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An investigation of the relationship between the flood wave speed and parameters in runoff-routing models

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Abstract An important aspect of flood management is the estimation of catchment response to storm events by simulation of flood wave propagation through the catchment storage. A derivation of the wave speed (c) is presented in terms of discharge (Q) and the exponent (m) in the storage (S)–discharge relationship $S = kQ^m$. The wave speed–discharge relationship has the form $c = (\alpha^m{}_c/m)Q^{1-m}$, where α_c is a parameter in the kinematic wave model. The parameters in this relationship, however, change with flow conditions in the cross section, especially as flow leaves the main channel to inundate the floodplain. This finding confirms that the parameters in the single storage–discharge relationship do not have discrete values as used in many runoff-routing models, and hence the use of single parameter values is not suitable for estimating floods in many situations. It is shown that a more reasonable relationship is $S = S_c + k^1LQ^{1/b}$, where S_c is a threshold storage, and the other parameters are functions of the flow conditions. The parameters k^1, L, b and also the parameters in the wave speed-discharge relationship change between inbank, overbank, and fully developed floodplain flow situations. This paper investigates the relationship between the kinematic wave exponent ($1/\beta_c$) and discharge that gives an estimate of ($1/b$) which is not provided by the wave speed–discharge relationship. This estimate is shown to be approximately equal to m, the exponent conventionally used in the storage–discharge relationship $S = kQ^m$. Further, the kinematic wave exponent ($1/\beta_c$) can be approximated by the ratio between the water velocity and the flood wave speed. This analysis is used to explain the complicated behaviour of wave speed and water velocity in channel, overbank, and fully developed floodplain flows.

Etude de la relation entre la vitesse d'onde de crue et les paramètres des modèles d'acheminement du ruissellement

Résumé Un aspect important de la gestion des inondations est l'estimation de la réponse du bassin hydrologique aux événements orageux, réponse obtenue en simulant la propagation de l'onde de crue a travers le réseau de stockage. Une expression de la vitesse d'onde a été établie en fonction de débit (Q) et de l'exposant (m) de la relation stockage–débit $S = kQ^m$. La relation vitesse d'onde–débit prend alors la forme $S = S_c + k^1LQ^{1/b}$, où α_c est un paramètre du modèle d'onde cinématique. Tous les paramètres de cette relation varient avec les conditions d'écoulement dans la section transversale, tout particulièrement quand le flux quitte le canal principal pour envahir la zone inondable. Ce résultat confirme que les paramètres de la relation univoque stockage–débit n'ont pas les valeurs discrètes utilisées par de nombreux modèles d'acheminement du ruissellement. Par conséquent l'utilisation des valeurs de ces paramètres uniques ne peut permettre l'estimation des inondations dans de nombreuses situations. Les auteurs montrent que la relation $S = S_c + k^1LQ^{1/b}$, où S_c est un seuil de retenue et où les autres paramètres sont fonctions des conditions d'écoulement, est plus réaliste. Les paramètres k^1, L, b ainsi que les

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paramètres de la relation vitesse d'onde–débit varient avec les situations d'écoulement: dans le lit de la rivière, débordement hors des berges ou complètement en plaine d'inondation. Cet article étudie une relation entre l'exposant d'onde cinématique ($1/\beta_c$) et le débit donnant une valeur approchée de $1/b$ ce que ne permet pas la relation vitesse d'onde–débit. L'étude montre que cette valeur estimée est approximativement égale à m, l'exposant conventionnellement utilisé dans la relation $S = kQ^m$. Plus précisément, l'exposant cinématique d'onde ($1/\beta_c$) peut-être assimilé au rapport entre la vitesse de l'eau et la vitesse de l'onde de crue. Cette analyse a été utilisée pour expliquer le comportement complexe de la vitesse d'onde et de la vitesse de l'eau lors d'écoulements en canal mais aussi hors du lit du canal voire complètement en plaine d'inondation.

INTRODUCTION

Of the many problems involving unsteady flows in catchments, the classical problem is the movement of flood waves through the catchment. The process of tracing this movement is referred to as flood routing which is described by Chow *et al.* (1988) as the procedure which is used to determine the time and magnitude of flow at a point on a watercourse. Alternative flood routing techniques place different emphases on the processes that influence flood wave motion. For many engineering purposes, simple procedures for estimating runoff and flood routing need to be adopted to explain complicated phenomena involving the unsteady and nonuniform flow in flood wave movement along channels. Hydrological flood routing techniques are the result of one such simplification. The procedure usually applied in these techniques is based on the basic differential equation describing continuity of mass. In the application of this equation in hydrological routing techniques, it is assumed that channel storage is a function of inflow to and outflow from the channel reach. A storage–discharge relationship linking the outflow rate to the storage in the system is required for hydrological routing techniques to be feasible. The form of the storage–discharge relationship widely used by Australian and Japanese hydrologists and being adopted more frequently by hydrologists in other countries has a nonlinear form which can be expressed algebraically as:

$$S = k\,Q^m \tag{1}$$

where S is the storage in the system with dimensions of L^3, Q is the discharge (L^3T^{-1}), k is a dimensional empirical coefficient (T when $m = 1$), and m is a dimensionless exponent related to the storage characteristics. Many numerical models have been created based on the assumption that catchment storage behaves in a nonlinear manner ($m \neq 1.0$). These numerical models include RORB developed by Mein *et al.* (1974), and WBNM developed by Boyd *et al.* (1979). As implemented in these models, a single function (i.e. only one set of parameter values) is used to describe the storage discharge relationship over the full range of discharges likely to be encountered during the period of flood wave simulation. This study, as one of its primary aims, considers the applicability of this assumption through an investigation of the relationship between the wave speed and parameters in the storage–discharge relationship.

