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A Modified Muskingum Routing Hydrograph Approach: Theory and Practice

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Abstract

Hydrological routing methods can be used to predict the influences on a flood wave as it passes through a catchment. The equation of continuity and the equation of momentum are used to derive the simpler data requirements for routing. It is useful for preliminary estimates of the time and peak discharge of a flood. This paper presents a modified linear Muskingum routing method where the flood peak attenuation and travel time reduction parameters. Developing a routing hydrograph with different flood peak and travel time reduction parameters in which change geometrical and resistance properties of varied upstream and downstream hydrographs is investigated. The flood attenuation and travel time (Muskingum routing) of the flood

Standard Muskingum routing was then used to develop hydrographs for each of the subcatchments (from 0 to 10.5d peak attenuations were again determined through upstream and routed downstream hydrographs and with these and the attenuation developed. Actual weighting of corresponding constant subsequently used to determine the peak attenuations determined from the model simulations. Validation and analysis, the complex and variables related to catchment and hydrological properties and expressions and determining that the properties were developed. The modified Muskingum routing method uses expressions for K and x was applied to a case study of the Reigoro Sui-gi river in the between measured and routed hydrographs was observed.

Keywords: Overbank; Flood Muskingum routing; hydraulic and hydrological methods; floodplains; Modeling and simulations.

1 Introduction

Hydraulic or hydrological flood routing techniques are commonly used by hydrologists to predict the temporal and spatial variations of a flood wave (Choudhury et al., 2002) methodologies that have been developed with more analytically rigorous methods having in mind to accommodate the dynamics and influences of floodplain behavior on the flood wave in a natural channel. The Muskingum method of flood routing is a popular approach. The popularity of the Muskingum method derives primarily from its simplicity. It requires knowledge of geographical catchment characteristics are not required.

1 understand the propagation of flood waves represented by a
 2 carried out using data by (Gumbel, 1977) Muskingum approach
 3 represents a flood routing technique based on equation of continuity
 4 and a relationship that describes the storage in the system. The
 5 full scale dynamic wave model based on the Saint Venant equations provide
 6 sophisticated hydraulic flood routing (Tung, 1985). Hydraulic routing uses
 7 equation of continuity and Saint Venant's momentum equation
 8 involving numerical solution of finite difference or characteristic method.
 9 Simplification of the momentum equation into approximate solutions (e.g. momentum
 10 wave, convective diffusion) that are easier to calculate and more accurate.
 11 Advantages and disadvantages of hydraulic routing techniques are
 12 routing techniques can more adequately represent the dynamics of flow in
 13 canals and rivers. These methods are more demanding in terms of
 14 information inputs and require data to accurately represent the
 15 characteristics of the main channel and flood plain. Boundary conditions are
 16 required. Computationally, inputs and computational procedures for
 17 techniques are simpler (Singh, 1988). These methods are useful when
 18 preliminary estimates of the time and shape of a flood wave are
 19 are required, or where budgetary constraints may not facilitate
 20 A modified Muskingum hydraulic routing method for flood plain
 21 flows is presented in this paper. It is based on the standard Muskingum
 22 However, rather than determining the attenuation of a flood wave as it
 23 upstream and downstream hydrographs for given inflows, the
 24 routing parameters are determined from empirical relationships based on
 25 attenuation of a flood wave travel time. This method therefore has similarities

1 Muskingumge method (Chow, 1959) at flood routing parameters determined
 2 from geometrical and resistance properties of the channel, the
 3 calibration process of the method involved a series of modifications
 4 hydraulic modelling and standard Muskingum method for a given storage
 5 and storage weight, for a range of catchment and hydrograph
 6 variate regression analysis was used to calibrate these comp
 7 properties and expressing the developed. The modified
 8 Muskingum routing method based on the same was applied to as
 9 case study of the River Suerig old alignment between measure
 10 hydrographs was the method offers a simple and inexpensive m
 11 estimating the time covered by a wave as it progresses along
 12 of low to moderate sinuosity and in which backwater and iner
 13 small.

14

15 2 Muskingum Routing

16 The Muskingum method (McCarthy, 1958), on the storage
 17 discharge relationship is extensively used in river engineering
 18 The method for a river system is in a sense a backwater influence
 19 small and the model parameters are chosen to represent the hydrau
 20 behaviour of the system (Chow, 1959). Muskingum is based on continuity
 21 and storage relationships expressed as:

$$22 \text{ Continuity } \frac{dS}{dt} = I_t - O_t \quad (1)$$

$$23 \text{ Storage } S = K \left[\frac{I_t + x O_t}{2} \right] \quad (2)$$

1 when S_i and Q_i are simultaneous amounts of storage, inflow and outflow
 2 given times, K is a storage constant expressing the ratio between storage and
 3 river reaches dimensionless weighting factor that varies between 0 and 1
 4 river. This weighting factor describes the relative importance of
 5 storage. The storage time t_s is constant, i.e. effectively constant through
 6 river reach (McCuen, 1988). If K and t_s are known, routing is performed using:

