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THE SANTA BARBARA URBAN HYDROGRAPH METHOD

James M. Stubchaer, F. ASCE
Flood Control Engineer
Santa Barbara County Flood Control
and Water Conservation District
Santa Barbara, California 93101

다음 논문은 1975년 7월 28일 미국 University of Kentucky에서 개최된 National Symposium on Urban Hydrology and Sediment Control에서 발표된 도시유역에서의 홍수량을 소형전자계산기에 의하여 계산하는 간단한 방법을 소개하는 논문이다. 이론적으로 유역추적의 개념을 사용하였으며 방법의 간편성으로 인하여 실무에서 쉽게 쓸 수 있는 방법이므로 여기에 논문 전문을 전개한다. <편집자>

INTRODUCTION

Hundreds of millions of dollars are spent each year for the construction of urban drainage facilities. Costs vary largely with the design flows, yet manystorm drains are designed on the basis of the Rational Method with runoff coefficients selected from national handbooks,(1) even though the coefficients frequently have a range of ±35% and a spread of 100%. The hydrologic methods used to develop hydrographs for the design of storm water storage facilities are even less standardized. Large agencies or large engineering firms may have specialized hydrologists and large computer facilities, but this is usually not the case with smaller agencies and firms. There is a real need for a hydrologic method which is simple, easily understood, does not require a large computer, and which can readily be applied to local situations to develop peak flows and hydrographs of known frequency for the design of urban drainage facilities.

The Rational Method has long been used to calculate peak flows, but it is not suitable for calculating complete hydrographs. The following questions are only a few of those which arise during the application of the Rational Method: Does the 10 years storm result in a 10 year storm result in a 10 year peak flow? What value of the runoff coefficient "C" should be used? Is "C" constant for all storms, or does it vary with rainfall intensity or return period? How are the effects of different watershed wetness values accounted for? Is the watershed time of concentration really the critical storm duration?

In calculating hydrographs, what loss rates and storm patterns should be selected? Are synthetic rainfall distribution curves, such as those published by the Soil Conservation Service, valid? Is it true, as most rainfall distribution curves assume, that the peak 20 minute intensity of a given frequency occurs during the 6 hour storm of the same frequency? How can these many variables be analyzed to obtain peak flows and runoff volumes for a given return period?

Probably the best means of obtaining answers to these questions is to analyze 20 or more years of rain and flow records from the watershed in question. Unfortunately, long term stream gage records are not available for most urban watersheds, though long term recording raingage records are fairly common. An obvious solution is to use a watershed model calibrated on a short period of concurrent rain and flow records to calculate hydrographs for the period of rain records, and then make a frequency analysis of the results. Which model should be used?

Complete, continuous, computer simulation systems require masses of data and very large computers. They are not easily understood by the non-specilista and can be cumbersome to use. These systems keep running accounts of soil moisture, evapo-transpiration, snow melt, and other items which are important for river basins. In smaller urban watersheds, however, a large portion of runoff comes from impervious areas hydraulically connected to the drainage systems so that soil moisture conditions are not dominant. The complete systems often cannot be used for small inlet areas, where the short times of concentration are less than the time increment used in the simulation programs.

The Unit Hydrograph Method is easy to understand, but clumsy to apply, especially to complex storms in small urban watersheds with short times of concentration. The method deals only with direct rainfall excess. In order to apply the method, the unitgraph for the watershed must be known or estimated.

In an effort to devise a hydrograph computation method which is simple, understandable, and easy to apply, the Santa Barbara County Flood Control and Water Conservation District developed in 1966 a computer program known as "HYDRO." The original program used rainfall distribution for a 6 hour strom from the Soil Conservation Service type "C" curve. A constant pervious loss rate was assumed and rainfall excess for each time increment of rainfall was calculated. This rainfall excess was then routed through an imaginary reservoir with a routing constant equivalent to the watershed time of concentration. At first,urban runoff data was not available fer testing this method. In 1968, a major new drainage system, the Victoria Street Interceptor Storm Drain (VSSD), was constructed in Santa Barbara and in 1970 a streamflow and rainfall recording station was installed on the drain where it discharges into Mission Creek. The data obtained from this station was utilized in evolving "HYDRO" into the Santa Barbara Urban Hydrograph method.

THE SANTA BARBARA URBAN HYDROGRAPH METHOD

The Santa Barbara Urban Hydrograph(SBUH) method is a simple means of developing runoff hydrographs from urban watersheds using local data. To use the method, the hydraulically connected impervious portion of the watershed is determined. Rain falling on this impervious portion is considered to be 100% runoff. Rain falling on or flowing from unconnected impervious areas across pervious portions is subjected to infiltration losses,the magnitude of which depend on antecedent rainfall. The resulting rainfall excess for each time period is multiplied by the watershed area in acres. The instantaneous hydrograph thus obtained is routed through an imaginary linear reservoir with a routing constant equivalent to the watershed time of concentration to obtain the final hydrograph.

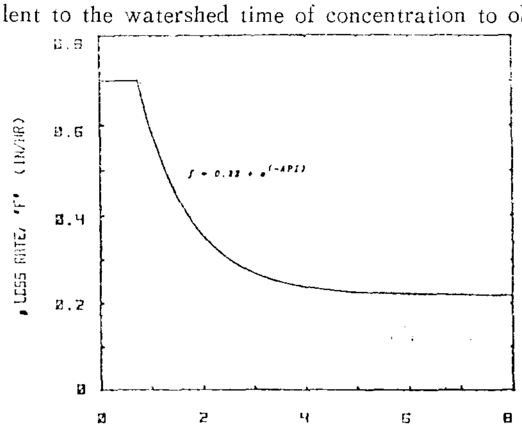

Fig. 1. -API versus f Curve

In the SBUH method, the watershed is not divided into zones of equal flow time. During calibration, it was found that a simple routing procedure gave satisfactory results without greater refinement. This is perhaps the only unique portion of the SBUH method. The other concepts involved were discovered and discussed many years ago (2, 3, 4, 5).

A step by step description of the basic SBUH method is given below:

1. A storm recorded in or near the watershed under study is selected and the precipitation records reduced to obtain rainfall amounts ($P(t)$ for at least 6 hours or the duration of the storm. The

time increment (Δt) used is in hours, and was 1/6 hour, or 10 minutes, for the VSSD study.

