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Real-Time Flood Forecasting by a Hydrometric Data-Based Technique

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1. INTRODUCTION

Flooding, an age-old global problem, has been increasing at a worrisome pace in recent years. However, natural flooding of large areas did not create more dangerous situation in a prehistoric world. With the expansion of anthropogenic activities, there is destruction of natural drainage systems and with increasing urbanization there is reduction in the opportunity for rainwater infiltration, resulting in an increase of surface runoff potential, and thereby increasing the loss of lives and economic damages due to floods. Further, global climate change induced extreme rainfall events trigger flash floods in both urban and rural watersheds making the severity of flood hazards many fold. According to the Report of the Sub-Group on Flood Management for the XI Five Year Plan (2007-2012), Central Water Commission, Ministry of Water Resources, Government of India (CWC, 2006), on an average, 7.55 million hectares of land in India is affected by flood annually, of which 3.54 million hectares is the cropped area. Besides these, 1589 lives and 94,839 heads of cattle are lost in addition to damaging 1.22 million houses annually. The value of damages to crops, houses, and public utilities alone accounts for about Rs. 1805 crore (US $361 million) per annum. Some recent instances of such type of devastating floods could be the 2008 Bihar flood in the Kosi River, 2007 Bangladesh flood in the Padma River as shown in Fig. 1 and 2005 Mumbai

Fig. 1 Map of South Asia showing some recently flood-affected areas.
(*Source:* http://news.bbc.co.uk/1/hi/world/south_asia/6927389.stm)

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Fig. 2 Areas prone to floods in the Indian Subcontinent (CWC, 2006).

flood in the Mithi River. The areas prone to floods in the Indian subcontinent are illustrated in Fig. 2 (shaded in red color).

Early efforts to reduce flood-related deaths and damages were primarily devoted to flood control measures such as construction of levees, flood walls, dams, storage reservoirs, channelization of rivers, channel improvements, drainage improvements, diversion of flood waters, and watershed management – all of these measures fall under the category of structural measures to control flood problems. Given the impossibility of building larger and larger structures to cope up with the extremely low probability flood events, the structural measures alone cannot completely circumvent risk of flood hazards. Hence, an important role is left to the non-structural measures to be compared, evaluated and actuated in real-time, which implies the need for accurate flood forecasts with a sufficient lead-time to allow for proper response action. Consequently, flood forecasting with sufficient lead-time has become an important non-structural measure for flood hazard mitigation and for minimizing flood related deaths. Therefore, it is essential that reliable flood forecasting methods be employed which is physically based, less data intensive and, over and above, is easily understood by the field engineers.

In light of the above facts, the analysis presented in this chapter focuses on this specific aspect of flood forecasting by studying the use of a variable parameter Muskingum stage-hydrograph (VPMS) routing method as a component model of a hydrometric data-based deterministic forecasting model. It would be shown later in this chapter that the use of a physically based component in a forecasting model enables the use of a simple stochastic error-updating model to estimate the forecast error. The estimation of forecast error is made in the proposed model using a two-parameter linear autoregressive model with its parameters updated at every time interval of 30 minutes at which the stage observations are made. The proposed forecasting model is tested considering several flood events which occurred in a 15 km river reach selected along the Tiber River, in Central Italy, bounded by upstream Pierantonio and downstream Ponte Felcino gauging stations.

This chapter deals with an overview of the flood forecasting and early warning system being adopted worldwide, technical aspects of the proposed hydrometric data-based VPMS routing model and its frameworks for floodplain flow condition as well as real-time application, and a case study demonstrating the application of the proposed model.

2. FLOOD FORECASTING AND EARLY WARNING SYSTEM: AN OVERVIEW

Flood forecasting and warning systems are cost-effective means of reducing the damaging impacts of floods. These real-time flood forecasting methods in practice can broadly be categorized as '*discharge forecasting*' and '*stage forecasting*'. The real-time discharge forecasting obtained by rainfall-runoff modeling is generally less accurate than that obtained by the channel routing of an upstream discharge hydrograph (Srikanthan et al., 1994). However, in headwater catchments with no information on discharge hydrographs, these forecasting methods are very much useful. The various rainfall-runoff modeling approaches used in flood forecasting are: (i) Unit Hydrograph (UH) approach for linear catchment modeling; (ii) Non-linear catchment routing models, viz., RORB model, Watershed Bounded Network Model (WBNM), RAFTS model, and URBS model (Carroll, 1992); (iii) Loss models, viz., Constant loss rate (Φ-index) method (Reed, 1982), Variable loss rate method, Constant proportional loss model (NERC, 1975), Variable proportional loss model (NERC, 1975), and Initial loss-continuing loss model of Bureau of Meteorology, Australia; (iv) Non-linear storage models, viz., Inflow-Storage-Outflow model (Lambert, 1969, 1972), Isolated event model (NERC, 1975), and Generalized non-linear storage model (O'Connell, 1980); (v) Conceptual models, viz., Sacramento model, SAMFIL model (Vermuleulen and Snoeker, 1991), IPH-II model (Bertoni et al., 1992), Australian Water Balance Model (AWBM) (Boughton, 1993), NAM model (Refsgaard et al., 1988), SBV model (Bergstrom, 1976, 1992), Tank model (Sugawara, 1979), Probability distributed model (Moore and Jones, 1991), Alabama rainfall-runoff model (Henry et al., 1988), Xinanjiang model (Zhao et al., 1980), ARNO model (Todini, 1996), and Variable Infiltration Capacity (VIC) model (Liang et al., 1996); (vi) Spatially distributed models, viz., SIMPLE model (Kouwen, 1988), TOPOG model (O'Loughlin et al., 1989), TOPMODEL (Beven and Kirkby, 1979), and SHE model (Abbott et al., 1986a, 1986b); (vii) Transfer function models (Harpin, 1982; Powell, 1985; Cluckie and Ede, 1985; Owens, 1986; Troch et al., 1991); and (viii) Statistical methods, viz., Constrained linear systems (Natale and Todini, 1977), IHACRES model, and Filter separation auto-regressive model.

Furthermore, the real-time discharge forecasting in long river systems can be obtained by different flood routing methods. These flood routing methods can be categorized as: (a) Statistical routing methods, (b) Artificial intelligence-based methods (e.g., Artificial Neural Network, Fuzzy logic, and Genetic Algorithm); (c) Hydrologic routing methods, viz., Muskingum method, Muskingum-Cunge method (Cunge, 1969), variable parameter Muskingum-Cunge method (Ponce and Yevjevich, 1978), multilinear Muskingum discharge routing method (Perumal, 1992), multilinear discrete cascade model (Perumal, 1994a), and non-linear reservoir-type channel routing method (Georgakakos and Bras, 1982); and (d) Hydraulic routing methods, viz. kinematic wave routing method, diffusion wave routing method, and variable parameter Muskingum discharge (VPMD) routing method (Perumal, 1994b, 1994c). However, there are a few stage routing methods available in the literature which can be used for real-time flood forecasting. The hydrologic stage routing methods include 'statistical methods', 'artificial intelligence-based methods', 'multilinear Muskingum stage routing method' (Perumal et al., 2009b), and 'multilinear discrete cascade model'. Similarly, the hydraulic stage routing methods include 'de Saint-Venant equations and their simplifications', and 'variable parameter Muskingum stage (VPMS) routing method' (Perumal and Ranga Raju, 1998a, 1998b, Perumal et al., 2007). Among the various models enlisted above, this chapter focuses only on the VPMS routing method amenable for real-time flood forecasting.

The current developments in computer technology, coupled with advances in telemetry system for automatic data acquisition, not only allows the improved forecasting of any magnitude of flow at any point in a watershed, but also enables the automatic operation of the hydro-systems affected by the forecasted flow. The development of hydrological forecasting system involves various sub-systems which deal with historical and real-time data collection, data transmission, database management, forecasting procedure (modeling), forecast dissemination, and forecast evaluation and updating. Technological advances in the field of flood forecasting under these subsystems, which deal with technical aspects of forecasting, can be mainly divided into three groups: (1) data collection, (2) transmission, and (3) analysis for developing a forecasting model. With the recently improved hydrological instrumentation, automatic acquisition system of a wide range of hydrological data that includes automated rain gauges and river stage recorders, radar to detect the likely areas that would receive precipitation and its intensity, satellite based methods, and radio and satellite telemetry for transmission of data, the modern flood forecasting service systems have reached at a high point of development. The relevance of the flood forecasting as a significant flood abatement measure can be recognized from the fact that many agencies are now involved in real-time flood forecasting services in the world as discussed briefly herein.

