韓國水資源學會誌 論文
第28卷 第4號·1995年 8月
pp.171～183

Cheat강 유역 홍수분석 및 조절을 위한 Snyder의 단위유량도법 적용

Applications of Snyder's Unit Hydrograph to the Cheat River Basin for Flood Control Analysis

전 무 갑* · Eli, R. N.**
Chun, Moo-Kab • Eli, R. N.

Abstract

The Snyder's Unit Hydrograph Method is applied to simulate the November 1985 Flood of the Cheat River Basin, which is located in the North-East region of West Virginia in United States. The entire basin is divided into many subareas according to the hydrologic and geologic characteristics. The overland flows are computed on each subarea and combined together along the streams. The flows are also routed by the Normal Depth Storage and Outflow Method in Modified Pulse option. The several structural flood control alternatives are examined. The study shows the OPTION Ⅲ which has the three moderately sized dam is ultimately suitable to control the flood. The HEC-1 computer model is used to analyze the flood.

요 지

본 연구에서는 미국 West Virginia주 동북부에 위치한 Cheat강 유역 일원에 1985년 11월에 발생한 대홍수를 Snyder의 단위유량도법을 적용, 재현시켰다. 전체유역을 수문 및 지형특성에 따라 각 소유역으로 나누어 각 소유역에 대해 지표면유출을 계산하였다. 유역 하천에 대해서는Modified Pulse Option에 있어서 평균수심저류 및 유출법으로 하도추적을 실시하였다. 또한 Cheat River 유역전체에 대한 홍수조절 계획이 본 연구에서 수립되었으며 Option Ⅲ에 명시된 3개 중규모 다목적댐 건설이 가장 적절한 것으로 고찰되었다. 홍수분석 및 계획수립에는 HEC-1 전산모형이 이용되었다.

* 농어촌진흥공사 농어촌연구원 책임연구원
** Associate Prof., Dept. of Civil Engr., West Virginia Univ., Morgantown, WV, USA.

1. Introduction

The subject engineering study evolved from the aftermath of the November 4–5, 1985 catastrophic flood on the Cheat subbasin of the Monongahela River Basin as shown in Fig. 1, hereafter referred to as the November flood.

Motivated by the November flood, the study is to formulate hydrologic model for the establishment of flood control scheme.

Since the upper half of the basin is more mountainous than lower half and has long narrow watershed as shown in Fig. 1, the Snyder's Synthetic Method of Unit Hydrograph (Viessman et al., 1977) was applied to determine hydrographs of the basin above Parsons, while the SCS Method of Unit Hydrograph was used for the area downstream of Parsons. Also, the methods were tested by computing the hydrographs at Davis on Blackwater River, Parsons on Shavers Fork and Rockville Big Sandy Creek and by comparing these to corresponding observed hydrographs.