2. The Antecedent Precipitation Index (API) for the 30 day period preceeding the storm is calculated as outlined by Linsley, pages 171~172(2), using a daily recession factor of 0.9. The apparent infiltration loss rate (f) is determined from a curve of f versus API. "f" varies from 0.22 to 0.7 inches/hour in the VSSD watershed (Fig. 1).

3. The impervious portion (I) of the watershed which is effectively hydraulically connected to the drainage system(streets with curbs, storm drains, channels) is estimated. If streamflow records are available, the runoff depths from smaller storms on dry watersheds may be used to determine I, as almost all the runoff will be from such impervious areas. In the absence of such data, I may be estimated as 1/2 of the total impervious areas. "I" will typically vary from 0.15 to 0.40 for urban areas. Rain which falls on this impervious portion is considered to be 100% runoff.

4. The watershed area and time of concentration are determined. The time of concentration is calculated in the conventional manner by summing the initial time, street travel time, and storm drain travel time. Initial times for residential lots generally vary from 7 minutes for houses with roof drains connected into the drainage system to 12 minutes for moderate size lots with unconnected roof drains. The use of overland flow methods is not recommended because of the non-planar surfaces, different types of surfaces, nonuniform slopes, difficulty in estimating roughness values for short low paths, and non-uniform rainfall intensities during real storms.

5. Runoff depths for each time period are calculated as follows:

Impervious area runoff, $R0 = I\ P(t)$ (1)

Pervious area runoff, $R1 = (1-I)(P(t)-f) >= 0$ (2)

Total Runoff depth, $R(t) = R0 + R1$ (3)

6. The "Instantaneous Hydrograph" is obtained by multiplying the runoff depth for each time period by the drainage area in acres and by the constant $1.008/\Delta t$ to convert units to cubic feet per second.

$$I(t) = 1.008\ R(t)\ A/\Delta t \quad (4)$$

7. The final hydrograph is obtained by routing the instantaneous hydrograph through an imaginary reservoir with a time delay equal to the time of concentration. This routing maybe done graphically (5) or mathematically, using the simple equation

$$Q(t) = Q(t-1) + KI(t-1) + I(t) - 2Q(t-1)) \quad (5)$$

where $K = \Delta t/(2t(c) + \Delta t)$. $I(t)$ is the instantaneous hydrograph flow at time(t), and $Q(t)$ is the final hydrogaph flow.

These seven steps will calculate the direct runoff hydrograph. The required computations can be performed manually or on a programmable calculator, though it is more convenient to use a computer.

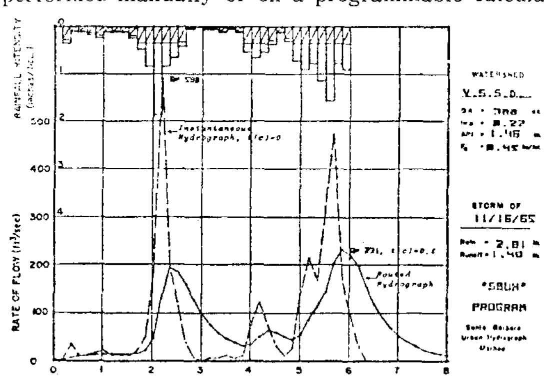

Fig. 2. -Plot of Results from "SBUH" for Storm of 11/16/65

A "BASIC" language computer program name "SBUH" is listed in the Appendix. 48 pieces of rainfalldata of Δt duration must be place between line 1991 and the end of the program. The other data needed "to run the program are requested through conversational statements. These are the watershed area in acres, the time of concentration in hours, the pervious loss rate in inches per hour, and the time increment in hours.

This program can be run on many time-sharing computer services or on small "BASIC" language computers, such as the Hewlett-Packard 9830 or the Wang 2200. An example of the output from this program for the November 16, 1965, storm is given in Fig. 3 and a plot is shown in Fig. 2. In this and subsequent similar figures, the rainfall histogram is plotted inverted at the top. Rainfall excess is shown by open rectangles, while infiltrated, or lost, rain is shown with diagonal lines through the rectangles of the histogram.

The version of the SBUH program used by the Santa Barbara County Flood Control District is called "HYD-SB". It performs the above steps and in addition, calculates the pervious area loss rate from the API and then adjusts f with time during the storm so that the f is higher that average initially, but decays with time. "HYD-SB" also adds in base flow equal to .066 times the surface runoff or the Δt recession coefficient times the base flow in the previous period, whichever is greater. A subroutine of the program scans the rainfall data and determines the maximum amount for each duration. The output hydrograph may be written into a file for further routing through a retarding basin, combining with other hydrographs, and other purposes.

CALIBRATION OF THE SBUH METHOD

The SBUH method was calibrated and tested using runoff records from the Victoria Street Interceptor Storm Drain in Santa Barbara for the period from 1970 to the present. The tributary watershed is 388 acres with a time of concentration of 0.60 hours. The watershed length is 11,500 ft. and the fall is 700ft. The watershed is predominantly older residential, with some institutional and commercial development in the lower areas and some open steep slopes in the upper areas. Rainfall data are from a recording gage located at the Santa Barbara Flood Control District office, which is near the center of the watershed, and other nearby gages. Runoff depths measured from small storms, when almost all the runoff is from hydraulically connected impervious areas, were divided by the rainfall depths and the results indicated that $I=0.18$ to 0.22. This is about half of the total area of streets, roofs, parking lots, etc., in the watershed, as measured from detailed topographic maps. Pervious area loss rates were determined for several storms by adjusting the loss rate and calculating the runoff depth until the recorded runoff was reproduced. These were plotted against API's for the storms and an empirical relationship derived (Fig. 1). A wide variation in the loss rate was observed, ranging from 0.7 in/hr on an initially dry watershed to 0.22in/hr on a saturated watershed..

```
    PROGRAM SBUH
    DELTA T (HRS) =?0.1667
    DRAINAGE AREA (AC)=?388
    T(C) (HRS) =?0.60
    IMPERVIOUS=?0.22
    PERVIOUS LOSS RATE (IN/HR)=0.45
RAINFALL=2.81      INCHES
    RUNOFF VOLUMES:
IMPERVIOUS=0.62   IN=19.99   AC-FT
RERVIOUS  =0.84   IN=27.11   AC-FT
TOTAL     =1.46   IN=47.09   AC-FT
```

