Water Resour Manage (2016) 30:5687–5698
DOI 10.1007/s11269-016-1438-z

Design and Application of an Adaptive Time Delay Model for Flow Routing in Prismatic Trapezoidal Geometry River Reach

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Received: 30 November 2015 / Accepted: 11 July 2016 /
Published online: 21 July 2016

Abstract Simplified flow routing model is favourably used for control-based application because it does not only present acceptable results but also is computationally inexpensive. Recently, the Time Delay model (TD) with two parameters, time constant and time delay has been developed in order to approximate the river flow in a very wide rectangular profile. This paper presents an advancement we thereafter call Adaptive Time Delay model (ATD) that expands the application scope of the TD Model by simulating the flow using a prismatic trapezoidal geometry. Firstly, the mathematical representation of the ATD model and the linearized Saint Venant model (SVE) are defined. Secondly, the transfer functions of the ATD model and the complex hydraulic model (SVE) are obtained by Laplace transformation. Finally, the Taylor expansion technique is used to find cumulants of the two transfer functions, and consequently equating the cumulants to derive time constant and time delay of the ATD model as functions of the complex hydraulic model parameters. By applying the fourth order Runge Kutta numerical scheme the flow rate and water level at downstream reach end are simulated. The innovation of this research is that both water stage and flow rate are derived through optimization. The performance of the ATD Model is also presented and compared to the TD Model in a case study. The extension of the time delay model does not only issue more accurate results but also introduces more outcomes like flow rate, and relation curves between time delay and time constant with discharge that might be useful in flood forecasting and other purposes in water resources operation.

Keywords Time delay · Time constant · Flow rate · Water level

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1 Introduction

One of the most frequent natural disasters is flood, which critically influences a large part of population on earth. The consequences of flood are loss of human lives and damage to properties, which have continuously risen over the years due to climate change. The intensity and duration of flood is significantly depending on rainfall regime, catchment area morphology, land use of a watershed, and operation policies of upstream dams. Because of the crucial role of flood in human life, different characteristics of flood have been comprehensively studied in years.

Flood wave propagation is very significant for hydraulic engineers to develop several methods for its analysis and simulation. The characteristics of physical phenomena including river morphology, initial condition and flow can be spatially and temporally illustrated by function of flow rate and water level. In his paper (Saint-Venant Ad 1871), Saint Venant derived a hyperbolic system expressed by partial differential equations to describe the one dimensional flow in open channels. Because of the inexistence of analytical solutions, a wide range of numerical approaches to resolve these equations have been developed, such as the method of characteristics (Abbott 1966), finite difference schemes (Abbott 1979; Harley 1967; Fread 1985; Garcia and Kahawita 1986; Stoker 1957), finite element schemes (Cooley and Moin 1976; Szymkiewicz 1991), finite volumes schemes (Cozzolino et al. 2011, 2014a, b, DAniello et al. 2015), spectral (finite) volume schemes (Wang 2002; Kannan and Wang 2012; Cozzolino et al. 2012), smoothed particle hydrodynamics (Monaghan 1992; Zhou et al. 2004; Liu and Liu 2010; Violeau and Rogers 2016). For practical purposes, numerous efforts have been made to simplify the full Saint Venant equation (SVE) in order to reduce computation times and data demand while ensuring a reasonable result. These include the well-known classical methods such as the Hayami model (Hayami 1951), Muskingum model (Cunge 1969), Kalinin-Milyukov method (Apollov et al. 1964). The theory of Hayami is originally recognized as an analytical solution of diffusive wave model with an assumption of constant celerity and diffusivity, and without lateral flow. Since many years, improvement of Hayami model for simulating and controlling open channel flow has been presented, e.g., in Bolea et al. (2010), Moussa (1996), Wang et al. (2014), and Cimorelli et al. (2013, 2014). The original Muskingum technique, which includes two parameters: travel time K and storage weighting factor x, is based on the equation of continuity and storage discharge relationship. It has been commonly applied and innovated in river flow simulation, for instance, by Bhuyan et al. (2015), Franchini et al. (2011), and OSullivan et al. (2012). The Kalinin-Miliyukov method defines a characteristic river reach by assuming that a discharge at a cross section of a reach is not influenced by lateral flow, and is also a linear function of water storage within that reach. It is extended for stream flow prediction by Szilagyi (2003, 2006).