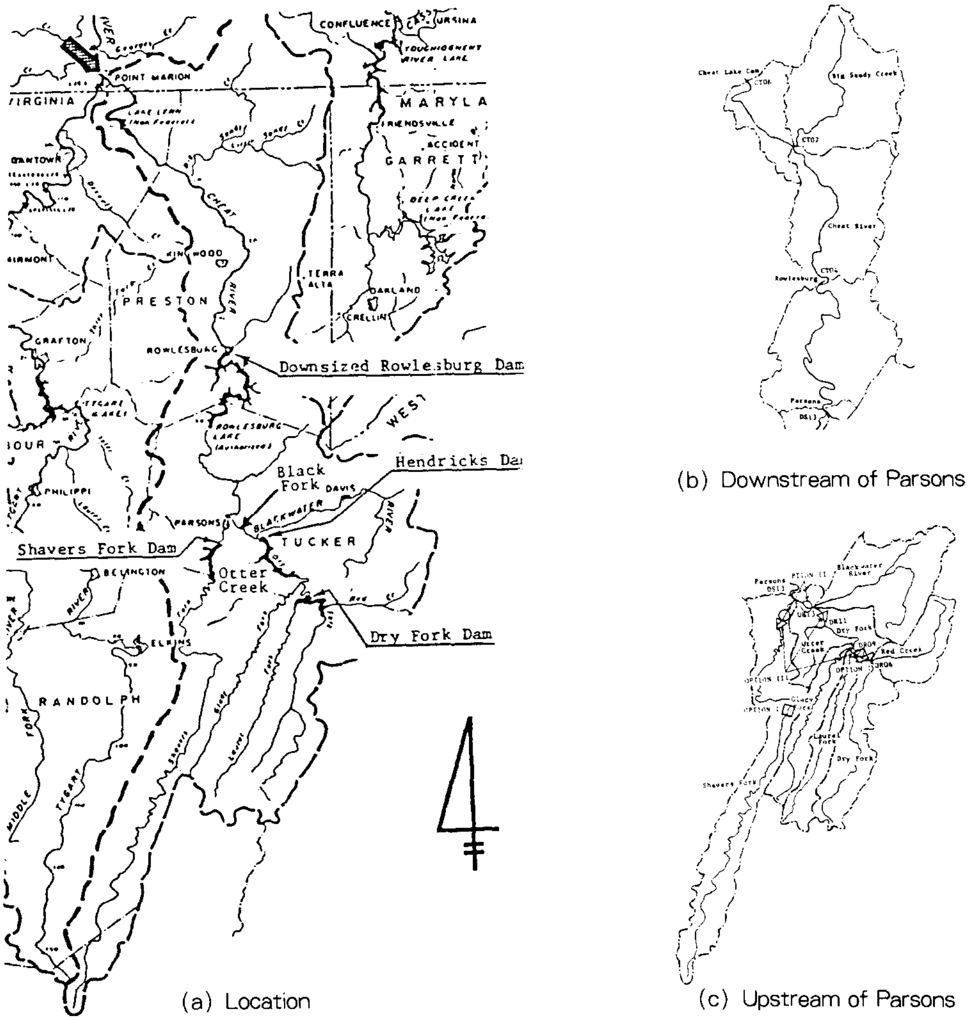


Fig. 1. Location Map and Subbasins (Dept. of Natural Resources, 1973)

The Snyder's Unit Hydrograph, intially, is based on the method developed by Snyder (1938) and expanded by Taylor and Schwartz (1952). Dalrymple (1965) relates the Snyder method to the flood peak and precipitation. U.S. Army Corps. of Engr. (1959) compares the SCS and Snyder's method. There are also a number of versions of the SCS model described in publications (Chow and Kulandaiswamy, 1982; Richard, 1984; Robert, 1980; SCS, U.S. Dept. of Agriculture, 1974).

According to the study, the installation of three moderately sized dam of option Ⅲ is selected as final plan.

2. Basin Descriptions

Consisting of 7 subbasins, the total area of the Cheat Basin is 3,630.2 km^2; 1,848.1 km^2 above Parsons and 1,782.1 km^2 below Parsons. The main stem is 193.1 km in length from Harper Knob, Randolph County to the Cheat Lake Dam site, Monongalia County; 62.0 km on Dry Fork, 6.0 km on Black Fork, and 125.2 km on Cheat River, respectively. The flow charts of the subareas and their connecting streams of the Cheat basin are included in Fig. 2. The slopes of subbasins and streambed are steeper upstream than downstream of Parsons. The subbasin areas and stream lengths of principal tributaries are shown in Table 1.

The pattern of stream drainage above Parsons is substantially different from what it is below. The upper half of the basin consists of long narrow subwatersheds that drain the areas between the parallel ridge lines that dominate the topography above Parsons. Shavers Fork, Glady Fork, Laurel Fork and Dry Fork can be characterized in this manner. The slopes are very steep, the soils are shallow and the drainage path to the main tributary channel is short. Red Creek is an exception in terms of shape, but identical in other respects. The Blackwater

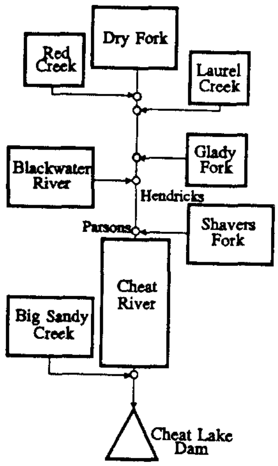


Fig. 2. Schematic Diagram of the Basin

Table 1. Subbasin Areas and Lengths

Tributaries	Areas (km^2)	Lengths (km)	Average Slopes(%)
Red Creek	159.0	27.7	1.8
Laurel Fork	155.7	54.0	0.9
Glady Fork	164.2	60.5	0.7
Otter Creek	75.1	20.9	–
Blackwater River	360.3	44.1	1.4
Shavers Fork	552.4	125.7	0.5
Big Sandy Creek	535.8	50.5	–
Main Stem & Othrs	1,627.7	193.1	–

River is the lone exception from this overall description. It drains a high altitude plateau that is dominated by the Canaan Valley, which contains unique hydrologic features in a setting of shallow sloping terrain, with significant area in marsh land.

3. Modeling of the River Basin

The method that can predict the effectiveness of a given structure design is to conduct a modeling study to test it against historical or hypothetical floods of a sufficient severity to meet the design requirements. In the study, the U.S. Army Corps of Engineers HEC-1 watershed hydrology model (HEC-1, 1985) is chosen to meet the modeling needs.

HEC-1 is used to simulate the November flood event on the Cheat Basin. The HEC-1 model enabled river discharge versus time to be simulated for the November flood at many points along the tributaries and main channel of the Cheat River where there are insufficient historical data available to decrease the flood to the extent required for a design study. This was accomplished by subdividing the entire drainage basin into several small subareas. The rainfall and infiltration were assumed to be uniform over subareas. The resulting rainfall excesses were then routed by the option to the outlet of the subbasin producing a runoff hydrograph.

