A WATERSHED BOUNDED NETWORK MODEL FOR FLOOD STUDIES

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ABSTRACT

A computer model for flood studies on rural and urban catchments is described. The model subtracts rainfall losses from rainfall hyetographs and calculates the resulting flood hydrographs. Both recorded floods and design floods can be calculated, the latter using built-in design rainfall intensity and temporal pattern data. The computer program includes built-in culvert and weir hydraulics, and level pool storage reservoir routing procedures. The model is tested using recorded data from 10 rural and 9 urban catchments in Australia.

INTRODUCTION

The Watershed Bounded Network Model WBNM was developed by Boyd, Pilgrim and Cordery (1979), and extended by Boyd et al (1987). It is described in Australian Rainfall and Runoff (Instn. Engrs. Australia, 1987). WBNM calculates flood hydrographs resulting from storm rainfall hyetographs, using runoff routing techniques. The model has recently been further developed into a comprehensive computer package for flood studies on natural and urban catchments. Its features now include built in design storms, optional rainfall loss models, spatially varying rainfall and losses, channel routing in watercourses, flood routing through storage reservoirs, runoff from impervious surfaces in urban catchments, and built in culvert and weir hydraulics. The computer program is menu driven, has full graphics capabilities, and is designed to satisfy quality assurance requirements. This paper describes the theoretical basis of the model, with a brief description of the computer program.

BACKGROUND TO THE MODEL

Model Structure

The catchment is divided into subcatchments using the stream network to define a branched network structure. Each subcatchment or watershed area is bounded by its dividing ridge, hence the name Watershed Bounded Network Model. Lag times for runoff routing are allocated to subcatchments depending on their size, therefore the number of subcatchments into which the catchment is divided is not crucial. The catchment can consist of a few large subcatchments with large lag times, or many small subcatchments with small lag times which, in combination, produce a large lag for the total catchment. Generally, small catchments will have few subcatchments (as few as one for small urban catchments) while large catchments will be divided into many subcatchments. Guidelines for division into subcatchments are given by Boyd et al (1987). Fig. 1 shows a typical catchment division.

Lag Relations for Subcatchments

The basic relation in runoff routing models is the lag time between centroids of inflow to the subcatchment and outflow from the subcatchment. Many studies have shown that the lag time is approximately proportional to the flow path length, which is itself related to the square root of catchment area (actually the power of area is closer to 0.6 than 0.5 : Hack, 1957; Leopold et al, 1964). Catchment area is the dominant variable, with catchment slope being of secondary importance.

In WBNM the lag time for transforming excess rainfall into a surface runoff hydrograph at the subcatchment outlet is based on the detailed studies of lag times in natural catchments by Askew (1970). The relation is

$$K = c\ A^{0.57}\ Q^{-0.23} \qquad (1)$$

where the lag time K is measured in hours, catchment area A in km^2, discharge Q in m^3/s, and c is a scaling parameter which controls the magnitude of the lag time.

Equation (1) shows that the lag time is not constant for all floods on a catchment or even during the one flood, rather it continually varies in inverse proportion to the stream discharge. The result is that runoff routing models using this relation are nonlinear, rather than linear as in unit hydrograph models.

Equation (1) refers to the transformation of excess rainfall into a runoff hydrograph at the catchment outlet. Because subcatchments are hydrologically similar to the larger catchment of which they are part, WBNM applies (1) to the individual subcatchments. This means that only the one scaling parameter c needs to be specified, and lag times are automatically calculated for all subcatchments.

Some subcatchments (3, 5, 7, 9, 10 in Fig. 1) not only transform the excess rain falling on their surfaces into a runoff hydrograph, but additionally transmit runoff from upstream subcatchments through the stream channel. Because flow velocities are higher in stream channels than for overland flow, the lag time for this channel routing will be less than the value for transformation of excess rainfall into runoff.

The decrease in stream slope in moving from the upper reaches of the catchment to the outlet is compensated by an increase inflow depth, so that flow velocities do not vary significantly along the channel. Studies of "between station" hydraulic geometry, for example by Leopold (1953), indicate that velocities remain relatively constant or increase slightly in the downstream direction. Therefore the lag time will be proportional to the length of the stream channel segment for the subcatchment divided by the flood wave speed. As before, the channel length is related to the approximate square root of subcatchment size, similar to the subcatchment length-area relation.