Wong & Laurenson (1983) concluded, in their empirical studies of wave speed for six Australian river reaches, that the form of the storage–discharge relationship in equation (1) inadequately described catchment storage characteristics. Their study attempted to understand the movement of the flood wave and to link it with runoff and flood routing estimates. They suggested that a better storage–discharge relationship would consist of two power functions, describing, respectively, inbank and floodplain flows, and a yet to be determined function describing flow between these two. The wave speed–discharge relationship, like the storage–discharge relationship is also different between inbank and floodplain flows. Further, this relationship needs to be defined for the transition region between these two flows. The present study aimed to investigate whether the wave speed–discharge relationship can be directly applied to the storage–discharge relationship, as suggested by Wong & Laurenson (1983), or whether some other form of relationship is needed.

The software package known as Rubicon (Haskoning, 1992) was used to estimate flood hydrographs and flood characteristics in the channel systems. The basis of Rubicon is an implicit finite difference numerical solution of the Saint Venant flow equations. Even though catchment storage consists of both overland flow storage and channel flow storage, in most natural catchments the channel system provides the major portion of the storage. This assumption has been the basis of studies by, for example, Bates & Pilgrim (1983), Wong & Laurenson (1983) and Yu & Ford (1993). Therefore, the link between the wave speed–discharge relationship and the storage–discharge relationship for river reaches developed in this present study should have direct applicability to both runoff and flood routing studies.

FLOOD ROUTING TECHNIQUES

Flood routing techniques can be classified into two major categories: hydrological routing and hydraulic routing techniques. While hydrological routing techniques are based solely on the use of the continuity equation, hydraulic routing techniques are based on the use of both the continuity equation and the dynamic equation of motion. For one-dimensional flow in open channels, the most commonly used form of these equations are the St Venant equations. Many simplifications of the one-dimensional St Venant wave equations are based on neglecting terms in the equation of motion; one such simplification is the kinematic wave. The basis of the kinematic wave is the expression of the equation of motion as:

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t} \qquad (2)$$

where y is the flow depth, v is the cross sectional average velocity, x is the longitudinal distance along the flow path, t is the time and S_f and S_o are the friction slope (energy gradient for steady flow) and bed slope respectively. In many flow situations, some of the terms in equation (2) can be neglected resulting in the kinematic wave which was shown by Eagleson (1970) and Henderson (1966) to be:

$$S_f = S_o \qquad (3)$$

or

$$Q = Q_n \tag{4}$$

where Q and Q_n are the discharge and the normal flow discharge respectively. This equation of motion can be formulated as a relationship between discharge and flow area (A) which, as shown by Bedient & Huber (1988), can be expressed as:

$$Q = \alpha_c A^{\beta_c} \tag{5}$$

where α_c and β_c are the kinematic channel-routing parameters that can be correlated with the resistance terms. The set of hyperbolic partial differential equations formed by equation (5) and the continuity equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{6}$$

can be expressed in a characteristic form where the characteristic path in the space-time reference plane is defined by:

$$\frac{\mathrm{d}x}{\mathrm{d}t} = \alpha_c \beta_c A^{\beta_c - 1} \tag{7}$$

The wave speed (c) of the flood wave is obtained from the slope of the characteristic path as:

$$c = \frac{\mathrm{d}x}{\mathrm{d}t} \tag{8}$$

Since the water velocity, from equation (5), is given by:

$$v = \frac{Q}{A} = \alpha_c A^{\beta_c - 1} \tag{9}$$

then the relationship between the celerity (c) and water velocity (v) is:

$$c = \beta_c v \tag{10}$$

An estimate of the value of β_c can be computed using either the Manning or Chezy resistance equations. For wide rectangular, triangular or parabolic channels, the estimated values for β_c will be, respectively, 1.67, 1.33, and 1.44 using the Manning equation, and, respectively, 1.50, 1.25, and 1.33 using the Chezy equation (Miller & Cunge, 1975).

CASE STUDY

The Herbert River in Northern Queensland, which drains an area of 9400 km^2 to the South Pacific Ocean, was used as a case study during this investigation. The location map of the study area is shown as Fig. 1 (after Cameron McNamara Consultants, 1980).

Continuous flood records in this catchment have been collected for the Herbert River at Ingham (GS116001) since 1916, and at Abergowrie (GS116006) since 1969. The drainage areas are, respectively, 8005 km^2 and 7530 km^2 for these stations.

