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RAPID FLOOD PEAK DETERMINATION ON SMALL WATERSHEDS

By

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1. INTRODUCTION.

Many thousands of estimates of spillway capacity for small structures must be made each year. Although the total cost of all such works on very small watersheds may make up considerable national expenditure, the cost of individual headwater structures is generally too low to warrant detailed individual hydrologic investigations. Instead of sophisticated and highly refined procedures of specialised hydrology a general method is required which can be rapidly applied by workers whose main activity lies in fields other than hydrology. A need exists to present the general practitioner with a method incorporating many of the latest theories and developments in hydrology but which is extremely simple and quick to apply.

The objectives of this paper may therefore be summarized as an attempt to present such a method capable of predicting flood peaks from ungaged rural watersheds ranging in size from 1/5 - 5 sq.miles. The material has been presented under three main headings; commencing with a brief description of how to apply the method. This is followed by describing tests which were applied to the method on the basis of observed floods. Finally the development of the method is described for those who may wish to read further than the mere practical application. Throughout there has been a choice of popular terminology rather than adherence to elegant statistics. Forgiveness is sought of the specialist hydrologists, but the paper is directed rather at the wider readership of practicing engineers. It is hoped that the somewhat bold assumptions on rather arbitrary aspects, will stimulate wide discussion by designers experienced in this type of work.

2. DESCRIPTION OF METHOD.

The three elements needed to make an estimate by this method are the maximum rainfall expected in half an hour; the basin characteristic; and the infiltration capacity. The former of these three elements can be obtained from published[1] maps for various return periods.

2.1. Basin characteristic. —— The design parameter which is to account for the speed with which flood waters would likely be propelled throughout the watershed is B. A value of B can be derived from Fig. 1 according to the interplay of H, the fall in feet from the top of the watershed to the site (not including waterfalls and gully heads), and ℓ, the length of the longest collector in the stream-system (continued out to the divide). This nomograph was presented by the Soil Conservation Service[2] for determining the "time of concentration", T_c. The present terminology "basin characteristic", B, has been substituted for the latter name so as to discourage any confusion with the classical concept involving speeds of travel of flood water or assumptions of channel roughness.

2.2. Infiltration capacity. —— The third design parameter, S, accounts for the various soils and the differences of plant cover between watersheds. Tables 1 and 2 which have been transcribed from the ASCE[3] manual provide an approximate means of estimating this infiltration capacity, S. The value of f_1 inches per hour, from Table 1, simulates infiltration capacity shown by a standard curve after applying excessive rainfall for 1 hour on bare soil. All this table does in effect is to divide the wide margin of guessing error from 0.01 to 1.00 into three classes. Selection of suitable values for f_1 will still be a vexed problem. Considerable judgement will be needed to consistently evaluate the field inspection of soil profiles. It is to be hoped that other workers will soon produce a highly portable infiltrometer which can readily give reliable on-site estimates of infiltration capacity. Thereby the problem of latitude in Table 1 and the vagaries of interpretation by its various users will be eliminated.

The modifying effects which differences in plant cover and land use have on infiltration are accounted for by F, selected from Table 2. It subdivides the cover types of permanent, close growing crops, or row crops each into three conditions. A classification "good" describes one which would strongly inhibit flood runoff. Table 3 presents a large number of F-values that were attributed to common cover conditions later in this study. It may be of interest to practicing engineers to compare their interpretation of Table 2 with that of the author. The product of f_1 and F yields an estimate of S. This combined estimate of infiltration capacity proved more strongly correlated with storm runoff than five other soil-landuse parameters in a previous study[4].

2.3. Design charts. —— Once the above three parameters have been estimated for a particular design the flood peak is obtained from the solid curves in Fig. 2. This figure is comprised of eight parts. Each part corresponds to one basin characteristic. Should a flood peak be required for an intermediate value of basin characteristic this may be achieved by interpolation between two bracketing values. These design charts should not be applied within the region shaded in Fig. 3. For these regions the ratio of P_{24h} to P_{30m} exceeds 4. The extent of underestimation that would occur in such regions can be appreciated from the dotted curves whose ratios are noted in parentheses.