To simplify the lag relations in WBNM, (1) is adopted for channel routing, but with a reduction factor to account for the reduced lag time. In a study of 10 natural catchments, ranging in size from 0.4 to 90 km^2, Boyd et al (1979) used (1) to allocate lag times for excess rainfall-runoff routing to all subcatchments depending on their size, then calculated the channel segment lag times required to reproduce the correct lag time for the total catchment. The average reduction factor was found to be 0.6. The lag time for channel routing adopted for natural catchments in WBNM is therefore

$$K = 0.6 \ c \ A^{0.57} \ Q^{-0.23} \tag{2}$$

Note that the combination of the reduction factor 0.6, together with the larger discharges in channels compared to overland flow, gives a significantly reduced lag time for channel routing.

Runoff routing calculations solve a conservation of mass and a storage-discharge equation:

$$I(t) - Q(t) = dS/dt \tag{3}$$

$$S = 3600 \ K.Q \tag{4}$$

where I is the inflow to the subcatchment (m^3/s), Q is the outflow from the subcatchment (m^3/s), S is the volume of water in storage at time t (m^3), and K is the lag time for the subcatchment (hours).

Equations (3) and (4) are solved to give the runoff routing equation:

$$Q_2 = ((I_1 + I_2)\ 0.5\ \Delta t \ + \ Q_1\ (K_1 - 0.5\ \Delta t))/(K_2 + 0.5\ \Delta t) \tag{5}$$

where subscripts 1 and 2 refer to values at the start and end of the time step Δt (hours).

For nonlinear routing, Q_2 depends on K_2, which is itself a function of Q_2 (equations 1,2). Therefore (5) is solved interactively at each time step until convergence occurs.

Application to Recorded Flood Data

After formulating the model structure, WBNM was tested using 129 recorded floods on 10 natural catchments in New South Wales, Australia (Boyd et al, 1979, 1987). The procedure was to first determine the rainfall losses (initial loss and continuing loss rate) by noting the start of hydrograph rise and by balancing volumes of excess rainfall and recorded surface runoff. Next, the model lag parameter c was determined to fit calculated and recorded hydrographs. The primary fitting measure was the peak discharge, but hydrograph shape was also considered.

Fig. 2 shows values of parameter c plotted against catchment size for all 10 catchments. The average value for all catchments is c = 1.68. Note that there is no systematic trend for c to increase or decrease with catchment size, indicating that the power 0.57 for A in (1,2) is correct.

Fig. 3 shows the percentage error in peak discharge relative to the recorded peak discharge for all 129 events, plotted against the recorded flood peak discharge. Again, there is no trend with event size, indicating that the nonlinearity power 0.23 in (1,2) is correct for this data. 15.4% of peak discharges were overestimated by more than 15%, and 26.4% were underestimated by more than 15%. The coefficient of variation of the error was 0.163. WBNM has also been applied to 33 catchments, ranging in size from 0.4 to 9000 km^2 (Sobinoff et al, 1983) with similar results.

The previous results refer to recorded rainfall and flood hydrographs. In many cases flood hydrograph models are used to calculate design floods from design storms. In this case, the aim is to use a design storm of specified frequency of occurrence to calculate a design flood with the same frequency. Boyd and Cordery (1989) used design flood procedures set out in Australian Rainfall and Runoff (I.E. Aust, 1987) to calculate values of lag parameter c which achieved this on 36 catchments, ranging in size from 0.04 to 1140 km^2. The procedure was to calculate the rainfall intensity of specified frequency of occurrence and critical duration for the catchment, distribute the rainfall into a design temporal pattern, subtract design rainfall losses and use WBNM to calculate the resulting design flood. The peak discharge was then compared with the value obtained from a log Pearson III frequency analysis of recorded flood data. Parameter c was adjusted so that the calculated flood peak agreed with the value from the flood frequency analysis. The average parameter c value for these catchments was found to be 1.80, substantially agreeing with the result for recorded rainfall and flood hydrographs.

Fig. 4a shows typical calculated and recorded hydrographs for a natural catchment.