At the beginning of modernization of flood forecasting services long before, only the government agencies were able to provide this service as they were the prime agencies responsible for the collection and dissemination of hydrologic data in real-time, its archival and processing needed for the forecasting purposes. However, with the rapid development of technology in data acquisition, telemetry, launching of many communication satellites and, above all, rapid development in computer technology resulting in cheaper desktop computers, it has become now that real-time satellite down-link systems for basin-wide hydrologic data are no longer the prerogative of government agencies. Affordable, easy-to-use hardware and software installations are within the reach of even small hydro-systems operators and they can get timely hydrologic forecasts for the efficient and environment friendly operations of the hydro-systems. While this is the scenario of flood forecasting technology in developed countries, the same is not prevailing in many developing countries. Since in most developing countries management of water resources is under the control of government agencies and flood evacuation and relief measures are their responsibility, the implementation of flood forecasting systems is mostly carried out by these government agencies or the agencies designated by them such as the World Meteorological Organization (WMO), aid agencies and, in some countries, by private companies.

The WMO Hydrological Operational Multipurpose Sub-program (HOMS) is intended to promote the transfer of hydrological technology between the member countries of WMO for use in their water resources projects. The technology is made available to users in the form of various kinds of components, for instance, manual and computerized techniques for data collection, processing and analysis; commonly used hydrological models; manuals describing field or office procedures; and instruments specifications. The WMO has been entrusted with technical supervision of the establishment of a large number of Forecasting Operational Real-Time Hydrological Systems (FORTH) in developing countries of Asia, Latin America, and Africa. However, over the past several years, many of these countries which have installed the hydrological forecasting systems with the technical assistance of WMO are modernizing these systems with the improved data acquisition systems, telemetry and advanced forecasting software, which have been well-tested and operational in developed countries. Some of the other factors responsible for the modernization are the assistance of the developed countries in installing improved flood forecasting systems in the form of bilateral aid, emergence of many private companies in developed countries for manufacturing electronic sensors and telemetry systems needed for real-time flood forecasting, and ever reducing costs of desktop computers. The notable countries which are playing major role in advancing the flood forecasting technology in other parts of the world, apart from their own countries, are the United States of America (USA), Canada, Denmark and The Netherlands. While the USA and Canada have developed easy-to-install advanced data collection instruments and telemetry systems, countries such as Canada, Denmark and The Netherlands have developed improved hydrological forecasting tools. A brief description of the activities of these agencies in real-time flood forecasting is presented herein.

The National Weather Service (NWS) of National Oceanic and Atmospheric Administration (NOAA) is the federal agency responsible to issue forecasts and warning of floods in the USA. Although many cities, counties or other local flood management agencies are involved now-a-days in the operation of local flood warning systems, the NWS is still the principal national agency responsible for flood forecasting and warning in the entire USA. The local agencies coordinate with NWS in getting the forecasting technology implemented in their area and continuously get technical guidance. The NWS uses the National Weather Service River Forecast System (NWSRFS) as the foundation of hydrologic forecast system. NWSRFS is a suite of hydrologic and hydraulic models that contains all the programs necessary to produce hydrologic forecasts for a river basin. The NWSRFS provides the basic framework for a national river forecast and flood warning system. The current available operations, which form the components of the NWSRFS are:

(a) Temperature index snow accumulation and ablation model
(b) Sacramento soil-moisture accounting model
(c) Antecedent Precipitation Index (API) rainfall-runoff models used in the Missouri Basin, Ohio basin, Middle Atlantic States, North-Central USA, and South-Western USA
(d) Unit hydrograph with a constant and variable baseflow option
(e) Lag and K, Muskingum, layered coefficient and Tatum routing procedures
(f) Flood Wave Model (FLDWAV)
(g) Reservoir model that allows the user to select and combine thirteen modes of regulation to simulate the operation of a single independently controlled reservoir
(h) Stage/discharge conversion using single-valued rating curves with log or hydraulic extensions and dynamically induced loop ratings

(i) Simple flow adjustment and blend procedure
(j) Simplified channel loss procedure
(k) Computation of mean discharge from instantaneous values
(l) Set timeseries values to zero
(m) Add and subtract timeseries
(n) Weight timeseries
(o) Change the time interval of a timeseries
(p) Plot instantaneous discharge
(q) Operational hydrograph display
(r) General timeseries plot
(s) Daily flow plots (calibration used only), and
(t) Statistical package (calibration)

While most of the precipitation data are collected by the NWS or its designated agencies in real time, the stream gauge data for most of the streams are received from the USGS stream network. Expanded use of telemetry at the USGS streamflow stations and refinement of telemetry equipment continue to improve the timeliness and reliability of data that are transmitted for forecasting purposes. The automated surface observatory systems are now replacing the manual weather observations, and advanced telecommunication systems are improving the integration and distribution of data. The US Army Corps of Engineers has developed the HEC-RAS and HEC-HMS modeling packages for flood forecasting in large river basins.

The Danish Hydraulic Institute (DHI) of Denmark has developed a range of most advanced flood management software, viz., MIKE 11, MIKE 11 GIS, MIKE 21, MIKE FLOOD WATCH, and MIKE SHE, and they have been used in many of the flood forecasting systems implemented for the large river basins in the world. In the majority of flood management projects carried out, DHI-Water and Environment has also been responsible for the overall project management at the implementation stage. Some of the flood forecasting projects implemented by DHI-Water and Environment are: Flood management in Poland; Flood management in Czech Republic; Anglican flow forecasting modeling system, UK; Flood forecasting for barrage operation, Malaysia; Inflow forecasting for hydropower optimization, Wales, UK; Expansion of flood forecasting and warning services, Bangladesh; Flood forecasting for middle river Yangtze, China; Real-time flood forecasting in Italy; Environment Waikato flood forecasting system in New Zealand; Ping River basin flood forecasting project in Thailand; and Pilot flood forecasting system for Lower Colorado River, Texas, USA.

Flood forecasting and warning in Canada has evolved into a network of forecast system across the country. There are five provisional streamflow centers in Canada. In addition, the power generation companies such as Hydro BC and Hydro-Quebec have developed advanced flood forecasting systems for operating a cluster of reservoirs and run-of-the river plants under their control for power generation. These companies are implementing the technology adopted in their forecasting systems for other countries also. Furthermore, some of the private companies such as Riverside Technology, Inc. (Rti), USA, and Water and Monitoring System International (WMSI), Canada are implementing the flood forecasting systems developed in the USA and in Canada, respectively, and also in other parts of the world.

Technologically, every flood forecasting model operates on two modes, viz., *simulation mode*, and *operation mode* (on-line forecasting). A flood forecasting model in the simulation mode attempts to produce the response of the system for the past recorded precipitation or upstream flow input. The response of the model is compared with the recorded response at the point of forecasting interest and, if both do not match, either the model structure is changed or the parameters are modified till the matching is done satisfactorily. Once the structure of the model and its parameters are identified during the calibration phase, the model can be used for forecasting purposes and it is said to be used in operational mode. While the basic structure of the model is not changed in the operational mode, the parameters need to be changed considering the current catchment conditions due to the variation of the input and subsequent change in other components of the rainfall-runoff process.

Moreover, the flood forecasting models are typically made up of two components: (i) *deterministic flow component*, and (ii) *stochastic flow component*. While the former is determined by the hydrologic/hydraulic model, the latter is determined based on the residual (error) series of the difference between the forecasted flow for a specified lead time and the corresponding observed one. The residual series reflects both the model error, due to the inability of the model used for forecasting to correctly reproduce the flow process, and the observational error while measuring the flow. It is imperative, therefore, to use an appropriate approach to reduce the model error.

The following section describes about the physical basis of the VPMS model used as a component model for flood forecasting.

3. VARIABLE PARAMETER MUSKINGUM STAGE-HYDROGRAPH ROUTING METHOD

The variable parameter Muskingum stage-hydrograph routing method, henceforth referred to as the *VPMS method*, was developed by Perumal and Ranga Raju (1998a, 1998b) directly from the Saint Venant equations. The form of the routing equation developed is same as that of the *Muskingum method*, by replacing the discharge variable by the stage variable and hence, the reason for adherence of the term "*Muskingum*". Further, the parameters vary at every routing time interval and they are related to the channel and flow characteristics by the same relationships as established for the physically based Muskingum method (Apollov et al., 1964; Cunge, 1969; Dooge et al., 1982; Perumal 1994b, 1994c). The detail development of this method is presented below (e.g., Perumal and Ranga Raju, 1998a; Perumal et al., 2007).