3.1 Subarea and Stream Network Development

The first important step in fitting the HEC-1 computer model to the Cheat River Basin is to subdivide the entire basin into relatively small homogeneous areas, and to select these subareas and stream segments so that they are of sufficient resolution accurately to reproduce the detailed variation of the rainfall-runoff process. The rainfall and other necessary data are determined for each subareas. The model is capable of computing a separate discharge versus time record for each of these subareas. It then accumulated these flows as it proceeded downstream.

The Normal Depth Storage and Outflow Method in Modified Pulse option (Viessman et al., 1977; HEC-1, 1985) was used for flood routing. In the option, the outflow rate in storage is computed as follows:

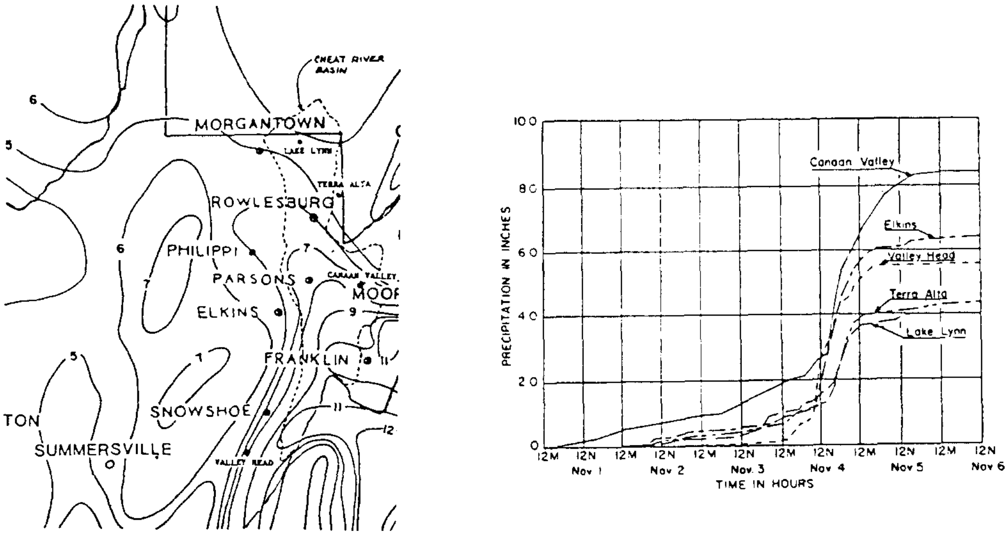
$$O=CYH^{x} \tag{1}$$

where O=outflow rate; Y=length of the spillway or cross-section; H=depth above spillway or cross-section; C=discharge coefficient; and x=exponent.

The input variables for this option are the hydraulic and geometric data of each section of the routing stream, Manning's n values, reach length, slopes of energy grade line, and the coordinates for an 8 point cross section which represents the shapes of the routing reach. Storage is cross sectional area times reach length. The outflows are computed for normal depth using Manning equation and then are added to the flow of the subarea downstream.

3.2 Storm Design Used in the Model

The Nove[illegible] with a magnitude exceeding the 500-year flood is selected for use as the design flood because of its extreme rarity. Actual frequency calculations based on the available historical record yield return periods values much greater than 500 years at many points along Cheat River. The decision is made not to use these values because of the potential large errors resulting from any computed frequency values exceeding the 500-year flood.



1 inch = 2.54 cm

(a) Isohyetograph (b) Precipitation Profile

Fig. 3. Accumulated Rainfall for the Period November 1-6, 1985 (SCS, Agricultural Dept., 1987)

Table 2. Rainfall Accumulations of Recording Stations

STATIONS	ACCUMULATIONS(mm)	PERIODS
Elkins	165	Nov. 1-6, 1985
Valley Head	144	same
Lake Lynn	102	same
Terra Alta	111	same
Canaan Valley	213	same

Therefore, the decision is made to label this flood simply as one "exceeding the 500-year flood". The discharge peak at the Rowlesburg Dam site exceeds the original Standard Project Flood (SPF) used in the design of the dam. This is another good indication that the November flood is a sufficiently severe test of any proposed structural flood control design.