Recommended Value of Lag Parameter c

Application of WBNM to a natural catchment simply requires that the catchment be divided into subcatchments and the area of each subcatchment be measured. Only parameter c, which controls the magnitude of lag time, needs to be specified. WBNM automatically calculates lag times for all subcatchments and stream segments using (1) and (2). If recorded rainfall and flood data are available these should be used to calibrate the model. If these are not available, values of lag parameter c from nearby catchments in the region should be used. From the studies described in the previous section, values of parameter c can be expected to be close to 1.7. Note that this corresponds to flood wave speeds of the order of 1.5 m/s in stream channels and 0.2 m/s for overland flow, for a flood of the order of 500 m^3/s at the catchment outlet.

DESIGN FLOOD ESTIMATION

Design flood estimation from design rainfall data involves the following steps:

- select the design frequency of occurrence
- calculate the rainfall intensity for this frequency
- distribute the design rainfall into a design temporal pattern
- subtract design rainfall losses
- calculate the design flood hydrograph resulting from this design storm

WBNM calculates design floods using procedures set out in Australian Rainfall and Runoff (I. E. Aust., 1987). The design rainfall intensity for various frequencies is given by a log Pearson type III distribution fitted to map values of 2 year and 50 year rainfall intensities for all of Australia. The design rainfall for various storm durations is given by interpolation between 1, 12, and 72 hour values, and extrapolation down to durations of 5 minutes. Design storm temporal patterns are provided for all of

Australia. WBNM has built in functions to calculate rainfall intensities, and a library of temporal patterns to calculate the design storms. The user simply has to nominate the design frequency and storm duration, and WBNM calculates the design storm and resulting design flood. Currently WBNM has built in design storms for Australia. It would be a simple matter however to include algorithms and temporal patterns for other countries.

WBNM calculates design storms in this way for average recurrence intervals of 1, 2, 5, 10, 20, 50, 100, 200, and 500 years. Additionally, Probable Maximum Floods are calculated for catchments less than 1000 km^2 and storm durations up to 6 hours. This uses the generalised short duration depth-duration-area data, plus associated temporal patterns, of the Bureau of Meteorology (1994), which in turn is based on US National Weather Service (1988) procedures.

RAINFALL LOSSES

The model includes four rainfall loss models: initial loss-continuing loss rate; initial loss-runoff proportion; Horton exponential; and initial loss-stepped loss rate where up to 3 loss rates can be specified, starting at different times during the storm. Rainfall losses can be global, applying to all subcatchments, or spatially varying for the different subcatchments.

FLOW DIVERSIONS

If flows in a stream exceed the channel capacity, flow spreads out onto the adjoining flood plain. In some circumstances the excess flow may be diverted out of the channel to flow down a floodway before rejoining the stream at a downstream subcatchment. WBNM models this by specifying a threshold discharge above which a portion of the excess flow is diverted. The downstream subcatchment is nominated, together with the time delay until the flow rejoins the stream channel system.

The portion of excess flow which is diverted can be calculated from open channel hydraulics. Fig. 5 shows schematic main channel flows and diverted flows at a point in the stream. For this example, the threshold discharge for flow diversion is 70 m^3/s, and 75% of excess flows are diverted.

RATING TABLES

WBNM allows rating tables of water level in the stream channel and the corresponding discharge to be entered for any subcatchment. The peak flood discharge at this point is then noted and used to give the maximum flood level at the location.

FLOOD ROUTING IN STORAGE RESERVOIRS

Puls' level pool routing is used for flood routing in reservoirs. Outlet structures can be culverts or spillways. The invert of the outlet can be above the bed of the storage (Fig. 6) in which case the "dead volume" below the outlet is first filled before outflow commences. The initial water level in the storage can be at any level, for example if a preceding flood has not completely drained, and reservoir routing commences from this level.

Reservoir routing requires information on the water level H-outflow discharge Q-stored volume S relation for the storage. This is supplied as a table of values. Alternatively, the size and type of the outlet structure can be specified, and WBNM calculates the H-Q relation from culvert and spillway hydraulic relations.

CULVERT AND SPILLWAY HYDRAULICS

Spillway hydraulics use the standard equation

$$Q = C_d \; L \; (H\text{-}HSPILL)^{1.5} \tag{6}$$

where the discharge coefficient is for SI units and is typically 1.7 for broad crest and 2.2 for ogee crest spillways.

