A DETERMINISTIC RUNOFF MODEL FOR USE IN FLASH FLOOD PLANNING

David C. Curtis

Office of Hydrology
National Weather Service, NOAA
Silver Spring, Md. 20910

Robert L. Mitchell
Office of Hydrology
National Weather Service, NOAA
Silver Spring, Md. 20910

John C. Schaake, Jr.
Office of Hydrology
National Weather Service, NOAA
Silver Spring, Md. 20910

1. INTRODUCTION

As the rate of flood plain intrusion in the United States increases, the upward trend of potential flood damages (economic as well as human) will undeniably continue. If communities are willing to assume the risks of flood plain occupation, it seems reasonable that they should invest in technologies that attempt to minimize expected losses from the inevitable flooding.

Flash floods are by definition short fused events. In order to effectively cope with flash floods, a community must be prepared. Each segment of the population must know what its responsibilities are and alternative responses as well. Communities should engage in pre-flood planning that identifies potential problem areas, quantifies potential flood flows or stages, and established the minimum response times of both watershed to rainfall and the community to the flood. Much emphasis should be placed on addressing the situation when, given that a significant flood may occur, how do the technicians monitoring the event effect the most appropriate community responses.

The following paragraphs discuss some of the important ideas to be reviewed during the pre-flood planning stages. Also, a catchment model is presented that can be very useful in identifying the important quanitification factors regarding a flash flood. Two detailed examples of model applications are included.

2. FLASH FLOOD PLANNING

The National Weather Service's flash flood planning begins by documenting communities and recreational areas that have a potential for flash flooding. An examination should then be made of the area's historical flood record, hydrologic regime, topography, flood control or water supply structures, data networks, and warning time required in the event of a flash flood. For many communities, a look at these factors will show that the NWS's existing flash flood warning program will do an adequate job.

In some cases, response time of the catchment may be too short or the NOAA observational networks cannot "read" the event and a flash flood catches a town or recreational area unprepared. In these latter cases, some type of warning system is needed that can be locally operated and maintained.

The NWS offices responsible for implementing flash flood warning systems need forecast procedures that can be used quickly and give good forecasts of the timing and height of the flood peak. These forecast procedures can be developed from hydrologic models with parameters adjusted to the flash flood prone basin and then pre-run for a variety of rainfall events. The accumulated results can be combined into a tabular or graphical look-up. If the office has access to a computer, the model could be run on demand provided that data input and computer turnaround are quick.

In those communities or areas with flash flood warning lead times too short for the NWS warning program to operate effectively, a locally operated system probably provides the only adequate warning time. The NWS could then offer a Local Flash Flood Warning System (LFFWS) to the community. This system is largely a NWS designed forecast procedure used in conjunction with a variety of data collection equipment. The forecast procedures, operated by individuals within the community, should be easy to use and give good results quickly. As with the NWS operated procedures, they can be developed from hydrologic models and put in a tabular or graphical look-up. Even though operated and maintained by communities, it is up to local NWS personnel to keep interest high through repeat visits and inspections on site.

The planning of a NWS or locally operated flash flood warning system must also consider its data network. Every effort should be made to get all existing rain and river gages into a real-time reporting network. The data collection equipment offered by NWS should cover a wide range of sophistication and cost and be reliable.

Flash flood planning is really a joint effort by the NWS and the community. The National Weather Service has the expertise to develop flash flood forecast procedures and also can make available to the community a variety of data collection equipment. The community in turn must work the procedures and equipment into a local operation that will take the flash flood forecast and quickly get it to the threatened individuals and businesses.

3. MODEL CONFIGURATION

The equations of continuity and momentum have been used for many years to describe fluid flow in long rivers. However, in recent years, certain simplifications or approximations to these equations (known as the kinematic approximation) have been used quite successfully to describe overland flows. Owing to the pioneering work of Lighthill and Whitham and Wooding in kinematic wave theory, investigators such as Harley, Schaake, and Woolhiser and Liggett have used kinematic wave theory to describe flows throughout a catchment.[3,5,8,11,12]

The catchment model used in this study is a version of the Deterministic Urban Model developed by Schaake for the Urban Water Systems Institute of Colorado State University.[8] The model utilizes kinematic wave theory to describe fluid flow over land surfaces as well as in stream channels. The availability of water for overland flow is determined by either the Hortonian concept of infiltration or the SCS Curve Number method. The following sections briefly describe the model representation of a catchment, kinematic wave theory, and the numerical solution procedure.

Catchment Representation

In a real catchment, the number of alternative flow paths available from the point of raindrop impact to the catchment outfall is extraordinary. To represent the system exactly, a model of enormous complexity would be required. Such a model would be expensive and unnecessarily difficult to use. Thus, many small details must be simulated in the aggregate, while still maintaining the integrity of the dynamics of surface runoff.