3.1 Concept

The VPMS method has been developed using the following concept: During steady flow in a river reach having any shape of prismatic cross-section, the stage and, hence, the cross-sectional area of flow at any point of the reach is uniquely related to the discharge at the same location defining the steady flow rating curve. However, this situation is altered during unsteady flow, as conceptualized in the definition sketch (Fig. 3) of the variable parameter Muskingum routing reach of length Δx, in which the same unique relationship is maintained between the stage and the corresponding steady discharge at any given instant of time, recorded not at the same section, but at a downstream section (section 3 in Fig. 3) preceding the corresponding steady stage section (mid-section in Fig. 3).

3.2 Theoretical Background

The routing method is derived from the Saint Venant equations, which govern the one-dimensional unsteady flow in channels and rivers without considering lateral flow and are given by:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \text{ (continuity equation)} \qquad (1)$$

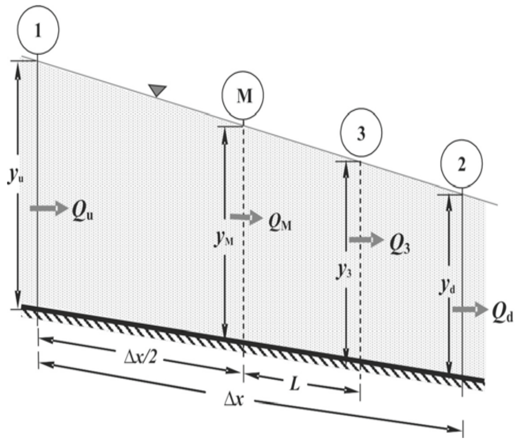

Fig. 3 Definition sketch of the stage–hydrograph routing method.

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t} \text{ (momentum equation)} \tag{2}$$

where Q = discharge; A = cross-sectional area of flow; S_o = channel bed slope; S_f = energy slope; g = acceleration due to gravity; v = average velocity over cross-section, and y = depth of flow. The notations x and t denote the space and time variables, respectively.

The derivation of the method involves the assumptions which enable the simplification of the unsteady flow process by assuming the channel reach to be prismatic and the gradients $\partial y/\partial x$, $(v/g)(\partial v/\partial x)$ and $(1/g)(\partial v/\partial t)$, which, respectively, denote the longitudinal water surface gradient, the convective and the local acceleration gradients, remain constant at any instant of time in a given reach. The latter assumption implies that the friction slope S_f is constant over the computational reach length at any instant of time and, hence, the flow depth varies linearly. It has been shown by Perumal and Ranga Raju (1998a, 1998b; 1999) that the use of the assumption of constant S_f and the Manning's friction law governing the unsteady flow enable to arrive at the simplified momentum equation expressed as:

$$\frac{\partial Q}{\partial x} = v\left[\frac{\partial A}{\partial y} + mP\frac{\partial R}{\partial y}\right]\left[\frac{\partial y}{\partial x}\right] \tag{3}$$

where m = an exponent which depends on the friction law used ($m = {}^2/_3$ for the Manning's friction law, $m = {}^1/_2$ for the Chezy's friction law); R = hydraulic radius (A/P); and P = wetted perimeter.

Using Eqn. (3), it may be shown that the celerity of the flood wave can be expressed as:

$$c = \frac{\partial Q}{\partial A} = \left[1 + m\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)\right]v \tag{4}$$

Note that the application of Eqn. (4) for unsteady flow in rectangular channels yields the same celerity relationship as given in the report of NERC (1975).

Using Eqns (1), (2), (3) and the expression of discharge using Manning's friction law, S_f can be expressed as follows:

$$S_f = S_o\left[1-\frac{1}{S_o}\frac{\partial y}{\partial x}\left\{1-\left[mF\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)\right]^2\right\}\right] \tag{5}$$

where F is the Froude number which is mathematically expressed as:

$$F = \left(\frac{Q^2\partial A/\partial y}{gA^3}\right)^{1/2} \tag{6}$$

Use of Eqn. (5) in the expression for discharge Q_M at the middle of the computational channel reach, using the Manning's friction law, and its simplification based on the binomial series expansion leads to the simplified expression for Q_M as:

$$Q_M = Q_3 - \frac{Q_3\left[1-m^2F_M^2\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)_M^2\right]}{2S_o\left.\frac{\partial A}{\partial y}\right|_3\left[1+m\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)\right]_{v_3}}\left.\frac{\partial Q}{\partial x}\right|_3 \tag{7}$$

Eqn. (7) expresses the discharge Q_M in terms of normal discharge Q_3, corresponding to y_M, the flow depth at the middle of the reach. The section where Q_3 passes corresponding to y_M is located at a distance L downstream of the midsection and is expressed as:

$$L = \frac{Q_3\left[1-m^2F_M^2\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)_M^2\right]}{2S_o\left.\frac{\partial A}{\partial y}\right|_3\left[1+m\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)\right]_{v_3}} \tag{8}$$

where the subscripts M and 3 attached with different variables denote these variables at midsection and section 3, respectively.

Use of Eqns (1), (3) and (4) leads to the following expression (Perumal and Ranga Raju, 1998a, 1998b):

$$\frac{\partial y}{\partial t}+c\frac{\partial y}{\partial x}=0 \tag{9}$$

It was pointed out by Perumal and Ranga Raju (1998a, 1998b) that although the form of Eqn. (9) is same as that of the well-known kinematic-wave equation (Lighthill and Whitham, 1955), it is capable of approximately modeling a flood wave in the transition range between the zero-inertia wave, governed by the convection-diffusion equation (Hayami, 1951) and the kinematic wave, including the latter. The characteristic of this new wave type governed by Eqn. (9), termed as the approximate convection-diffusion (ACD) equation, has been investigated in detail by Perumal and Ranga Raju (1999).

Applying Eqns (4) and (9) at section 3 of Fig. 3 and its simplification leads to (Perumal and Ranga Raju, 1998a, 1998b) the governing differential equation of the Muskingum type routing, using stage as the operating variable in place of discharge, and it is expressed as:

$$y_u - y_d = \frac{\Delta x}{\left[1+m\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)_3\right]_{v_3}}\times\frac{\partial}{\partial t}\left[y_d+\left(\frac{1}{2}-\frac{L}{\Delta x}\right)(y_u-y_d)\right] \tag{10}$$

where y_u and y_d denote the stages at the upstream and downstream of the reach, respectively. Using the similarity between the governing differential equation of the Muskingum method in discharge formulation and that of Eqn. (10), it is inferred that the travel time K of the Muskingum type stage routing method can be expressed as:

$$K = \frac{\Delta x}{\left[1 + m\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)_3\right] v_3} \tag{11}$$

and the weighting parameter θ, after substituting for L from Eqn. (8), can be obtained as:

$$\theta = \frac{1}{2} - \frac{Q_3\left[1 - m^2 F_M^2\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)_M^2\right]}{2S_o \left.\frac{\partial A}{\partial y}\right|_3 \left[1 + m\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)_3\right] v_3 \Delta x} \tag{12}$$

The product term $(\partial A/\partial y)_3[1 + m(P\partial R/\partial y)/(\partial A/\partial y)]_3\, v_3$, present in the denominator of Eqn. (12), can be replaced by the simple expression deduced from Eqn. (4) as:

$$\left.\frac{\partial A}{\partial y}\right|_3 \left[1 + m\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)_3\right] v_3 = \left.\frac{\partial Q}{\partial y}\right|_3 \tag{13a}$$

Expressing in terms of c_3, Eqn. (13a) is modified as:

$$\left.\frac{\partial Q}{\partial y}\right|_3 = \left.\frac{\partial A}{\partial y}\right|_3 c_3 \tag{13b}$$

Using Eqn. (13a), Eqn. (12) is modified as:

$$\theta = \frac{1}{2} - \frac{Q_3\left[1 - m^2 F_M^2\left(\frac{P\partial R/\partial y}{\partial A/\partial y}\right)_M^2\right]}{2S_o \left.\frac{\partial Q}{\partial y}\right|_3 \Delta x} \tag{14a}$$

After neglecting inertial terms of the Saint Venant equations, Eqn. (14a) can be expressed as:

$$\theta = \frac{1}{2} - \frac{Q_3}{2S_o \left.\frac{\partial Q}{\partial y}\right|_3 \Delta x} \tag{14b}$$

where the subscript 3 attached with these variables denote the section 3, wherein the discharge passing is the normal discharge corresponding to the stage at the middle of the Muskingum reach.