Culvert hydraulics use equations fitted by Boyd(1987) to the US Dept. of Transportation (1965) culvert charts and found to also apply to other published data, for example in Chow(1959) and French (1987). For inlet control, these are of the form

Rectangular Box culvert $Q = a\ B^b\ D^c\ (H\text{-}HCULV)^d$ (7a)

Circular Pipe culvert $Q = a\ D^c\ (H\text{-}HCULV)^d$ (7b)

where discharge Q is in m^3/s, box culvert width B is in metres, box culvert height and pipe culvert diameter D are in metres, and the coefficients a, b, c and d vary depending on whether the entrance is submerged or not.

URBAN CATCHMENTS

Catchment changes from natural to urban land use are modelled in two ways: changes in surface runoff from impervious compared to pervious surfaces; and changes of lag times in channels.

Plots of runoff depth versus rainfall depth (Miller et al., 1978; Jacobsen and Harremoes, 1981; Pratt et al.,1984; Boyd et al, 1993) indicate that runoff in urban catchments can consist of two parts: impervious surface runoff with small rainfall losses, short lag times and consequently rapid response to rainfall; and pervious surface runoff with higher rainfall losses, longer lag times and slower response to rainfall. In WBNM these are modelled separately, with hydrographs calculated for impervious and pervious surfaces for each subcatchment. Pervious area runoff uses the same rainfall loss models and lag equations discussed earlier. Impervious area runoff has a specified small initial loss and zero continuing losses, and a reduced lag time.

The lag time for impervious surface runoff is based on studies by Rao et al (1974), Aitken (1975) and NERC (1975). These show that lag times for urban catchments are generally reduced below lag times in natural catchments according to the ratio $(1+URB)^2$, where URB is the fraction of the catchment which is urbanised. The urban fraction URB will contain a mix of impervious surfaces such as roads and roofs, plus grassed surfaces such as lawns and gardens. The impervious fraction IMP will therefore be less than the urban fraction URB.

Note that a fully urbanised catchment (URB=1) gives a reduction factor of 0.25, but this fully urbanised catchment will still have some grassed surfaces with longer lag times. The lag time reduction on fully impervious surfaces will be even less than 0.25.

For nine urban catchments in Australia, the following relation for lag time on impervious surfaces was derived

$$K_{imp} = 0.1 \ c \ A_{imp}^{0.25} \qquad (8)$$

where A_{imp} = A.IMP is the impervious area of the subcatchment in km^2, and the scaling parameter c is the same as for natural catchments. Note that (8) calculates a fixed lag time for the impervious surfaces of each subcatchment depending only on its size and not on the discharge Q. This is because runoff from impervious surfaces of urban catchments has been found to behave linearly (Willeke, 1966).

Three options are provided for channel routing in urban catchments. Firstly, nonlinear routing using (2) and (5) but with a reduced lag time, where the lag reduction factor 0.6 is changed in inverse proportion to the flow velocities. For example, if the urban channel velocity is two times the natural channel velocity, the lag reduction factor becomes 0.3. Secondly, Muskingum routing, where the lag parameter K is determined from the stream channel length divided by the flood wave celerity, and the distributed routing parameter x is selected to account for the delay in the peak beyond the recession limb of the inflow hydrograph. Thirdly, the inflow hydrograph at the top end of the channel can be simply delayed by a specified number of minutes, depending on the flow velocity.

Fig. 4b shows typical calculated and recorded hydrographs for an urban catchment.

COMPUTER PROGRAM WBNM

The computer program is based around three modules, a menu system module, a computation module, and a graphics module. The menu system allows efficient data file handling, including creating, copying and editing. A single data file contains all information for the run, including catchment, storage reservoir, culvert and spillway, storm rainfall, rainfall loss, and parameter details. All results are written to a meta file to preserve a complete record for quality assurance purposes. The computation module performs all numerical calculations. The graphics module allows a schematic layout of the catchment to be viewed, as well as hydrographs for all subcatchments. For each subcatchment, runoff hydrographs from pervious and from impervious surfaces are displayed, as are diverted hydrographs. The hydrograph at the top end of the stream channel, and at the bottom end, after channel routing is displayed. Various options in the graphics are easily selected using mouse clicks or arrow keys.

