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COMPARISON BETWEEN EXPERIMENT AND COMPUTED FLOOD FLOW CHARACTERISTICS IN A COMPOUND MEANDERING CHANNEL

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Extensive development in river flood plains should be achieved in parallel with minimization of potential inundation risk of over-bank flooding. Flood risks of property damage and loss of life can be eliminated through accurate flow predictions and by proper application of mitigating measures such as flow regulation structures. This paper presents a comparison between experimental results and a 2-dimensional unsteady flow numerical model simulations of flood propagation in a compound meandering channel. The model is based on a finite volume discretization on a staggered grid with upwind scheme in flux is presented, it handles drying and wetting process for a flood plain, complex geometry and discontinuities, which are the main requirements for modeling compound channel flows. The main objective of this research is to advance the existing knowledge on flood flow phenomenon in compound channels through comparative studies of unsteady flow experiments and numerical simulations. Comparison between measured data and model simulations show generally good agreement.

Keywords: *Flood propagation, compound meandering channel and unsteady flow*

1. INTRODUCTION

Natural channels/rivers are seldom straight and in some cases compound sections are depicted: two-stage channel consisting of deep main channel with flood plains. Compound channels are most advocated due to their efficiency hydraulically compared to single channels. In an effort to understand compound channel flows, extensive research efforts have been made in quantifying of stage-discharge relationships in channels with inundated flood plains. Although great advances have been achieved to understand flow interactions between main channel and flood plains, most numerical investigations have concentrated on steady flow situations. Among few unsteady flow studies performed on compound meandering channel, investigation on flood flow storage and attenuation of peak discharge was performed[1].

Unsteady flow in compound channel involves a stage in which the flow overtops the bank and inundates the flood plain so that part of the flow is carried by the flood plains. In those situations the relative depth varies from zero to some value depending on the discharge. With low relative depths, momentum exchange between main channel and flood plain is high due to slow moving flood plain flow and fast moving main channel flow. On the contrary, with high relative depths momentum exchange is weak, since the fluid has no slow moving regions. Additionally, simple resistance relationships have been shown not to hold for compound channel flows. Many researchers have tried to explain either of these phenomenon's through compound channel steady flow experiments and 2-or 3-dimensional steady flow numerical models[1-9]. A peculiar categorization of compound channel flows was identified through experiments and field data analysis, that, over-bank flow with relative depth of up to 0.26 still had characteristics of single channel flow [1-4].

The flow in compound channels is highly 3-dimensional created by bed generated turbulence and velocity variation transversally and longitudinally, specifically in the interface region between main channel and flood plain. Flow analysis in natural rivers is challenging; the flow complexity arises due to morphological river changes, bed formations resulting from scouring and depositions, changes in discharges with time, roughness variability due to numerous reasons such as bed size material and vegetation. Additional factors that affect flow in natural rivers are existence of dead storage areas in the flood plain, flow short circuits on the meandering path and flow interaction between main channel and flood plain of compound

$$\frac{\partial}{\partial t}\left(\frac{h}{J}\right)+\frac{\partial}{\partial \xi}\left(\frac{Q^{\xi}}{J}\right)+\frac{\partial}{\partial \eta}\left(\frac{Q^{\eta}}{J}\right)=0.0 \qquad [1a]$$

$$\frac{\partial}{\partial t}\left(\frac{Q^{\xi}}{J}\right)+\frac{\partial}{\partial \xi}\left(\frac{UQ^{\xi}}{J}\right)+\frac{\partial}{\partial \eta}\left(\frac{VQ^{\xi}}{J}\right)-\frac{uh}{J}\left(U\frac{\partial \xi_x}{\partial \xi}+V\frac{\partial \xi_x}{\partial \eta}\right)-\frac{vh}{J}\left(U\frac{\partial \xi_y}{\partial \xi}+V\frac{\partial \xi_y}{\partial \eta}\right)=$$

$$-gh\left(\frac{\xi_x^2+\xi_y^2}{J}\frac{\partial z}{\partial \xi}+\frac{\xi_x\eta_x+\xi_y\eta_y}{J}\frac{\partial z}{\partial \eta}\right)-\frac{\tau_b^{\xi}}{\rho J}+\frac{\xi_x^2}{J}\frac{\partial}{\partial \xi}(\tau_{xx}h) \qquad [1b]$$