Recently, control theory methods have been applied to river flow modeling. The fundamental approach is to transform the linearized SVE (or its reduced form) to frequency domain and then to approximate its transfer function by different mathematical techniques (Baume et al. 1998). In this way new models for describing flow movement can be obtained. A popular example is an Integrator Delay Method (ID), which was firstly applied in water systems for controller design in low frequencies by Schuurmans et al. (1995, 1999). An upgrade named Integration Delay Zero (IDZ), in which a zero is placed in a system transfer function in order that the model is capable to simulate flow in high frequencies was developed by Litrico and Fromion (2004a). By focusing on the resonance sensitivity of waves in channels, the Integrator resonance model (IR) was derived for modeling rivers, which are short, flat and deep (van Overloop et al. 2010, 2014). One of the most recent work by Cimorelli et al. (2015) introduces a new model which can be used for real time forecasting and optimization. In which, the analytical solution for a cascade of diffusive channel has been derived in Laplace Transform domain accounting for downstream boundary conditions.

In addition, the application of the first order time delay model for river flow simulation has been also developed. For instance, using the first order time delay model with two constant parameters, time delay and time constant to simulate the outflow of a river reach is proposed by Rauschenbach (2001) and Pfuetzenreuter and Rauschenbach (2005). A similar, but more advanced approach is that transforms the linearized SVE into the frequency domain and approximate its solution by a transfer function of first order delay system by Dooge et al. (1987) and Litrico and Fromion (2004b). The relation between parameters of both models is that time constant and time delay are functions of the physical parameters of the river, such as roughness, bed slope, surface width, river reach length, and so forth so that those can vary in response with changes of flow rate. Despite the significant enhancement, the approach with its assumption of the river cross section with infinite width is not applicable to rivers where the cross section has a width smaller than 10 times the flow depth (Chow 1959). Furthermore, due to the significance of reservoirs system management in general as well as flash flood forecasting in particular, it is vital to extend the model to prismatic trapezoidal channel cross sections in order to not only broaden the models applicability, but also to improve its accuracy. Other studies of Cimorelli et al. (2013) and Cimorelli et al. (2014) introduce a reduced model of SVE named Linear Parabolic Approximation (LPA) developed by ignoring inertial terms of SVE. An analytical solution is then obtained in terms of discharge and water level variations, taking account of backwater effect and downstream lateral flow. This has not been mentioned in previous time delay models. Therefore, the different point of the adaptive time delay model (ATD) does not only approximate the full SVE but also use both water level and the flow rate to calculate the model parameters. This can be considered as an advancement of previous time delay models (Schuurmans et al. 1999; Pfuetzenreuter and Rauschenbach 2005; Litrico et al. 2010). Based on a similar approach to Litrico et al. (2010) the paper illustrates a methodology for a simplified prismatic trapezoidal river cross section that produces water level and discharge as outputs. Moreover, the advantage over the model in Rauschenbach (2001) is also proven by comparing the goodness of fit of both in a case study.

The remainder of this paper describes the methodology applied in Section 2, while the model will be applied to a case study and its performance evaluated in Section 3.

2 Methodology

The methodology is motivated by the fact that for a trapezoidal geometry, the surface width changes in response to the fluctuation of water level in a cross section in each time step, whereas it is constant in the case of a rectangular channel. This indicates that water level is a very significant variable, which should be adapted accordingly during the calculation of the parameters of the ATD model. First, we will implement the ATD method based on the scheme outlined in Litrico et al. (2010) and Munier et al. (2008). The algorithm consists of 3 steps:

– Define the two physical models (linearized SVE and ATD);
– Derive the transfer functions of linearized SVE and the adaptive time delay (ATD) model and

– Determine the parameters of the ATD model from parameters of the linearized SVE by Taylor expansion and moment matching.