$$Q_{t+\Delta t} = C_1 I_t + C_2 I_{t+\Delta t} + C_3 O_{t+\Delta t} \quad (3)$$

8 in which C_1 , C_2 and C_3 are routing coefficients given by:

$$C_1 = \frac{0.5\Delta t - Kx}{\Delta t(\Delta t - x)K + 0.5\Delta t} \quad (4)$$

$$C_2 = \frac{Kx + 0.5\Delta t}{\Delta t(\Delta t - x)K + 0.5\Delta t} \quad (5)$$

$$C_3 = \frac{\Delta t - 0.5\Delta t - \Delta t(\Delta t - x)K}{\Delta t(\Delta t - x)K + 0.5\Delta t} \quad (6)$$

12 where parameters K and t_s are defined as the time lag and
 13 are the inflow and outflow discharge coefficients (which sum to unity)
 14 are determined by using repeated hydrographs, time.

15 Values K and t_s that describe the storage characteristics of a river
 16 derived from observed upstream and downstream hydrographs
 17 records. These methods are well known and broadly represented in five classes
 18 (a) graphical method; (b) least squares method; (c) method of
 19 cumulative direct optimisation; and (d) sequential least squares
 20 and (McCann, 1980). Recently, Yoon and Padmanabhan (1993) proposed a further
 21 three methods for linear model parameter estimation and a
 22 backward stepwise regression outliers filtering estimation method and
 23 quadratic programming algorithm.

Graphical methods are commonly applied for graphical
 approach by McCarthy (1938) for the linear Muskingum routing model
 $\frac{dS}{dt} = \frac{1}{K} \left(\frac{dI}{dt} - \frac{dO}{dt} \right)$, known as the weighted gain method, routed storage for
 different assumed values. Different values of K produce a family of curves
 vary from being heavily looped to being reasonably linear. The
 the narrowest and best with a straight line is considered the best estimate
 of K . The inverse of this is the given value of K . Although the graphical
 method is generally used by Chow (1959), Linsley (1975), and Wilson (1995),
 some constraints exist. Furthermore, no objective criteria
 exists for choosing the appropriate value of K . A level of
 subjective interpretation is required to value the optimum K (Gelegenis
 and Ser2007, Yoon and Padmanabhan and Chaudhary 1993). Muskingum routing
 parameters have also been estimated using a method based on minimising
 sum of squares of the deviations between observed and
 inflow and outflow hydrographs (Gidamis 1978, 1997, Imbeldi
 Esen, 2000). The early routing procedure graphical and the latter the
 same and both methods simultaneously produce the method of moments
 and the method of cumulants as a result of fitting the first and second
 moments or cumulants of the hydrograph in the Muskingum reach
 to the Muskingum routing (Kang and Dunge 1991). The method of direct
 optimisation based on minimising the difference between observed
 hydrographs and directly the routing of the Muskingum routing model
 explicitly (Kang (Gelegenis and 2005), Errano

More recent advances in computer technologies have allowed the
 Muskingum routing method with hydrodynamic software packages

analysis of surface water drainage (see information in appendix C and the TOPMDE hydrological models) are based on the equations that are derived from the principles of conservation of mass and momentum in their dimensional form

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + V \frac{\partial V}{\partial y} = C \quad (7)$$

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g A \frac{\partial V}{\partial x} + g (S_0 - S_f) = C \quad (8)$$

(I) (II) (III) (IV)

where V is the flow velocity, g is the gravity acceleration, S_0 is the gravity river bed slope, S_f is the slope of the energy grade line, x is the longitudinal distance and t is time. These equations are simultaneous ordinary differential equations of the hyperbolic type and do not have an analytical solution. The first term in Eq. 8 is the local acceleration, the second term is the convective acceleration, the third term is the pressure difference, the fourth term is the friction and bed slopes. Numerical methods for solving the equations are broadly classified in two categories: approximate methods and complete numerical approximation methods. The approximate methods are based on the equations of continuity and momentum.

Kinematic and diffusion models can be constructed by Eq. 8 assuming that the acceleration term is negligible (Mousa and Bocquillon, 2006). Models that neglect the acceleration term are known as diffusion wave models (Chow, 1959; Bagchi and Barry, 1999; Mousa and Bocquillon, 2009). Models that neglect both inertial and pressure terms are known as Saint Venant equations. Full-Saint Venant equations have been used in many applications using numerical techniques in channels and wide open channels.

cross-sectional geometries with simplifying assumptions (Amein and
1970; Dooge et al., 1982; Wang et al., 2006)

Cunge (1969) deduced the effects of geometrical and resistance
reach in the Muskingum method (M-C) model.