2.4. Adjustment for A.P.I.. —— If one is making an estimate in a region where five days of the small-area flood-season preceeding the storm are likely to produce more than 4 inches of rain, then one should increase the previously derived value of q_e by 20%.

2.5. Adjustment for late-peaking storms. —— The highest flood peaks are generally caused by storms which have their highest intensities after considerable rain has already fallen. These are somewhat uncommon among small-area convective thunderstorms. Sometimes the work is located in a region where such late-peaking storms are particularly common. On other occasions one may wish to take additional precautions with a work of above-average importance. On such occasions this aspect of late-peaking storms warrants the addition of 50% to the values obtained from the design charts in Fig. 2. In instances where both the A.P.I. -correction and the correction for late-peaking storms are applicable an overall safety factor of 1.8 = 1.2 x 1.5 may be used.

2.6. Applicable regions. —— Calculations required for the development of this method were performed with eighteen pairs of 30-minute and 24-hour rainfall extremes. Further studies[5] showed that there were certain regions of the continental United States in which the method could be expected theoretically to give significant error. Estimates should not be attempted in these regions which are shown in Fig. 3.

3. TESTS OF METHOD.

After the theoretical development of this method had been completed, the United States Department of Agriculture[6] published some new data on observed floods from small watersheds. It therefore became possible to test this new method on eighty-three floods which occurred on twenty-nine widely spaced watersheds as a result of fifty separate storms. Of these eighty-three events only twenty-eight were among the forty-seven used in any way in the development of the original study[4]. Thus the present test may be considered virtually independent of any empirical notions involved in the theoretical development of the present method. The particular storms used are listed in Table 4 together with the event numbers by which they are referred to in subsequent illustrations. Agricultural Research Service numbers are indicated for each of the watersheds which ranged in extent from 130 to 4,380 acres. The localities have been mapped onto Fig. 3, whence it can be seen they are all outside the region to which the method is inapplicable.

Values obtained from the design charts, Fig. 2, will be compared to the eighty-three observed flood peaks. These design charts already contain a 14% increase above the theoretical values which was found necessary to compensate for the overall underestimation of the theoretical values. Defining the output of Fig. 2 as our base for further evaluation, will clarify how the adjustment percentages suggested in sections 2.4 and 2.5 were obtained.

3.1. Antecedent precipitation. —— The aggregate of all rain which fell on five days prior to the storm, including any light rain which preceded the actual rain storm on the same date, was taken as the antecedent precipitation index, A.P.I.. It was shown to be more closely linked to inordinately large flood peaks than were any of the total rains for either the 1-, 2-, or 3-preceding days. The ratio of the observed peak to the estimated peak may be considered as a measure of the excessive flooding that may occur under certain conditions. This ratio was studied according to the following arbitrary subdivision of A.P.I. : 0 to 2 inches, 2 to 4 inches, 4 to 6 inches and 9 to 11 inches. The corresponding average ratios of observed peaks to predicted peaks were 0.76, 1.04, 1.16 and 1.23. Moreover the corresponding scatter diagram, Fig. 4, showed very clearly that provided A.P.I. was less than 4 inches a great deal of random variation above and below the average value occurred. Once A.P.I. values greater than 4 inches were exceeded the peaks observed were consistently higher than those estimated by Fig. 2. Thus it can be seen why section 2.4 advocated only a simple correction of 20% for cases where A.P.I. exceeded 4 inches.

3.2. Time distribution of rainstorms. —— Whereas the theoretical basis of this method was developed for an early-peaking design storm, the eighty-three observed values only contained thirty-four such storms, designated Type A. Thirty storms were of a distinctly late-peaking nature, called Type L. A third major group of storms, Type E, commenced with low intensities, rose to high intensities half way through the storm, and then decayed symmetrically to low intensities before ceasing. The effects which these time distributions of storms have upon observed peaks can be seen in a general way from Fig. 5. The radically different influences of the separate storm types can be appreciated by considering the average $\frac{q_o}{q_e}$ ratios which are .91 for the A-type and 1.20 for the

L-type. The line obtained in Fig.5 by adding a 50% safety factor is seen to virtually envelop all but the most severe floods.