These steps will be described in detail in the following sections.

2.1 Definition of Physical Models

Firstly, the structure of the ATD model is described as in Eq. 1. It is a nonlinear system with delay

$$\begin{cases} \dot{q}(t) = f(q(t), Q_{in}(t)) \\ Q_{sim}(t) = h(q(t - T_d(q))) \end{cases} \tag{1}$$

According to the Lemma 1 in Litrico et al. (2010) the system (1) is linearized round an equilibrium point Q_o to derive the linear system with delay as follows.

$$\begin{cases} T_c \frac{dq}{dt} + q(t) = Q_{in}(t) \\ Q_{sim}(t) = Gq(t - T_d) \end{cases} \tag{2}$$

where G is the gain, T_c is the time constant, T_d is the time delay, $Q_{in}(t)$ is the inflow, $q(t)$ is the state of the system and $Q_{sim}(t)$ is the simulated outflow. Then the system (2) is used to approximate the linearized SVE.

Secondly, the linearized SVE is written for a river of length $L(km)$ assuming uniform flow and absence of lateral flow (Litrico and Fromion 2004b) as follows:

$$B\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = 0 \tag{3a}$$

$$\frac{\partial q}{\partial t} + 2V\frac{\partial q}{\partial x} + \frac{2gS_b}{V}q + \left(C^2 - V^2\right)B\frac{\partial y}{\partial x} - gB(1+K)S_b = 0 \tag{3b}$$

where $q(x,t)(m^3/s)$ is the deviation of flow rate from equilibrium value Q_o, $y(x,t)(m)$ is the deviation of water level from equilibrium value Y_o, $C(x)(m/s)$ is the celerity, $V(x)(m/s)$ is the mean velocity, $g(m/s^2)$ is the gravitational acceleration, $S_b(m/m)$ is the river bed slope, and $B(m)$ is the water surface width.

The boundary conditions of the system includes: $q(0,t)$ is the upstream inflow at station $(0\ \ km)$, and $q(L,t)$ is the outflow discharge at the downstream station. The parameter K in Eqs. 3a–3b is presented as follows:

$$K = \frac{7}{3} - \frac{4A}{3BP}\frac{\partial P}{\partial y} \tag{4}$$

with the wetted perimeter $P(m)$ and the cross sectional area $A(m^2)$.

2.2 Derivation of Transfer Functions

In this step the physical models are transferred into frequency domain. Main aim of transferring these models to frequency domain is to explicitly represent the relationship between the inflow and outflow. Following a similar approach as in Litrico et al. (2010) and Munier et al. (2008), the Laplace Transformation is applied to Eqs. 2 and 3a–3b to derive the transfer functions given in Eq. 5b for the ATD model and Eq. 5d for the linearized SVE model, respectively.

$$q\,(L,s) = \frac{Ge^{-sT_d}}{1+sT_c}q(0,s) \tag{5a}$$

$$TF_{ATD}(L,s) = \frac{Ge^{-sT_d}}{1+sT_c} \tag{5b}$$

$$q\,(L,s) = e^{\varepsilon_2 L}q(0,s) \tag{5c}$$

$$TF_{SVE}(L,s) = e^{\varepsilon_2 L};\ \varepsilon_2 = as + b - \sqrt{cs^2 + ds + b^2} \tag{5d}$$

$$a = \frac{Fr}{C(1-Fr^2)};\ b = \frac{(1+K)BS_b}{2A(1-Fr^2)};\ c = \frac{1}{C^2(1-Fr^2)}^{2} \tag{5e}$$

$$d = \frac{S_b B}{VA}\frac{(2+(K-1)Fr^2)}{(1-Fr^2)^2};\ Fr = \sqrt{\frac{q^2 B}{gA^3}} \tag{5f}$$

Where $q(L,s)$ is the outflow at station $L, q(0,s)$ is the inflow at station 0, s is the Laplace operator, Fr is the Froude number for trapezoidal channel. In Eqs. 5a–5f, the backwater effect is ignored by assuming that the river has infinite downstream length.