The runoff model conceptualizes a natural catchment (figure 1) as a set of flow segments (figure 2). Each flow segment is considered to have a uniform set of flow parameters (i.e., uniform roughness, infiltration, slope, etc.). The segments are generally described as overland flow segments or as channel segments. Each overland segment is an inclined plane of a given slope, surface roughness, and precent imperviousness. Also, the appropriate infiltration parameters are given for the previous area within an overland flow segment. The channel segments are either open channels or closed conduits. The open channels are troughs of triangular or rectangular shape. The closed conduits represent sewer flow and are either rectangular or circular in shape. The model is conceptually simple, but a set of flow segments can be easily arranged into a network that will represent many complex catchments.

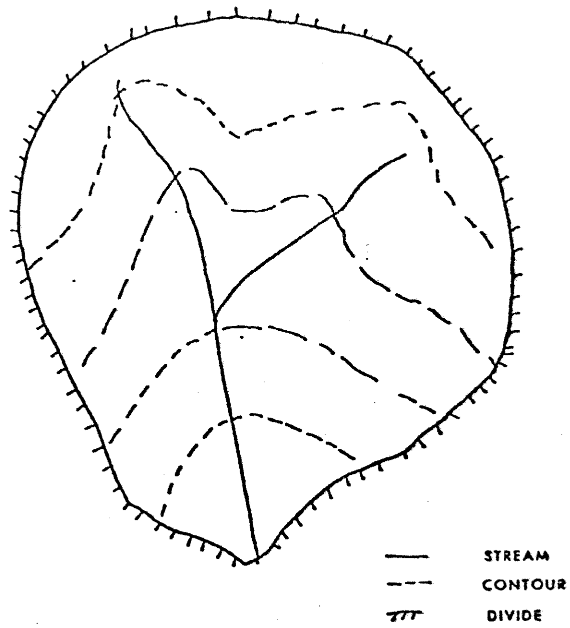

Figure 1. Natural Catchment.

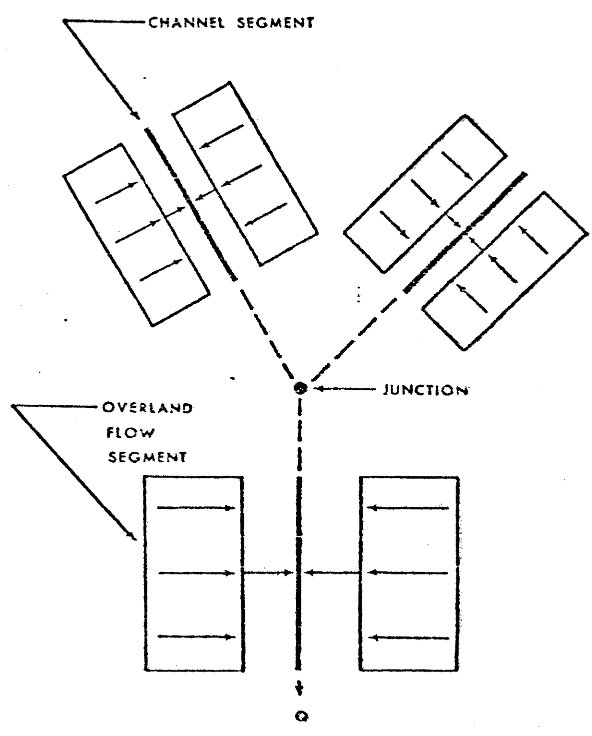

Figure 2. Conceptualized Catchment.

Kinematic Wave Equations

The rainfall/runoff model is based upon the kinematic approximation of the fluid continuity and momentum known as the St. Venant equations. The St. Venant equations are:

Continuity Equation

$$\frac{\partial q}{\partial x} + \frac{\partial y}{\partial t} = q_1 \qquad (1)$$

Momentum Equation

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

where q is the flow rate per unit width, y is the depth, q_1 is the lateral inflow, S is the channel bottom slope, v is the flow velocity, S_f is the friction slope, and g is the gravitational acceleration constant.

Analytical solutions to equations (1) and (2) are not possible due to the non-linearities in equation (2) and the complex nature of the boundary conditions. The St. Venant equations can be solved numerically and these techniques are well established. However, application of the St. Venant equations to overland flow would require solution on an extremely small scale both spatially and temporally. The resulting computer requirements would be excessive.

Lighthill and Whitham have shown that movement of a flood wave in a river is composed of dynamic and kinematic effects.[5] They also indicated that the dynamic component decays exponentially for Froude numbers less than two. Woolhiser and Liggett have also indicated that the dynamic effects could be neglected if:

$$\frac{SL}{yF} > 10 \qquad (3)$$

where S is the channel bottom slope, y is the depth, L is the length, and F is the Froude number.[12] By neglecting the dynamic effects, the momentum equation (equation 2) is approximated as:

$$S = S_f \qquad (4)$$

Equation 4 is the steady flow form of the momentum equation, which can also be written as:

$$q = \alpha y^m \qquad (5)$$

where α and m are the kinematic flow parameters.