Cunge showed that the Muskingum formula for solving flood routing
a finite difference approximation of the linearised diffusion wave

being valid for a SVE and a subsonic by neglecting the inertial terms.

flood routing, the necessity of calibration of Muskingum parameters

is not required and the routing parameters are determined from hydrographs of

the routing:

$$K = \frac{\Delta x}{c} \quad (9)$$

and

$$x = \frac{1}{2} \frac{Q}{B S_0} \quad (10)$$

where parameters are as described above and Δx is the

longitudinal distance in the channel discharge averaged over

the channel

The time Δt used in the routing procedure is appropriately chosen

to define the shape of the inflow hydrograph and the dependency of

routing procedures and the coefficient of resistance n is not

significantly smaller than the distance travelled by the flood

This interval in the M-C routing method is based on Q , the channel

top width and longitudinal channel slope wave, is obtained from

the slope of the discharge curve given Q . Details of the method

1 are discussed in Volume III of the Flood Forecasting and Control (FFC) Report (a) (1999a) investigated the performance of the flood routing, using
 2 (1999a) investigated the performance of the flood routing, using
 3 hypothetical flood hydrographs in a prismatic channel with slope
 4 indicated that the method differs from loss of outflow which depends on bed
 5 slope and roughness. Furthermore, it was observed that an
 6 and trailing edge oscillation occur in the rising and recession
 7 respectively. These oscillations can be minimized by increasing the
 8 increases, but gradually disappear with decreasing bed slope.
 9 The standard linear Muskingum routing method assumes
 10 and these are determined by measured inflow and outflow hydrographs. The
 11 method therefore, does not accommodate changes in these parameters
 12 accurately the routing of storm sequences in the river reach
 13 (Kundzewicz and Strupczewski, 1982). More recently, the method has
 14 do allow for parameter variability with changing characteristics
 15 for example, Peumal, Singh, and Singh, 1992; and Peumal and Esen, 2006).
 16 Peumal (1992) developed a linear Muskingum routing method based on
 17 distribution scheme. The physically based Muskingum method
 18 model in this method and the parameters are varied at each
 19 prescribed flow zones in the routing hydrograph (1992) proposed
 20 versions of the linear Muskingum method with variable parameters
 21 reach travel time (which depends on the storage, channel characteristics)
 22 obtained from a simplified routing method. Humoud and Esen (2006)
 23 proposed two approximate methods for the estimation of linear
 24 parameters. The first method requires the computation of the
 25 hydrographs at the point of intersection, and the computation of

1 within the The second method requires the computation of t
2 hydrographs at two specific points.

3 Although the current paper is based on that line at Mubkingu
4 between $S(t) = K[S_0 - S(t)]$ and is usually linear and Gill (1978) for the
5 storage relationship in Muskingum model:

$$6 \quad S(t) = K[S_0 - S(t)]^n \quad (1)$$

7 where n is an exponent that defines the behavior of the arc calculation
8 and weight K is a parameter directly determined from inflow and
9 hydrograph and the alternative parameter estimation is presented
10 for example, Tung (1985), Pradhan and Mohan (1999), Kim et al.
11 (2006), Chah (2009), Gill (1978) proposed a segmented linear
12 model based on segmented curves where the coefficients are determined
13 squares method where the technique is somewhat arbitrary process of selecting
14 three points in the segment and using the continuity and
15 storage equation to solve the parameters (Tung 1985). Tung (1985) proposed
16 procedures using the (H) pattern technique which is a simple linear
17 regression (LR), the conjugate gradient (CG), Powell (DF)
18 techniques and used the state variable technique for computing
19 with Gintology which showed that the DF and H techniques produced
20 estimations of the routing parameters (Pradhan and Mohan 1999) proposed
21 least squares regression technique which directly fits the nonlinear
22 procedure iteratively to find the parameters using the Marquardt
23 algorithm (Marquardt, 1963). In addition, this method also gives the data
24 and provides reasonably accurate estimates of the parameters to be estimated.

expedite the optimisation process. Moilanen (1997) suggested a calibration technique
 determined by a genetic algorithm. It avoids the need to make any assumptions
 about the parameters. Kim et al. (2001) proposed a search algorithm to estimate the
 same parameters. They observed that the reformulated genetic algorithm and
 mathematical algorithms, evolutionary programming and genetic programming.
 (2001) presented a hybrid genetic algorithm (BFGS) for parameter estimation in
 nonlinear Muskingum. The BFGS algorithm is a branch of Newton's method based on
 mathematical gradient descent for solving unconstrained nonlinear optimisation
 problems. Chau (2009) developed a Fuzzy Inference System (FIS) implemented in an
 adaptive network model to estimate the outflow hydrograph. A calibration procedure
 for finding the coefficients of the three parameters to determine this outflow is
 proposed (Kim et al., 2001)

3 Method

A multi-stage procedure (Fig. 1) that includes the HEC-RAS modelling of a
 generalised river reach, standardising Muskingum as a standardised for
 developing expressions for storage and weighting factors (in the
 modified Muskingum method). The process begins with implementation of a
 generalised HEC-RAS model using input flow hydrographs and geometric data to
 determine travel times and relative attenuations which are then used
 to develop equations for estimating Muskingum model parameters. The process
 is explained in detail.