2.3 Determination of the ATD Model Parameters from the Linearized SVE Model

To compute outflow of the river, the TF of SVE must be transferred to time domain, which is not that simple with this hyperbolic system. In this paper we apply the approach suggested in Dooge et al. (1987) and Litrico and Fromion (2004b) approximating the TF of SVE by TF of ATD model in which the cumulants of both transfer functions are equated. As the SVE will be approximated by the first order delay model, the accuracy of the simulated discharge will be enough by determining the first three cumulants of both TFs as described in the following steps:

– Firstly, Taylor expansion is applied to both TFs up to the second order as in Eq. 6.

$$TF\,(x,s) = M_1\,(x) + M_2\,(x)\,s + M_3\,(x)\,s^2 + 0\left(s^3\right) \tag{6}$$

– Secondly, the cumulants of both TFs are derived by taking the logarithm of the TFs to second order (r = 2).

$$Cu\,[TF\,(x,s)] = (-1)^R\frac{d^r}{ds^r}(\ln[TF(x,s)])_{s=0} \tag{7}$$

– and finally the cumulants of the TF of SVE are matched to those of the ATD model and consequently the ATD parameters are defined as follows:

$$T_d = \frac{2L}{(1+K)V} - T_c;\ T_c = \sqrt{\frac{4-(K-1)^2Fr^2}{gS_bFr^2(1+K)^3}2L};\ G = 1 \tag{8}$$

These parameters will then be applied in the ATD model (2) in order to simulate outflow of a river reach. For a trapezoidal channel, Fr and K in Eq. 8 are defined as a function of y, and Q. With an assumption of uniform river flow, the water level y which is related to Q

will be estimated by the method of the section factor $AR^{2/3}$ for uniform flow computation (Chow 1959):

$$\mathrm{AR}^{2/3} = \frac{\mathrm{nQ_{sim}}}{\sqrt{\mathrm{S_b}}};\ \mathrm{A} = (\mathrm{b} + \mathrm{my})\,\mathrm{y};\ \mathrm{R} = \mathrm{A/P};\ \mathrm{P} = \mathrm{b} + 2\mathrm{y}\sqrt{1 + \mathrm{m}^2} \qquad (9)$$

The left side of Eq. 9 is a section factor $AR^{2/3}$ depending on the geometry of the water area (water level y, side slope m, and bottom width b) while the right sides is determined by Manning coefficient n, discharge Q_{sim}, and bed slope S_b. To find the value y, optimization technique is applied to solve the set of equations. Water level y is then used to approximate instantaneous hydraulic characteristics of the channel, and T_c, T_d subsequently. Afterwards, the fourth order Runge Kutta is utilized to integrate the ATD system in Eq. 2 to the next system state.

3 Case Study

To illustrate the performance of the ATD model, a small river reach located on an upstream part of Thu Bon River in central area of Vietnam is selected which is shown in Fig. 1. The