CONCLUSIONS

This paper has described the background to the Watershed Bounded Network Model which calculates flood hydrographs from storm rainfall hyetographs. The model has been successfully applied to a wide range of natural and urban catchments. A computer program WBNM is available which includes flood routing through storage reservoirs, built in culvert and spillway hydraulics, diversion of excess flows, and built in design storms, including probable maximum flood estimates.

Copies of the computer program WBNM can be obtained, free of charge from the first author at Dept. of Civil and Mining Engineering, University of Wollongong, Australia, 2522, fax +61 42 213238, email m.boyd@uow.edu.au.

APPENDIX I. REFERENCES

Aitken, A.P. Hydrologic investigation and design in urban stormwater drainage systems.(1975). Australian Water Resources Council Technical Paper 10. Australian Govt. Publishing Service. ISBN 0 642 501 343.

Askew, A.J. (1970). "Variation in lag time for natural catchments." Proc. ASCE, Vol. 96 (HY2), 317-330.

Boyd, M.J., Pilgrim, D.H. and Cordery, I.(1979) "A storage routing model based on catchment geomorphology". J. Hydrology, 42, 209-230.

Boyd, M.J., Bates, B.C., Pilgrim, D.H. and Cordery, I.(1987). "WBNM-a general runoff routing model". University New South Wales Water Research Laboratory Report 170. ISBN 0 85824 379 2.

Boyd, M.J.(1987). "Head-discharge relations for culverts". Proc. 21st Congress International Association for Hydraulic Research. Vol. 6. Keynote Addresses, 118-122. ISBN 0858252805.

Boyd, M.J. and Cordery, I.(1989). "A study of design parameters for flood hydrograph models". Instn. Engrs. Australia, Hydrology and Water Resources Symposium. National Conference Publication 89/19, 119-123.

Boyd, M.J., Bufill, M.C. and Knee, R.M.(1993). "Pervious and impervious runoff in urban catchments". Hydrological Sciences Journal, 38(6), 463-478.

Bureau of Meteorology (1994). The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method. Australian Govt. Publishing Service. ISBN 0 644334193

Chow, V.T.(1973). Open Channel Hydraulics. McGraw Hill, ISBN 007 085906X.

French, R.H. (1987). Open-Channel Hydraulics. McGraw-Hill ISBN 007 Y663424.

Hack, J.T.(1957). "Studies of longitudinal stream profiles in Virginia and Maryland"., U.S. Geol. Survey Prof. Paper 294-B.

Institution of Engineers Australia.(1987). Australian Rainfall and Runoff - A Guide to Flood Estimation. 2 volumes. ISBN 085825 434 4.

Jacobsen, Per and Harremoes, P.(1981). "Significance of semi pervious surfaces in urban hydrology, Proc. 2nd Intl. Conf. Urban Storm Drainage, ed. B.C. Yen, Water Resources Publications, Littleton, Colorado, 424-433.

Leopold, L.B.(1953) "Downstream change of velocity in rivers". American J. Science, 251, 606-624.

Leopold, L.B., Wolman, M.G. and Miller, J.P.(1964). Fluvial Processes in Geomorphology. Freeman, San Francisco.

Miller,R.A., Mattraw, H.C. and Jennings, M.E.(1978). "Statistical modelling of urban storm water processes". Proc. Intl. Symp. on Urban Stormwater Management, ed. C.T. Haan, Univ. KentuckyPublications, 269-273.

Natural Environment Research Council.(1975). Flood Studies Report Vol. 1 Hydrological Studies. ISBN

Pratt, C.J., Harrison, J.J. and Adams, J.R.W.(1984). "Storm runoff simulation in runoff quality investigations". Proc. 3rd Intl. Conf. Urban Storm Drainage, ed. P.Balmer, Chalmers Univ. Technology, Goteborg, 285-294.

Rao, A.R., Delleur, J.W. and Sarma, P.B.S.(1974). "Conceptual hydrologic models for urbanising basins". Proc. ASCE, 98(HY7), 1215-1220.