$$+\frac{\xi_x\eta_x}{J}\frac{\partial}{\partial \eta}(\tau_{xx}h)+\frac{\xi_y^2}{J}\frac{\partial}{\partial \xi}(\tau_{yy}h)+\frac{\xi_y\eta_y}{J}\frac{\partial}{\partial \eta}(\tau_{yy}h)+\frac{\xi_x\eta_y+\xi_y\eta_x}{J}\frac{\partial}{\partial \eta}(\tau_{xy}h)+\frac{2\xi_x\xi_y}{J}\frac{\partial}{\partial \xi}(\tau_{xy}h)$$

$$\frac{\partial}{\partial t}\left(\frac{Q^{\eta}}{J}\right)+\frac{\partial}{\partial \xi}\left(\frac{UQ^{\eta}}{J}\right)+\frac{\partial}{\partial \eta}\left(\frac{VQ^{\eta}}{J}\right)-\frac{uh}{J}\left(U\frac{\partial \eta_x}{\partial \xi}+V\frac{\partial \eta_x}{\partial \eta}\right)-\frac{vh}{J}\left(U\frac{\partial \eta_y}{\partial \xi}+V\frac{\partial \eta_y}{\partial \eta}\right)=$$

$$-gh\left(\frac{\xi_x\eta_x+\xi_y\eta_y}{J}\frac{\partial z}{\partial \xi}+\frac{\eta_x^2+\eta_y^2}{J}\frac{\partial z}{\partial \eta}\right)-\frac{\tau_b^{\eta}}{\rho J}+\frac{\xi_x\eta_x}{J}\frac{\partial}{\partial \xi}(\tau_{xx}h) \qquad [1c]$$

$$+\frac{\eta_x^2}{J}\frac{\partial}{\partial \eta}(\tau_{xx}h)+\frac{\xi_y\eta_y}{J}\frac{\partial}{\partial \xi}(\tau_{yy}h)+\frac{\eta_y^2}{J}\frac{\partial}{\partial \eta}(\tau_{yy}h)+\frac{\xi_x\eta_y+\xi_y\eta_x}{J}\frac{\partial}{\partial \xi}(\tau_{xy}h)+\frac{2\eta_x\eta_y}{J}\frac{\partial}{\partial \eta}(\tau_{xy}h)$$

meandering channel. Numerical analysis of these flows, requires 3-dimensional model[4] or depth averaged k-ε models that have performed well[6, 10]. However, for practical applications a 2-dimensional depth averaged model is chosen to simplify the analysis. By depth averaging the 2-dimensional model ignores the secondary flow effects. Secondary flow-induced stresses makes significant contribution to the total apparent shear stress[8] used in 1-dimensional approach engineering designs. For two stage channels the secondary flows greatly extend the lateral shear layer width on the flood plain.

This paper presents a comparison between unsteady flow laboratory experimental results and 2-dimensional numerical simulations. It describes an improved numerical model, which earlier version was developed by Nagata et. al [11]. The model is formulated based on finite volume on a staggered grid with upwind scheme in flux. Steady and unsteady flows can be simulated; different flow regimes such as sub-critical, supercritical and discontinuities can be handled. The method has been adopted due to its capability of handling wetting and drying process of the flood plain, which is important for modeling unsteady compound channel flows. Original Nagata et. al's approach could not simulate well laboratory unsteady compound meandering channel flow results. Two major items of the method were modified. The mathematical formulation of turbulent shear stress terms was changed from the semi-empirical Reynolds approach of Nezu and Nakagawa to commonly used parabolic eddy viscosity formula, which assumes Boussinesq eddy-viscosity concept. In addition, the ways of implementation of open boundary conditions and initial conditions was changed. For upstream and downstream open boundaries, zero-order extrapolation method was changed to method of characteristic that behaves well according to the direction of waves[12]. The wall condition was explicitly assigned no flux crossing through both wall sides and steady state condition was used as input for initial condition. The model is used to simulate laboratory experimental results[2] in a sine generated compound meandering channel. Comparison between measured data and model simulations show generally good agreement. This shows how useful the model can be in prediction of flood propagation in channels. However for application of the model to natural rivers further investigation is needed because of different factors involved in natural rivers.