3.3. Watershed size. —— The question may arise as to whether the method gives consistant results throughout the full range of watershed sizes. The $\frac{q_o}{q_e}$ ratios of 1.08, 1.0, 0.85, 0.785 which correspond to watersheds of $<\frac{1}{2}$, $\frac{1}{2}$-1, 1-2 and >2 sq. miles, respectively, show that the fluctuations are small. The greatest deviation of 21% exists as a safety factor for catchments greater than 2 sq. miles. No amendment was therefore recommended.

3.4. Very short bursts of most intense rainfall. —— In some of the eighty-three observed storms rainfall intensity over such short periods as 2 minutes averaged rates of up to $10\frac{1}{2}$ inches per hour. It was initially considered likely that such intense pulses of rainfall may produce inordinately high flood peaks when compared to those whose storm intensity never rose above 3 inches per hour. No systematic influence could however be found on observed peaks.

3.5. Watershed shapes. —— It is commonly expected that the shape of a watershed and the conformation of its tributaries influence the size of its floods. For instance a long narrow watershed is considered to produce lower peaks than would arise from an otherwise similar but fan-shaped watershed. Thus it was decided to group the twenty-eight watersheds on which the eighty-three floods were observed into five classes according to shape. The average values of $\frac{q_o}{q_e}$ for these classes varies from 0.67 to 1.08. The variability within each class and the small samples involved precluded the derivation of adjustment factors. A significant field of research appears to lie in studying the interplay between storm- and catchment-types on a much larger sample. Thus far sufficient evidence has been obtained to establish that no serious underestimation will result from applying Fig. 2 regardless of catchment shape.

4. DEVELOPMENT OF METHOD.

Six broad aspects contributed towards developing this new procedure. Firstly mathematical simplifications for a single triangular hydrograph were considered as valid approximations to floods in general. Secondly the resulting algebraic equations were coupled to an empirical expression for storm runoff, which had been derived from observations of forty-seven floods. Thirdly a typical mass curve was assumed to relate the rainfall at any time after the commencement of the storm to the maximum 30-minute rain, P_{30}. Fourthly equations, which resulted from the above reasoning, were evaluated for about 12,000 combinations of values for their five variables. Fifthly the resulting array of 12,000 values of peak rate of runoff was studied in an attempt to eliminate any unimportant variables and to find relationships between the remaining influences. Finally an overall adjustment was made to these theoretical predictions in terms of eighty-three observed flood peaks.

4.1. Peak of S.C.S. triangular hydrograph. —— The theoretical basis of this approach hinges around Eqn. 1.

$$q_{csm} = \frac{484\ W}{0.6B + \frac{D}{2}} \qquad (1)$$

This is obtained from geometrical considerations of the peak value of a triangular approximation to hydrograph shape. Certain observed average relationships between the base lengths of hydrographs and basin characteristics and durations of the causative rainfall pulses are also involved. Provided one knows the amount of storm runoff, W inches, then a prediction of peak rate of runoff q could be made for paired values of B and D.

4.2. Runoff volume. —— A method which is to predict peaks from ungaged watersheds can not depend upon measured runoff volumes. General predictions of W must therefore be obtainable on the basis of infiltration, rainfall and other assessable factors. The following empirical relationship between W and P which was incorporated into the present study had been developed[4] in 1962 after considering thirty-six possible causative factors, for fourteen United States watersheds ranging in size from $\frac{1}{4}$ to 4 square miles :

$$W = .1315 - .5792\,S + .1902\,B + .4261\,P \qquad (2)$$

Substituting Eqn. (2) into Eqn. (1) gives :

$$q_{csm} = \frac{484\,(0.1315 - 0.5792\,S + 0.1902\,B + 0.4261\,P_d)}{0.6\,B + \frac{D}{2}} \qquad (3)$$

This can be evaluated provided one knows : the infiltration capacity S, the basin characteristic B, and the rainfall P_d likely to occur over the duration D. It is now necessary to relate P_d to D.