Having made the decision to utilization the November flood as SPF for this flood control feasibility study, it is useful to examine the precipitation which lead to the flood. As shown in Fig. 3, the accumulated rainfall depth for the total storm event is highest in the ridge and valley mountain topography of the upper watershed. The storm is obviously influenced by the elevation of the ridges in the upper reaches of the drainage basin. Although the lines of equal rainfall depth indicate maximum at the 28 cm depth level, it is known that local depths exceeded 35 cm at several points along the ridges on both sides of the West Virginia border. Overall, the depth decreases in the northwest direction, receding to approximately the 10 cm level near the outlet of the watershed at Lake Lynn. This pattern is consistent with many storms that result from moist air from the south meeting cooler dry air from the north. This latter collision is sufficient within itself to produce large depths of rainfall over a short time period. The ridge lines intensify the effect by acting as a lifting mechanism, forcing the moist air to higher levels, resulting in a greater rate of condensation, and hence havier rainfall. Historical rainfall data, averaged over time, reflects the same general distributions shown in Fig. 3. Therefore, it appears that the rainfall accumulation pattern resulting in the November flood is consistent with storms that have occurred in the past, and might again occur in the future.

The rainfall records, during the flood period, of 4 stations located in Elkins, Valley Head of Randolph Co., Terra Alta of Preston Co., Lake Lynn of Monongalia Co., respectively, are used to provide rainfall distribution data from which the storm's total precipitation for each subareas could be interpolated. Table 2 shows the accumulated precipitation. It should be noted that the rainfall distribution for Canaan Valley in Fig. 3 is simulated on the basis of discharge records at Davis on the Blackwater River, and the Canaan Valley total precipitation accumulation. The Canaan Valley is too far from other available recording station to apply their distribution records.

3.3 Applied Unit Hydrograph

For the Snyder's Method (Snyder, 1938; Taylor and Schwartz., 1952), the two main parameters, Tp and Cp, are input to the model. By assuming the coefficient, C_t, the basin lag, Tp, in hours can be expressed by

$$T_p = 0.75C_t(LLc)^{0.3} \tag{2}$$

where L=length of main stream from outlet to divide (km); L_c=distance from outlet to a point on the stream nearest the centoid of the basin (km); and C_t=coefficient; function of basin slope (lower value for steeper slope). C_p is the coefficient which influences the unit-hydrograph peak, q_p, given by

$$q_p(cms) = 2.75C_pA/T_p \tag{3}$$

where A=drainage area (km^2), and C_p= corfficient; function of basin slope (higher values for steeper slope).

The empirical coefficients, C_t, and C_p are calibrated on the Blackwater River and Shavers Fork, respectively, and applied to other subbasins according to their similarity and steepness. The discharge computed by applications of these assumed coefficients is also com-

Table 3. The Parameters for the Snyder's Method in the Model

Subareas	C_t	L(km)	L_c(km)	T_p(hrs)	C_p	A(km^2)
Red Creek	2.21	10.98	5.25	8.90	0.82	159.0
Laural Fork	2.01	14.91	6.89	9.82	0.82	156.0
Glady Fork	1.84	12.70	6.63	5.18	0.83	164.0
Otter Creek	2.00	12.50	5.53	5.30	0.83	75.0
Blackkw R.	2.66	10.81	5.24	6.74	0.70	360.0
Shavers Fork	1.96	12.80	6.20	5.28	0.83	552.0
Dry Fork	1.88	7.74	3.54	3.74	0.83	381.0

pared with observed flow rate at the mouth of Cheat River. The parameters applied to the model for Snyder's Method are as shown in Table 3.

For SCS Unit Hydrograph Method, the lag times, T_L, and SCS curve numbers are input to the model for the computation of peak times, Tp, and peak discharges expressed by

$$T_p(\text{hrs})=0.5\Delta t + T_L \quad (4)$$

$$Q_p=2.08A/T_p \quad (5)$$

where $T_L=0.6T_c$; T_c=time of concentration; Δt=the duration of effective rainfall; and A= drainage area (km^2).

The lag times and curve numbers applied to the subareas are shown in Table 4. The SCS curve numbers were calibrated in the Big Sandy Creek on the basis of rainfall and observed discharge and applied to other subareas according to their cover condition.

Table 4. Lag Times and SCS Curve Nos.

Subareas	T_L(hrs)	CN's
Cheat4	3.0	74
Cheat7	2.1	74
Cheat8	3.8	74
Big Sandy	4.5	73