Use of Eqns (11) and (14b) in Eqn. (10) leads to a form similar to that of the Muskingum routing equation, but using stage as the operating variable instead of discharge, and it is expressed as follows:

$$y_{d,j+1} = C_1 y_{u,j+1} + C_2 y_{u,j} + C_3 y_{d,j} \tag{15}$$

where $y_{u,j+1}$ and $y_{d,j+1}$ denote the upstream and downstream stages at time $(j+1)\Delta t$, respectively; $y_{u,j}$ and $y_{d,j}$ denote the upstream and downstream stages at time $j\Delta t$, respectively, where Δt is the routing time interval; and the coefficients C_1, C_2 and C_3 are expressed as:

$$C_1 = \frac{-K\theta + 0.5\Delta t}{K(1-\theta) + 0.5\Delta t} \quad (16a)$$

$$C_2 = \frac{K\theta + 0.5\Delta t}{K(1-\theta) + 0.5\Delta t} \quad (16b)$$

$$C_3 = \frac{K(1-\theta) - 0.5\Delta t}{K(1-\theta) + 0.5\Delta t} \quad (16c)$$

4. EXTENSION OF THE VPMS METHOD FOR ROUTING IN A TWO-STAGE COMPOUND CROSS-SECTION CHANNEL REACH

4.1 Channel Reach Details

It is assumed that the channel reach is characterized by a two-stage uniform compound cross-section with a trapezoidal main channel flow section and an extended trapezoidal floodplain section as shown in Fig. 4. It is, further, assumed that the entire channel reach is characterized by a uniform or representative Manning's roughness coefficient irrespective of main or floodplain channel. This assumption may not be strictly valid in practice. Since the main aim herein is to develop a simplified hydraulic routing method using stage as the main routing variable, such an assumption helps to reduce complications in the development of the method.

4.2 Development of Celerity-Stage Relationship

Stage-hydrograph routing using the VPMS method, either in a single section (main channel section) or a compound section channel reach, involves the use of Eqns (11), (14b), (15), and (16a–c). Estimation of the parameters K and θ, given by Eqns (11) and (14b) at every routing time interval, involves the variables Q_3, c_3, and $(\partial Q/\partial y)_3$. One of the important parameters in flood routing process is the celerity at which the flood wave travels along the river reach downstream. The average celerity of a flood wave can be estimated as the average travel time of the flood peaks of the hydrographs recorded at either end of a reach (Wong

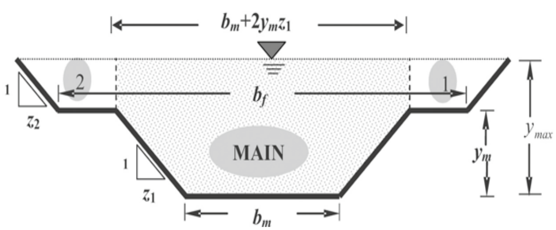

Fig. 4 Compartmentalization of the compound channel section into a main channel (shade) and two floodplains (1 and 2) for celerity computation.

and Laurenson, 1983, 1984). Celerity, corresponding to any discharge, Q can also be estimated using the rating curve at a particular cross-section as:

$$c = \frac{\partial Q}{\partial A} = \frac{\partial Q}{\partial y} \bigg/ \frac{\partial A}{\partial y} \tag{17}$$

Alternatively, Q can be estimated analytically by Eqn. (4).

Note that the wave celerity relationship is not unique during unsteady flow due to differing relationships established when the flood is in the rising and falling stages. Following Eqn. (21), it is expected that the celerity versus stage relationship to have a discontinuity at the intersection of the full-bank flood level (corresponding to the top width of the main channel cross-section) and the bottom width of the floodplain cross-section, due to sudden increase of the wetted perimeter.

4.2.1 Celerity-Stage Relationship for the Main Channel

When the depth of flow in the channel ranges between zero and y_m (see Fig. 4), the unsteady flow corresponds to the main channel flow traversing within the simple trapezoidal section. The celerity of flow at any section of the main channel flow reach can be expressed using Eqn. (4) as:

$$c = \left[\frac{5}{3} - \frac{2}{3}\frac{R_{main}\left(\partial P_{main}/\partial y\right)}{\left(\partial A_{main}/\partial y\right)}\right]\left(\frac{Q_{main}}{A_{main}}\right) \tag{18}$$

where Q_{main} is the discharge at the section where celerity is computed; and

$$R_{main} = \frac{A_{main}}{P_{main}} \tag{19a}$$

$$A_{main} = b_m y + z_1 y^2 \tag{19b}$$

$$P_{main} = b_m + 2y\sqrt{1+z_1^2} \tag{19c}$$

$$\frac{\partial A_{main}}{\partial y} = b_m + 2yz_1 \tag{19d}$$

$$\frac{\partial P_{main}}{\partial y} = 2\sqrt{1+z_1^2} \tag{19e}$$

The longitudinal gradient of water depth $\partial y/\partial x$ can be estimated using backward difference scheme as:

$$\frac{\partial y}{\partial x} = \frac{y_d - y_u}{\Delta x} \tag{19f}$$

where y_u and y_d, respectively, correspond to flow depths, at any instant of time, at the upstream and the downstream of the conceptual reach length Δx.

4.2.2 Celerity-Stage Relationship for the Floodplain Channel

When the flow depth exceeds y_m, the flow occupies the floodplain. For the estimation of celerity under this condition, the flow is compartmentalized into flow in the main channel and in the two symmetrical compartments above bank-full level as shown in Fig. 4. It may be noted that the main channel flow section above the bank-full level corresponds to a rectangular section having the width corresponding to the top-width of the main trapezoidal channel section. Accordingly, the compound channel discharge encompassing the flow in the floodplain is expressed as follows:

$$Q_{compound} = Q_{main} + Q_1 + Q_2 \tag{20a}$$

where Q_{main} is the flow in the main channel, and Q_1 and Q_2, respectively, are the flow components of the floodplain channel compartments 1 and 2. Equation (20a) may be written in terms of the flow area and velocity of respective compartmentalized sections as:

$$Q_{compound} = A_{main}\, v_{main} + A_1 v_1 + A_2 v_2 \tag{20b}$$

where v_{main}, v_1, and v_2 are the velocities in the main channel section, and in the floodplain channel compartments 1 and 2, respectively.

Using Eqn. (17), the wave celerity for the compound section may be expressed as:

$$c_{compound} = \frac{\partial Q_{compound}}{\partial A_{compound}} = \left(\frac{\partial Q_{compound}}{\partial y}\right) \Big/ \left(\frac{\partial A_{compound}}{\partial y}\right) \tag{21}$$

The expression for ($\partial Q_{compound}$) may be estimated from Eqn. (20a) as:

$$\frac{\partial Q_{compound}}{\partial y} = \frac{\partial Q_{main}}{\partial y} + \frac{\partial Q_1}{\partial y} + \frac{\partial Q_2}{\partial y} \tag{22}$$

Each of the derivatives of the right hand side of Eqn. (22) may be expressed using Eqn. (13a) as

$$\frac{\partial Q_{main}}{\partial y} = \left[\frac{5}{3}\frac{\partial A_{main}}{\partial y} - \frac{2}{3}\frac{A_{main}}{P_{main}}\frac{\partial P_{main}}{\partial y}\right] v_{main} \tag{23a}$$

Similarly, the second and the third terms of the Eqn. (22) can be expressed as:

$$\frac{\partial Q_1}{\partial y} = \left[\frac{5}{3}\frac{\partial A_1}{\partial y} - \frac{2}{3}\frac{A_1}{P_1}\frac{\partial P_1}{\partial y}\right] v_1 \tag{23b}$$

$$\frac{\partial Q_2}{\partial y} = \left[\frac{5}{3}\frac{\partial A_2}{\partial y} - \frac{2}{3}\frac{A_2}{P_2}\frac{\partial P_2}{\partial y}\right] v_2 \tag{23c}$$

where the velocities of unsteady flow in different compartments of the compound channel section can be expressed using the Manning's friction law and the momentum Eqn. (2), after neglecting the acceleration terms, as follows:

$$v_{main} = v_{main,\,0}\sqrt{1 - \frac{1}{S_o}\frac{\partial y}{\partial x}}\ ;\ v_1 = v_{1,\,0}\sqrt{1 - \frac{1}{S_o}\frac{\partial y}{\partial x}}\ ;\ \text{and}$$

$$v_2 = v_{2,\,0}\sqrt{1 - \frac{1}{S_o}\frac{\partial y}{\partial x}} \tag{23d,e,f}$$

where

$$v_{main,0} = \frac{\sqrt{S_o}}{n}\left(\frac{A_{main}}{P_{main}}\right)^{2/3};\ v_{1,\,0} = \frac{\sqrt{S_o}}{n}\left(\frac{A_1}{P_1}\right)^{2/3};\ \text{and}\ v_{2,\,0} = \frac{\sqrt{S_o}}{n}\left(\frac{A_2}{P_2}\right)^{2/3} \tag{23g,h,i}$$