PEAK FLOW =230.7 CFS AT 5.83 HRS

TIME(HRS)	FAIN(IN)	FLOW(CFS)	TIME(HRS)	FAIN(IN)	FLOW(CFS)
0	0	0	0.67	0.03	9.56
0.17	0.06	3.78	0.83	0.04	11.64
0.33	0.02	7.89	1	0.02	12.58
0.5	0.02	8.49	1.17	0.02	12.03

TIME(HRS)	FAIN(IN)	FLOW(CFS)	TIME(HRS)	FAIN(IN)	FLOW(CFS)
1.33	0.02	11.61	4.83	0.12	52.05
1.5	0.04	12.56	5	0.15	83.13
1.67	0.08	18.16	5.17	0.13	109.49
1.83	0.14	43.2	5.33	0.19	140.86
2	0.31	127.95	5.5	0.26	201.79
2.17	0.14	192.02	5.67	0.13	230.68
2.33	0.11	183.23	5.83	0.15	221.04
2.5	0.07	157.68	6	0.06	197.08
2.67	0.01	124.25	6.17	0	152.78
2.83	0	94.57	6.33	0	115.51
3	0.01	72.13	6.5	0	87.38
3.17	0.01	55.79	6.67	0	66.03
3.33	0.02	44.07	6.83	0	49.92
3.5	0.01	35.21	7	0	37.74
3.67	0.02	28.51	7.17	0	28.54
3.83	0.09	31.82	7.33	0	21.57
4	0.11	47.81	7.5	0	16.31
4.17	0.09	59.89	7.67	0	12.33
4.33	0.06	58.07	7.83	0	9.32
4.5	0.02	48.94	8	0	7.05
4.67	0.05	41.41			

Fig. 3. -Printout of Results from "SBUH" for Storm of 11/16/75

The results of the calibration for the January 18, 1973, storm are shown in the upper part of Fig. 4. This storm had the 5th largest peak flow and the 7th greatest runoff depth in the annual series of data from 1953 to date. The first peak is 17% high, the second peak is close, but the third peak could not be accounted for in any of the raingage charts examined.

The calibration had just been completed when the storm of December 3, 1974, occurred on a dry watershed. This was the 2nd largest 6 hour storm and the 4th largest 40 minute storm during the period of records. The flow calculated by the unrecalibrated "HYD-SB" program and the observed flow for this storm are shown in the lower part of Fig. 4. Overall, the calculated flow agrees quite well with the observed flow. Because the watershed was so dry, most of the runoff during the first 3 1/2 hours was from streets, and the actual $t(c)$ during this period was lessthan 0.6 hrs, so the observed peaks from small raintall "bursts" were higher than calculated. The peak flow and the runoff depth both rank 6th in the series of 22 annual maxima.

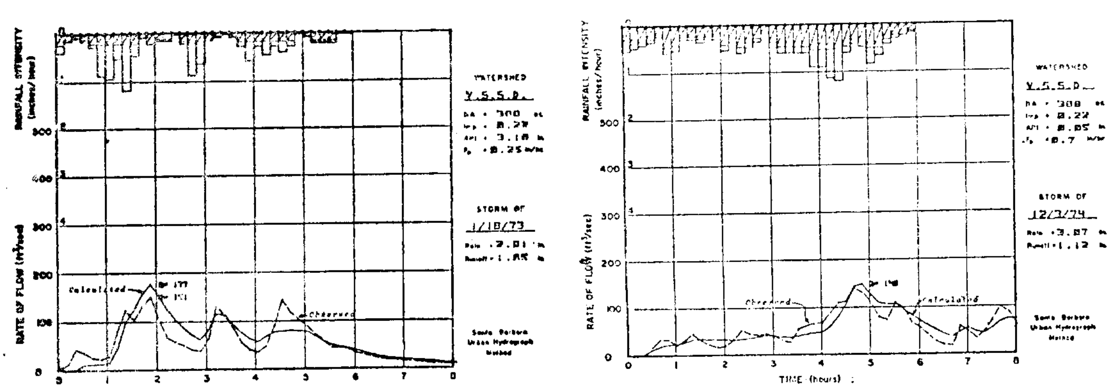

Fig. 4. Observed and Calculated Hydrographs for 2 Storms

It is fortunate that two significant storms occurred during the period of concurrent records, along with a number of lesser storms. The confidence in the method is enhanced by a wide variation in the storms simulated. The accuracy of the calculated results appears to be consistent with the accuracy of streamflow measurements and the variation of rainfall intensities over the watershed.

In an effort to test the SBUH method on watersheds other than the VSSD, engineering literature was searched for urban runoff data. One set of data was found in a paper by Eagleson(7) for an urban watershed of 141 acres in Louisville. Kentucky. The watershed was 83% impervious with an observed runoff depth of 44% of the rainfall. From watershed data given, $t(c)$ was estimated to be 0.35hrs. The rainfall data for a Δt of 1/12hr (5min) was run through the SBUH program with $I=0.44$ and $f>$observed intensities to duplicate the volume determination method of Eagleson's paper.

The results are plotted on Figure 6. The solid line is the observed flow, the circles the results from Eagleson's unitgraph method, and the semi-dashed line the results from the SBUH program. The fit appears to be quite good. The Louisville watershed has a much different shape than the VSSD watershed and other parameters are quite different.

Additional data on urban runoff for two storms on the Oakdale Avenue Basin in Chicago. Illinois, was found in papers by Chien and Saigal (8) and Terstriep and Stall (9). Rainfall data for the storms of July 2, 1960, and July 7, 1964, from the first paper was prorated over 2 minute time intervals for use with this Δt in the SBUH program. The watershed area is 12.9 acres, of which 5.8 acres is impervious area. The second paper contains a time-area diagram which indicates the maximum $t(c)$ from this impervious area is 12 minutes. The paper by Chien and Saigal estimates sewer flow time as 5 mintes and overland flow time for 50 feet of turf as from 16 to 60 minutes, depending on rainfall intensity. Because the impervious area runoff is dominant in the runoff process for this watershed, the shorter $t(c)$ is believed to be more accurate and was used as input data to SBUH. The impervious runoff fraction of 0.35 was obtained from the second paper. The pervious area loss rate was adjusted to approximate observed volumes in the absence of API and f data. The loss rates used were 0.8 in/hr for the 1960 storm and 1.8 in/hr for the 1964 storm. The results of the SBUH runs are plotted on Figure 5.