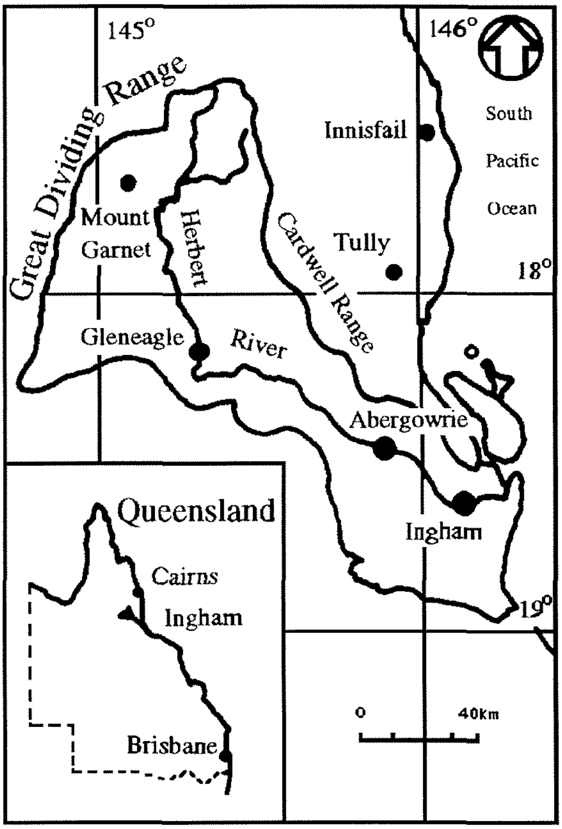

Fig. 1 Location map of the Herbert River catchment (after Cameron McNamara Consultants, 1980).

Between these two main stream gauging stations, there is another gauging station, located on a tributary at Stone River (GS116017), which has a drainage area of 168 km^2 and has continuous records since 1970.

Located near these three stations are five rainfall stations (at Abergowrie, Ingham, Upper Stone, Paluma and Cardwell) with hourly data. Moreover, geometric data were available for cross sections between the gauging stations at Abergowrie and Ingham. A schematic of all cross section locations on the Herbert and Stone Rivers is shown as Fig. 2.

METHODS OF FLOOD ESTIMATION

Simulation of flood hydrographs and flood characteristics for all cross sections between the upstream station at Abergowrie and the downstream station at Ingham was undertaken by using the Rubicon software package. Rubicon was developed by Haskoning (1992) for the simulation of steady and unsteady flows in open channel systems, and is based on the solution of the Saint Venant equations with a highly

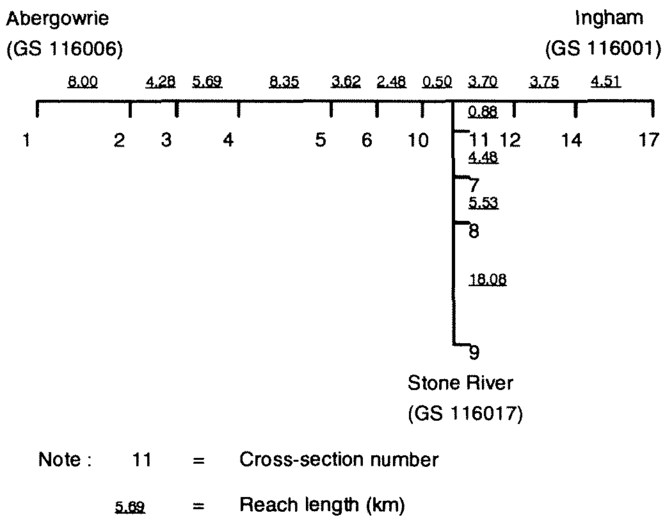

Fig. 2 Schematic of all cross section locations on the Herbert River used in the case study.

accurate and efficient modification of Preissmann's implicit finite difference scheme. It can be used, therefore, to solve a wide range of hydraulic engineering problems including complex flow over floodplains.

Results of hydrograph simulation

Four flood events which occurred in 1973, 1980, 1984 and 1986 were used for this investigation. The maximum flood discharges for these events were respectively, 3237, 4063, 1343 and 9510 $m^3 s^{-1}$. A flood-frequency study suggests that the 3-year flood at the site has a peak discharge of 4000 $m^3 s^{-1}$. The estimated flood hydrographs calculated using Rubicon compare very well with the observed hydrographs at the downstream gauging station located at Ingham for all four flood hydrographs. A comparison between the calculated and observed hydrographs for one of the four

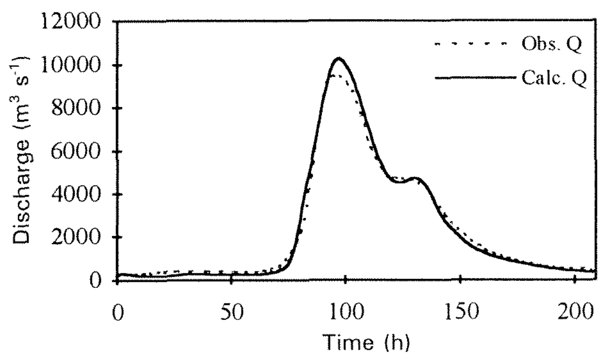

Fig. 3 Calculated and observed hydrographs, Ingham.

flood events (the 1986 event) is shown as Fig. 3. Data used for subsequent analysis of the storage–discharge relationship was extracted from these simulation results. Therefore, it was essential to ensure that the flood wave propagation obtained using Rubicon adequately reproduced the actual flood wave propagation.

INVESTIGATION OF THE STORAGE–DISCHARGE RELATIONSHIP

Both the discharge and storage within a reach were found to be related to the cross-sectional area at the downstream end of the reach. Analysis of the results shows that the storage–discharge relationship was governed more by the geometry of the flow cross section than by the magnitude of the flood flows or any other catchment feature such as catchment size. The exponent of the storage–discharge relationship (equation (1)) was found to change dramatically once overbank flow commenced. This finding suggests that the use of a single value for the exponent of the storage–discharge relationship throughout a flood may be inappropriate for estimating floods by runoff-routing (Sriwongsitanon *et al.*, 1994).