Substituting Eqns. (23a–f) in Eqn. (22) leads to

$$\frac{\partial Q_{compound}}{\partial y} = \left[\frac{5}{3}\frac{\partial A_{main}}{\partial y} - \frac{2}{3}\frac{A_{main}}{P_{main}}\frac{\partial P_{main}}{\partial y}\right] v_{main,\,0}\sqrt{1-\frac{1}{S_o}\frac{\partial y}{\partial x}}$$
$$+\left[\frac{5}{3}\frac{\partial A_1}{\partial y} - \frac{2}{3}\frac{A_1}{P_1}\frac{\partial P_1}{\partial y}\right] v_{1,\,0}\sqrt{1-\frac{1}{S_o}\frac{\partial y}{\partial x}}$$
$$+\left[\frac{5}{3}\frac{\partial A_2}{\partial y} - \frac{2}{3}\frac{A_2}{P_2}\frac{\partial P_2}{\partial y}\right] v_{2,\,0}\sqrt{1-\frac{1}{S_o}\frac{\partial y}{\partial x}} \tag{24}$$

Substituting Eqn. (24) in Eqn. (21), the celerity of the flow when it exceeds the main channel section may be modified as (Perumal et al., 2007; Sahoo, 2007):

$$c_{compound} = \left[\left(\frac{5}{3}\frac{\partial A_{main}}{\partial y}\right) v_{main,\,0}\sqrt{1-\frac{1}{S_o}\frac{\partial y}{\partial x}}\right] \Big/ \left[\frac{\partial A_{compound}}{\partial y}\right]$$
$$+\left[\left(\frac{5}{3}\frac{\partial A_1}{\partial y} - \frac{2}{3}\frac{A_1}{P_1}\frac{\partial P_1}{\partial y}\right) v_{1,\,0}\sqrt{1-\frac{1}{S_o}\frac{\partial y}{\partial x}}\right] \Big/ \left[\frac{\partial A_{compound}}{\partial y}\right]$$
$$+\left[\left(\frac{5}{3}\frac{\partial A_2}{\partial y} - \frac{2}{3}\frac{A_2}{P_2}\frac{\partial P_2}{\partial y}\right) v_{2,\,0}\sqrt{1-\frac{1}{S_o}\frac{\partial y}{\partial x}}\right] \Big/ \left[\frac{\partial A_{compound}}{\partial y}\right] \tag{25}$$

where

$$A_{compound} = (b_m + y_m z_1) y_m + \left(b_f + z_2\left(y - y_m\right)\right)\left(y - y_m\right) \tag{26a}$$

$$\frac{\partial A_{compound}}{\partial y} = b_f + 2\left(y - y_m\right) z_2 \tag{26b}$$

$$A_{main} = (b_m + y_m z_1) y_m + \left(b_m + 2 y_m z_1\right)\left(y - y_m\right) \tag{26c}$$

$$P_{main} = b_m + 2 y_m \sqrt{1 + z_1^{\,2}} \tag{26d}$$

$$\frac{\partial P_{main}}{\partial y} = 0 \tag{26e}$$

$$A_1 = A_2 = 0.5\left(\left(b_f - b_m - 2 y_m z_1\right) + z_2\left(y - y_m\right)\right)\left(y - y_m\right) \tag{26f}$$

$$P_1 = P_2 = 0.5\left(b_f - b_m - 2 y_m z_1\right) + \left(y - y_m\right)\sqrt{1 + z_2^2} \tag{26g}$$

$$\frac{\partial A_1}{\partial y} = \frac{\partial A_2}{\partial y} = 0.5\left(b_f - b_m - 2 y_m z_1\right) + z_2\left(y - y_m\right) \tag{26h}$$

$$\frac{\partial P_1}{\partial y} = \frac{\partial P_2}{\partial y} = \sqrt{1 + z_2^2} \tag{26i}$$

4.3 Routing Procedure

The procedure described below is adopted while routing using the VPMS method.

The stage at the outlet of the reach is estimated using the recursive Eqn. (15). The parameters K and θ vary at every routing time interval in a two-step process: In the first step, by estimating the unrefined stage estimate $y_{d,j+1}$ for the current routing time interval using the values of K and θ, estimated at the previous time step and, subsequently, using this estimate and $y_{u,j+1}$, the flow depth at the middle of the reach is estimated as

$$y_M = (y_{u,j+1} + y_{d,j+1})/2 \tag{27}$$

The initial values of K and θ are estimated using the initial steady flow in the reach.

Similarly, the depth of flow at section 3 of Fig. 3 is computed as:

$$y_3 = \theta\, y_{u,\,j+1} + (1-\theta)\, y_{d,\,j+1} \tag{28}$$

Using y_M, given by Eqn. (27), the discharge at section 3 is computed in the following manner depending on whether y_M is within the main channel section or in the compound channel section:

$$Q_3 = \begin{cases} \dfrac{A_M}{n}\left(\dfrac{A_M}{P_M}\right)^{2/3} S_0^{1/2}, \text{ when } y_M \le y_m \\ \left[A_{main}\, v_{main}\right]_M + \left[A_1 v_1\right]_M + \left[A_2 v_2\right]_M, \text{ when } y_M > y_m \end{cases} \tag{29a,b}$$

where $A_{main,M}$, $A_{1,M}$ and $A_{2,M}$ are evaluated at the midsection of the reach using Eqns (26c) and (26f), respectively, and

$$v_{main,M} = \frac{\sqrt{S_o}}{n} R_{main,\,M}^{2/3};\; v_{1,\,M} = \frac{\sqrt{S_o}}{n} R_{1,\,M}^{2/3}; \text{ and } v_{2,\,M} = \frac{\sqrt{S_o}}{n} R_{2,\,M}^{2/3} \tag{30a,b,c}$$

When $y_3 \le y_m$, the wave celerity c_3 is estimated by using Eqn. (18) corresponding to y_3; and when $y_3 > y_m$, the wave celerity c_3 corresponding to flow depth y_3 of the compound channel section is estimated by using Eqn. (25).

Corresponding to the estimated values of y_M and y_3, obtained using Eqns (27) and (28), respectively, the value of $(\partial Q/\partial y)_3$ is estimated using Eqns (13b), (18) and (19a–f) or (24) and (26a–i) depending on whether y_M is within the main channel section or extends into the floodplain channel section, respectively. In the second step, using these values of Q_3,c_3 and $(\partial Q/\partial y)_3$, the refined values of K and θ are estimated using Eqns (11) and (14b), respectively, for the current routing time interval, which are, then, used to estimate the refined stage-hydrograph estimate using Eqns. (15) and (16a–c).

5. APPLICATION OF VPMS MODEL FOR REAL-TIME FLOOD FORECASTING

The variable parameter Muskingum stage routing model applicable for forecasting purposes is written by modifying Eqn. (15) as (Perumal et al., 2009a):

$$\hat{y}_{d,(j\Delta t+T_L)} = C_1 y_{u,\,j\Delta t} + C_2 y_{u,\,j\Delta t} + C_3 \hat{y}_{d,((j-1)\Delta t+T_L)} + e_{est,(j\Delta t+T_L)} \tag{31}$$

where $j\Delta t$ is the time of forecast, $\hat{y}$ denotes the forecasted stage values and T_L is the forecasting lead time. The minimum T_L is Δt, the routing time interval at which the stage measurements are made, and this corresponds to one time interval ahead of forecast. The maximum lead time interval that can be adopted depends on the accuracy of the obtained forecast. The larger the T_L, the poorer would be the accuracy of the forecast.