Equations 2 and 5 can easily be solved numerically and are used as the basis for the mathematical description of both overland and channel flows by the model. The kinematic wave equations for an overland flow segment are:

$$\frac{\partial q}{\partial x} + \frac{\partial y}{\partial t} = q_1 = i - f \qquad (6)$$

$$q = \alpha_o y^{m_o} \qquad (7)$$

where i is the rainfall intensity, and f is the infiltration rate. The quanity i - f is the rainfall excess, and the subscript "o" refers to the overland flow plane.

The corresponding equations for a channel segment are:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q_1 \qquad (8)$$

$$Q = \alpha_c A^{m_c} \qquad (9)$$

where A is the cross sectional area of flow; Q is the discharge rate; and q_1 is the lateral inflow rate of overland flow. The subscript "c" refers to the channel segment.

The kinematic wave parameters α and m can be estimated by Manning's formula. In the case of overland flow:

$$q = \frac{1.49}{n_o} Y_o^{5/3} S_o^{1/2} \qquad (10)$$

where

$$\alpha_o = \frac{1.49}{n_o} S_o^{1/2} \qquad (11)$$

$$m_o = 5/3 \qquad (12)$$

In the case of a triangular channel:

$$Q = \frac{1.82}{n_c}\left[\frac{\sqrt{Z}}{1+\sqrt{1+Z^2}}\right]^{2/3} A^{4/3} S_c \qquad (13)$$

$$\alpha_c = \frac{1.82}{n_c}\left[\frac{\sqrt{Z}}{1+\sqrt{1+Z^2}}\right]^{2/3} S_c^{1/2} \qquad (14)$$

$$m = 4/3 \qquad (15)$$

and Z is the channel side slope parameter.

Numerical Solution Procedure

The kinematic wave equations can be combined to yield:

$$\frac{\partial A}{\partial t} + mA^{m-1}\frac{\partial A}{\partial x} = q_1 \qquad (16)$$

(Note: Equation 16 could apply to overland flow or channel flow.) This equation has only one dependent variable and can be solved for A in terms of x, t, and q. The model solves equation 16 numerically by replacing the partial derivatives with the appropriate finite difference approximations. The result can be combined with an equation of the form of equation 9 to determine the corresponding discharge, Q.

The numerical solution procedure used in the model has been described elsewhere and will not be discussed in detail here.[10] However, one significant feature of the solution procedure is worth mentioning. To avoid the convergence

Table 1. South Parking Lot #1 Catchment Parameters

Segment	Upstream Segments	Adjacent Segments	Type	NDX	Length (FT)	Slope FT/FT	Manning's Roughness	Other 1	Parameters 2	SCS CN
OF1			5[1]	5	36.	0.0190	0.012	1.0[3]	1.0[5]	98.
OF2			5	3	20.	0.0167	0.012	1.0	1.0	98.
OF3			5	3	25.	0.0190	0.012	1.0	1.0	98.
SW4		OF1 OF2	3[2]	5	165.	0.0148	0.020	113.0[4]	--	--
SW5	SW4	OF1 OF2	3	5	100.	0.0213	0.020	113.0	--	--
SW6	SW5	OF2 OF3	3	3	50	0.0213	0.020	113.0	--	--

1 Overland flow segment
2 Triangular channel
3 Overland flow area parameter
4 Top width of channel at 1 foot depth
5 Percent impervious

Table 2. Weights for Raingage

Segment	1
OF1	1.0
OF2	1.0
OF3	1.0

Table 3. Kinematic Flow Parameters

Segment	ALPHA	M
OF1	17.115	1.67
OF2	16.046	1.67
OF3	17.115	1.67
SW4	1.774	1.33
SW5	2.128	1.33
SW6	2.128	1.33

Table 4. Rainfall Data - South Parking Lot #1

Time (Min)	Intensity[1] (in/hr) at Gage 1
1.	0.60
4.	0.20
5.	1.20
6.	1.80
7.	2.40
8.	1.80
9.	3.00
11.	1.20
12.	0.60
13.	1.20
14.	3.00
15.	3.60
16.	4.20
17.	1.80
18.	1.20
19.	0.60
25	0.20
30.	0.10
Total Volume =	0.53 in

1 Uniform intensity since the time of the previous data entry.

and stability problems that can occur with particular numerical grid spacings (i.e., the relative sizes of Δt and Δx), two numerical solutions procedures are used. The choice of which solution procedure to use is made internally and depends upon the ration of the kinematic wave speed to $\Delta x/\Delta t$. Together the two solution procedures allow the user to obtain stable and convergent solutions for any arbitrarily chosen Δx and Δt.

4. MODEL APPLICATION

Two vastly different applications of the model are presented. The first is a simulation of a very small urban catchment and the second is simulation of a much larger catchment in a mountainous setting. The catchments were chosen to demonstrate the range of catchment types to which the model can be applied.