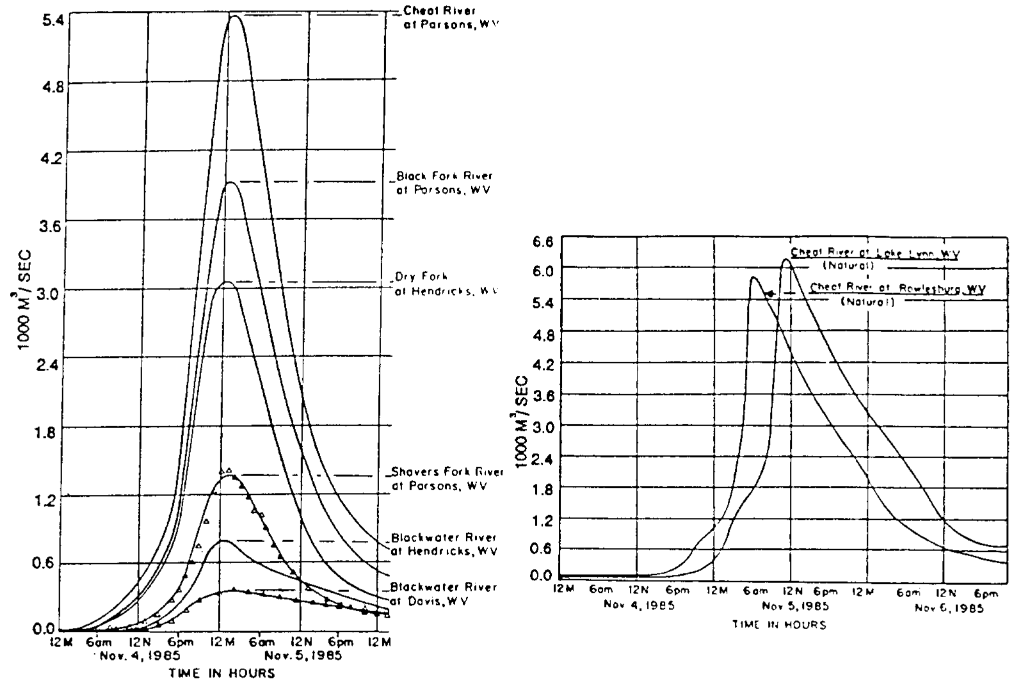
The antecedent moisture condition for each basin was assumed to correspond to full saturation because of the rain which fell for 48-72 hours before the flood. This phenomenon temporarily reduced the basin's ability to absorb the rainfall; thus higher values of runoff and peak discharge occurred above Parsons. The exponential loss function option of HEC-1 model was applied to estimate rainfall losses, and the parameters for the option were obtained by employing the optimization option using observed runoff data at Davis on the Blackwater River and rainfall distribution at Canaan Valley. These optimized parameters were then applied to other subareas.

3.4 Stream Flow Analysis

The HEC-1 model requires a large number of parameters to describe the watershed so that it can accurately simulate the discharge along the tributaries and main channel. Where possible these parameters are fine-tuned to match existing records of the November flood. For example, flood records for the Shavers Fork and the Blackwater River are matched with computed hydrographs as explained in previous section. Table 5 illustrates the excellent agreement between the model and the actual flood records on these basins. The Δ's in Fig. 4 mark the observed hydrograph on Blackwater River and Shavers Fork.

Table 5. The Comparison of the Computed and Observed Hydrograph

GAGE	CLASS	SUMS (m^3)	MEANS (m^3)	PEAKS (m^3)	PEAK TIME(HRS)
Blckwater River	Cmptd	8,776	170	340	26 : 00
	Obsvd	8,838	177	354	26 : 00
Shavers Fork	Cmptd	21,289	424	1,302	25 : 00
	Obsvd	20,305	406	1,354	25 : 00
Big Sandy Creek	Cmptd	4,841	85	198	26 : 00
	Obsvd	4,568	91	202	26 : 00



(a) Tributaries above Parsons (b) At the Lower Half of the Cheat Basin

Fig. 4. Natural Streamflows of Tributaries above Parsons

Table 6. The Characteristics of Hydrograph on Tributaries and Main Channel

LOCATIONS	PEAK DISHS.(m^3)	PEAK TIMES(HRS.)	BASIN AREAS(km^2)	REMARKS
Blkw. R.	764	23 : 00	360	
Dry F.	2,887	24 : 00	898	Excluding Blk F. R.
Blk F. R.	3,708	24 : 00	1,296	
Shvr. F.	1,302	25 : 00	552	
Cheat River at Parsons	5,000	25 : 00	1,848	Including Shavers F.
Cheat R. at Rowlesburg	5,548	31 : 00	2,422	
Big Sandy Cr. Rckvl.	198	26 : 00	536	
Cheat R. at Lake Lynn	255	35 : 00	3,654	

All indications are that the simulated flood is an accurate representation of the actual flood event, both in terms of discharge record and total volume of runoff.

The computed discharge peaks of the tributaries are shown in Table 6. The flow of Blackwater River, Dry Fork, and Shavers Fork at Parsons, resulted in a total peak discharge of 5,000 m³.

The town of Parsons lies precisely at a natural convergence point of all of the drainage in the upper half of the Cheat Basin. All of the stream and rivers converge at this point, as evidenced by the narrow neck in the overall Cheat drainage basin shape. This natural "funnel" produces a great flooding potential for the town of Parsons and points downstream. This natural convergence of the drainage in the upper Cheat basin is further enhanced by an apparent equal basin lag time from each of the tributaries discussed above to the convergence point at Parsons. An equal basin lag time for all of the tributaries to the Cheat at Parsons simply means that the individual flood peaks on each of the tributaries tend to arrive at approximately the same time, and therefore are directly additive at Parsons as shown in Fig. 4.