In order to apply the VPMS routing method in a river reach for real-time flood forecasting purposes, an error updating model also needs to be developed for estimating the forecast error, which when added to the model estimated forecast for a given lead time would yield the final forecasted flow at the site of interest. It is proposed to use a second-order linear autoregressive error updating model of the following form for forecasting the error at time $(j\Delta t+T_L)$ (Perumal et al., 2009a):

$$e_{est,(j\Delta t+T_L)} = a_1\, e_{obs,j\Delta t} + a_2\, e_{obs,(j-1)\Delta t} + \varepsilon_{(j\Delta t+T_L)} \tag{32}$$

where $e_{obs,j\Delta t}$ and $e_{obs,(j-1)\Delta t}$ are the forecasting errors estimated at time $j\Delta t$ and $(j-1)\Delta t$, respectively, and $\varepsilon_{(j\Delta t+T_L)}$ is the random error (white noise). However, the flow depth forecasting can be made only after the lapse of certain initial period of the forecasting event, known as the warm up period. The difference between the observed stage and the VPMS routed stage in the warm up period is considered as the actual error and its series is assumed to be stochastic in nature. The initial parameters a_1 and a_2 of the error update model are assessed using this error series estimated in the warm up period. The duration of initial warm up period considered for developing the error update model should not be long to render the forecasting exercise to be of no practical use for forecasting the given event, and, at the same time, it should not be too short resulting in numerical problem while estimating the parameters a_1 and a_2 using the least squares approach. The parameters a_1 and a_2 are updated in real-time on the basis of the last available observations.

6. CASE STUDY AND RESULTS

The proposed forecasting model consisting of the VPMS routing method as the basic model, and the second-order linear autoregressive model as the error updating model, is applied for forecasting flow in a 15 km reach along the Tiber River of Central Italy (Fig. 5). The selected reach is bounded by Pierantonio and Ponte Felcino gauging stations and has an average bed slope (S_o) of 0.0016. This average bed slope is estimated from the elevation difference of the two gauging stations considered herein.

Note that the approximation of the VPMS method for routing a given stage-hydrograph in a river reach requires the use of an equivalent prismatic channel reach; this involves the approximation of the actual river reach sections at the two ends to an equivalent prismatic section with a one-to-one relationship established between the flow depth of the actual section of a given flow area with the corresponding flow depth of the prismatic channel section of the same flow area. Based on the surveyed cross-sections at the ends of the actual river reach, it was considered appropriate to approximate the actual reach by a compound trapezoidal section reach. Accordingly, the surveyed cross-sections of the actual reach were overlapped and a two-stage trapezoidal compound section geometry with $b_m = 25$ m, $y_m = 5$ m, $b_f = 59.5$ m and $z_1 = z_2 = 2.5$ (see Fig. 6) as required for the prismatic channel reach conceptualization of the VPMS routing method was finalized by a trial and error approach by fitting the best relationships between the actual flow depths and the equivalent trapezoidal section ones as: ($y_{u\text{-}trap} = 0.8887\, y_{u\text{-}actual} + 0.11$) for Pierantonio section and ($y_{d\text{-}trap} = 1.0582\, y_{d\text{-}actual} - 0.1308$) for Ponte Felcino site. $y_{u\text{-}trap}$ and $y_{d\text{-}trap}$ are the equivalent upstream and downstream flow depths in the trapezoidal channel section corresponding to the flow depths $y_{u\text{-}actual}$ and $y_{d\text{-}actual}$ in the actual river section. Using the upstream section relationship, the observed stage hydrograph of any event was converted to equivalent trapezoidal section stage hydrograph to enable the routing using the VPMS method and, using the relationship ($y_{d\text{-}actual} = 0.945\, y_{d\text{-}trap} + 0.1236$), developed on the basis of the downstream relationship, the routed hydrographs of the equivalent trapezoidal section was converted to the actual end section estimated hydrograph.

For studying the applicability of the proposed forecasting model, 12 flood events recorded concurrently at Pierantonio and Ponte Felcino stations were used. The details of these events, each recorded at half-

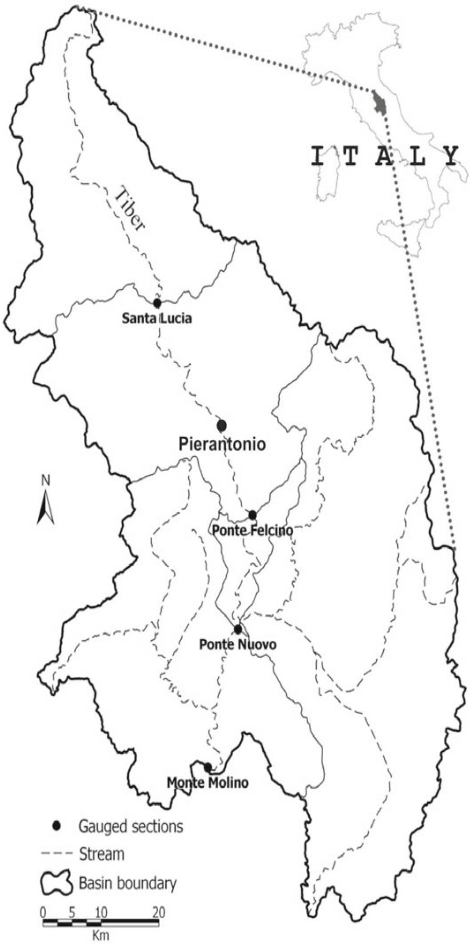

Fig. 5 Index map of the upper Tiber River in Central Italy.

an-hour intervals, are shown in Table 1, where also the details of wave travel time, percentage of lateral flow and actual and equivalent trapezoidal peak flow depths at both stations are reported. The accuracy of the proposed forecasting model was studied using a warm up period of five hours and considering five forecast lead times (1.0, 1.5, 2.0, 2.5 and 3.0 hours). The efficiency of the forecast was evaluated using two criteria: Nash-Sutcliffe (NS) criterion (Nash and Sutcliffe, 1970) and Persistence criterion (PC). As the NS criterion is well known in hydrological literature, only the Persistence criterion is explained herein. The Persistence criterion compares the prediction of the proposed model against that obtained by the no-model, which assumes steady state over the forecasting lead time, and is evaluated as follows:

$$PC = \left(1 - \frac{\sum_i \left(y_{i\Delta t} - \hat{y}_{i\Delta t} \right)^2}{\sum_i \left(y_{i\Delta t} - y_{(i\Delta t - T_L)} \right)^2} \right) \times 100 \tag{33}$$

where y and $\hat{y}$ denote the observed and the forecasted flow depth values, respectively.

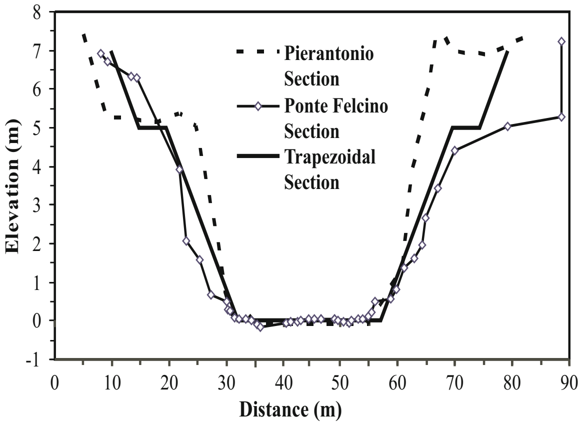

Fig. 6 Cross sections of the upper Tiber River at Pierantonio (upstream) and Ponte Felcino (downstream) gauging stations.

Table 1. Pertinent characteristics of the flood events studied

Flood event	*Wave travel time (h)*	*Lateral inflow (%)*	*Pierantonio section*		*Ponte Felcino section*	
			Actual peak stage (m)	*Equivalent trapezoidal peak stage (m)*	*Actual peak stage (m)*	*Equivalent trapezoidal peak stage (m)*
December 1996	1.50	1.90	4.74	4.32	4.22	4.33
April 1997	1.50	6.50	5.07	4.62	4.57	4.70
November 1997	1.00	5.40	4.22	3.86	3.81	3.90
February 1999	2.00	4.40	5.06	4.61	4.52	4.65
December 1999	0.00	24.70	2.71	2.52	2.79	2.82
December 2000	2.00	Flooding	5.92	5.37	5.25	5.42
April 2001	2.00	0.20	3.68	3.38	3.23	3.29
November 2005	2.5	Flooding	7.10	6.42	6.92	7.19
3rd December 2005	1.00	3.60	5.10	4.64	4.42	4.55
5th December 2005	1.00	5.70	5.49	4.99	4.76	4.91
30th December 2005	2.00	1.90	4.99	4.54	4.34	4.46
February 2006	1.50	28.40	2.28	2.14	2.64	2.66

Tables 2 to 6 show the forecasting results for peak flow depth forecast at Ponte Felcino station for all the selected flood events and for all the investigated lead times. The results also include the accuracy of peak stage reproduction, error in time to peak stage, Nash-Sutcliffe (NS) efficiency and Persistence criterion (PC) efficiency. Some of the floods studied herein are characterized by the flooding events (December 2000 and November 2005) with flow spilled over the main channel almost in the entire stretch of the reach and also received unaccounted lateral flow.