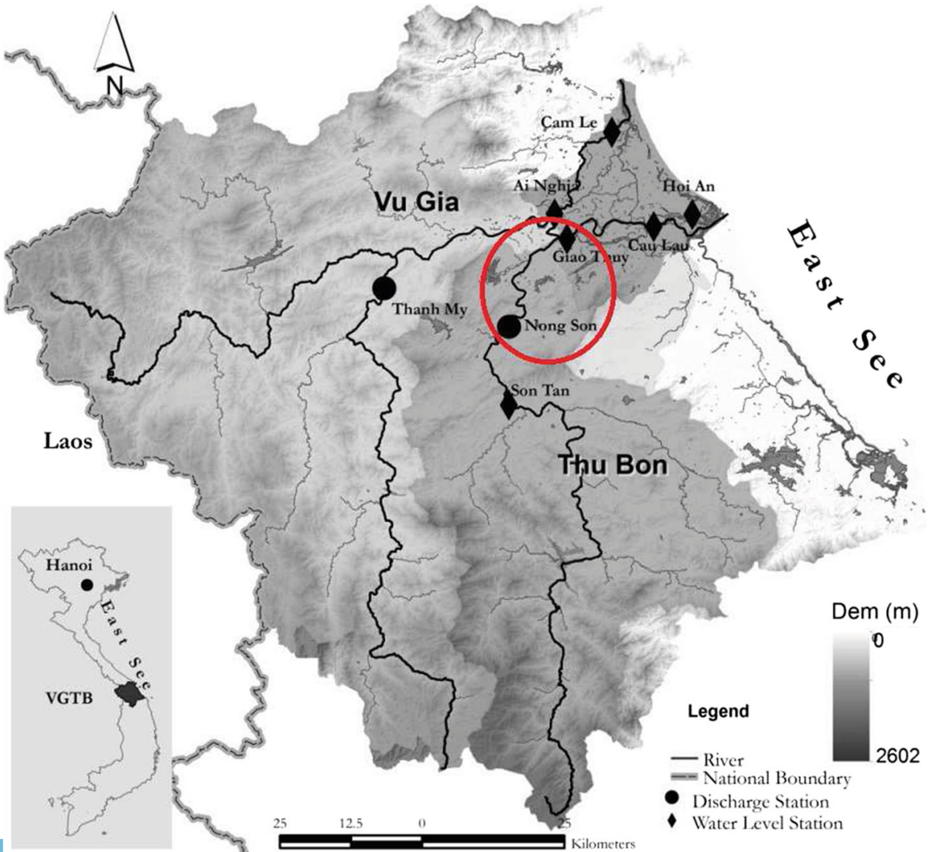

Fig. 1 Location of the selected river reach at Vu Gia Thu Bon river basin in Central Vietnam

Table 1 Characteristic of studied river reach at upstream Vu Gia Thu Bon river basin

Geometry data			Flow data	
L (km)	b (m)	m	Daily upstream observed flow rate at Nong Son station	Daily downstream observed water level at Giao Thuy station
22	40	2	1 year period of 2010	1 year period of 2010

reach starts from Nong Son gauge station and ends at Giao Thuy gauge station. The geometry data of the river reach are listed in Table 1. 1-year-period upstream and downstream flow data at both gauge stations are considered as referenced data for the simulations in this paper. According to the collected data at Giao Thuy, water depth reaches from 5.0 to 8.0 m during main flood season. This indicates that ATD with trapezoidal geometry should be utilized to enhance good accuracy in estimating the flood peak in this narrow but deep stream. The idea is that the ATD model parameters will be calibrated using the referenced data for 3 months of main flood season (October–December) to obtain the relation curves Q & T_c and Q & T_d for a wide range of discharge Q. After calibration the model is validated using reference data from the drought season (January-June) and compared to the model given in Rauschenbach (2001).

3.1 Model Calibration and Validation

The model calibration is basically an optimization technique that minimizes a quadratic error of simulated and observed outflow Q or water stage y. The two possible objective functions are expressed in Eq. 10. As can be seen in the data Table 1, many physical parameters such as the Manning coefficient n, and river bed slope S_b required to estimate the