The outflow hydrograph from a particular reach was found by Sriwongsitanon *et al.* (1994) to be directly related to the downstream cross-sectional area (A) by:

$$Q = a\,A^b \tag{11}$$

where parameters a and b change with cross section geometry but are constant for a particular reach. It was possible also to relate storage within the reach to the cross section by the empirical relationship:

$$S = L(c_o + d_o A) \tag{12}$$

where L is the reach length between two adjacent cross sections, c_oL represents the S intercept in the storage *vs* cross-sectional area relationship, d_oL is the slope of the relationship, and c_o and d_o are constant for a river reach.

A relationship between storage and discharge can be derived by equating the cross-sectional area A in equations (11) and (12), resulting in:

$$S = L\left[c_o + \left(\frac{d_o}{a^{\frac{1}{b}}}\right)Q^{\frac{1}{b}}\right] \tag{13}$$

Equation (13) can be rewritten as:

$$S = S_c + k^1 L Q^{\frac{1}{b}} \tag{14}$$

where $S_c = c_oL$ and $k^1 = d_o/a^{1/b}$. The value of S in equation (1) is the absolute storage above a specific datum such as cessation of flow (Pilgrim, 1987). In equation (14) the specific datum of the storage volume is S_c, the threshold storage. Since storage routing is concerned only with changes in storage, the value of this threshold is irrelevant and, consequently, is ignored in flood estimation using runoff-routing techniques. Consequently, from equations (1) and (14), kQ^m and $k^1LQ^{1/b}$ are equivalent and thus:

$$\frac{1}{b} = m \tag{15}$$

The value of $1/b$ normally varies from 0.60 to 1.0 for inbank flows and increases to about 3.0 for overbank flows. The larger $1/b$ values reflect the large increases in storage in the reach as overbank flow increases. This storage increase is accompanied by momentum transfer between the main channel and the floodplain, with a consequent reduction of water velocity and wave speed. When the flow depth increases further, the water velocity and wave speed begin to increase again and the exponent $1/b$ falls. Experiments conducted by Nalluri & Judy (1985) showed that as the roughness on the floodplain increases, the velocity decreases and the difference between the main channel and the floodplain mean velocities becomes larger, exhibiting a stronger interaction. Therefore, the difference in roughness and, hence, flow resistance between the main channel and the floodplain influences the variation of the exponent $1/b$ (Sriwongsitanon *et al.*, 1994).

Relationship between the parameter b in the storage–discharge relationship and the kinematic wave routing parameter β_c

Sriwongsitanon *et al.* (1998) showed, using Figs 4 and 5, that the parameter b in equation (11) is very close to the kinematic wave routing parameter β_c in equation (5) for both inbank and overbank flow situations. With a kinematic model, the parameter β_c can be calculated from equation (10) where the wave speed, c, is determined from the rating curve for the cross section as:

$$c = \frac{\mathrm{d}Q}{\mathrm{d}A} = \left(\frac{1}{B}\right)\frac{\mathrm{d}Q}{\mathrm{d}y} \tag{16}$$

where B is the free surface width.

Shown in Fig. 4 is the discharge–($1/\beta_c$) relationship at cross section 4 for the February 1986 flood where only inbank flows occurred. The average value of $1/\beta_c$

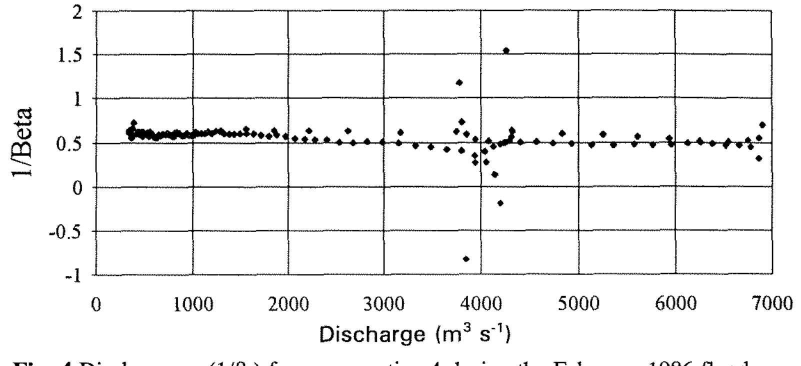

Fig. 4 Discharge *vs* ($1/\beta_c$) for cross section 4 during the February 1986 flood.

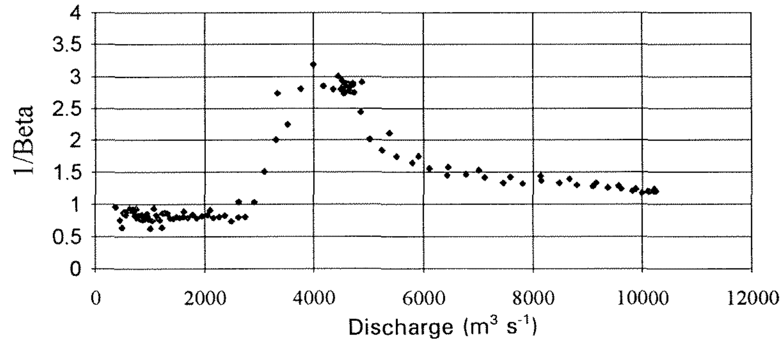

Fig. 5 Discharge *vs* ($1/\beta_c$) at Ingham (GS116001) for the February 1986 flood.

was determined to be equal to 0.57 which is approximately equal to 0.58, which was the value of $1/b$ determined using the storage–discharge relationship (equation (16)) for the same flood and cross section by Sriwongsitanon *et al.* (1994).