Figures 7 to 10 show some typical forecasted events for various lead times. The given inflow hydrograph and the corresponding observed outflow hydrograph are also shown in these figures. It is inferred from the results given in Tables 2 to 6 that up to a lead time of 3.0 h, only two flood events (December 1999 and February 2006) could not be successfully forecasted as reflected by their PC estimates (<50%). These two events are characterized by significant lateral flows (>25% of inflow hydrograph volume). As the proposed forecasting model has been developed using the assumption of no lateral flow

Table 2. Results of the forecasting model for a lead time of one hour (err_y_{peak} = percentage error in peak stage; err_t_{peak} = error in time to peak stage)

Event	err_y_{peak} *(%)*	err_t_{peak} *(h)*	*NS (%)*	*PC (%)*
December 1996	0.08	–1.50	99.82	93.80
April 1997	–0.20	–0.50	99.95	97.80
November 1997	0.97	–3.00	99.87	96.15
February 1999	–0.77	–0.50	99.90	96.59
December 1999	1.95	1.00	99.79	78.68
December 2000	–0.64	0.50	99.80	90.11
April 2001	–0.61	0.50	99.67	95.66
November 2005	0.06	0.00	99.87	90.54
3rd December 2005	–1.29	1.00	99.74	95.26
5th December 2005	–0.17	0.50	99.80	93.66
30th December 2005	0.02	0.00	99.91	92.60
February 2006	1.50	1.00	99.62	81.56
Mean absolute value	0.69	0.83	99.81	91.87

Table 3. As for Table 2, but for a lead time of 1.5 hours

Event	err_y_{peak} *(%)*	err_t_{peak} *(h)*	*NS (%)*	*PC (%)*
December 1996	0.53	–1.00	99.70	95.33
April 1997	–0.77	0.00	99.85	97.01
November 1997	1.86	–2.50	99.79	97.13
February 1999	–0.27	0.50	99.93	98.94
December 1999	2.53	1.50	99.49	75.67
December 2000	–0.82	–1.00	99.66	92.10
April 2001	1.06	–0.50	99.57	97.44
November 2005	–0.38	0.00	99.66	88.97
3rd December 2005	–0.48	–0.50	98.89	90.59
5th December 2005	0.39	0.00	99.59	94.12
30th December 2005	0.96	0.00	99.87	94.95
February 2006	3.40	0.00	98.86	74.01
Mean absolute value	1.12	0.63	99.57	91.36

Table 4. As for Table 2, but for a lead time of 2.0 hours

Event	*err_y*$_{peak}$ *(%)*	*err_t*$_{peak}$ *(h)*	*NS (%)*	*PC (%)*
December 1996	0.96	–0.50	99.33	94.06
April 1997	–0.31	–2.00	97.38	92.83
November 1997	2.60	–3.50	99.40	95.33
February 1999	0.43	0.50	99.62	96.54
December 1999	2.94	2.00	98.79	66.30
December 2000	–0.12	–8.50	99.30	90.53
April 2001	3.72	0.00	97.79	92.29
November 2005	–0.65	1.00	99.36	88.06
3rd December 2005	1.62	–8.00	95.54	77.98
5th December 2005	1.51	–3.00	98.58	88.34
30th December 2005	1.17	1.50	99.67	92.65
February 2006	5.70	0.50	97.20	62.68
Mean absolute value	1.81	2.58	98.50	86.47

Table 5. As for Table 2, but for a lead time of 2.5 hours

Event	*err_y*$_{peak}$ *(%)*	*err_t*$_{peak}$ *(h)*	*NS (%)*	*PC (%)*
December 1996	3.82	–4.50	97.81	87.22
April 1997	0.77	–1.50	97.93	84.36
November 1997	5.78	–4.00	98.17	90.65
February 1999	3.16	–4.00	98.18	89.21
December 1999	3.63	2.50	97.04	45.34
December 2000	3.95	–8.50	98.17	83.65
April 2001	7.92	–1.00	90.73	78.61
November 2005	–0.94	2.00	98.86	86.05
3rd December 2005	7.74	–7.50	86.25	54.76
5th December 2005	5.87	–4.00	94.66	71.37
30th December 2005	1.75	0.50	98.88	83.90
February 2006	7.81	1.50	94.07	47.23
Mean absolute value	4.43	3.46	95.90	75.20

in the considered reach, it is expected that the efficiency of the model would be poorer in forecasting the flow depth when that event is associated with significant lateral flow. Though the error update model can, to some extent, improve the forecasts in the event of lateral flow, it may not give reliable forecasts when there is significant lateral flow in the reach.

It can be seen from Figs 7 to 10 and from the forecast results of other events (not shown herein) that for almost all the events studied the update error model overestimates the forecast error when the rate of increase of rising limb suddenly decreases resulting in increased forecast error around this time zone. The minimum PC estimated for the forecasted events is greater than 60%, except for three events (December 1999, 3rd December 2005 and February 2006) out of which two events are characterized by significant lateral flow.

Overall, the results presented herein show that the hydrometric data-based VPMS model can be efficiently used as the forecasting model for practical river engineering problems. The other advantages

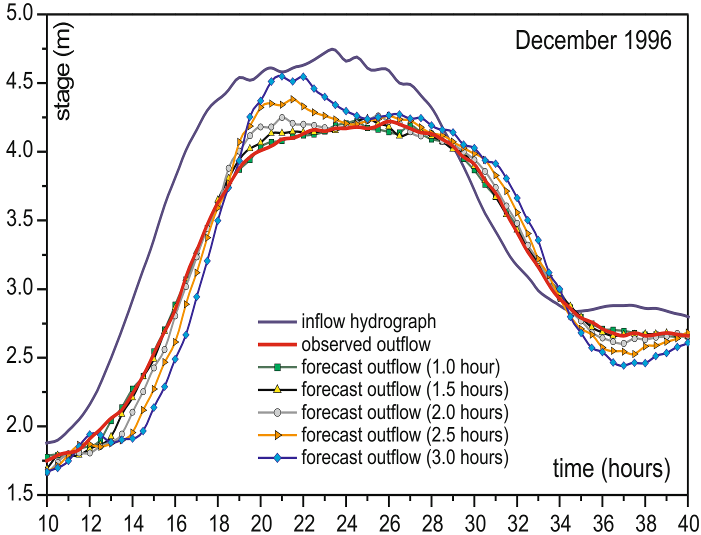

Fig. 7 December 1996 event: comparison between the observed and forecasted stage hydrographs for different lead times at Ponte Felcino section. The input stage hydrograph at Pierantonio site is also shown.

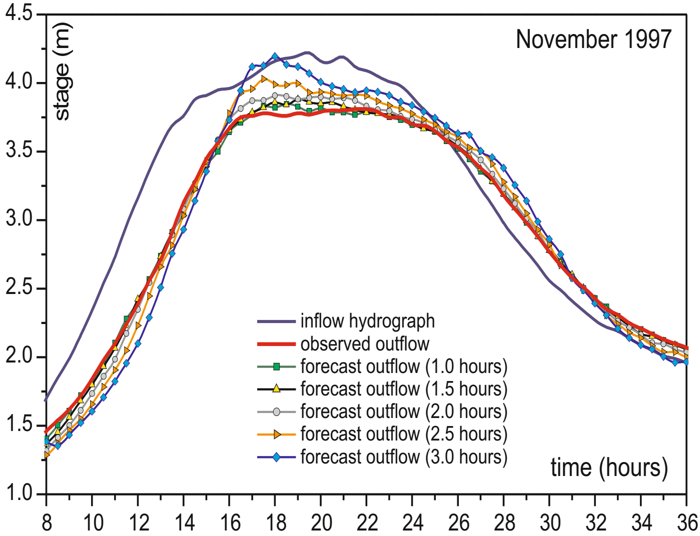

Fig. 8 As for Fig. 7, but for the event of November 1997.