4.3. Preselected design storm. —— Some characteristic sequence had to be hypothesized regarding the time it took for increasing proportions of the rain to fall. The method developed here for predicting flood peaks from small watersheds is intended for application to areas subject to short convective downpours. These localized rainstorms frequently commence with a high intensity. The major share of the rain generally occurs within the first half-hour. The actual amounts occurring in the early stages of many rains are similar. Differences in the total- or 1 hour - amounts result from one rain continuing at lower intensity for an hour or somewhat longer, while another storm may stop abruptly after a very similar first half-hour. If the rain falling during the most intense half-hour is taken as the common denominator for preparing percentage mass curves, then the important high-intensity portions of many storms will appear closely similar. Previously[7,2] percentage mass curves have always been drawn to rejoin at 100% either on the 60 minute- or the 6 hour-abscissa. Fig. 6 shows how the above deviation from standard practice unifies the percentage behaviour among different storms. The continuous and dotted curves of this figure were derived from Jennings[7] mean mass curves, showing the accumulation of the rain throughout 1 - hour storms.

Extensive studies by Hershfield[1] of the relation between rainfall extremes of 5, 10, 15, 45 and 60-minutes show them to be on an average 0.37-, 0.57-, 0.72-, 1.15- and 1.26- times the 30-minute extreme. Although these so called Hershfield ratios are not identically synonymous to the progression of percentages already discussed in Fig. 6, their plotting on to it as crosses yields interesting results. These Hershfield ratios form an approximate upper envelope to the typical percentage mass curve. They have been used in this theoretical part of the study to obtain the rainfall of 60 minutes and less in terms of the 30-minute extremes, $P_{30\ m}$.

Rainfall extremes for durations longer than one hour are a function of both the 1-hour rainfall and the 24-hour rainfall. A study of available nomographs[8] led to the following multipliers for durations of 1.33, 1.67, 2, 2.5, 3, 4, 6, 9 and 12 hours respectively :- 0.68, .123, .171, .232, .279, .364, .5, .642 and .747. The difference between the 24-hour and 1-hour extremes, times the appropriate multiplier must be added to the 1-hour extreme to estimate the rainfall for the appropriate duration. Since the 1-hour rainfall was established as 1.26 times the 30-minute rainfall it was easy to write a computer program which obtained P_d, the precipitation for any of the fifteen durations, based on $P_{30\ m}$ and $P_{24\ h}$. This was considered to be the design storm for the theoretical portion of this method.

4.4. Interplay of various factors in maximization. — There are four factors which effect the magnitude of the flood peak, q, determined by Eqn. (3). These are S, B, D, and P_d (which in turn is dependent on D and the two rainfall values P_{30m} and $P_{24\ h}$). The interaction and relative importance of these factors was assessed by studying the 12,000 values of q obtained by substituting various combinations of S, B, D, $P_{30\ m}$ and $P_{24\ h}$ into Eqn. (3) and the rest of the program. Seven values of S ranging from 0.02 to 5.0 inches per hour were substituted. All combinations were repeated for B = 0.25, 0.5, 1.0, 1.5, 2, 3 and 4 hours. Durations, D, evaluated by the computer were .25, .5, .75, 1, 1.33, 1.67, 2, 2.5, 3, 4, 6, 8 and 12 hours. Shorter or intermediate durations were used in manual calculations where closer definition was necessary. The pairs of rainfall values used are as follows :-

$P_{30\ m}$: 0.56, 0.56, 0.79, 0.83, 1.07, 1.37, 1.58, 1.58, 1.58, 1.58,
$P_{24\ h}$: 2.00, 3.80, 3.80, 2.00, 3.80, 3.80, 3.20, 3.80, 4.40, 5.60,

$P_{30\ m}$: 1.58, 1.58, 1.87, 2.33, 2.74, 2.77, 3.00, 4.53
$P_{24\ h}$: 7.60, 10.60, 3.80, 5.60, 7.60, 5.60, 10.60, 16.00