The combined discharge at Parsons is then passed down the Cheat River while accumulating the remaining tributary runoff as it was encountered an routed to the mouth of the Cheat at Point Marion, PA.

The lower half of the Cheat Basin will mostly often receive less rainfall than the upper half because of its lower elevation. Although slopes are steep and soils are poor, their influence is less sever much of the drainage area above Parsons. The peak discharge at Parsons is increased by 18% as it traveled to the mouth of the Cheat River below Lake Lynn, as shown in Fig. 4. Therefore in the case of the November flood, 82% of the peak discharge experienced at the mouth of the Cheat River occurred at Parsons, to which 50% of the total drainage area contributes.

The conclusions of this portion of the study, supported by the above observations, are that the hydrologic response of the Cheat River Basin to storm rainfall input is strongly dictated by its unusual basin shape and drainage characteristics, and that the severity of this response is strongly biased toward the upper half of the Cheat Basin. The storm rainfall pattern producing the November flood is considered to be representative of the most likely pattern for severe storm events that will in turn produce the worst case floods along the main stem of the Cheat River. These are additional reasons for selecting the November flood as the design event.

4. Examinations of Structural Flood Control Alternatives

In this study, the criterion to control the flood was assumed to be the 10-year frequency flow. These levels are 812 cms on Black Fork and 410 cms on Shavers Fork (U.S. Dept. of Housing and Urban Develop., 1978). These flows are safe to the extent that they minimize possible damage to all populated areas by keeping the water surface elevations to levels at/or slightly above bankfull (U.S. Army Engineer Dist. Pitts., 1978).

River channel improvements such as realignments and enlargements are quickly rejected from consideration as a primary means of flood control. The November flood, chosen as the design flood, simply is far in excess of any much smaller flood level that could reasonably be handled by these means.

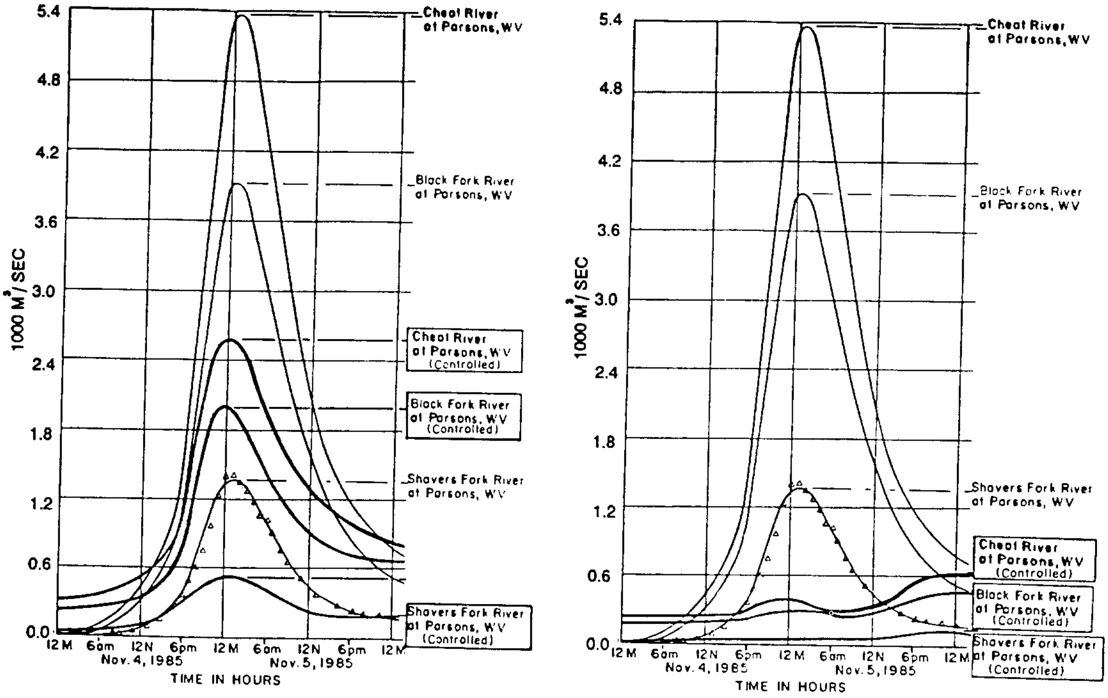
The final three alternatives as shown in Fig. 1 involve the use of various sizes and numbers of dams. First, to provide the level of protection deemed necessary, most of the drainage area above Parsons has to be effectively controlled. By use of the HEC-1 model, it is quickly determined that uplands SCS dams would provide insufficient control of drainage area. At least 50 % or more of the area must be controlled in order for there to be even a chance of success. Second, the terrain and streambeds are so steep as to require dam size that quickly rose above those limits normally associated with SCS dams. This latter situation resulted when an effort was made to move dam into downstream locations where they could jointly control more drainage area. The need for increased dam sizes and a more downstream location precluded involvement of the SCS in providing the major component of flood protection.