Sobinoff, P., Pola, J.P. and O'Loughlin, G.G.(1983). "Runoff routing parameters for the Newcastle-Sydney-Wollongong region". Instn. Engrs. Australia, Hydrology and Water Resources Symposium, National Conference Publication 83/13, 28-32.

U.S. Dept. Transportation (1965). Hydraulic Charts for the Selection of Highway Culverts. Federal Highway Admin. Hydraulic Engg. Ciurcular No.5. US Govt. Printing Service.

U.S. National Weather Service (1988). Probable Maximum Precipitation Estimates - U.S. between the Continental Divide and the 103rd meridian. Report No. 55A.

Willeke, G.E (1966). "Time in urban hydrology". Proc. ASCE, 92 (HY1), 13-29.

APPENDIX II. NOTATION

The following symbols are used in this paper :

A	=	catchment or subcatchment area (km^2)
A_{imp}	=	impervious area (km^2)
B	=	box culvert width (m)
C	=	scaling parameter controlling magnitude of lag time
Cd	=	spillway discharge coefficient (for SI units)
D	=	height of box or pipe culvert (m)
Δt	=	time step in routing calculations (hour)
H	=	water level above bed of storage reservoir (m)
HCULV	=	invert level of culvert (m)
HSPILL	=	invert level of spillway (m)
I	=	inflow to subcatchment (m^3/s)
K	=	lag time between centroids of inflow and outflow hydrographs (hour)
L	=	length of spillway crest (m)
Q	=	discharge (m^3/s)
S	=	volume of water in temporary storage (m^3)
URB	=	fraction of catchment which is urbanised (range 0-1)
IMP	=	fraction of catchment which is impervious (range 0-1)
x	=	Muskingum channel routing distributed parameters (range 0 - 0.5)

KEY WORDS

Flood hydrograph, Urban catchments, Design flood, Storage reservoir routing, Detention basin, Culvert hydraulics.

FIGURES

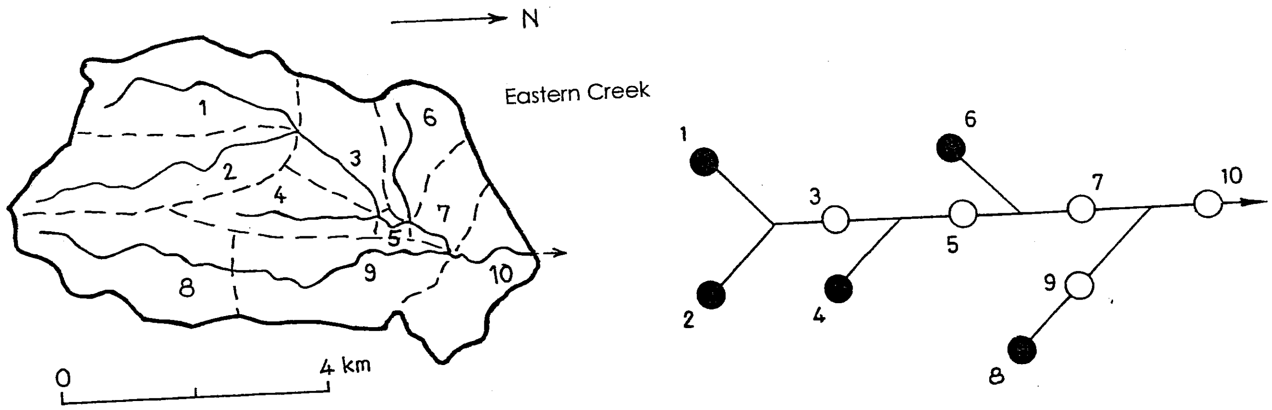

FIG. 1. Typical Catchment and Model Structures.

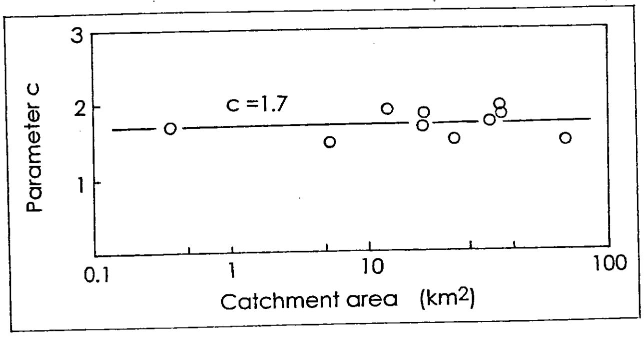

FIG. 2. Lag Parameter c versus Catchment Size.