2. MODEL DESCRIPTION

(1) Governing Equations

Two-dimensional depth averaged continuity equation and momentum equations are derived from depth integration of 3-dimensional Navier Stokes equations. From the 2-dimensional depth integrated momentum equations, transformation to natural coordinate system was done, and by mathematical manipulation using metrics, to obtain the equations [1a]-[1c] above with contravariant fluxes. Where, t is time, h and z (=h+z_b) are water depth and water level elevation respectively, z_b is the ground level elevation. u and v are depth averaged velocities in x and y directions respectively, g is the acceleration due to gravity. The turbulent shear stress terms in x,

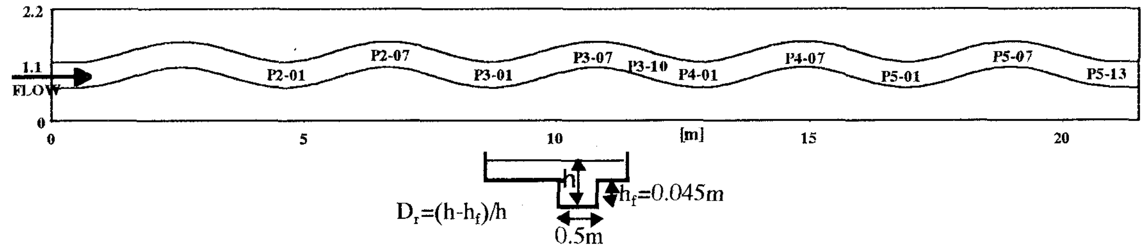

Fig.1 Channel layout plan and measuring points and cross section

$$\tau_{xx} = 2\varepsilon\left(\frac{\partial u}{\partial x}\right);\ \tau_{xy} = \tau_{yx} = \varepsilon\left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}\right);$$
$$\tau_{yy} = 2\varepsilon\left(\frac{\partial v}{\partial y}\right);\ \varepsilon = \frac{\kappa h u_*}{6};\ u_* = \sqrt{\tau_{bx}/\rho}, \qquad [2a]$$

$$\tau_{bx} = \frac{\rho g n^2 u\sqrt{u^2+v^2}}{h^{1/3}};\ \tau_{by} = \frac{\rho g n^2 v\sqrt{u^2+v^2}}{h^{1/3}};$$
$$n = \frac{h^{1/6}}{\sqrt{g}\left(6.0 + 5.75\log_{10}\left(\frac{h}{k_s}\right)\right)} \qquad [2b]$$

$$\xi_x = J y_\eta;\ \eta_x = -J y_\xi;\ \xi_y = -J x_\eta;\ \eta_y = J x_\xi;$$
$$J = 1/\left(x_\xi y_\eta - x_\eta y_\xi\right)$$
$$U = \xi_x u + \xi_y v;\quad V = \eta_x u + \eta_y v; \qquad [2c]$$
$$Q^\xi = \xi_x uh + \xi_y vh;\quad Q^\eta = \eta_x uh + \eta_y vh$$
$$\tau_b^\xi = \xi_x \tau_{bx} + \xi_y \tau_{by} \text{ and } \tau_b^\eta = \eta_x \tau_{bx} + \eta_y \tau_{by}$$

x-y and y directions are computed as shown in equation [2a].

Despite the deficiencies of zero-equation models, depth averaged turbulent eddy viscosity coefficient, ε has been adopted due to computational ease of application and its contribution to many practical applications. κ is the Von Karman constant (=0.41) and u_* is the shear velocity. Expressions for boundary shear stress and Manning's roughness coefficient are shown in [2b]. k_s is the equivalent roughness. The metrics and contravariant velocities, discharges and boundary shear stress terms are defined as shown below.

(2) Method of solution

Equation [1a] to [1c] were solved by an explicit numerical scheme that is 2nd order accurate in time and 1st order accurate in space. The numerical scheme is an improved version Nagata et. al's approach[11] that uses Adam's Bashforth method [6] and upwind scheme in flux for time and space advancement respectively. Finite volume discretization was employed on a staggered grid arrangement for velocity components in order to avoid pressure discontinuities at the interface. The grid used was non-orthogonal. On the open boundaries, upstream and downstream boundaries, a unidirectional flow condition was implemented. Additionally, on the upstream and downstream boundaries measured discharge hydrograph and measured water depth hydrograph was given respectively. The stream-wise velocity on both boundaries was computed by 1-dimensional method of characteristics. For the solid wall boundaries, a slip condition was implemented and no flux crossing through the wall.