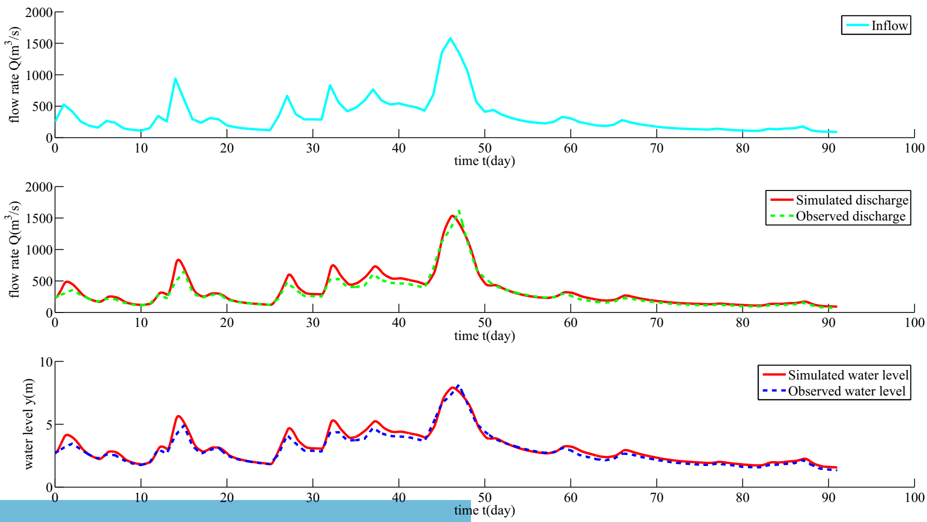

Fig. 2 Model calibration results for the two objective functions for discharge and water level

Table 2 Results of calibration, validation and estimated model parameter

Criteria	Calibration		Validation		Model parameter	
	Q	y	Q	y	n	Sb
NSE	0.94	0.94	0.91	0.9	0.035	0.0014
PBIAS	−10.5	−6.5	0.68	4.06		

outputs y_{sim} and Q_{sim} were not available. Therefore, these were also estimated during the optimization process to achieve a best match between simulated and observed outputs.

$$\min \sum_{i=1}^{n} \left(y_{obs}^{i} - y_{sim}^{i} \right)^2 ; \min \sum_{i=1}^{n} \left(Q_{obs}^{i} - Q_{sim}^{i} \right)^2 \tag{10}$$

The ATD model calibration results are depicted in Fig. 2. The model uses geometry data in Table 1 and 3 months of upstream main flood to simulate the downstream flood of which returns both flow rate and water level. By comparing the results with referenced data at downstream, the model presents a quite well goodness of fit as expressed through Nash-Sutcliffe Efficiency (NSE) (11) and Percent bias (PBIAS) (12).

$$\mathrm{NSE} = 1 - \left[\frac{\sum_{i=1}^{n} \left(Q_{obs}^{i} - Q_{sim}^{i} \right)^2}{\sum_{i=1}^{n} \left(Q_{obs}^{i} - Q_{mean}^{i} \right)^2} \right] ; \mathrm{NSE} = 1 - \left[\frac{\sum_{i=1}^{n} \left(y_{obs}^{i} - y_{sim}^{i} \right)^2}{\sum_{i=1}^{n} \left(y_{obs}^{i} - y_{mean}^{i} \right)^2} \right] \tag{11}$$

$$\mathrm{PBIAS} = 1 - \left[\frac{\sum_{i=1}^{n} \left(Q_{obs}^{i} - Q_{sim}^{i} \right) 100}{\sum_{i=1}^{n} Q_{obs}^{i}} \right] ; \mathrm{PBIAS} = 1 - \left[\frac{\sum_{i=1}^{n} \left(y_{obs}^{i} - y_{sim}^{i} \right) 100}{\sum_{i=1}^{n} y_{obs}^{i}} \right] \tag{12}$$