Each of the major tributaries of the Cheat above Parsons is examined for feasible dam sites along their lower reaches, where at least 50% of the drainage area would be controlled. Red Creek, Laurel Fork, and Glady Fork had considerable discharge magnitude and subarea. However it is impossible to find a dam site on Red Creek because the slope is steep with population areas around. Detailed examination of the topographies and discharges of Laurel Fork and Glady Fork also causes rejection of the possible dam location on these streams, because both streams have valleys that are too narrow and slopes that are too steep. This problem resulted in too little reservoir storage in comparison to dam size. No suitable dam sites could be located on the Blackwater River either since the channel slopes are very steep downstream of Davis as on Laurel Fork and Glad Fork. The Blackwater's hydrologic response to rainfall input is more sluggish in comparison to the other tributaries. In spite of the very heavy rainfall accumulation during the November flood, the Blackwater peak discharge is only 764 cms, as indicated in Table 6.

It is realized that full control of Dry Fork and Shavers Fork might be sufficient for the flow of the Blackwater to pass through Parsons and into the Cheat without significant damage to any populated areas. The watershed model late showed that this could be accomplished. With Blackwater to remain uncontrolled, it is all important to control the remaining tributaries to the fullest extent possible. This was relatively easy to accomplish on Shavers Fork, while Dry Fork proved to be very difficult. Locating a single moderately sized dam on the lower Dry Fork proved to be impossible because of the excessive steepness of the river channel. These latter limitations do not permit sufficient reservoir storage to contain the flood without an excessively large dam. The ultimate solution is to locate two dams of moderate size in series, one below the other such that they jointly created enough storage to contain the November flood fully. The most downstream dam is located just upstream of the mouth of the Blackwater River, above the town of Hendricks. The upstream dam is located within a narrow bend in the river at a point just upstream of the mouth of the Glady Fork tributaries. Both of these latter dams are able to work together to contain the November flood on Dry Fork fully. Three main options, which are examined by the HEC-1 computer model to get optimum results above Parsons, are introduced below.

(1) OPTION I

This plan involved controlling the design flood by siting two medium sized dams; one on Dry Fork and other on Shavers Fork. The Dry Fork Dam is sited 1.12 km below the confluence of Dry Fork and Laurel Fork where the streams have a broad valley with a mild slope upstream. The catchment area of the dam is 281 km^2 and



(a) Option Ⅰ

(b) Option Ⅱ

Fig. 5. Natural versus Controlled Flows in Option I and Ⅱ

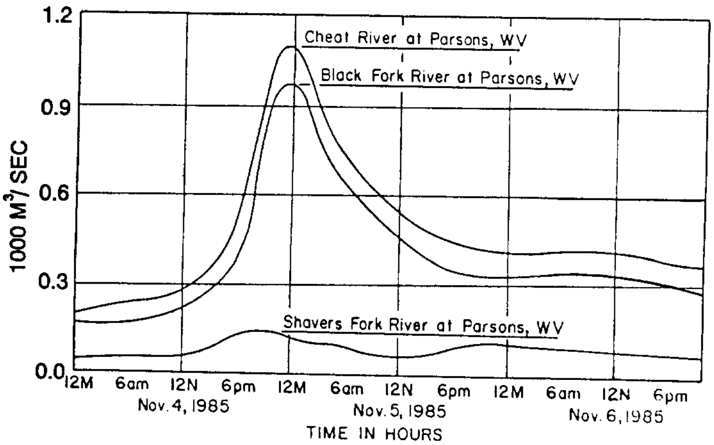


Fig. 6. Controlled Flows in Option Ⅲ

natural peak flow at this site is 1,952 cms. However, the remaining controlled peak flow at Parsons is still high at 1,898 cms, as shown in Fig. 5.

On Shavers Fork, a dam was located near Weese to protect population area downstream, such as Boden and Faulkner. The selected dam site has enough reservoir storage to impound most of the flow from upstream. The catchment area is 353 km^2. The controlled peak flow on Shaves Fork at Parsons is 506 cms as compared to the natural peak 1,303 cms. Thus the total controlled discharge on Cheat River at Parsons is 2,405 cms. The reduction ratio is 52%, which is short of the ultimate goal of the study, even though other secondary protection measures could be taken. So, it is assumed that a more intensive mitigation plan is necessary to control the November flood.