4.1 South Parking Lot #1

The urban catchment, a parking lot, is one gaged by the Johns Hopkins Storm Drainage Research Project. The gaged runoff data was not used to calibrate the model. Rather, model parameters were directly chosen on the basis of the physical features of the of the area. This small catchment is especially interesting since all of the major flow path components can be realistically represented by the model.

The natural topographic features of the parking appear in figure 3 along with the structural representation of the catchment as used by the model. The model parameters appear in tables 1, 2, 3, and 4. Figure 4 shows the rainfall input, the gaged outlet hydrograph and the simulated outlet hydrograph.

Agreement between the observed and simulated discharge is quite good. The timing of the two hydrograph peaks is correct and the match of the second simulated peak discharge is nearly exact. The only discrepancy is that the initial peak was overestimated. Perhaps small amounts of water were lost through small cracks in the pavement early in the storm or flow resistence properties changed during the storm. due to the cleaning action of the runoff.

4.2 Big Thompson River

The second catchment presented is a segment of the Big Thompson River Basin located on the eastern slopes of the Colorado Rocky Mountains. This catchment was chosen for presentation due to general interest in the particularly savage flash flood that recently occured on the Big Thompson River and due to a special hydrologic investigation that took place during

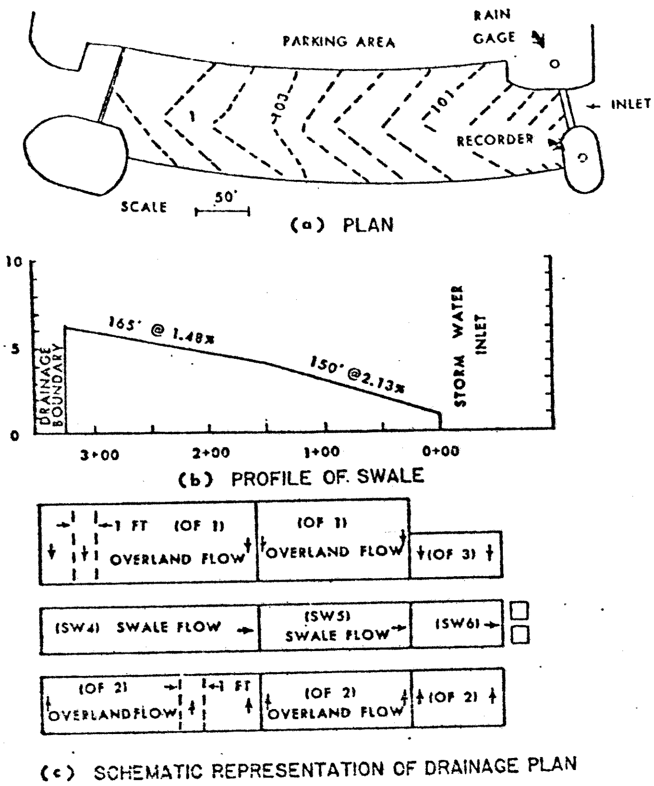

Figure 3. Representation of South Parking Lot #1.

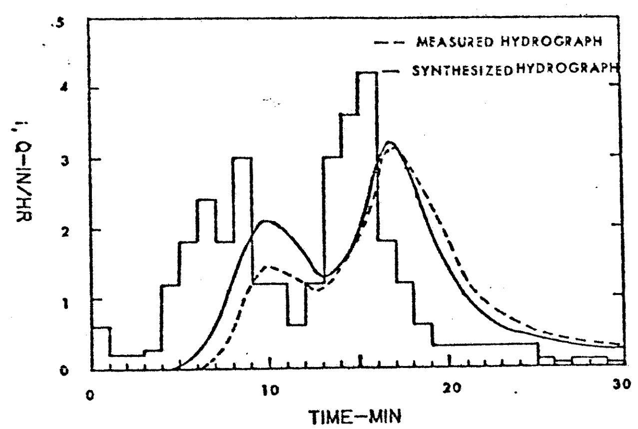

Figure 4. Input-Output Graphs for South Parking Lot #1.

the post-flood hydrometeorological analyses conducted by various public agencies.*

Basin Description

The Big Thompson River traverses a variety of terrain as it begins at the Continental Divide and drains eastward through the Front Range of the Rocky Mountains in North-Central Colorado and into the plains where it meets the South Platte River near LaSalle, Colorado. Of interest in the study is the reach of the Big Thompson below the Olympus Dam and Reservoir (Lake Estes) near Estes Park and above Drake (figure 5). This area experienced extensive damage due to the flooding.