Wetting and drying was considered by predefining a minimum water depth (**h_{st} = 0.001m**). When the water depth in the cell under consideration and its surrounding cells was less than **h_{st}** then fluxes in that cell during that time step were assigned a value of zero, then computation advanced to next time step. In the transverse direction the flow was evaluated based on both **h_{st}** and water elevation, because sometimes gravity effect takes place, so the fluxes cannot be assigned zero. Stability requirement was not explicitly accounted for in the numerical model, time step was chosen sufficiently small and constant throughout the computation time based on trials.

(3) Model Application

Numerical simulation was done on experimental data[2] for the compound meandering channel in **Fig.1**. The experimental channel was 21.5m long, 0.045m-bankfull depth, and 0.5m wide sine generated main channel with one wavelength 4.1m. Total width of the channel including flood plains was 2.2m and channel slope of 1/500. Measurement points were located as P2-01, P2-07, P3-01, P3-07, P4-01, P4-07, P5-01, P5-07 and P5-13. The measured water depth and velocity on these points were compared with computed results. Two sets of experiments (Exp.1 and Exp.2) in the same channel conditions were used to verify the simulation results. Input discharge hydrograph with maximum discharge of 18l/s in Exp.1 and input discharge hydrograph with maximum discharge of 30l/s in Exp.2 with corresponding measured downstream water depth hydrographs are shown in **Fig. 2**.

3. RESULTS AND DISCUSSION

(1) Effect of roughness

In these numerical simulations, water depth on the upstream boundary condition was computed by uniform flow Manning's formula using input

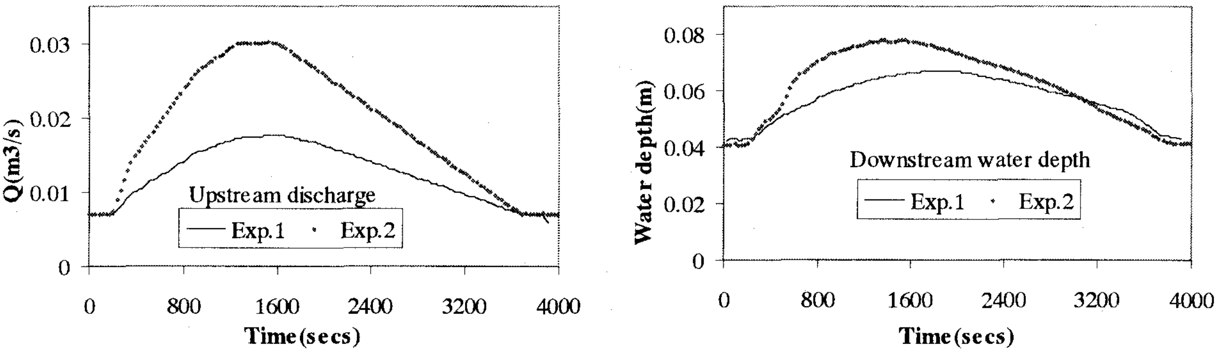

Fig.2 upstream input discharge hydrograph and downstream water depth hydrograph for Exp.1 and Exp.2

discharge hydrograph. The stream-wise velocities for upstream boundary computed by 1-dimensional method of characteristic, required accurate Manning's roughness coefficient distribution with time. Composite temporal Manning's roughness coefficient for the compound section at the upstream boundary was calculated from measured water surface profiles using one-dimensional non-uniform flow equation. These distributions with time are shown in **Fig.3** for Exp.1 and Exp.2.