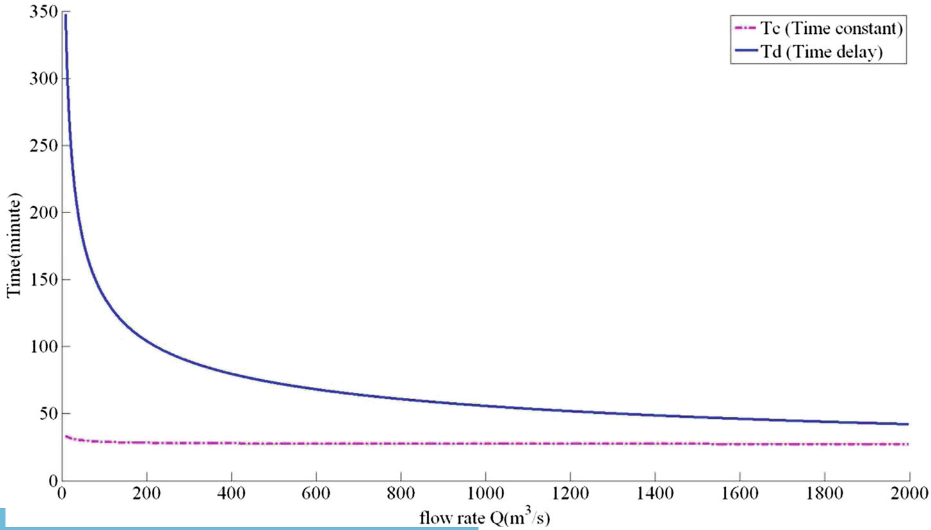

Fig. 3 Relation of Time constant T_c and Time delay T_d with inflow Q

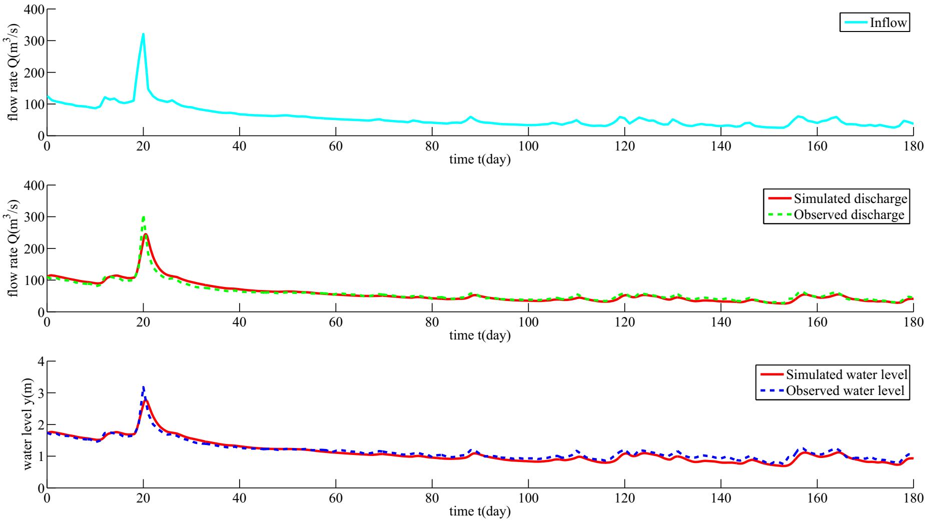

Fig. 4 Model validation in response with discharge and water level

The Table 2 shows the NSE coefficient equals to 0.94 for both Q and y while PBIAS reaches -10.5 for Q and 6.5 for y. This indicates that the ATD model returned a very good result according to the guidelines of evaluating stream flow models in Moriasi et al. (2007).

During calibration the nonlinear relationships between the time constant T_c, the time delay T_d with discharge Q are determined as depicted in Fig. 3. The linearly interpolated parameters values from these curves are directly used by ATD model to simulate downstream flood in different upstream flood scenarios. This approach significantly reduces the computation time as well as raises feasibility for real time flood simulation application. In the Table 2, n and S_b were also determined as 0.035 and 0.0014, respectively. Based on