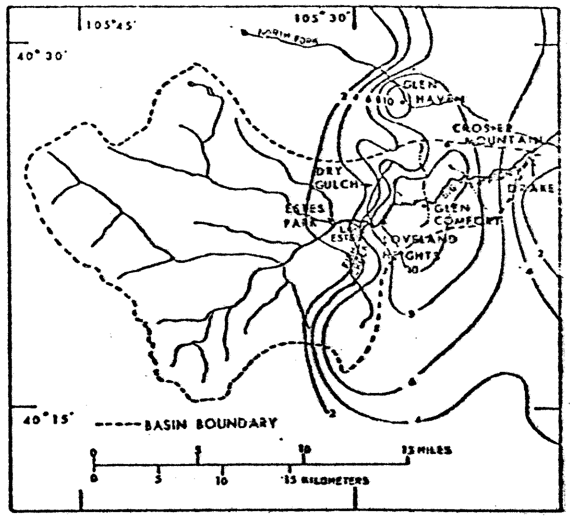

Figure 5. Big Thompson Catchment and Isohytograph for July 31-August 2, 1976, Storm.

The 34 square mile catchment that contributes to the reach between Olympus Dam and Drake is typical of many mountain catchments. Elevations are generally between 6000-9000 feet and the slopes are steep. There are a few areas with heavy stands of timber at the higher elevations. Below about 8000 ft ground cover is sparse with scattered patches of prarie grass and sage brush.

Storm Data

Data availability was by far the biggest problem to overcome in the study. To begin, there were no official rain gages (recording or non recording) in the areas of heavy precipitation. Total storm depths were estimated from the results of a post storm bucket survey (see figure 5). Isohyetal analysis shows that about 10 inches of rain fell over a major portion of the catchment.[6]

Radar and satellite information were of little value in verifying ground estimates of total rainfall. A WSR-57 radar was operating at Limon, Colorado, approximately 100 nautical miles from the flood site. At this range, the WSR-57 radar is near its effective operating limit due to the curvature of the earth and due to the natural spreading of the conical bean. At 100 nmi the center of the 2^{o} conical beam probably

*After an especially disasterous flood event, several public agencies will often cooperate to analyze the hydrometeorological conditions that led to the flood. In the case of the Big Thompson flood as with other similar events, the NWS was a lead agency in surveying the meteorological aspects while a major hydrologic analysis was conducted by the USGS. Due to the lack of data available to verify remote observations, a special study was initiated in the Hydrologic Research Laboratory of the NWS to determine whether the rainfall estimates being generated were realistically in accord with the discharge estimates for the basin. The idea was to use a physically based catchment model to ascertain the link, if any, between the rainfall and runoff estimates.

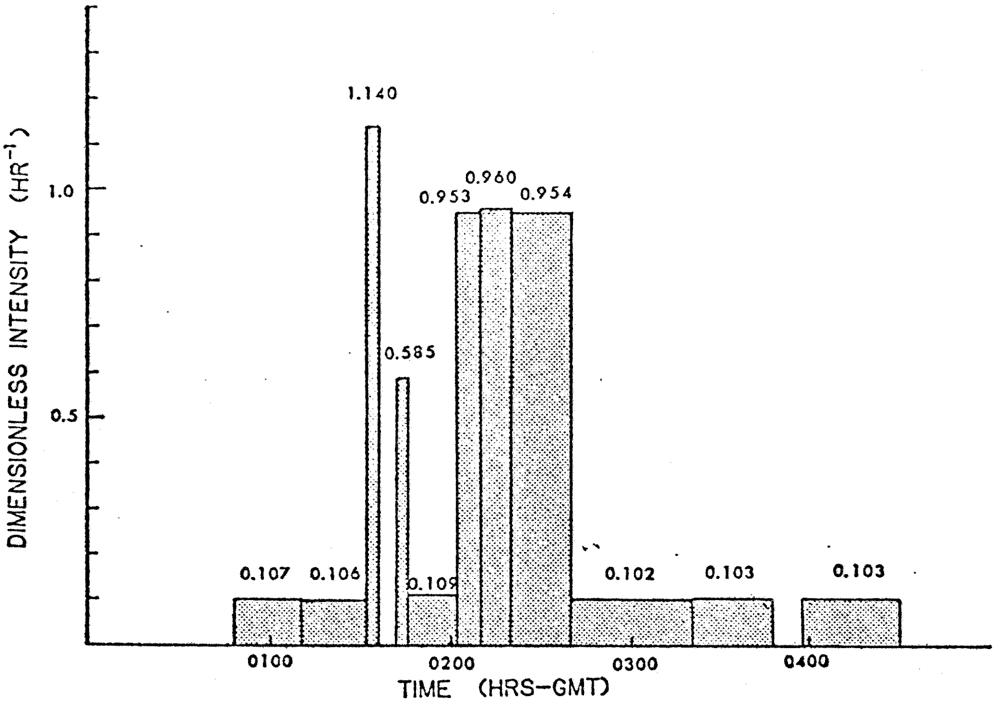

Figure 6. Rainfall Distribution Histograph.

was never much lower than 15,000 ft with a radius of about 10,000 ft. With a beam diameter of well over 3 nmi, much of the small scale activity escaped radar detection. Integration of the radar reflectivity data from the Limon facility yielded only 0.98 in from 0000 to 0600 GMT at Glen Comfort which according to the isohyetal analysis may have received closer to 10 in of rainfall during that period.