Previous unsteady flow studies in compound channel on Manning's roughness coefficients are scarce. Nevertheless, Fukuoka and Fujita[5] and Myers and Brennan[7] performed steady flow studies, in these studies the variation of Manning's roughness coefficient with relative depth was noted. Both studies observed a decrease of Manning's roughness coefficient at small relative depths, depending on the method of discharge analysis. Later, an increase to normal values as the relative depth increased further. This happens as explained by those studies to be partly caused by the reduction of hydraulic radius at small relative depths. A relatively small increase in discharge, when the flow is above bankful causes large increase in wetted perimeter for almost constant area, thus reduction in hydraulic radius. As the relative depth increases further, large increase in area is accompanied by almost same wetted perimeter, thus increase in hydraulic radius. An increase or decrease in hydraulic radius results to an increase or decrease of Manning's roughness coefficient respectively.

In this study, it was observed that varying the temporal distribution of Manning's roughness coefficient at the upstream boundary improved the results of computed water depth hydrograph at other sections. For both simulations upstream roughness distribution with time showed great variations during transition from in-bank to over-bank and the other way round. The trend agrees with previous steady flow studies.

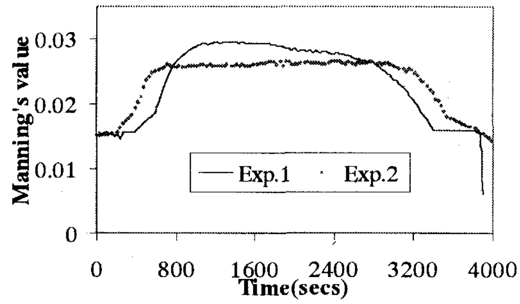

Fig.3 Temporal distribution of upstream Input Manning's roughness coefficient

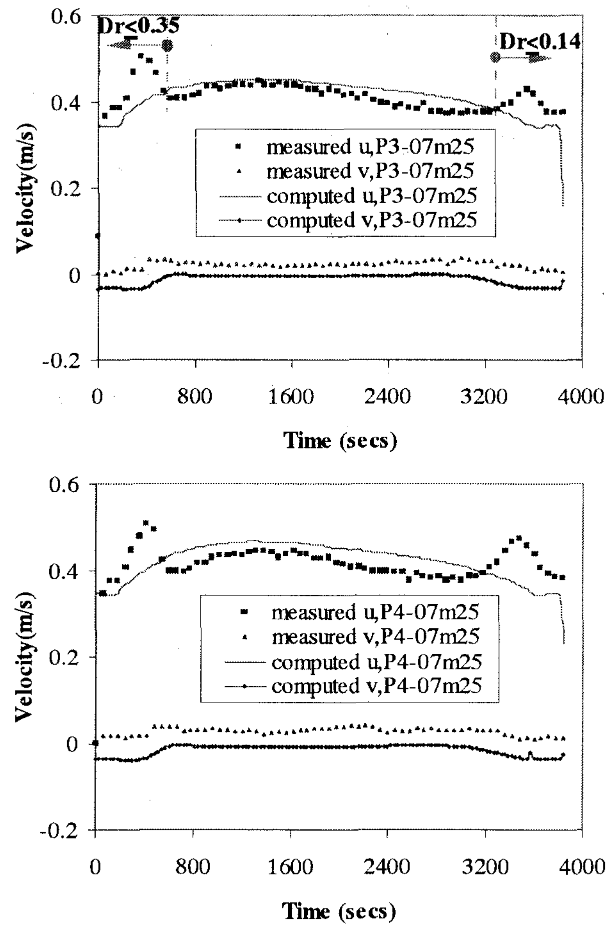

Fig.4 Computed & Experiment 1 Velocity-ydrograph

(2) Water depth-hydrographs

Fig.5 and **Fig.6** present the comparison of simulated water depth hydrograph and experimental results respectively, at respective locations indicated in **Fig.1**. The results are for the centerline of the main channel denoted by m25. The agreement between the comparison is generally good. The model can be used for prediction of flood wave height as it propagates in channels.