Shown in Fig. 5 is the discharge–($1/\beta_c$) relationship at Ingham (GS116001) where overbank flows occurred for the February 1986 event. The values of $1/\beta_c$ tend to be constant (about 0.85) for inbank flows ($A \leq 1792$ m^2 or $Q \leq 2600$ m^3 s^{-1}). As bankfull flow is approached, however, the value of $1/\beta_c$ increases to a maximum of 3.0 and then falls to an approximately constant value of 1.4 at very high flows. The average values of 0.85 and 1.4 are close to the values of $1/b$ for inbank and fully developed floodplain flows obtained by Sriwongsitanon *et al.* (1994) who fitted the storage–discharge relationship given by equation (14) to the data shown in Fig. 5.

The results illustrated in Figs 4 and 5 reveal that the parameter b in the storage–discharge relationship of equation (14) and the kinematic wave routing parameter β_c are approximately equal. Therefore, reproduction of the flood hydrograph and associated flow characteristics, such as flood wave speed and water velocity, shows that the kinematic routing model and the inherent assumption of a single form of relationship between discharge and storage can be applied to runoff-routing for flood estimation.

WAVE SPEED–DISCHARGE RELATIONSHIP

The link between the wave speed–discharge relationship and the storage–discharge relationship has been investigated previously by Wong & Laurenson (1983). In their study the wave speed was calculated as reach length divided by travel time of the hydrograph peak. The corresponding discharge was calculated as the average of the peak discharges for the inflow and outflow hydrographs. However, the lowest wave speeds which occur for discharges at or above the bankfull discharge were estimated by theoretical construction because insufficient data were obtained for these conditions. Therefore, the wave speed–discharge relationship cannot be characterized at the transition between inbank and floodplain flows.

In the present study, the wave speed was calculated using results obtained from the Rubicon software package for every value of discharge, including those at or above bankfull discharge. The wave speed (c) was obtained using equation (16) which is the rating curve at each cross section. Consequently, the wave speed–discharge relationship could be generated for the whole range of flows, including overbank and floodplain flow conditions. Therefore, the link between the storage–discharge relationship and wave speed–discharge relationship could be investigated at all cross sections in the study area.

A wave speed–discharge relationship was derived by substituting and rearranging equations (5), (9), and (10) to give:

$$c = \alpha_c \beta_c \left(\frac{Q}{\alpha_c} \right)^{1-\frac{1}{\beta_c}} \quad (17)$$

As shown by Sriwongsitanon *et al.* (1998), by substituting $m = 1/\beta_c$ into equation (17), the relationship can be presented as:

$$c = \left(\frac{\alpha_c^m}{m} \right) Q^{1-m} \quad (18)$$

Equation (18) has a similar form to the relationship derived by Wong & Laurenson (1983), which was:

$$c = a_1 Q^{b_1} \quad (19)$$

where a_1 was assumed to be independent of m. They obtained values for b_1 in equation (19) from a regression of recorded data. However, comparison of equations (18) and (19) reveals that the parameter a_1 is dependent on m. Similarly, the values of b_1 obtained from equation (19) with a_1 constant will not give accurate estimates of m using the relationship $b_1 = 1 - m$ as suggested by Wong & Laurenson (1983).

For instance, Fig. 6 presents the wave speed–discharge relationship corresponding to equation (18) at cross section 4 where there is only inbank flow for the February 1986 flood. At this cross section, the value of $1/\beta_c$ (Fig. 4) or $1/b = m$ was found to be approximately constant. By using the average values of $\alpha_c = 0.018$ and $\beta_c = 1.72$ (or $1/b = m = 0.58$, Sriwongsitanon *et al.*, 1994) the wave speed–

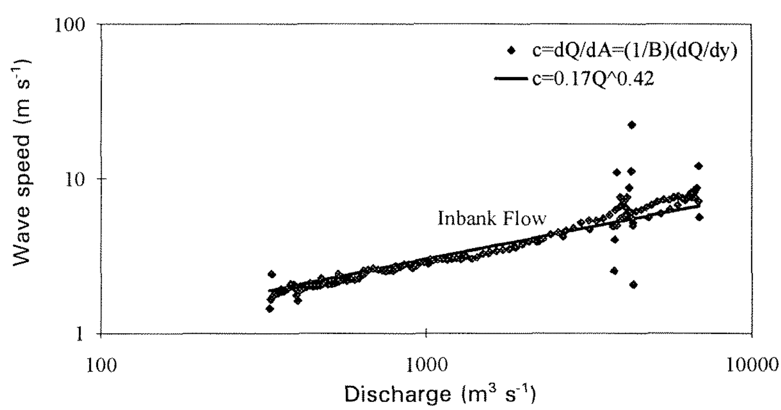

Fig. 6 Wave speed–discharge relationship cross section 4, February 1986 flood.

discharge relationship according to equation (18) will be:

$$c = \frac{0.018^{0.58} Q^{0.42}}{0.58} = 0.17\,Q^{0.42} \tag{20}$$

This relationship fits reasonably well with values obtained from Rubicon model runs. The best fit to the wave speed–discharge data points shown in Fig. 6, of the relationship presented by Wong & Laurenson (1983), was $c = 0.10Q^{0.49}$. Thus, m, using Wong & Laurenson's (1983) relationship was found to be 0.51 (i.e. $1 - b_1$), instead of 0.58. This represents a 12% difference in the estimated value of m. Equation (18) provides a better relationship to explain the wave speed–discharge interaction in terms of $1/b$ (= m) since it was derived from a consideration of the fundamental hydraulic processes.