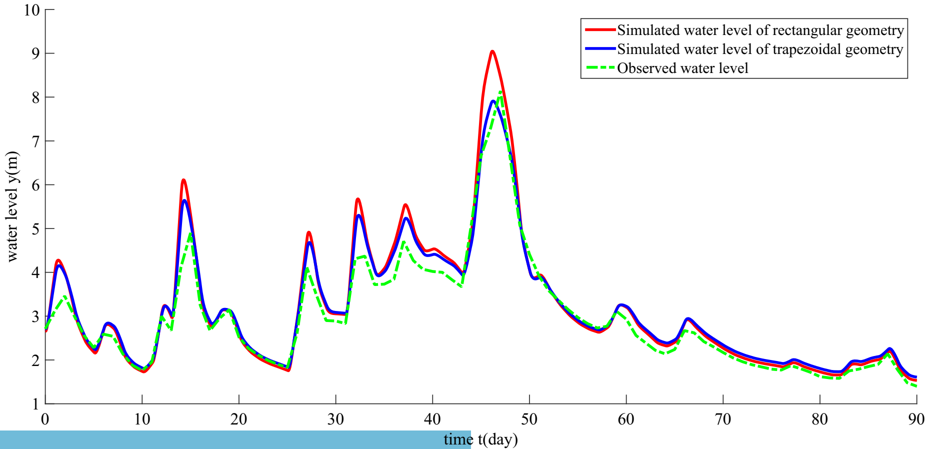

Fig. 5 Comparison between ATD model with trapezoidal and rectangular geometry

Table 3 Result of 2 models comparison

Criteria	Trapezoid	Rectangular
NSE	0.94	0.89
Deviation (m)	0.2	1

estimated parameters, it should be a deep river with stone and weeds at the bottom that is in accordance with attributes of the selected river reach located in mountainous area of Vu Gia Thu Bon basin at which agriculture and forest land exist (Mai 2009).

The validation task is executed by applying the calibrated curves of T_c , and T_d to simulate outflow for 6 months of drought period. The result is depicted in Fig. 4, where the model shows a very good result presented by the NSE of 0.91 for Q and 0.90 for y as well as by $PBIAS$ of 0.68 for Q and 4.06 for y. As in Moriasi et al. (2007) this proves that the model is valid for application to flow routing of this river reach.

3.2 Comparison for both Time Delay Models

The performance of the ATD model in comparison to the model in Rauschenbach (2001) based on rectangular profile is also evaluated for the main flood season. The measured water level is used to evaluate the goodness of both models. The technique for approximating water level from section factor (Chow 1959) is applied to simulate the water level from the flow rate estimated by the model in Rauschenbach (2001). The results are shown in Fig. 5 and Table 3. It can be seen that the water level of the ATD model matches with observed stage better than the water level of the model in Rauschenbach (2001). In terms of NSE, the accuracy of ATD model for this case study is 0.94 compared to 0.89 of the model in Rauschenbach (2001). In Fig. 5, it is clear that assuming a rectangular profile overestimates the water level peak of 8.1 m in period time t from the day 45 to 47 by approximately 1.0 m compared to the ADT model which underestimates by only 0.2 m. Therefore, the ATD model returns more accurate results for the same hydrological conditions. This has very important role for decision maker who can determine an effective plan of flood prevention and excavation.

4 Conclusion

In this paper an adaptive time delay model based on a prismatic trapezoidal geometry was introduced. The model is an extension of the river model developed in Rauschenbach (2001) for very wide rectangular cross section channel. The method uses moment matching to derive the parameters of the model from linearized SVE model. The application scope of the time delay models is also now opened for small deep rivers. After calibration of the model to obtain the nonlinear relation curves of T_c and T_d with Q, the outflow simulation for different scenarios becomes very simple, fast and accurate. Therefore, this method can be utilized to simulate, and design control strategies for river systems. However, taking account of backwater effect in this ATD model is not considered in this work. This subject would be studied thoroughly. Although the application of the model in case study showed very promising results, the further improvement with downstream boundary conditions may expand the applicability of the model.

Acknowledgments The great acknowledgement is expressed to DAAD for financial support and to my colleagues at Fraunhofer Institute IOSB-AST, Ilmenau University of Technology, and at Institute for Technology and Resources Management in the Tropics and Subtropics, Cologne University of Applied Sciences. The authors thank the anonymous reviewers for their significant remarks.

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