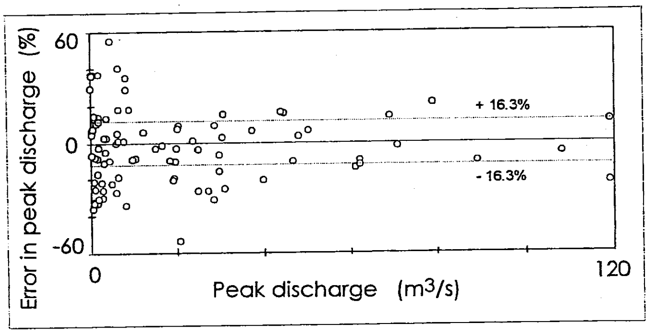

FIG. 3. Accuracy of Model Prediction.

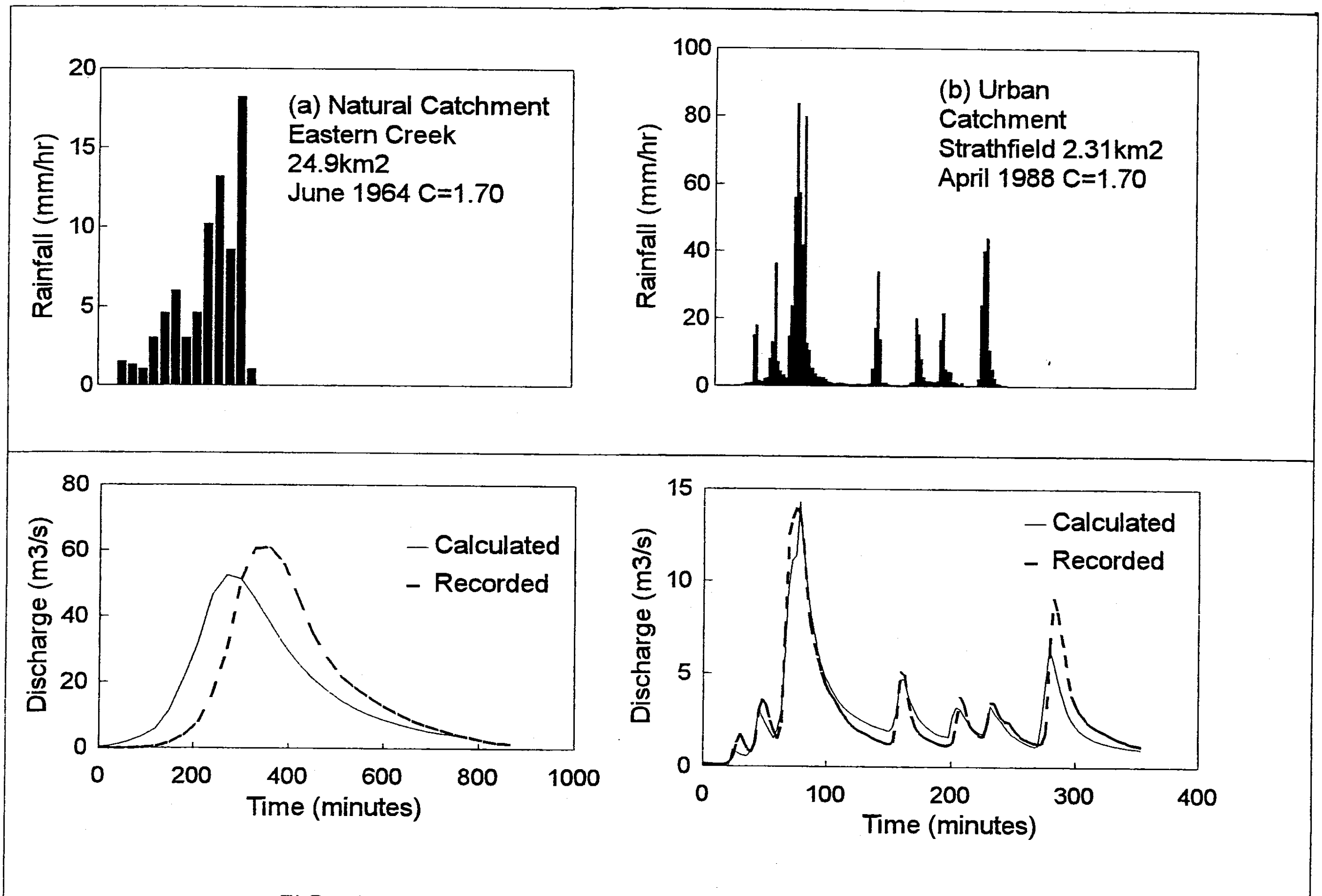

FIG. 4. Calculated and Recorded Hydrographs.