Fig. 1

3.1 HECRAS Model of General Reach Riv

The HECRAS model of the generalised river reach was developed for an extensive range of geometrical hydraulic properties. The HECRAS model is a one-dimensional link and node river Engineering River Analysis System developed by the US Army Corps of Engineers. It solves the dynamic Venant equations using an implicit, finite difference hydrograph method. The model also hydrographs during a peak flow are key to the model.

3.1.1. Hydrographs of Varying Peak Flow

Hydrographs of varying peak flow were developed using a methodology and associated software package 3.1 of the Irish Flood Studies Programme (O'Connor and O'Connell, 1990). The model is a record of flow data.

Fig. 2

Any gauged location of good quality data was available was therefore suitable and a good site was chosen. Hydrograph development is as follows:

(1) The annual exceedence flood event for the selected site identified from data (as shown for example in Fig. 2 (a))

(2) Annual exceedence flood hydrographs were isolated from the complex segments on each side of the peak, having the component (Fig. 2. (b))

(3) The isolated flood hydrographs to have a peak value of unity

dividing all its flow ordinates by the peak flow.

(4) This unit peak is assumed to represent a hydrograph percentile flow
were determined percentiles of 98, 95, 90 & .10 and 5.

(5) Widths corresponding to these flow percentiles were averaged
exceedence series and the rise in flood by of the graph was
approximated by fitting a form of the Gumbel curve. The
of the full unit hydrograph was determined by a potential recession
drawn from the point of inflection of the curve (Fig. 2)

(6) The required graphs were generated by derived unit hydrograph
peaks of different return periods (2, 5, 25, 50, 100, 500 and
A base flow the particular watershed peak ordinate (Fig. 2)

Fig. 3

Annual maximum flood for the catchment of the extreme
Value (GEV) Type I distribution (NERIS, 1975) and quantiles
for the hydrograph (Fig. 2) from 9/1s. 40 m³/s event to 153.90
m³/s for the 100 flood.

3.1.2 Hydrographs of Varying Duration

The hydrographs of varying duration have the same base length
therefore flood volume is determined by the relationship
between flood volume and flood peak is not fully defined. Flood
processes and to fully account for the random nature of their
included if flood is too accurately related to the peak

1 trapezoidal geometries in both the main channel and floodplains. The base and top widths of the
 2 of the main channel and floodplains were expressed in terms of Manning's roughness coefficient n and
 3 assigned values of 0.03 and 0.25 respectively. The high base width was chosen to
 4 ensure that measurable attenuations were obtained. The base width of the main channel was 50 km and the
 5 longitudinal slope of the main channel was 1 in 1000. The floodplains were also 1000 m wide.
 6 total, 65 variations of these basic properties were considered, designated by A to H and in which
 7 the effect of channel length was varied at a time (Table A1). Case A was the base case. Case B
 8 the effect of channel length was investigated by varying the floodplain slope (Case C varied the
 9 slope of the floodplains, Case D the floodplain width, Case E, the floodplain slope and Case F
 10 hydraulic resistance. The effect of peak inflow (Q_p) and flood duration (T_p) was
 11 explored by the two sets of hydrographs (Fig. 2). The length and Froude number of the
 12 the generalised model in the Case G and Case H simulations. In the Case G simulations,
 13 the effect of channel property on flood attenuation was examined by comparing input and output hydrographs.

16

17 3.2 Standard Muskingum Model

18 The travel time of the peak of the flood wave determined from the input and output hydrographs in the
 19 standard Muskingum routing model was assumed to be constant. The Muskingum routing model involves
 20 corresponding weighting factors involved. For a combination of
 21 with assumed values of x (increasing incrementally from 0 to 0.5 where 0 is
 22 high attenuation and vice versa), standard Muskingum flood routing hydrographs were
 23 hydrographs were repeatedly performed using Eq. 3, together with Eq. 4 to generate a series
 24 a series of outflow hydrographs. Peak outflows were determined from the

with the peak inflow hydrograph (Fig. 1B) (allowing series of relative attenuation to be determined, from:

$$\% \text{ Relative attenuation} = \frac{Q_1 - Q_2}{Q_1} \quad (14)$$

where Q_1 and Q_2 are the peak inflow and outflow hydrographs (Fig. 1B).