These results seem to demonstrate a general applicability of the SBUH method and to indicate that the laborious time-area and unitgraph methods may not be required in urban watersheds.

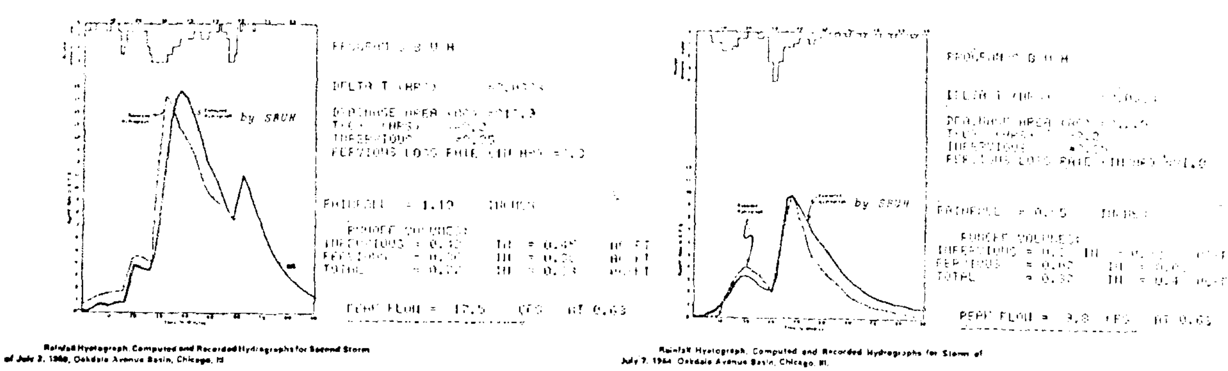

Fig. 5. Results of SBUH Method for 2 Storms on Oakdale Avenue Basin, Chicago, Illinois.

APPLICATION OF THE SBUH METHOD TO THE VICTORIA STREET STORM DRAIN WATERSHED

The calibrated SBUH method was used to calculate the peak events for each water-year for which recording raingage charts were available. Recording raingage charts from 1953~54 to 1974~75 were scanned to determine the most intense annual storms. Ten minute rainfall amounts for 6 hour durations and API's were obtained for each storm and run through "HYD-SB." The storm which resulted in the highest peak flow in each year was selected for further analysis. Rainfall amounts for the maximum 10, 20, 30, 40, 60, 90, 120, 180, 240, and 360 minute periods were extracted and subjected to frequency analysis using the Gumbel method (2). The annual series of peak flows and runoff depths was analyzed similarly. The plots of the frequency analyses are shown in Fig. 7.

Table 1 contains a summary of the "HYD-SR" runs and the frequency analyses, with the return periods(RP) for the 0.6 hour rainfall intensities, the peak flows, the 6 hour rainfall amounts, and the runoff depths shown in parentheses. All the data for a water year are from the same 6 hour storm.

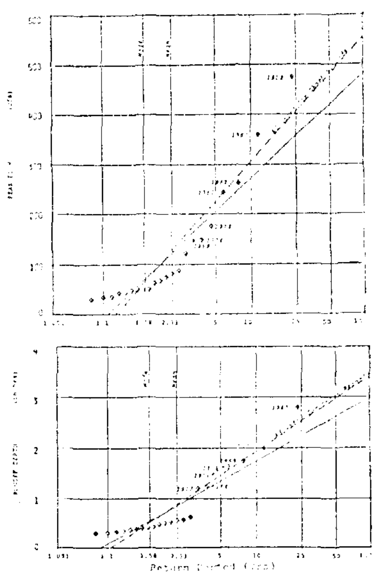

Fig. 6. -Plot of results of Unitgraph Method & SBUH Method on Louisville Data·

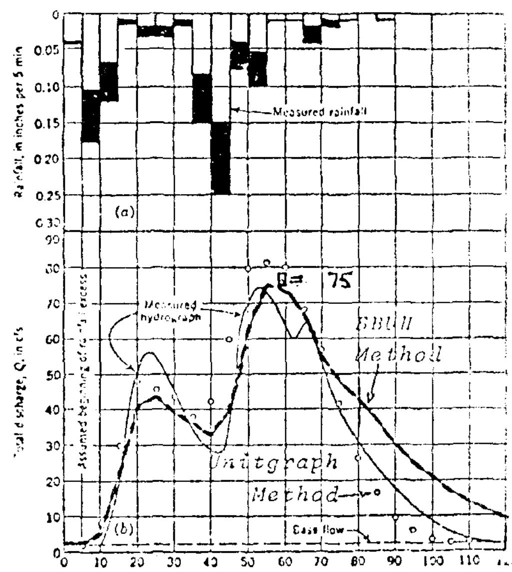

Fig. 7-Peak Flow and Runoff Depth Frequency Curves for VSSD.

TABLE1-Summary of Results from "HYD-SB" for the V.S.S.D. Watershed

Water Year	API	0.6 Hr Rain	R.P.	Peak Flow		R.P.	C Peak	6Hr Rain	R.P.	Runoff Depth	R.P.	C Vol
	in	in/hr	yrs	in/hr	cfs	yrs		in	yrs	in	yrs	
53-54	.54	.60	(1.6)	.12	48	(1.4)	.20	1.59	(1.8)	.42	(1.5)	.26
54-55	1.28	.51	(1.4)	.11	42	(1.4)	.22	1.27	(1.4)	.32	(1.3)	.25
55-56	2.27	.35	(1.2)	.07	29	(1.3)	.20	1.44	(1.6)	.34	(1.4)	.24
56-57	.91	.55	(1.5)	.13	49	(1.4)	.24	1.45	(1.6)	.39	(1.4)	.27
57-58	2.33	2.11	(80)	1.22	476	(48)	58	1.59	(1.8)	1.54	(5.7)	.96
58-59	.09	.65	(1.8)	.12	47	(1.4)	.18	2.09	(3.3)	.50	(1.6)	.24
59-60	.56	.48	(1.4)	.11	43	(1.4)	.23	1.68	(2.0)	.41	(1.4)	.24
60-61	2.46	.57	(1.6)	.22	67	(1.6)	.39	1.20	(1.3)	.56	(1.7)	.47
61-62	5.41	.93	(3.3)	.67	262	(6.8)	.72	2.66	(7.2)	1.99	(11)	.75
62-63	.16	.77	(2.3)	.21	81	(1.7)	.27	2.00	(2.9)	.62	(1.8)	.31
63-64	.61	.51	(1.4)	.13	50	(1.5)	.25	1.00	(1.2)	.29	(1.3)	.29
64-65	3.62	.40	(1.2)	.16	62	(1.5)	.40	.71	(1.0)	.30	(1.3)	.42