Similarly, satellite information did not have the necessary resolution to verify ground measurements. Satellite estimate showed a probable rainfall in a 300 st mi^2 area of the Big Thompson basin of 3.22 inches.

As mentioned above, there were no recording rain gages located in the basin. In any catchment the temporal structure of the rainfall is important. But, in mountain areas with fast response times, the timing of the rainfall is crucial to the simulation of a flood event. The best information available regarding the storm distribution is the radar reflectivity data from the WSR-57 at Limon. Adjustments have been made to the radar data to account for the problems discussed previously and a cummulative mass curve was estimated for Glen Comfort.[6] This curve was normalized and transformed into a storm distribution histogram that appears in figure 6. This function is used to distribute mean areal amounts determined by the isohyetal analysis resulting from the bucket survey.

Storm hydrographs do not exist for the point of interest at Drake. Gaging stations near Drake were either washed away or rendered inoperable by the flood. The only information available results from peak discharge estimates derived from high water marks surveyed after the storm.[1] Estimates of the timing of the flood peak were determined in part from eye witness accounts.[1,4] Thus, it is estimated that a peak discharge of about 30,000 cfs occurred near Drake between 0300 and 0330 GMT. This discharge was generated entirely by the 34 square mile watershed between Drake and Olympus Dam. An additional 155 square miles drains into Lake Estes but this did not figure in the actual flood since there was no outflow from Lake Estes during the storm.

To put this storm in perspective, the average annual rainfall in the area is about 14-16 inches and the average flow for a 304 sq mi area discharging at the mouth of the Big Thompson Canyon is less than 200 cfs for this time of year. The previous flood of record at the mouth of the canyon was 7,600 cfs. (July 19, 1945)

Catchment Representation

The natural catchment was conceptulized as the series of overland flow planes and stream segments shown in figure 7. Three stream segments were chosen to reflect the variation of slope and cross-sectional characteristics of the main stem. Two additional stream segments were included to represent major point inflows to the main channel. Two overland flow planes with identical properties are included for lateral input to each stream segment.

All of the model parameters were estimated from the physical characteristics of the basin. Most of the information was obtained from a topographic map of the area. In this study, the exact length of the main stem was obtained from a post storm survey. However, this too could have been estimated from a good topographic map. Model parameters appear in tables 5, 6, and 7.

Table 5. Big Thompson Catchment Parameters

Segment	Upstream Segments	Adjacent Segments	Type	NDX	Length (FT)	Slope (FT/FT)	Manning's Roughness	Other 1	Parameters 2	SCS CN
OL1			5[1]	4	1000.	0.1500	0.60	9.92[3]	0.0[5]	75.
OL2			5	4	800.	0.2500	0.60	9.92	0.0	75
OL3			5	4	1000.	0.2000	0.60	8.02	0.0	75
OL4			5	4	800	0.3000	0.40	8.02	0.0	80
OL5			5	4	800.	0.2750	0.40	8.02	0.0	80
S1		OL1	3[2]	60	39450.	0.0127	0.07	50.004	--	--
S2		OL2	3	40	20000.	0.0750	0.07	30.00	--	--
S3	S1 S2	OL3	3	40	17840.	9.0120	0.08	40.00	--	--
S4		OL4	3	40	20000.	0.1000	0.07	30.00	--	--
S5	S3 S4	OL5	3	40	19880.	0.0339	0.08	20.00	--	--

1 – Overland flow segment
2 – Triangular channel
3 – Overland flow area parameter
4 – Top width of channel at 1 foot depth
5 – Percent impervious

Table 6. Weights or Rain Gage – Big Thompson

Segment	1	2
OL1	1.0	0.0
OL2	1.0	0.0
OL3	0.0	1.0
OL4	0.0	1.0
OL5	0.0	1.0

Table 7. Kinematic Flow Parameters – Big Thompson

Segment	ALPHA	M
OL1	0.208	1.67
OL2	0.269	1.67
OL3	0.277	1.67
OL4	0.509	1.67
OL5	0.488	1.67
S1	0.616	1.33
S2	1.775	1.33
S3	0.565	1.33
S4	2.050	1.33
S5	1.196	1.33

The model applies a spatially averaged rainfall to each overland flow segment. The actual rainfall for a segment is determined by a weighted average of up to five "rain gages" referenced to that segment. Two "rain gages" time series were developed from the isohyetal map and storm distribution histogram. The first "rain gage" time series was derived by finding a mean storm depth of 9.15 in over approximately the upper 60% of the basin and distributing the volume according to the distribution function determined for Glen Comfort. The same procedure was followed to get the second time series for the remaining 40% of the basin but with a mean storm depth of 7.31 in. (Table 8)

It is understood that the storm depths indicated in figure 5 are for a period covering more than two days. However, most of the rain fell during the six-hour period 0000 to 0600 GMT. It is not known exactly how much rain fell after 0600 GMT but in some locations it may have been as much as 1.0 to 2.0 inches.