Linear relationships between these relative attenuations from assumed weighting factors were developed. These relationships were actual weighting factors of attenuation by comparison of the inflow and outflow hydrographs (Fig. 1B) produced a weighting factor for each of the 65 simulation iterations which were directly determined, covering the full range of properties that were assessed for the different inflow hydrographs.

3.3 Regression analysis

Using univariate regression, the coefficients of storage constant and weighting factors were correlated to catchment and hydrograph properties. Expressions for determining these properties were developed.

4 Results

Estimated values for K and α from the RASCH model and standard Muskingum coefficients and variations resulting from parameters with catchment and hydrograph properties are shown respectively. Variations in both K and α (or β) are shown to have only a small influence on storage and weighting factors in the analysis.

Fig4

Fig5

Results confirm increasing floodplain length (Case A) and width (Case B) noted by Wolff and Burges (1994) as the capacity of the overbank and delay the propagation of a flood wave along a channel. Also important, steep catchments have the capacity to convey floodwaters more mildly and the increased conveyance is reflected in the (Fig. (4)) and attenuation (Fig. (5)). These trends are consistent with Wolff and Burges (1994) (1999b) have cited (2006) the large attenuations with a reduction in the variability of the cumulative distribution gradient. Taseg et al (1999b) reported that the Muskingum Curve method suffers a certain amount of distortion in longitudinal slope channels creating a lateral slope (Case B) results in a geometry in which overbank flow is continually redirected back towards the main channel with steep lateral slopes a with increased proportion of the flood wave in the channel. Furthermore, floodplain resistance in the generalised that in the channel the reduction in resistance and is provided reduced travel time (Fig. (4)) in geometries with increasing lateral slopes. It is noted that a diminishing proportion is being influenced by the high roughness in the main channel and floodplain values (Case C) produce increased attenuation and travel time.

The full influence of floodplains on flood wave attenuation influenced by flow magnitude Q . Consider the resulting overbank flow period (typically less than 2 years), flows will not significantly increase will not be affected by the additional discharge. Fig 4(g) indicates that wave speed is high for relatively fast flows that produce low overbank deposits (see Fig 5a) and floodplain influences increase attenuation and travel time. Simulations assessed flood duration attenuation and flood wave travel time (duration) (define the flood volume with hydrograph sharp peaks short duration experience significantly higher attenuation than those with that are characterised by high rise limb of the hydrograph which occupy floodplain storage that is available and once occupied is available for the remainder of the flood. The attenuation process is thus limited. In contrast, hydrographs with low rise limb but of the flood volume and short duration are comparatively high downstream attenuations.

5 Development of Modified Parameters

The influences of friction, resistance and hydraulic properties Fig 4 and Fig 5 were included in a multiple regression analysis to generate for these parameters the floodplain width to the bankfull width (B/B_{bf}) expressed as a single parameter (B/B_{bf}) defined as consistent with the bank flow equation (ample Knight and S. Thieme 1996).

$$K = 0.794 \frac{L^{0.24} n_p^{0.42} n_{mc}^{0.60} T_B^{0.07}}{S_p^{0.53} Q_p^{0.09} (h_p + B)^{0.06}} \quad (15)$$

$$x = 0.035 \frac{L^{0.03} S_p^{0.16} T_B^{0.39}}{(h_p + B)^{0.05} (h_{mc})^{0.06} n_p^{0.0001}} \quad (16)$$

Application of Eq 15 requires data and therefore horizontal flood profile represented by a value S . Similarly, the equation assesses flood peak flow should exceed bankfull discharge capacities in a reach should be noted that Eq 16 are based solely on the influences of the parameters relative attenuations and delays by the HRAS peaks detection modelling of a reach of the generalised river reach. The values of n_p are therefore based on the simulated data only and as with regional parameters that may intuitively be considered to be important for in the analysis of the influence of floodplain roughness on the flood point it would be expected that significant floodplain roughness, large storage and yield low storage weighting. As a result of modelling, increasing floodplain resistance proportion increased the being conveyed in channel for all flows investigated, with the result that simulation performance of the shown in Fig 6. These the routing parameters, plotted on line against the calculated Eq 15 and Eq 16.