The wave speed–discharge relationship at Ingham (cross section 17) for the February 1986 flood is shown in Fig. 7. At this cross section, which has both inbank and overbank flows, the wave speed–discharge relationship cannot be predicted from a single relationship because the $1/\beta_c$ values change with discharge as shown in Fig. 5. Values of $1/b = m$ for inbank flow will be fairly constant as shown in Figs 4 and 5. However, for overbank flow, values of $1/b=m$ (and therefore $1/\beta_c$) are continually changing until flow over the floodplain becomes fully developed where $1/b = m$ once again approaches an approximately constant value.

Regression analysis revealed the best relationships to describe Wong & Laurenson's assumed wave speed–discharge relationships (equation (19)), at each cross section. From left to right in Fig. 7, these relationships would be respectively, $c = 0.28Q^{0.24}$, $c = 2.9E11Q^{-3.29}$ and $c = 0.018Q^{0.33}$, with m values of 0.76, 4.29, and 0.67 respectively. However, using equation (18) values of $1/b = m$ for the first and third relationships were 0.85 and 1.4 representing, respectively, 12% and 109% variation from the values estimated using Wong & Laurenson's approach.

Further, consideration of Fig. 5 shows that $1/b = m$ continually varies for the transition between inbank and fully developed floodplain flows. At cross section 17 and for these flow conditions, the value of $1/b = m$ increases from 0.85 to a maximum value of 3.0 and then decreases to an approximately constant value of 1.4.

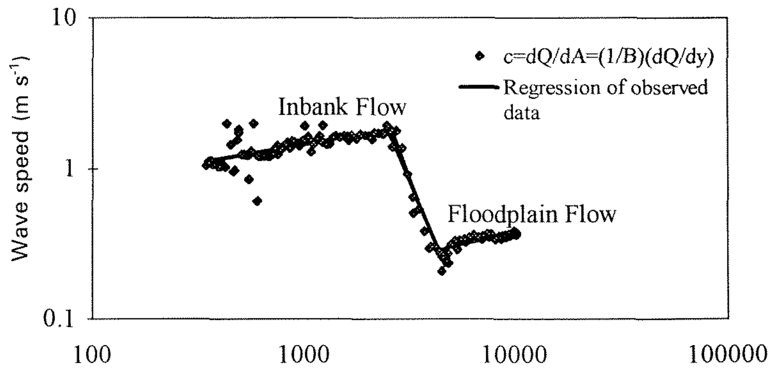

Fig. 7 Wave speed–discharge, Ingham (GS116001), February 1986 flood.

Consideration of Fig. 5 reveals that the wave speed–discharge relationship, $c = (a_c/m)Q^{1-m}$ at the transition, continually changes as $1/b = m$ changes. Therefore, Wong & Laurenson's approach assuming a constant m value does not explain the wave speed–discharge relationship in this transition region.

The investigation reported herein shows that the wave speed–discharge relationship is related to the storage–discharge relationship, via the common power function m in $S = kQ^m$ and $c = (a^m_c/m)Q^{1-m}$. However, the wave speed–discharge relationship cannot be directly applied to the storage–discharge relationship as was suggested by Wong & Laurenson (1983). Further, it has been shown that the use of $1/\beta_c$ would be a better choice for estimating the exponent $1/b = m$, which can then be used in the storage–discharge relationship. Flood wave speed is an important parameter in runoff and flood routing and, therefore, it is important that appropriate values for this parameter should be identified. This underlines the importance of having an understanding of the wave speed characteristics as a flood wave moves through a catchment.

Investigation of flood wave speed data such as that shown in Fig. 5 indicates that the wave speed gradually increases as the discharge increases for inbank flow, then continuously decreases as the flow increases beyond the bankfull stage, and then increases as the floodplain flow becomes fully developed. The data shown in Fig. 5 indicate changes in the value for the exponent $1/b = m$ which can be used in the storage–discharge relationship. In the transition between inbank and fully developed floodplain flow conditions, the flood wave speed rapidly decreases but the channel storage increases, and therefore the exponent $1/b = m$ increases. The rate of increase of channel storage again decreases as the discharge approaches the fully developed floodplain flow condition and $1/b = m$ decreases and becomes approximately constant.

Bhowmik & Demissie (1982) explain in their study of the carrying capacity of floodplains of the Sangamon River and Salt Creek, Illinois, that at the stage when a river reaches bankfull, the floodplain acts as a combination of conveyance channel and storage reservoir. As the flood magnitude increases further to the fully developed floodplain flow situation the stream channel and the floodplain effectively become a large conveyance channel.

Therefore, flood wave speed plays a significant role in identifying the amount of storage in the channel and floodplain and this can be estimated using the appropriate form of the storage–discharge relationship. Moreover, this investigation of the wave speed–discharge relationship reveals that the storage–discharge relationship varies from one cross section to another and is different between inbank, overbank, and fully developed floodplain flow situations; this was suggested previously by Sriwongsitanon *et al.* (1994).