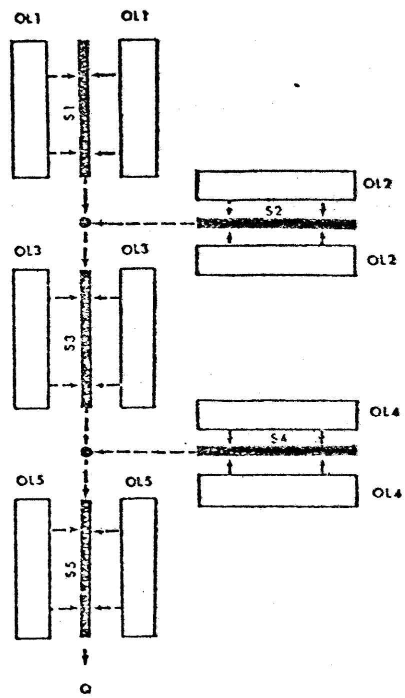

Figure 7. Big Thompson Representation and Flow Segment Identification.

Using the full storm volume indicated in figure 5 is likely to produce results on the high side. However, for lack of a definitive technique to determine the appropriate reduction, the full storm volume will do, at least for the first run. Later, the rainfall depths can be easily scaled to check other possible depths such as 80 or 90% of the values derived from figure 5.

Simulation Results

Simulated hydrographs for Drake are shown in figure 8. Storm depths equal 100%, 90% and 80% of that shown in figure 5 where used to generate the indicated hydrographs. In all cases, the distribution function of figure 6 was used.

Table 8. Rainfall Data - Big Thompson

Time (Min)	Intensity (IN/HR)[1] at Gage 1	2
41.	0.00	0.00
70.	0.98	0.78
92.	0.97	0.77
96.	10.43	8.33
102.	0.00	0.00
106.	5.35	4.28
122.	1.00	0.80
130.	8.72	6.96
140.	8.78	7.02
150.	8.73	6.97
160.	8.73	6.97
180.	0.93	0.75
200.	0.93	0.75
220.	0.94	0.75
238.	0.00	0.00
270.	0.94	0.75
	9.15 in	7.31 in

1 - Uniform intensity since the time of the previous data entry

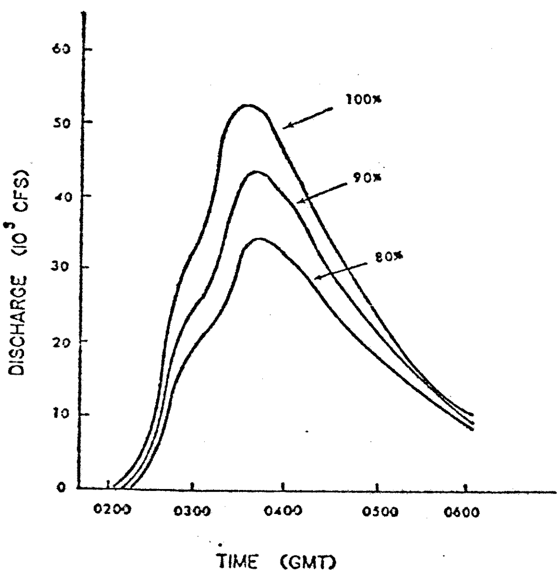

Figure 8. Simulated Hydrographs at Drake for Selected Percentages of Total Rainfall Depth for July 31-August 2, 1976, Storm.

As expected, the hydrograph peak resulting from the 100% rainfall level was high (53,000 cfs as compared to an estimated 30,000 cfs peak at Drake). The hydrograph peaks resulting from the 90% and 80% rainfall levels (44000 cfs and 35000 cfs) were successively in better agreement with the estimated flows.

The observed peak flows were estimated to have occurred between 0300 and 0330 GMT at Drake. The simulated peaks occurred between 0330 and 0340 GMT. Such agreement seems highly satisfactory given that the same temporal structure of the storm was assumed at each point in the catchment and the spatial structure was varied in as much as the creation of two mean areal precipitation areas allowed.

The exact temporal and spatial structure of the storm also has a direct bearing on the peak discharges. In fact, for this storm the storm distribution is probably more important than discriminating whether eight or ten inches fell during the six-hour period of interest. Eyewitness accounts supported by some scattered data show a highly variable storm in time and space. From eyewitness accounts, it was evident that the peaks from individual gullies, washes, and tributaries to the Big Thompson were highly non-synchronous. It has been established, for instance, that the estimated peak discharge from Dry Gulch, a tributary entering the main stem just below Olympus Dam, occurred at 0430 GMT[1] with a smaller peak as late as 1030 GMT. The use of areal averages in space and time for simulation purposes will tend to coordinate the flows and give higher peaks than those estimated at Denver.