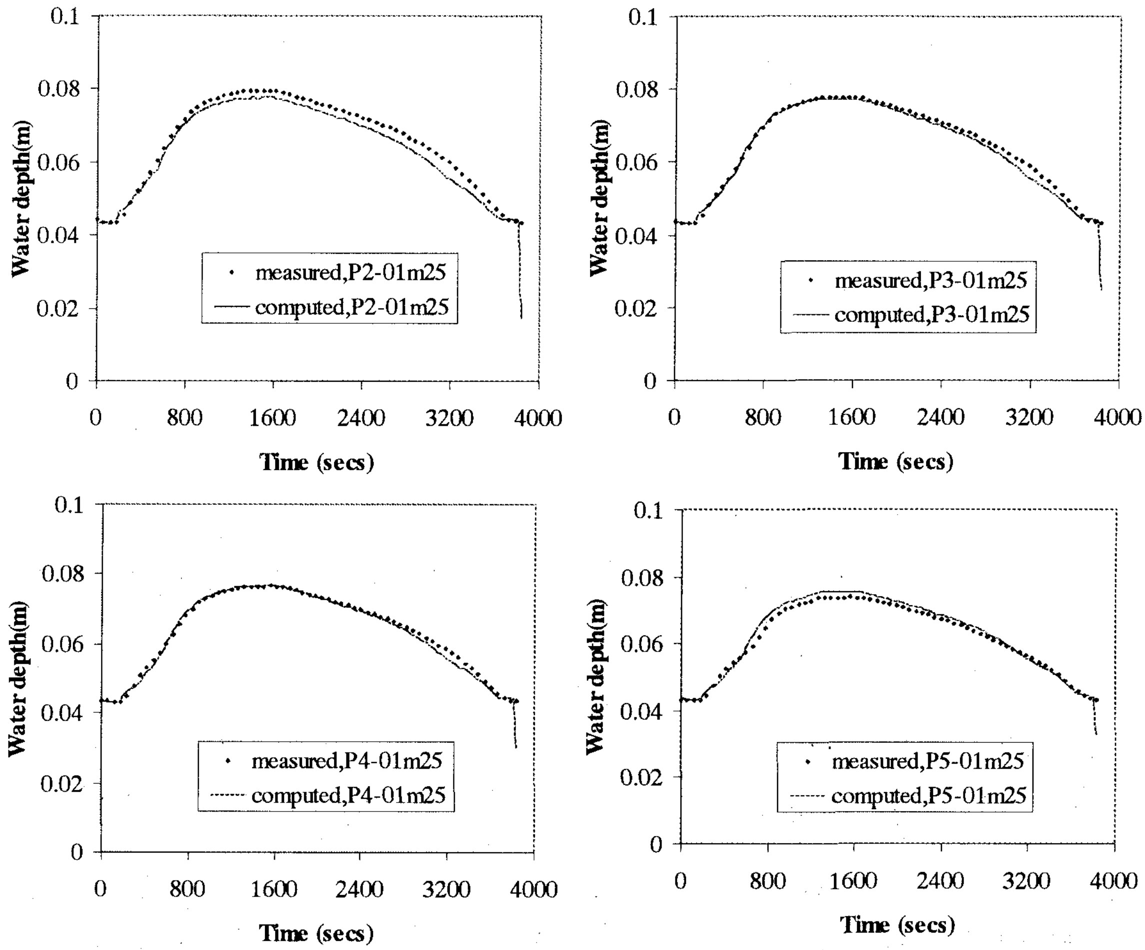

Fig. 5 Comparison between water depth-hydrograph for experiment 1 and computations