65-66	1.46	1.13	(5.4)	.62	243	(5.8)	.55	2.81	(9.0)	1.63	(6.5)	.58
66-67	4.36	1.20	(6.5)	.92	359	(16)	.77	3.62	(31)	2.80	(39)	.77
67-68	.23	.49	(1.4)	.09	35	(1.4)	.18	1.54	(1.7)	.37	(1.4)	.24
68-69	10.66	.56	(1.6)	.37	146	(2.7)	.67	2.34	(4.6)	1.74	(7.6)	.74
69-70	2.00	.56	(1.6)	.22	87	(1.8)	.39	1.31	(1.4)	.52	(1.6)	.40
70-71	.76	.71	(2.0)	.10	74	(1.7)	.27	1.64	(1.9)	.43	(1.5)	.29
71-72	2.10	.45	(1.3)	.09	34	(1.4)	.20	1.36	(1.5)	.38	(1.4)	.28
72-73	3.18	.90	(3.1)	.45	175	(3.3)	.50	2.01	(3.0)	1.19	(3.5)	.59
73-74	.26	1.01	(4.0)	.31	121	(2.2)	.30	1.58	(1.3)	.57	(1.7)	.36
74-75	.05	1.06	(4.5)	.38	148	(2.7)	.36	3.07	(13)	1.22	(3.6)	.40
Mean	2.06	.76			122			1.78		.84		
Std Dev	2.37	.23			115			.69		.68		

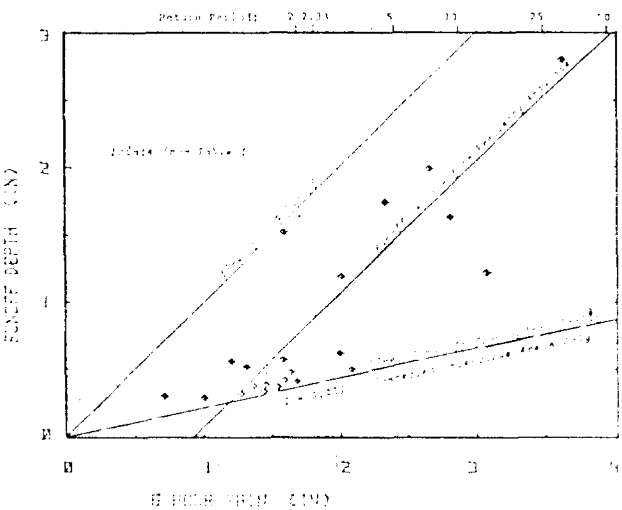

Fig. 8. -Runoff Depth versus 6 Hour Rainfall.

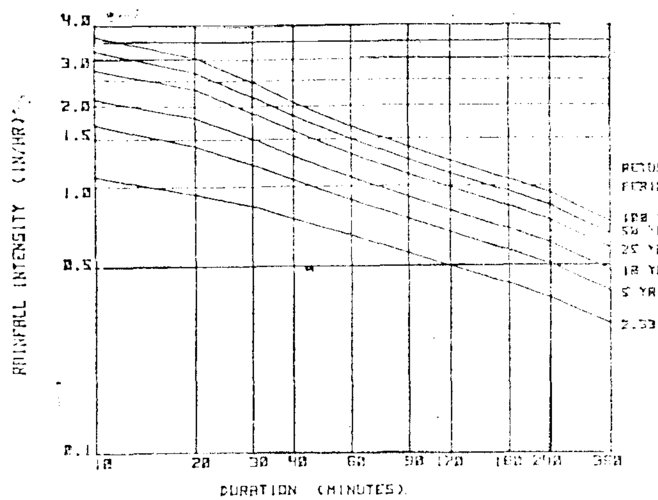

Fig. 9. -Rainfall Intensity-Duration Curves from VSSD Data.

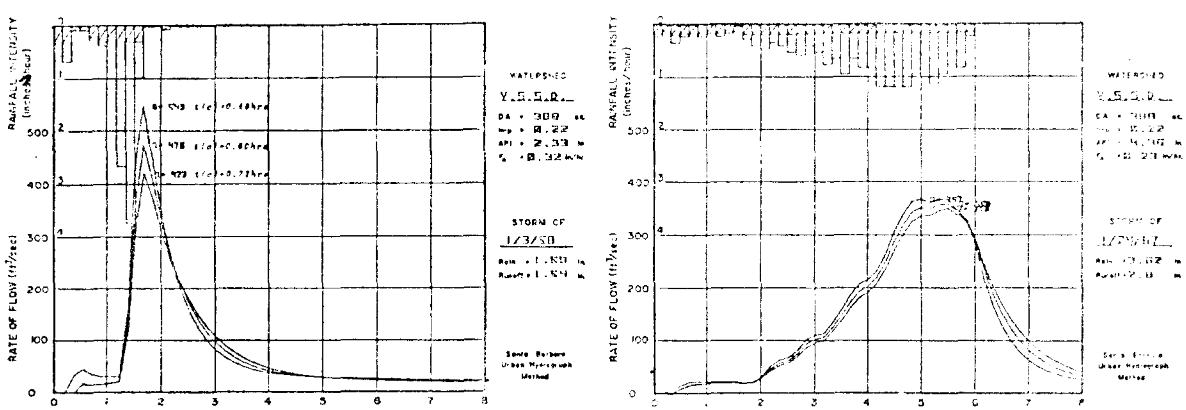

Fig. 10. Effects of 20% Changes in $t(c)$ for 2 Storms

A review of the data in Table 1 leads to the conclusion that the most intense 0.6 hour rainfall intensities do not have the same return period as the same 6 hour storms. In 1958, the RP of the short storm was 80 years while the longer duration storm had an RP of 5.7 years. In 1967, the short storm RP was 6.5 years and the long storm RP was 39 years. This pattern appears typical for storms with RP's exceeding 5 years, which indicates that many synthetic rainfall distribution curves are not valid, because they are constructed from intensity-duration curves with all durations having the same RP.