Given the problems with identifying the required rainfall and discharge data for this storm, the model has produced hydrographs that seem very reasonable in terms of the magnitude and timing of the discharge peak. In addition other factors were present that limit efforts to further refine the results. These matters include but are not limited to: 1) Tremendous channel degradation took place during the flood followed by subsequent deposition as the flood subsided, 2) There were some estimates that up to 40% of the flow volume was debris, and 3) Peak discharges were computed by the slope-area method with a probable wide margin of confidence. One point to remember when considering differences in discharge measurements, however, is that the important variable, stage, is generally a function of discharge to a power less than one. Thus, discharge differences will be damped when converted to stage. The wider the flood plain at the point of interest the more pronounced the effect will be.

The Big Thompson flood was four times as great as the previous record (7600 cfs, 1946) and records have been kept there since 1887. Even though the magnitude of the flood of 1976 was far beyond what many people felt imaginable, it is far below what may be theoretically possible. The probable maximum precipitation for a 34 square mile watershed in the Big Thompson area is over 20 inches in six hours. Using a storm depth of 20 inches and distributing it according to figure 5, a peak discharge of over 200,000 cfs was computed at Drake. The peak is likely to be higher if any outflow (or failure due to the great storm depth) from Lake Estes. Although it may not be pertinent to design all structures to such a rare event, it is certainly not appropriate to totally ignore its potential either.

5. CONCLUSIONS

The point of these procedures may not be to produce the most accurate indications of the eventual hydrograph. Flash flood procedures must supply enough timely information to decision makers so that damage prevention activities may be carried out.

Decisions generally serve to allocate resources in quantum amounts. For instance, when a disaster strikes, governors don't continuously call up individual National Guardsmen as the situation worsens. In practice, guardsmen are called in discrete units of a hundred or a thousand at a time. The decision to allocate these resources in such amounts is based on the expected achievement of a certain level or threshold of danger. In the final analysis then, it may be less important to accurately define the magnitude and timing of the flood peak than to quickly identify the fact that critical danger thresholds will be surpassed.

6. REFERENCES

1. Grozier, Richard U.; Larry F. Lang et al, 1976, The Big Thompson River Flood of July 31 - August 1, 1976, Larimar County, Colorado, prepared by the U.S. Geological Survey and the Colorado Water Conservation Board.

2. Gruntfest, Eve C.; Thomas E. Downing; Gilbert F. White, 1978, Big Thompson Flood Exposes Need for Better Reaction System to Save Lives, Civil Engineering, Vol. 48, No. 2, pp. 72-73.

3. Harley, B. M.; F. E. Perkins; P. S. Eagleson, 1970, A Modular Distributed Model of Catchment Dynamics, MIT Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Report 133.

4. Henz, John F.; Vincent R. Scheetz; Donald O. Doehring, The Big Thompson Flood of 1976 in Colorado, Weatherwise, pp. 278-285.

5. Lighthill, M. J.; G. B. Whitham, 1955, On Kinematic Waves- -I. Flood Movement in Long Rivers, Proc. Ry. Soc. London, A., Vol. 229, pp. 281-316.

6. Maddox, Robert A, et al, 1977, Meteorological Aspects of the Big Thompson Flash Flood of 31 July 1976, NOAA Technical Report ERL 388-APCL 41.

7. National Oceanic and Atmospheric Administration, 1976, Big Thompson Canyon Flash Flood of July 31-August 1, 1976, Natural Disaster Survey Report 76-1.

8. Schaake, John C., Jr., 1971, Deterministic Urban Runoff Model, Chapter VIC, Treatise on Urban Water Systems, Ed. by Albertson, M. L., et al., Colorado State University.

9. U. S. Weather Bureau, 1960, Generalized Estimate of Probable Maximum Precipitation Over the United States West of the 105th Meridian for Areas to 400 Square Miles and Durations to 24 Hours, Tech. Paper 38.

10. Wilson, Charles; Igancio Rodriguez-Iturbe, 1975, Joint Usage of Rainfall Ronoff Models and Rainfall Generation Models, MIT Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics.

11. Wooding, Robin A., A Hydrologic Model for the Catchment-Stream Problem,

    a. I. Kinematic Wave Theory, 1965, Journal of Hydrology, Vol. 3, Nos. 3/4, pp. 254-267.

    b. II. Numerical Solutions, 1965, Journal of Hydrology, Vol. 3, Nos. 3/4 pp. 268-282.

    c. III. Comparison with Runoff Observations, 1966, Journal of Hydrology, Vol. 4, pp. 21-37.

12. Woolhiser, D. A.; J. A. Liggett, 1967, Unsteady, Flow over a Plane - the Rising Hydrograph, Water Resources Research, Vol. 3, No. 3, pp. 753-771.