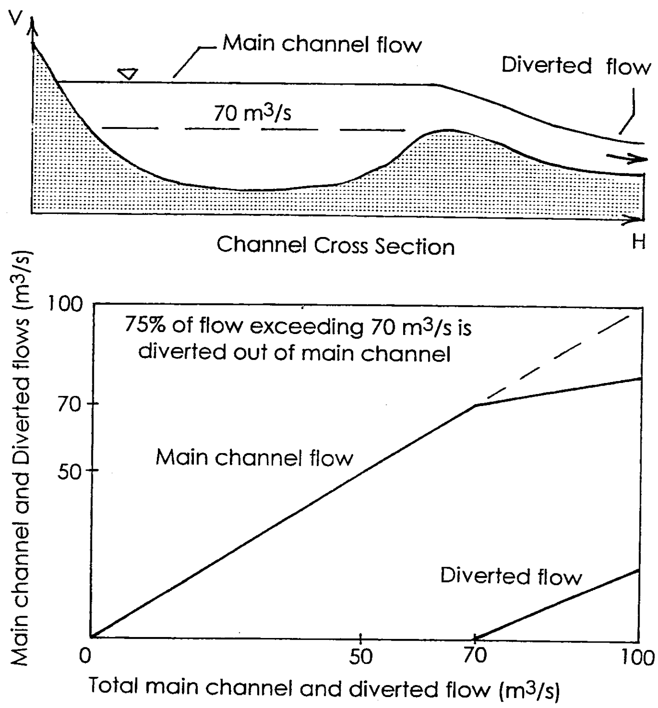

FIG. 5. Diversion of Flows Exceeding Stream Channel Capacity.

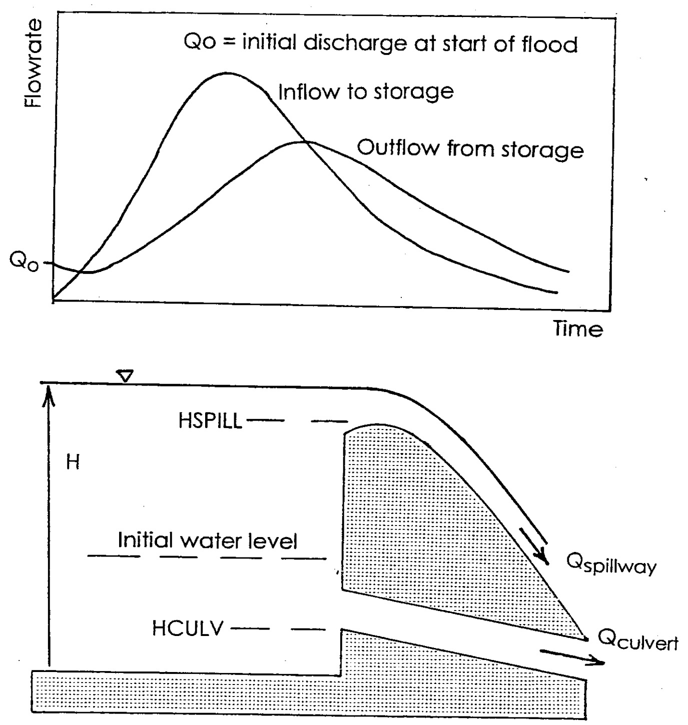

FIG. 6. Flood Routing through Storage Reservoirs