A plot of runoff depths versus 6 hour rain is shown in Fig. 8. The points are bounded by the upper line of 100% runoff and the lower line of impervious area runoff only. Within this range, the points are widely scattered and no unique relationship is apparent. A curve based on frequency relationships is shown lcoints on this curve represent 6 hour rainfall amounts and runoff depths of the same return period.

Rainfall intensity-duration curves compiled from the rainfall records used in this analysis are shown in Fig. 9. Precipitation frequency curves from the same data are shown in Fig. 11.

EFFECTS OF ERRORS IN ESTIMATING TIME OF CONCENTRATION

The SBUH method was applied to the 1958 and 1967 VSSD storms with times of concentration increased and decreased by 20% to determine the sensitivity of the peak flows to errors in the time used. The results are shown in Fig. 10. The hydrographs with the highest peaks have the 0.48 hour time and the hydrographs with the lowest peaks have the 0.72 hour time. A comparison is given in Table 2.

The effect of under or over estimating the time of concentration quite pronounced with the 1958

Table 2- Effect of 20% Change in $t(c)$ on Peak Flows

Storm	$t(c)$	Peak Flow	Difference
1958	0.48hrs	549cfs	+15.3%
	0.60hrs	476cfs	—
	0.72hrs	422cfs	−11.3%
1967	0.48hrs	367cfs	+2.2%
	0.60hrs	359cfs	—
	0.72hrs	350cfs	−2.5%

storm, which had a very high intensity rain for a short period. Here the routing through the watershed had a significant effect. On the other hand, the same variation in concentration times had little effect on the 1967 storm, which was of longer duration so that runoff rates were nearer equilibrium and the routing was not so important. Most of the observed storms were similar in pattern to the 1967 event, so it is believed that small errors in estimating times of concentration are not significant.

APPLICATION OF RESULTS TO RATIONAL METHOD

The Rational Method, $Q=C_IA$, is widely used in the design of storm drains and is easy to apply. The major unknown in this procedure is the runoff coefficient, "C." In order to apply the results of the SBUH method to the Rational Method, a plot of the derived "C" values versus the rainfall intensity for the VSSD watershed was prepared (Fig. 12). The points were widely scattered and no line of best fit was apparent. The coefficients were relatively low for short duration, high intensity storms, such as 1958, and were relatively high for long duration, even intensity storms on wet watersheds, such as 1967. It was not possible to obtain single value coefficients of observed rainfall intensities because of the wide variation in rainfall distribution and antecedent moisture conditions. Runoff coefficients were then calculated for various return periods by using the results of the frequency analyses of rainfall and runoff from the SBUH results. Peak runoff rates were divided by $t(c)$ rainfall intensities for the same return periods. The results are given in Table 3 and by a curve in Fig. 12.

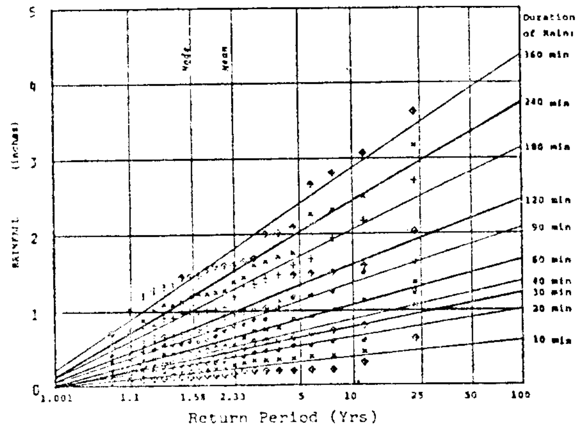

Fig. 11. -Precipitation Frequency Curves from VSSD Data.

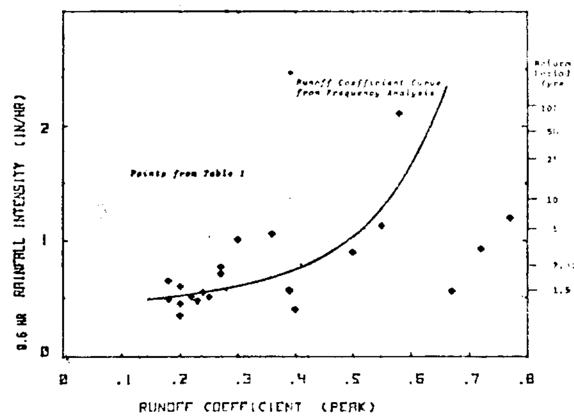

Fig. 12. -Runoff Coefficient versus Rainfall Intensity for 0.6 Hour Duration.

Table 2-Runoff Coefficients for Various Return Periods for the V.S.S.D. Watershed in Santa Barbara, California

Return Period (yrs)	0.6hr Rain (in/hr)	Peak Runoff (in/hr)	Runoff Coefficient
1.5	.57	.16	.28
2.33	.79	.32	.41
5	1.11	.57	.52
10	1.37	.78	.57
25	1.72	1.04	.60
50	1.96	1.23	.63
100	2.19	1.42	.65

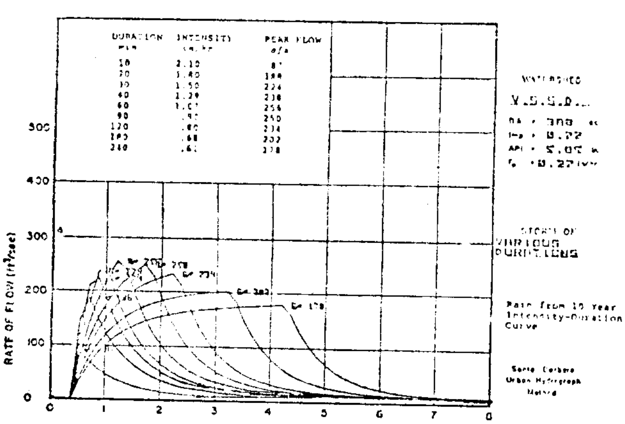

Fig. 13. -"HYD-SB" Results from Rainfall of Various Intensities and Durations.