Fig6

Fig 6 indicates that the data reproduced reasonably well the simulation for most of the geometrical, resistance and hydrographical parameters. Simulating the flood process shown to vary most significantly with duration. (The poor fit may result from the assumption of independence between the flood peak and the flood duration). The fit was made when including duration as a parameter in the regression model. A plot of Fig 6 indicates the low influence of the parameters determined using the equations

6 Illustration of the Modified Muskingum Routing Method

The routing procedure was applied to the River Suir, Tipperary. The River Suir is typical of the low main channel floodplain situated between the New Bridge (Station 16008) and Caher Park (Station 16009) gauging stations. In addition a third station at Killardry (Station 16007), where the flow of the River Suir joins the Suir River and New River is measured, are characterised by good quality flood data from 1854 to 2000. The less significant tributaries of the river between these stations but are catchment areas to the New Bridge are 16.2 km² and 11.6 km² respectively and the area to Killardry is 16.2 km². The flood history of the river in this area also indicates that significant

Fig. 7

1 Illustrating the hydrologic application of the Muskingum method
 2 Eq15 and Eq16 to a selection of hydrographs at New through the
 3 River Suir reach and at Caher Park obtained both measured
 4 data from a HECAS model of the river. The HECAS model was developed
 5 from 35 recently obtained cross-sections at New and Caher Park that define
 6 the main channel geometry and floodplains to widths of approximately
 7 channel banks. The Suir and Aherlowe data was augmented by LIDAR data to
 8 further define the floodplain topography to widths of approximately
 9 main channel. Longitudinal distances between sections were
 10 approximately 400 m and this resolution in the section was increased by
 11 interpolation. The lower reach of the River Aherlowe was increased to
 12 85% and 70% of the Suir and Aherlowe catchments respectively
 13 pasture and this land use is the floodplain. A Manning's n of 0.05 for
 14 grassland pasture with areas of brush described the hydraulic resistance
 15 a coefficient of 0.04 that is typically for a reach with some
 16 some obstructions and marginal vegetation defined the main channel
 17 (Chow, 1956; Chow, 1959; Hollinrake and Millington, 1994).
 18 The majority of natural hydrographs are complex and are
 19 multiple peaks that reflect both the temporal variability of the
 20 heterogeneity of the catchment. It is theoretically possible to resolve a
 21 hydrograph into a series of simpler hydrographs from
 22 isolated storm events. This was done for the three hydrographs
 23 events as a laborious task assisted by FSU hydrograph processing
 24 facilities identified a number of hydrographs from three gauging stations for

specific events. These events related to periods in December 1986/September 2004/February 2004 (Fig 8).

Fig8

Measured outflow hydrograph at Caher Park was compared by a number of tributary flow (Fig 7) for which no flow data is available on the peak and timing of the hydrographs in the main River Suir, therefore ensure that the outflow graph at Caher Park is not unduly influenced by these tributaries that measured at a distance of 10 km. The modified method measured hydrographs at Newfair (Fig 8) were routed through the HEC RAS model of the river system to the tributary where the data is available and compared to observed hydrographs at Caher Park. The difference between the hydrographs in the RAS model and those measured at Caher Park for the three events (Fig 9) indicates that the contribution of the other than that from the Aherlow River is negligible.

Fig9

Parameters 5 and 6 apply only to a single reach and cannot be extrapolated to a river system with a tributary network. For validation of the modified method, the Caher Park hydrographs for the three storm events were compared to the Aherlow River hydrographs. The difference between the routing of the observed hydrographs through the HEC RAS model of the river from this location to Caher Park and subtracting the Aherlow hydrograph

The resulting hydrographs adjusted for reach differences through this process are somewhat artificial and backwater effects from interaction of the floodplain of the main channel and tributaries are included. It is likely to be local and in the context of a 16.8 km reach, the adjusted hydrographs are compared with those determined by measured hydrographs at New Bridge using the modified Muskingum performance of the approach to be illustrated.

The testing of the Muskingum Eq 15 and Eq 16 was based on assigning appropriate values to parameters that describe the geometry of the channel and floodplains together with the characteristics of the flow. Geometrical properties of the main channel and floodplains were determined from survey data and where necessary, average values were used. The floodplain slope length (L) for the ESU catchment was described by numerical values. Main channel and floodplain resistances were estimated to be 0.001 s/m and flood peaks and durations were determined from hydrographs at New Bridge. This data is summarized in Table 1. The 1954/55, 1964/65, and 1964/65 floods were used for the initial testing of the model. The flood hydrographs were computed by averaging 400 m intervals over the 16.8 km reach. The predicted hydrographs from the H-RAS model when routing the measured hydrographs are compared with the

Table

Caher Park hydrographs using the modified Muskingum method. With these average values shown with Fig. 10.

Caher Park hydrograph and hydrographs derived from H-RAS model referred to as H-RAS are also shown for comparative purposes.

