (1988). 'Towards a specific approach of urban hydrology in Africa' In: Hydrologic Processes and Water Management in Urban Areas. Proceedings Duisburg Conf., Urban Water 88, April 231-237.

DRUCE, D.J., 2001. Insights from a history of seasonal inflow forescating with a conceptual hydrologic model. *Journal of Hydrology* Volume 249 p102-112.

DUNNE, T. (1986) 'Urban Hydrology in the Tropics: problems solutions, data collection and analysis' in: Urban Climatology and its application with special regards to tropical areas. Proceedings. Of the Mexico tech Conf. Nov. 1984 World Climate Programme. WMO.

GEORGAKAKOS, P.K.; HUDLOW, M.D., 1984. Quantitative Precipitation Forecast Techniques for use in Hydrological Forecasting Bull. Am. Meteorol.Soc.65, 1186-1200.

HAMILTON, S.K. SOUZA, O.C. COUNTINHO, M.E., 1995. Dynamics of Floodplain inundation in the alluvial fan of Taquari River (Pantanal, Brazil*), Proceeding of the International Society of Theoretical and Applied Limnology*, 1995 Congress, São Paulo Brazil,

IBITT, R.P.; HENDERSON, R.D.; COPELAND, J.; WRATT, D.S., 2000. Simulationg mountain runoff with meso-scale weather model rainfall estimates: a New Zealand experience. Journal of Hydrology 239 (2000) 19-32

IPCC(2001) Climate Change 2001: Impacts, Adaptation and Vulnerability. A Report of Working Group II of Intergovernmental Panel on Climate Change.

KITANIDIS, P.K.; BRAS, R.L., 1978. Real Time forecasting of river flow., Department of Civil Engineering, Cambridge, Rep. No 235. Massachusetts Institute of Technology.

KRZYSTOFOWICZ, R.; 1995. Recent advances associated with flood forecast and warning systems. Ver. Geophys. Vol 33 Suppl.

JICA, 1995. *The master study on utilisation of water resources in Parana State in the Federative Republic of Brazil.* Sectoral Report vol H- Flood Control.

MINE, M., 1998. *Método determinístico para minimizar o conflito entre gerar energia e controlar cheias.* Tese de Doutorado . Instituto de Pesquisas Hidráulicas. UFRGS.

RUITER, W. (1990) 'Watershed: Flood protection and drainage in Asian Cities'. Land & Water In'l 68:17-19.

SILVEIRA, C.A C. 1996. *Previsão de Volumes de Espera em Tempo Real*, dissertação de mestrado Instituto de Pesquisas Hidráulicas. UFRGS december.

STEFFEN, 1988. Previsão de cheia a curto prazo em trechos de rio: modelos não-lineares. Dissertação de mestrado. IPH/UFRGS.

TUCCI, C.E.M., 1978. Hydraulic and Water Quality Model for a River Network, PhD Dissertation, Civil Engineer Department, Colorado State University, USA.

TUCCI, C.E.M. Modelos Hidrológicos. Editora da UFRGS. ABRH. 652p

TUCCI, C.E.M.; CLARKE, R.T. 1980. Adaptive forecasting with a conceptual rainfall-runoff model. Hydrological Forecasting. Proc. Oxford Symp., IAHS n.129 pp425-454

TUCCI, C.E.M.; GENZ, F.; 1996 Previsão e controle de inundações. in: Hidrossedimentologia do Alto Rio *PARAGUAY*, Instituto de Pesquisas Hidraulicas, UFRGS, FEMA/MT Fundação do Meio Ambiente do Mato Grosso, SEMA, Secretaria do Meio Ambiente do Mato Grosso do Sul, Ministério do Meio Ambiente.

TUCCI, C.E.M.; LANNA, A E; SIMÕES LOPES, M; ENDRES, L, l987,Comparaison entre Modeles de Prevision de Crues, Revue Hidrologie Continentale, vol.2 e n.1.

TUCCI, C.E.M.; ORDONEZ, J.S.; SIMÕES LOPES, M. 1981. Modelo Matemático Precipitação-Vazão IPH II Alguns Resultados. *Anais* IV Simpósio Brasileiro de Recursos Hídricos. ABRH. Fortaleza.

TUCCI, C.E.M.; PORTO, R.L., 2001. *Storm hydrology and urban drainage.* In: Urban Drainage in Humid Tropics. IHP. UNESCO Capítulo 4 p69-28

TUCCI, C.E.M; VILLANUEVA, A , 1997. Controle de enchentes das cidades de união da Vitória e Porto União. CORPRERI, 117 p.

TUCCI, C.E.M.; DIAS, P.S.; CLARKE, R.T.; COLLISCHONN, W. 2002. Previsão de médio prazo de vazões afluentes a reservatórios no rio Uruguai. IPH/IAG/ANEEL 150 p.

VILLANUEVA, A O N, 1997. *Dynamic floodplains simulations: compound channels and wetlands.* PhD thesis IPH-UFRGS (in portuguese)

VILLANUEVA; ZAMANILLO, ED.; TUCCI, C.E.M 1987. Previsão de Vazão para Irrigação. VII Simpósio Brasileiro de Recursos Hídricos Anais Vol 1 p536-549.

WARK, J. B. ; SLADE, J.E. RAMSBOTTOM, D. M., 1991. *Flood Discharge Assessment by the Lateral Distribution Method.* Report SR 277, Dez 1991. Hydraulics Research Wallingford

WHO (1988) 'Urbanization and its implications for Child Health: Potential for Action'. World Health Organization. Geneve