The results indicate that "C" is not constant, does not vary uniquely with rainfall intensity, does depend on the type of storm, and does vary with watershed wetness. It appears that "C" varies with frequency in a predictable manner. Therefore, it is believed that the Rational Method can be reliably used in the VSSD and similar watersheds in Santa Barbara for the design of storm water conveyance systems for selected flow frequencies, using the "C" values from Table 3. "C" values for other watersheds can be derived in a similar manner. The Rational Method is especially convenient for the design of storm drain inlets and laterals. If no storage basins are included, it can be used for the design of entire systems and the results should be consistent with the results obtained from using the SBUH method. The Rational Method cannot be used for design of storm water retarding basins and it cannot be used to compute the runoff from observed storms on an individual basis.

CRITICAL STORM DURATION FOR PEAK FLOW

In the Rational Method, the assumption is made that the critical storm duration is the time of concentration of the watershed, and the rainfall intensity used is obtained from an intensity-duration curve or this time. In order to test the validity of this assumption, 10 year rainfall intensities for 10, 20, 30, 40 60, 90, 120, 180, and 240 minute durations, as determined by the rainfall frequency analyses discussed earlier, were run through "HYD-SB" for the VSSD watershed, with $t(c)=0.6$ hrs. The results are shown in Fig. 13, with the peaks from left to right in order of the storm duration. The highest peak flow, 256 cfs, was obtained from the 60 minute duration. The peak flow for the 36 minute $t(c)$ is, by interpolation, 232 cfs, which is 9% lower. The hydrographs from the 10 and 20 minute duration storms do not approach equilibrium runoff conditions, hence the lower peak flows. The hydrographs from 120, 180, and 240 minute durations approach equilibrium runoff, but the peak flows are lower because of lesser rainfall intensities.

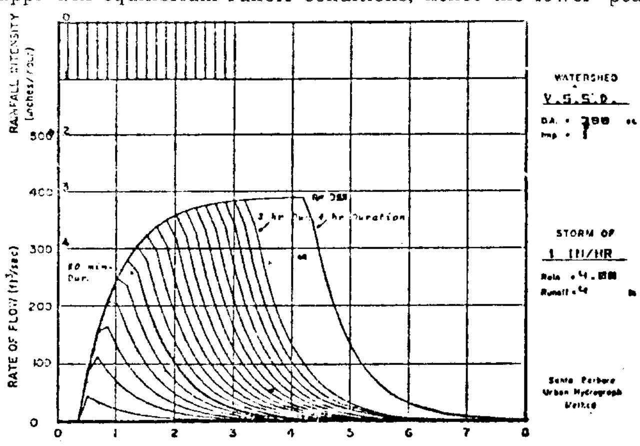

Fig. 14. -"HYD-SB" Results from Continuous Uniform Net Rain of 1"/hr

Though storm durations somewhat longer than the watershed time of concentration yield the maximum peak flows, it is believed that the peak flow from the duration equal to the $t(c)$ is close enough to the maximum to justify the assumption made in the Rational Method regarding critical storm duration. It should be noted that runoff volumes increased with storm duration. It should also be noted that the 10 year peak flow from the frequency analysis is 305 cfs, considerably higher than any of the peaks from the SBUH runs with uniform intensity precipitation.

S-CURVES AND UNITGRAPHS FROM SBUH METHOD

The results of a uniform intensity storm with a net rain of 1 inch/hr for from 1 to 18 consecutive 1/6 hour periods, together with the hydrograph from a 4 hour storm of the same intensity, are shown in Fig. 14. If an inch per hour(net) storm, continued indefinitely, an equilibrium runoff rate of $1.008\times388=391$ cfs would be reached. Fig. 14 shows how this equilibrium rate is approached. Equilibrium is nearly reached after 4 hour of rain. The 4 hour hydrograph to the left of the recession limb is the mass curve, or "S-Curve," of the series of unit storms. The hydrograph from the 1 hour rain (sixth from the left) is the unitgraph from the SBUH method, as it results from 1 inch of net rain in 1 hour. SBUH unitgraphs for otherdurations can easily be calculated.

CONCLUSIONS

The Santa Barbara Urban Hydrograph (SBUH) method is a simple means of developing hydrographs for urban watersheds. Most civil engineers are familiar with the parameters used, and the computational effort involved is low. Even with the great simplification involved, observed hydrographs are satisfactorily reproduced. Effects of watershed changes can be evaluated by adjusting the time of concentration and portion impervious.

The SBUH method may be used with data from recording raingage charts to simulate annual maximum peak flows and runoff volumes. These results can be analyzed statistically to determine flows and volumes of designated return periods for the design or urban drainage facilities. Statistically valid coefficients for use in the Rational Method can be derived.

APPENDIX Ⅰ.-REFERENCES

1. "Design and Construction of Sanitary and Storm Sewers," American Society of Civil Engineers, New York, 1970, pp. 42—53.
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8. Chien and Saigal, "Urban Runoff by Linearized Subhydrographic Method," *Journal of the Hydraulics Division*, ASCE, Vol. 95, No. HY6, 1969, pp. 1809—1834.
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APPENDIX Ⅱ-NOTATION

The following symbols are used in this paper;

A = watershed area (acres)

API = antecedent precipitation index (inches)

C = runoff coefficient for the rational method

f = infiltration rate (inches/hour)

i = rainfall intensity (inches/hour)

I = impervious portion of the watershed effectively hydraulically connected to drainage system (decima fraction)

$I(t)$ = instantaneous hydrograph flow at time t (cfs)

K = routing constant $\Delta t/(2t(c)+\Delta t)$

$P(t)$ = rainfall depth in time increment t

$Q(t)$ = final hydrograph flow at time t (cfs)

RO = impervious area runoff depth in a time increment Δt

$R1$ = pervious area runoff depth in a time increment Δt

$R(t)$ = total runoff depth in time increment t

Δt = time increment used in SBUH (hours)

$t(c)$ = time of concentration of watershed (hours)