Fig.01

Although strong correlation between the H-RAS routed hydrograph and the use of the simplified method as presented limited that the floodplain widths are derived from hydraulic predictive capacity for flood plain attenuation indicators (FAIs) that developed for the Irish Flood Studies Update (FSU) using that definition of active river flood plain for the 100 year (Q_{100}) and the 400 year (Q_{400}) floods from normal depth modelling at FSU nodes (approximate intervals of 500 m) on the main channel network. The assumption that the time with a flood in a river is equivalent to bankfull flow in a river. bankfull recurrence intervals in many order of years (see for example, 1982, Piquet, 1997, and Castro Jackson), this is a simplification. The assumption that the flood is determined using an FSU relationship and Murphy, 1999, and catchments given

$$Q_{med} = 1.237 \sqrt{AREA} BFI^{0.937} SAAR^{0.922} FARL^{1.206} DRAIN^{0.2217} DS^{0.0311} S^{0.812} \quad (17)$$

where $AREA (km^2)$ is the catchment area of the river to the outlet point, $S (m/km)$ the average slope of the river between the outlet and the outlet, $SAAR (mm)$ is the annual average catchment rainfall, $FARL$ is a flood attenuation factor for reservoirs and lakes, $DRAIN (km)$ is the baseflow

simple index that relates the length of the stream to the area of the
 the gauged catchment. The index is defined as the percentage area of the
 defined as the percentage area of the catchment that is within the

Simple multiple regression analysis was used to determine the appropriate growth curve factors defined by
 magnitudes of Q_0 and Q_{100} . Flood flows were determined by subtracting
 values from these flood quantiles and corresponding floodplain elevations
 iteratively at all nodes using the Manning equation to determine the geometry at that node.
 resistance coefficients were consistent with the known values. Incorporating these
 depths into a Digital Terrain Model (DTM) produced a map of flood plain
 Q_0 , Q_{100} and Q_{1000} for the river network.

The return periods for the 1205044/155, d 19186 the River Suir
 between 5 and 100 years. The three most relevant polygon from which
 estimate floodplain widths and on wave damage data from the flood
 extent at all nodes. The three most relevant polygon from which
 modified Muskingum approach using this floodplain width of 100
 with those generated from hydraulic routing and those developed

Fig.1 1

6.1 Discussion of Results

Visual comparison of the hydrographs in Fig.1, although somewhat
 subjective, provide a quick and simple means of assessing the performance of the
 Muskingum routing method presented in this paper. The hydrographs for the
 hydrograph in Fig.0, correlate closely with the Muskingum method in
 which floodplain widths were extracted from the routing results. However,

1 given that floodplain widths are based on our approach is of a hydro
 2 limited use. More meaningful assessments of the method
 3 hydrographs showing in Heggen, a green is not gain observed
 4 hydrographs from modified Muskingum in which floodplain widths are
 5 determined from the FAI catchment. The FAI is not just hydrographs.
 6 The goodness between the Muskingum and hydrographs is qualitatively and
 7 less subjective in getting the goodness criterion recommended by
 8 Schulze (1999) is statistical tests measures estimated data input
 9 from an observed input if the tests are applied to assess di
 10 component of mean square (RMSE) term the magnitude of error in the
 11 computed hydrographs is indicated using the relationship by Schulze
 12 given by:

$$13 \quad RMSE = \sqrt{\frac{\sum_{i=1}^n (Q_{comp}(t) - Q_{obs}(t))^2}{n}} \quad \text{for } i = 1, 2, 3, \dots, n \quad (18)$$

14 where $Q_{comp}(t)$ and $Q_{obs}(t)$ are the computed and observed discharge in different
 15 time steps. Even though it is important in hydrology, percent
 16 error in computed and observed peak flow rates, peak time
 17 determined using the following equations (Gweiny & Stephens, 1995),

$$18 \quad E_{peak} = \frac{Q_{comp} - Q_{obs}}{Q_{obs}} \times 100 \quad (19)$$

$$19 \quad E_{time} = \frac{t_{comp} - t_{obs}}{t_{obs}} \times 100 \quad (20)$$

$$20 \quad E_{volume} = \frac{V_{comp} - V_{obs}}{V_{obs}} \times 100 \quad (21)$$

where E_p , E_v and E are percentage errors in peak flow, timing and volumes respectively. Q_{p-obs} and Q_{p-comp} are observed and computed peak flows, t_{p-obs} and t_{p-comp} are computed and observed peak times, Q_{obs} and Q_{comp} are observed hydrograph volumes.

Even though the RMSE and E statistics may model performance effectively, the peak computed and observed hydrographs not be accounted for. To overcome this, Nash and Sutcliffe (1970) proposed a dimensionless coefficient (E_f), given as: efficiency

$$E_f = \frac{\sum_{i=1}^n (Q_{obs,i} - Q_{comp,i})^2}{\sum_{i=1}^n (Q_{obs,i} - \bar{Q}_{obs})^2} \quad (2)$$

in which $\bar{Q}_{obs} = \frac{1}{n} \sum_{i=1}^n Q_{obs,i}$ and $\bar{Q}_{comp} = \frac{1}{n} \sum_{i=1}^n Q_{comp,i}$.