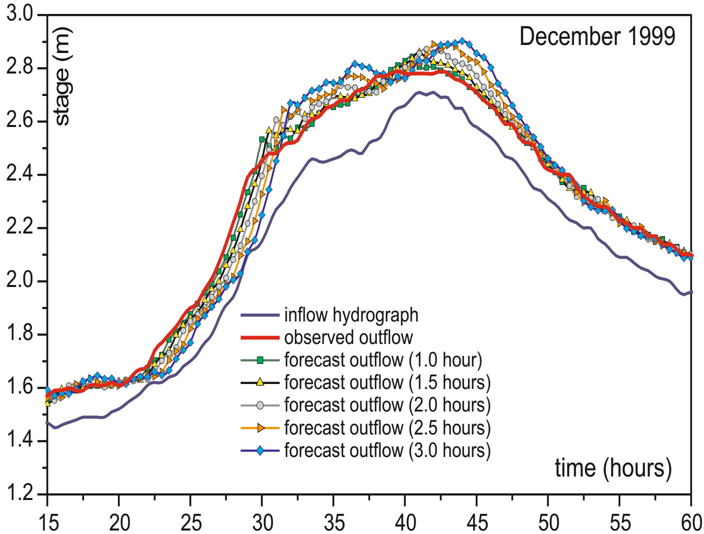

Fig. 9 As for Fig. 7, but for the event of December 1999.

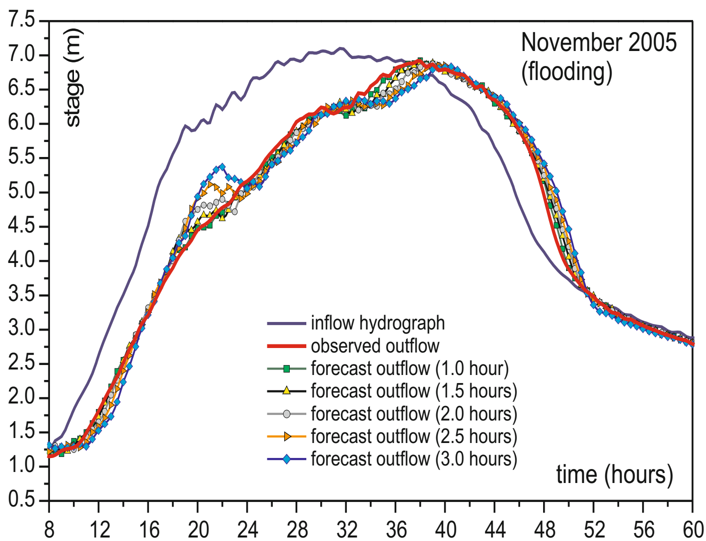

Fig. 10 As for Fig. 7, but for the event of November 2005.

Table 6. As for Table 2, but for a lead time of 3.0 hours

Event	*err_y$_{peak}$ (%)*	*err_t$_{peak}$ (h)*	*NS (%)*	*PC (%)*
December 1996	7.79	–5.00	94.74	78.10
April 1997	2.65	–7.50	95.48	75.72
November 1997	10.09	–3.50	96.26	86.55
February 1999	11.26	–3.50	95.27	80.04
December 1999	4.02	4.50	95.87	45.41
December 2000	8.79	–8.50	96.23	75.88
April 2001	13.58	–0.50	79.09	65.16
November 2005	–1.22	2.50	98.15	84.07
3rd December 2005	13.06	–6.50	74.75	39.59
5th December 2005	10.42	–3.50	90.50	63.89
30th December 2005	2.46	0.00	97.80	77.68
February 2006	9.85	2.00	90.88	41.20
Mean absolute value	7.93	3.96	92.09	67.77

of this model includes its capability to compute the downstream discharge hydrograph corresponding to the routed or forecasted downstream stage hydrograph, establishment of sectional rating curves, and celerity-stage, celerity-discharge relationships in ungauged and semi-gauged river basins (e.g., Perumal et al., 2007).

7. APPLICABILITY CRITERIA FOR THE VPMS MODEL

Identifying a suitable simplified method for application to a given flood routing problem is a difficult task. Several researchers have attempted to provide criteria for the selection of the appropriate routing methods (Henderson, 1966; Woolhiser and Liggett, 1967; Ponce et al., 1978; Daluz, 1983; Fread, 1985; Ferrick, 1985; Price, 1985; Dooge and Napiorkowski, 1987; Marsalek et al., 1996; Moussa and Bocquillon, 1996; Singh, 1996; Tsai, 2003) for the application to a given routing problem with or without considering any downstream boundary condition. Among these criteria, the one introduced by Ponce et al. (1978) has found its place in standard textbooks (French, 1986; Ponce, 1989; Chaudhry, 1993; Viessman and Lewis, 1996; Singh, 1996). However, the criteria by Ponce et al. (1978) were established on the basis of at least 95% accuracy in the wave amplitude when compared with the dynamic wave after one propagation period. The linear stability analysis used in arriving at these criteria considers the first-order approximation of the shallow water wave propagation which is treated as an infinitesimal disturbance imposed to the initially steady uniform flow. The common features of these criteria include the assumptions of a prismatic channel and a sinusoidal wave of arbitrary amplitude. However, in reality, flood waves found in natural rivers differ significantly from the assumption of sinusoidal shape and also they exhibit nonlinear behavior. Hence, the assumptions behind the development of these criteria are inherently contradictory with the characteristics of real life flood waves.

Ferrick and Goodman (1998) pointed out that large amplitude flow increases of practical interest must be described by the nonlinear equations. Since linear stability theory is valid for small perturbations from the reference flow, and the real world flood waves are frequently very large in amplitude, the linear analysis used in the development of the Ponce et al. (1978) applicability criteria is questionable (Crago and Richards, 2000). Further, Zoppou and O'Neill (1982) tested the criteria of Ponce et al. (1978) for a real life flood routing problem of a 33.2 km reach of the Australian river Yarra between Yarra Grange and Yering, for assessing the applicability of the diffusive and kinematic wave models as approximations to the dynamic wave model. A good agreement was obtained in all cases studied using the kinematic wave model, despite the criteria of Ponce et al. (1978) predicting that it would be unsuitable for routing under these circumstances. On the basis of these considerations, Zoppou and O'Neill (1982) and Perumal and Sahoo (2006) cautioned the river engineers and hydrologists about the limitations of these criteria. In light of subsequent development of improved simplified methods, which have moved from the domain of complete linear models to that of variable parameter models, duly accounting for the non-linear characteristics of a flood wave, this caution seems to have a greater significance. Hence, the applicability criteria advocated by Ponce et al. (1978) for identifying an appropriate flood wave model for a given routing problem may be replaced by alternative criteria with physical significance, given the non-linear mechanism of flood wave propagation in real-world rivers.

Perumal and Sahoo (2007) showed that one of the logical ways of developing these alternative criteria can be by directly incorporating the magnitude of the scaled water profile gradient $(1/S_o)(\partial y/\partial x)$, which is used for the classification of flood waves (Henderson, 1966; NERC, 1975) as kinematic or diffusive. In fact, such an applicability criterion was advocated by Price (1985) for the simplified routing method developed by him, but it is too restrictive with $|(1/S_o)(\partial y/\partial x)| \leq 0.05$. The hydrograph characterized by the presence of $(1/S_o)(\partial y/\partial x)$ signifies a diffusive flood wave and its absence signifies a kinematic flood wave (NERC, 1975). The scaled gradient can be estimated at every routing time level of the given hydrograph at the inlet of the routing reach.

On the basis of an extensive study by Perumal and Sahoo (2007), it is revealed that the applicability of the VPMS model to be assessed at the inlet of the reach for routing a given hydrograph requires satisfying the criteria $(1/S_o)(\partial y/\partial x)_{\max} \leq 0.79$ and $(1/S_o)(\partial y/\partial x)_{\max} \leq 0.63$ for stage routing and discharge computation, respectively.

8. CONCLUSIONS

River stage forecasting at any downstream ungauged site plays a vital role in a comprehensive and coordinated planning for flood hazard mitigation and evacuation work. The adaptive parameter estimation methods employing the Kalman filtering technique may not be worth the effort for real-time flood forecasting (Ahsan and O'Connor, 1994). In such a scenario, the application of the simplified physically based model like the variable parameter Muskingum stage-hydrograph routing (VPMS) method along with a simple error updating technique, such as the one proposed in this chapter, is much more useful for real-time flood forecasting at a river gauging site. Based on the forecasting performance for different events investigated, it can be inferred that the proposed model has the potential for practical forecasting applications in hydrometric data-based modelling provided that the adopted forecasting lead time is not longer than the mean wave travel time of the selected river reach. Further investigations through different case studies need to be carried out in order to verify the proposed forecasting model accuracy. Furthermore, as a future study, the model formulation presented in this chapter may be extended by accounting for significant lateral flow contribution entering along the selected river reach.

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