APPENDIX Ⅲ. -LISTING OF PROGRAM"SBUH"

```
100 DIM R[49], I[49], P[49], Q[49]
110 X=P9=19=S=Y9=0
120 DEF FNP(X)=INT(100*×+0·5)/100
130 PRINT
140 PRINT TAB(9); "PROGRAM SBUH"
150 PRINT
160 PRINT
170 REM READ RAINFALL LOOP
180 FOR T=2 TO 49
190 READ P[T]
200 P9=P9+P[T]
210 NEXT T
220 PRINT TAB(9); "DEL TA T (HRS) ="
230 INPUT R2
240 PRINT
250 PRINT TAB(9); "DRAINAGE AREA (AC)
    =";
260 INPUT A
270 PRINT TAB(9); T(C) (HRS) =":
280 INPUT T2
290 PRINT TAB(9); "IMPERVIOUS =";
300 INPUT I
310 PRINT TAB(9); "PERVIOUS LOSS RATE
    (IN/HR) =";
320 INPUT F
330 F=F*R2
340 K1=1.00B*A/R2
350 REM NEXT LOOP CALCS AND SUMS
    RUNOFF DEPTHS
360 FOR T=2 TO 49
370 R1=P[T]-F
380 IF R1>=0 THEN 400
390 R1=0
400 R[T]=1*P[T]+R1*(1-I)
410 I9=I9+I*P[T]
420 S=S+R[T]
430 I[T]=R[T]*K1
440 NEXT T
450 PRINT
460 PRINT
470 REM K2=ROUTING CONSTANT
480 K2=R2/(2*T2+R2)
490 REM NEXT LOOP CALCS HYDROGRAPH
500 O[1]=I[1]=P[1]=R[1]=0
510 FOR T=2 TO 49
520 O[T]=Q[T-1]+K2 * (I[T-1]+I[T]-2 * O
    [T-1])
530 IF Y9>=Q[T] THEN 560
540 Y9=0[T]
550 T9=R2*T
560 NEXT T
570 PRINT
580 PRINT TAB(8);"RAINEALL=": FNP(P9);
    "INCHES"
590 PRINT
600 A8=A/12
610 PRINT TAB(11); "RUNOFF VOLUMES:"
620 PRINT TAB(8); "IMPERVIOUS="; FNP
    (I9); "IN=FNP(19*A8); "AC-FT"
630 PRINT TAB(8); "PERVIOUS="; FNP(S-
    19); "IN="; FNP((S-19)*AB);"AC-FT"
640 PRINT TAB(8); "TOTAL =": FNP(S);
    "IN="; FNP(S*AB); "AC-FT"
650 PRINT
660 PRINT
670 Y9=INT(10*Y9+0.5)/10
680 PRINT TAB(11); "PEAK FLOW="; Y9
    "CFS AT"; FNP(T9); "HRS"
700 PRINT TAB(11); "..............."
720 PRINT
730 PRINT "TIME RAIN FLOW"
740 PRINT "(HRS) (IN) (CFS)"
750 PRINT
760 FOR T=1 TO 49
770 PRINT TAB(8); FNP(T*R2-R2); TAB(18);
    P(T);TAB(28); FNP(Q[T])
780 NEXT T
790 STOP
1990 REM RAINFALL DATA FROM LINE 2000
2000 DATA 0.1, 0.2, 0.25, 0.57, 0.89, 0.45, 0.36
2010 DATA 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0,
     0, 0, 0, 0, 0, 0, 0, 0, 0, 0
2020 DATA 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0
9990 END
```

QUESTION AND ANSWER SESSION

Question by Cecil Leonardo, Fresno Metropolitan Flood Control District, 600 Rowell Building, Fresno, California 93726:

Can you provide a sample manual calculation of the SBUH method?

Stubchaer:

Example: Apply SBUH to Storm of 5/22/73 on Clays Mill Watershed, with data given in Haan's paper D2, and rainfall scaled from Fig. 3, page D-22.

Given: $A=890\text{ac}$

$I=1/2\times 32.2\%=0.16$

$\Delta t=15\text{min.}=0.25\text{hrs.}$

Estimate: $t(c)=1.00\text{hrs.}$

$f=1.5\text{in/r}=0.375\text{in/15min.}$

Routing Constant: $K=\frac{\Delta t}{2t(c)+\Delta t}=\frac{0.25}{2+0.25}=0.111$

Computations are shown in table below. The final hydrograph, $Q(t)$, is obtained from Eq. 5, which is applied only once for each time increment. For instance,

$$Q(3)=Q(2)+0.111*[I(2)+I(3)-2*Q(2)]$$
$$=200+0.111*[1130+377-2*200]$$
$$=319\text{cfs}$$

Time (hrs)	Time Increment t —	Rainfall Depth $P(t)$ Data (in)	Impervious Runoff $R0$ Eq. 1 (in)	Pervious Runoff $R1$ Ep. 2 (in)	Total Runoff $R(t)$ Eq. 3 (in)	Instant Hydrograph $I(t)$ Eq. 4 (cfs)	Final Hydrograph $Q(t)$ Eq. 5 (cfs)
.25	1	0					
.50	2	.42	.067	.038	.105	377	42
.75	3	.63	.101	.214	.315	1130	200
1.00	4	.41	.066	.029	.095	341	319
1.25	5	.12	.019	—	.019	68	294
1.50	6	.10	.016	—	.016	57	243
1.75	7	.06	.010	—	.010	36	199
2.00	8	.03	.005	—	.005	18	161
2.25	9	.05	.008	—	.008	29	131
2.50	10	.03	.005	—	.005	18	107
2.75	11	.03	.005	—	.005	18	87
3.00	12	.05	.008	—	.008	29	74
3.25	13	.05	.008	—	.008	29	64
3.50	14	.07	.011	—	.011	39	57
3.75	15	.04	.006	—	.006	22	51
4.00	16	.03	.005	—	.005	18	44
4.25	17	.05	.008	—	.008	29	39
4.50	18	.02	.003	—	.003	11	35
4.75	19	—	—	—	—	—	28
5.00	20	—	—	—	—	—	22
5.25	21	—	—	—	—	—	17
Totals		2.19in.	.351in	.281in	.632in	2270 cfs—1/4hrs	2214 cfs—1/4hrs