The coefficient of efficiency provides a well accepted measure of fit between computed and observed hydrographs, as it varies toward unity as the simulated hydrograph progressively improves (Green and Stephenson, 1986). Nash and Sutcliffe (1970) considered to reflect a good adjusted model. The Muskingum and HECRAS hydrographs in Figure 1 are compared to the adjusted hydrographs developed from observed data.

Table

Table shows that the Muskingum method produces outflow hydrographs compare favourably with the adjusted hydrographs developed through HECRAS modelling. The trends between both the Muskingum HECRAS hydrographs and the adjusted hydrographs from the si-

1 assumptions in the momentum exchange between the main channel and floodplain more
 2 exchange the River Shurensay flood wave attenuation and decrease
 3 However, modified routing parameters were developed from analysis of
 4 data from modeling generated in which the energy losses from
 5 interactions not included. Furthermore, the influences of geometrical,
 6 hydrograph properties in this analysis were assessed as independent
 7 Similarly, main channel and interactions are unaccounted for in
 8 hydrographs developed from modeling of the River Shurensay reach.
 9 these interactions, however, approach the relative importance of
 10 considered a reasonable estimate for the presented method. The
 11 satisfactory performance of the method further implies that the storage
 12 Muskingum routing methods is a substitute for the momentum
 13 approach in typical Irish rivers therefore it is reasonable to attribute the
 14 to channel and flow characteristics (Perumal, 1992)

15 It should be noted however that the limitations of the applied
 16 hydrological or Muskingum methods and therefore, the results
 17 should be confirmed if applied to river reaches where backwater
 18 significant, where floodplain sinuosity is excessively high or
 19 momentum exchange between main channel and floodplain zones are
 20 method however, does provide a simple and inexpensive method
 21 estimates of the time and shape of a flood hydrograph as a first
 22 reach

23

7 Conclusions

A modified Muskingum routing method is presented in this paper. The method is based on relationships in river systems and can satisfactorily reproduce systems where inertia effects and backwater influences are significant. The routing parameters that describe the storage characteristics of a river reach are usually derived analytically from observed hydrographs and extracted from historical flow records. The expressions for K and x are described in terms of standard geometrical and resistance properties of floodplains together with a procedure for determining the relationships were based on regression analysis of computational data generated from the modelling of a generalised river reach. The expressions are:

$$K = 0.794 \frac{L n_p^{0.24} n_{mc}^{0.42} Q_p^{0.60} T_B^{0.07}}{S_p^{0.53} Q_p^{0.09} (h_m + B)^{0.06}} \quad x = 0.035 \frac{L S_p^{0.16} T_B^{0.39}}{(h_m + B)^{0.05} n_p^{0.06} n_{mc}^{0.006} n_p^{0.006}}$$

The method requires a x and K and therefore floodplains are represented by a x value. Furthermore, the method does not take into account the effects of bank clogging. The method was applied to a reach of the River Suir, Co. Tipperary, Ireland and the outflow hydrographs that compared favourably with the observed records.

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Appendix

Table A1

1 List of Figures

2 Fig. 3 Stages in the development of the modified flooding parameter

3 Fig. 2 Development of theoretical flood hydrographs for routing

4 Fig3. Notation describing cross geometry in model of generalise

5 Fig. 4 Influence of hydrograph and floodplain properties on Mus

6 (K).

7 Fig5. Influence of hydrograph and floodplain properties on Mus

8 factor. (

9 Fig6. Comparison of simulated storage (K) and storage parameter (st

10 (x) to that calculated using empirical methods on lin

11 Fig7. Study region of the River Suir catchment.

12 Fig8. Hydrographs at Killardry, New Bridge (1954) and 15C (after 1986) for

13 2004 storm events

14 Fig9. Measured and routed outflow hydrographs at Callow Park for a

15 contribution from ungauged tributaries

16 Fig.10. Routed hydrographs of New Bridge (1600) at Callow Park

17 data adjusted HECAS modelling (SHEC) and the modified Muskingu

18 method (Muskingum) using floodplain model (HECAS model

19 simulations.

20 Fig.1.1 Routed hydrographs of New Bridge (1600) at Callow Pa

21 data adjusted HECAS modelling (SHEC) and the modified Muskingu

22 method (Muskingum) using floodplain model (HECAS model

23 attenuation indicators (FAIs).

24

25

1 List of Tables

2 Table 1 Catchment description for 1954/55, 1986 and 2004 flood events

3 modified Muskingum routing method

4 Table 2 Goodness of fit for modified Muskingum (Muskingum RAS and HEC

5 hydrographs comparison with other direct hydrographs

6

7 Table 3 Summary of simulation results for Muskingum routing parameters

8 modelling of generalised river reach

9

10