Another interesting phenomenon shown in Figs 4 and 5 is that the wave speed for the channel flows (inbank) fluctuates about the fitting lines more than is evident for the overbank and floodplain flows. This phenomenon is caused by the dynamic waves which have a greater effect on the inbank flows than the overbank and floodplain flows. Therefore, the kinematic model, in general, will be more accurate for the overbank and floodplain flow situations than for inbank flows.

WAVE SPEED AND WATER VELOCITY BEHAVIOUR IN CHANNEL AND FLOODPLAIN FLOWS

As mentioned, the kinematic routing parameter $1/\beta_c$ in the equation $Q = \alpha_c A_c^{\beta c}$ or the exponent $1/b = m$ in the relationships between storage and discharge,

$$S = L\left[c_o + \left(\frac{d_o}{a^{\frac{1}{b}}}\right)Q^{\frac{1}{b}}\right]$$

and

$$S = kQ^m$$

can be approximated by the ratio between water velocity and flood wave speed as given by equation (10). An understanding of the behaviour of wave speed and water velocity in the main channel, overbank, and fully developed floodplain flow conditions can be obtained by examining these relationships further.

Figure 8(a) shows the water velocity–wave speed relationship at cross section 5 (Fig. 8(b)) for the February 1986 flood. At this cross section there was no overbank flow for this event. Both water velocity and wave speed increase as the discharge increases. The water velocity, which can be estimated using Manning's equation, increases because the hydraulic radius (R) increases as discharge increases, while the slope (S) and roughness coefficient (n) are approximately constant. The wave speed, which can be computed by equation (16), also increases as the discharge increases

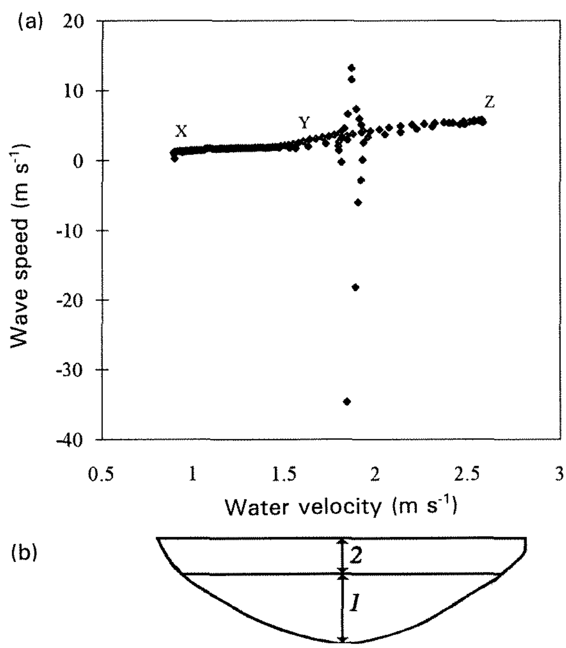

Fig. 8 (a) Water velocity–wave speed relationship at cross section 5 for February 1986 flood; and (b) schematic of cross section 5.

because the increasing rate of dQ/dy is larger than that for the increment of top width (B), where the channel banks are almost vertical, as shown in Fig. 8(b).

However, as can be seen in Fig. 8(a), the slope of the water velocity–wave speed relationship at this cross section is separated into two segments which intersect at the point Y where the water velocity is about 1.5 m s^{-1}. The first segment is between points X and Y on Fig. 8(a) (corresponding to region 1 in Fig. 8(b)), and the second segment is between Y and Z in Fig. 8(a) (corresponding to region 2 in Fig. 8(b)). The average value of $1/\beta_c$ or $1/b = m$ for the first segment is 0.73 and for the second segment it is 0.52 (Sriwongsitanon *et al.*, 1994). The parameter β_c for the first segment is smaller than that for the second segment because in the second segment the banks of the cross section become steeper which reduces the increment of the top width but increases the increment of the wave speed as discharge increases.

Large fluctuations in the wave speed occur at water velocities between 1.8 m s^{-1} and 1.9 m s^{-1}, as shown in Fig. 8(a). This shows the effect of the dynamic wave behaviour which is caused by the second rise of this flood hydrograph as shown in Fig. 3 at approximately 125 h. The dynamic behaviour causes flood waves to propagate in both upstream (minus sign of the wave speed) and downstream (plus sign of the wave speed) directions for a short time and then the kinematic behaviour dominates the flood wave motion again.

The water velocity–wave speed relationship at Ingham (section 17) for the February 1986 flood is shown in Fig. 9(a). In contrast to cross section 4 (Fig. 8(a)) both inbank and overbank flows occur at this cross section. Here, the water velocity–wave speed relationship has three distinct segments.

The first segment starts at the bottom of the cross section (corresponding to M in Fig. 9(a)) and continues up to the water stage number 1 shown in Fig. 9(b) (shown in Fig. 9(a) as N on the rising limb and R on the falling limb of the hydrograph). In this segment wave speed tends to increase as water velocity increases, as usually occurs for inbank flows and similar to what was described previously for cross section 5. The average water velocities in segment M–N (rising limb) are in the range 1.0–1.9 m s^{-1}, and in segment R–M (falling limb), in the range 1.0–1.5 m s^{-1}. The average value of $1/\beta_c$ in the first segment, as given by Sriwongsitanon *et al.* (1994) is 0.85. This approximate value, which is the same as $1/b = m$ is confirmed in Fig. 5 and in Fig. 9(a) by segments M–N and R–M.