(2) OPTION Ⅱ

To minimize the flow rate at Parsons, the dam sites are selected at the lowest point possible on both streams, Black Fork and Shavers Fork. A dam sited below Hambleton on the Black Fork River controlled 1,258 km^2 of the catchment area. The dam is able to reduce the 3,702 cms natural peak flow to 475 cms on the Black Fork River at Parsons. By siting the two dams as described above, the total controlled flow at Parsons is 598 cms as shown Fig. 5. However, the Option II is rejected promptly because of conflicts with populated areas, and the valuable properties that would be flooded by the proposed dam.

(3) OPTION Ⅲ

As explained in a previous part of the this article, complete control of the flow on the Dry Fork is absolutely necessary since suitable dam sites do not exist on the Blackwater River. Adding one more dam, named Hendricks Dam, to Option I is planned, locating it 1.6 km upstream of Blackwater River mouth. The Hendricks Dam is able to improve the adverse stream runoffs from Glady Fork and Otter Creek which are released downstream without any regulation and thus made the peak flow rate at Parsons too high in Option I. The population areas such as Hendricks and Hambleton, to be flooded in Option II, would also be protected from the floods. Even though the reservoir site has steep slopes and a narrow valley, enough storage is provided to impound the desired amount of runoff, 1,093 cms, from the Dry Fork Dam, Glady Fork subbasins, and Otter Creek Subbasin. Using this medium sized dam, the peak flow rate on Black Fork at Persons was reduced to 926 cms; 75% reduction; 10 years frequency flow.

The controlled flow rate on Shavers Fork at Parsons is 125 cms. Thus, the total controlled flow at Parsons is 1,046 cms as compared to the natural flow of 5,000 cms. Fig. 6 describes the controlled flow of streams above Parsons.

5. Conclusions

The final design options are heavily influenced by the constraints placed on the study by the local citizens of Tucker County and surrounding counties. The constraints required that little or no homes or farms be displaced, hence greatly limiting potential reservoir locations. The town of Parsons and surrounding communities must be heavily protected.From the present investigation, the following conclusions have made:

(1) The computer model to analyze the November floods of Cheat River Basin is completed by HEC-1 model.
(2) Because of basin's characteristics the Snyder's Unit Hydrograph is applied on the upstream of Parsons while SCS method is used for the lower half of the basin.
(3) The Snyder Method gives a more accurate fit at Parsons and Davis.
(4) The results computed by the Snyder's Unit Hydrograph are also matched remarkably well with those of the observed

hydrographs on the upstream of Parsons. These results are illustrating the Snyder's method is suitable in the areas of steeper slopes and highlands.

(5) According to the analysis, the final proposed plan to mitigate Cheat River Basin flood hazard consists of the use of the three moderately sized dams of Option Ⅲ.

References

Chow, V.T., and Kulandaiswamy, V.C. (1982). "The IUH of general hydraulic system model." *J. of Hydr. Engr.*, ASCE, Vol. 108, No. 7, pp. 830–844.

Comprehensive survey of the Monongaheal river West Virginia. (1973). Department of Natural Resources, Vol. Ⅰ.

Dalrymple, T. (1965). "Flood peak runoff and associated precipitation in selected drainage basins in the United Stated." *U.S. Geol. Surv. Water Supply paper No. 188.*

Flood-hydrograph analysis and computations. (1959). Disign Mannuals, Em. 1110-2-1405, U.S. Army Corps of Engr.

Flood insurance study town of Parsons WV. (1978). U.S. Dept. of Housing and Urban Development, Federal Insurance Administration.

HEC-1 flood hydrograph package user manual. (1985). The Hydrologic Engineering Center, U. S. Army Corps of Engineers.

Parsons and vicinity, West Virginia local flood protection projects Shavers Fork, Black Fork, and Blackwater rivers. (1965). U.S. Army Engineer District, Pittsburgh.

Robert, M.R. (1980). "Runoff synthesis using landsat and SCS model." *J. of Hydr. Div.*, ASCE, Vol. 106, No. HY9, pp. 667–678.

Snyder, F.F. (1938). "Synthetic unit hydrographs." *Trans. Am. Geophys. Union*, Vol. 19, pp. 447–454.

Taylor, A.B., and Schwartz, H.E. (1952). "Unit hydrograph lag and peak flow related to basin characteristics." *Trans. Am. Geophys. Union*, Vol. 33, pp. 223–246.

The flood report. (1987). SCS Agricultural Dept. State Office, 74 High Street, Morgantown, WV.

Urban hydrology for small watershed. (1974). Technical Release No. 55, SCS, Engineering Division, U.S. Department of Agriculture.

Viessman, J.J. Jr., Knapp, J.W., Lewis, G.L., and Harbaugh, T.E. (1977). *Introduction to hydrology*, 1EP-Dum-Dommelly Harper and Row Publishers, New York.

〈접수 : 1995년 4월 8일〉