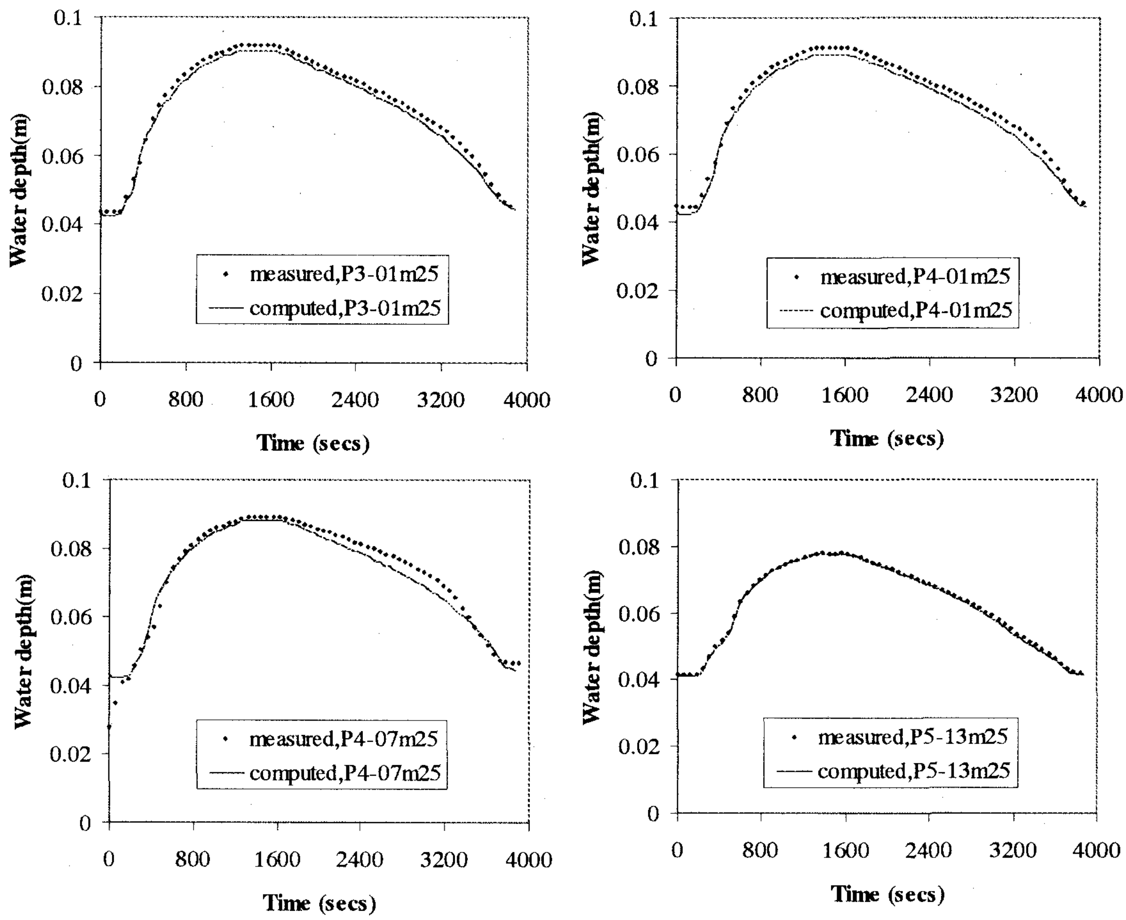

Fig. 6 Comparison between water depth-hydrograph for experiment 2 and computations

(3) Velocity –hydrographs

Computed velocity hydrographs presented in **Fig. 4** shows reasonably good agreement for large relative depths (800s $\leq$ t $\leq$ 3300s). With small relative depth there is a discrepancy in the velocity hydrograph. However, for river engineering design purposes, the values of the velocity at high relative depths are important compared to low relative depth values. Nevertheless, in order to improve the numerical model, investigation to account for discrepancy at small relative depth is undertaken. Some of the reasons for this are: first, roughness variability with water depth. During transition from in-bank flow to over-bank flow, large variation of roughness coefficient was observed, this confirms other studies that, roughness formulas such as one in [2b] do not hold for small relative depths. Second, order –1 upwind schemes are not sufficiently accurate in most practical purposes. Effort is made to upgrade the numerical scheme to second order accuracy in space. Third explanation is the depth averaged turbulent eddy viscosity coefficient is inaccurate at the transition from in-bank flow to over-bank flow near the main channel flood plain interface. This can be explained by strong flow interaction due to fast moving main channel flow and slow moving flood plain flow, when the flow inundates the flood plain.

4. CONCLUSION

An improved two-dimensional model, which earlier version was developed earlier by Nagata et. al [11], based on finite volume approach was able to simulate well the water depth hydrograph. The good agreements between measured and computed water depth hydrograph demonstrate how useful it can be for flood prediction in channels. However for application of the model to natural rivers further investigation is needed.

Spatial variation of hydraulic variables and parameters such as water depth, velocity, roughness coefficient and depth averaged turbulent eddy viscosity coefficient have been a focus of many steady flow studies. This unsteady flow study shows additional temporal change of those hydraulic variables and parameters need to be considered during flood flow.

In these computations the variability of Manning's roughness coefficient with time is highly significant for small relative depths, when the flow interaction is a large. In addition, the accuracy of upstream and downstream input data is very important for unsteady flow computations.

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