The second segment is between water stages 1 and 2, in Fig. 9(b), or from N to O for the rising limb and Q to R for the falling limb, in Fig. 9(a). For the rising limb (N–O), as discharge increases the water velocity gradually declines but wave speed reduces rapidly from about 1.8 m s^{-1} to 0.6 m s^{-1}. For the falling limb (Q–R), as discharge reduces, the water velocity gradually increases while wave speed increases rapidly between 0.5 m s^{-1} and 1.5 m s^{-1}. For the rising limb (N–O), the top width B increases quickly as discharge increases and, hence the wave speed quickly drops. The water velocity also drops a little because the hydraulic radius decreases and the roughness increases, but this only has a small effect on discharge. For the falling limb (Q–R), the variations of wave speed and water velocity are in the direction opposite to those for the rising limb. This segment is at the intersection between the main channel and the floodplain flows where the values of $1/\beta_c$ (Fig. 5) or $1/b = m$

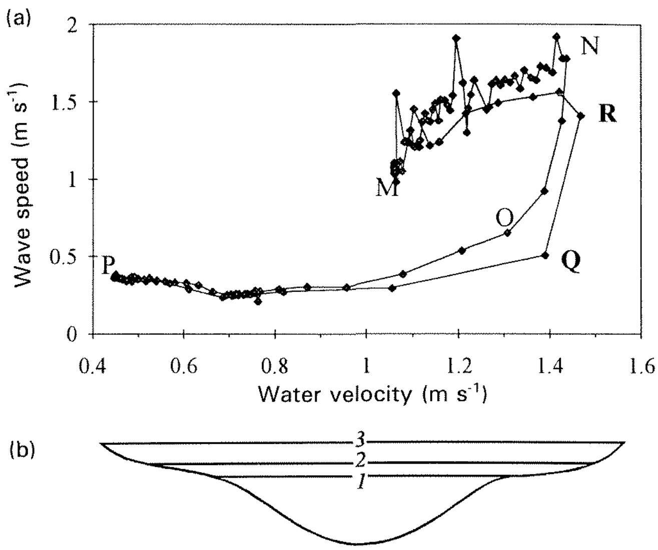

Fig. 9 (a) Water velocity–wave speed relationship at Ingham (GS116001) (cross section 17) for February 1986 flood; and (b) schematic of cross section at Ingham (GS116001) (section 17).

continue to increase above 0.85 and where water just enters the segment causing $1/b = m$ to reach a maximum value of around 3.0. The value of $1/\beta_c$ then falls to a value of 1.4 at the beginning of the third segment.

The third segment was between water stages 2 and 3, as shown in Fig. 9(b). The water velocity rapidly reduces from 1.3 m s^{-1} to 0.45 m s^{-1} for the rising limb (*O–P* in Fig. 9(a)) and rapidly increases from 0.45 m s^{-1} to 1.4 m s^{-1} for the falling limb (*P–Q*). For the rising limb (*O–P*), the wave speed gradually decreases at the beginning and slowly increases at the end. The water velocity rapidly reduces in this segment because the roughness increases and the hydraulic radius decreases, which have opposing effects on discharge. The top width increment in this segment increases much less than in the second segment and therefore the wave speed reduces slowly at the beginning and has a small increment at the end when the sides of the cross section become steeper. For the falling limb (*P–Q*), the variations of wave speed and water velocity are in the direction opposite to the rising limb. In this segment the parameter $1/\beta_c$ (Fig. 5) decreases slowly, the average value for $1/b = m$ being 1.4, as shown by Sriwongsitanon *et al.* (1994).

The characteristics of the wave speed–water velocity relationship, as discussed above, show that the behaviour of these two variables has a big effect on the variation of the exponent $1/b = m$ which is also a parameter in the storage–discharge relationship. Therefore, the water velocity–wave speed relationship as presented in equation (10) which has the approximate value of $1/\beta_c$, should be a more reasonable relationship with which to identify the exponent $1/b = m$ rather than using the wave speed–discharge relationship defined by equation (18). This is because the wave speed–discharge relationship cannot clearly indicate the exponent $1/b = m$ used in the storage–discharge relationship, particularly at the transition between inbank and fully developed floodplain flow situations where the exponent varies considerably.

CONCLUSION

This investigation of the wave speed–discharge relationship shows different forms from one cross section to another, and particularly the differences between inbank, overbank, and fully developed floodplain flow situations. This behaviour is reflected also in the form of the storage–discharge relationship as expressed in equation (14) which was suggested by Sriwongsitanon *et al.* (1994). Therefore, the single parameter storage–discharge relationship $S = kQ^m$ usually used in runoff-routing models needs to be modified, especially when the flow leaves the channels to inundate the floodplain. The variation in the parameter values between the different flow regimes, as shown in this paper, can be obtained from a kinematic relationship between the flow area and the discharge.

Flood wave speed in the wave speed–discharge relationship plays a significant role in identifying the amount of storage described by the storage–discharge relationship. However, the wave speed–discharge relationship cannot define the exponent $1/b = m$, particularly at the transition between inbank and fully developed floodplain flow conditions for which the exponent $1/b = m$ changes continuously. The relationship that can define the exponent m in the storage–discharge relationship is the one between water velocity and flood wave speed. The relationship between discharge and parameter$1/\beta_c$, where $1/\beta_c$ is the ratio between the water velocity and flood wave speed, has been shown to be the best relationship to identify $1/b = m$ and consequently the variations in the storage–discharge relationship.

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