If D is regarded as the only factor that may vary then any one of the curves in Fig. 7 shows, for the case of a particular B, how the peaks of single triangular hydrographs (evaluated according to Eqn. (3), etc.) commence to grow in magnitude while longer durations of storms are being considered. A maximum peak is achieved for an optimum duration. Thereafter further increases in D, which appears in the denominator of Eqn. (3), effect a reduction in q because the relative growth of P_d has slowed down according to the latter portion of Fig. 6. A lack of knowledge about this optimum duration has previously impeded the application of single triangular hydrographs. Trial and error solutions could be used. The optimum duration would differ from one case to another, even for the same rainfall regime, because it depends upon B and S. The constraint of optimum duration, which consequently yields the absolute maximum q for a particular set of B, S-, $P_{30\ m}$ - and $P_{24\ h}$ - values is a requirement set in the present study. These maximum flood peaks similar to the five values marked by crosses in Fig. 7, will be designated by q_m. Attention throughout the analysis of these results was restricted to the 882 values of q_m that were thus available.

The first factor which could be discarded as unimportant was $P_{24\ h}$. This could only be done in areas where the ratio between $P_{24\ h}$ and $P_{30\ m}$ was less than 4. This is the reason for demarcating certain areas of inapplicability in Fig. 3. This simplification therefore left the theoretical maximum flood peaks q_m, to be functions of only $P_{30\ m}$, S and B.

4.5. Graphical correlation and overall adjustment. —— Graphical correlation of the 882 maximum flood peaks with the three factors $P_{30\ m}$, S, and B, gave rise to eight sets of graphs, similar to those shown in Fig. 2. They differed from Fig. 2 only by giving the theoretical outcome of the method, q_m, instead of q_e.

Initial tests showed that these theoretical predictions were on an average 1.14 times smaller than eighty-three observed floods. One theoretical simplification employed in the calculation of peaks is suspected of producing consistant under-estimation. In reality the hydrographs for longer durations of rainfall should be obtained by summing the ordinates of two or more separate triangles lagged behind each other. This would lead to a compound peak somewhat higher than obtained from the single triangle of longer base length assumed in this study. The y-axis of Fig. 2 was therefore drawn so that $q_e = 1.14\ q_m$. Under average conditions q_e, the expected flood obtained from Fig. 2, has thus been corrected for latent bias in the method and its assumptions.

4.6. Comparison with a possible alternative. —— The process of maximization could have been applied to Eqn. (1) with the one change being the choice of another means of predicting W. The Soil Conservation Service[2] have popularized the relationship between runoff and rainfall amounts as a family of curves, each corresponding to a particular soil cover complex number. These numbers range downwards from 100, which represents a smooth steel sheet, and are frequently used in practice. It was considered interesting to obtain a comparison between the S values described earlier and these runoff curve numbers, C_n. To do this the calculations involved in preparing Fig. 2. were repeated with W obtained from the Soil Conservation Service runoff curve numbers, instead of from Eqn. (2). The results of a few cases are shown in Fig. 9. The general type of behaviour appears similar to that obtained by the use of infiltration capacities. This figure enables one to cross reference the two means of accounting for runoff producing characteristics of watersheds.

5. CONCLUSIONS.

1. A method has been presented for estimating the magnitude of flood peaks likely to occur on watersheds ranging in size from 1/5 to 5 sq. miles.

2. Estimates can be made over a large part of the continental United States on the basis of the 30-minute rainfall maxima available from published maps for the appropriate return period.

3. The second parameter required for making the estimate has been called the basin characteristic. It depends only on the length of the longest collector and the fall across the watershed which can be readily obtained in practice.

4. The remaining parameter essential to the estimate concerns the influence of soil and cover on infiltration. It can presently be obtained from previously published tables. Considerable scope for refining the method hinges around this factor.

5. Application of the method to eighty-three observed floods shows that the accuracy achievable is satisfactory for many purposes.

6. Special conditions which produce higher observed floods are discussed under the following headings : high antecedent precipitation; various patterns of mass curves; sizes of watersheds; very short bursts of most intense rain; and diverse shapes of watersheds and their drainage tributaries. An addition of 50% to the values given by the design charts for average conditions is shown to adjust for most of these conditions.

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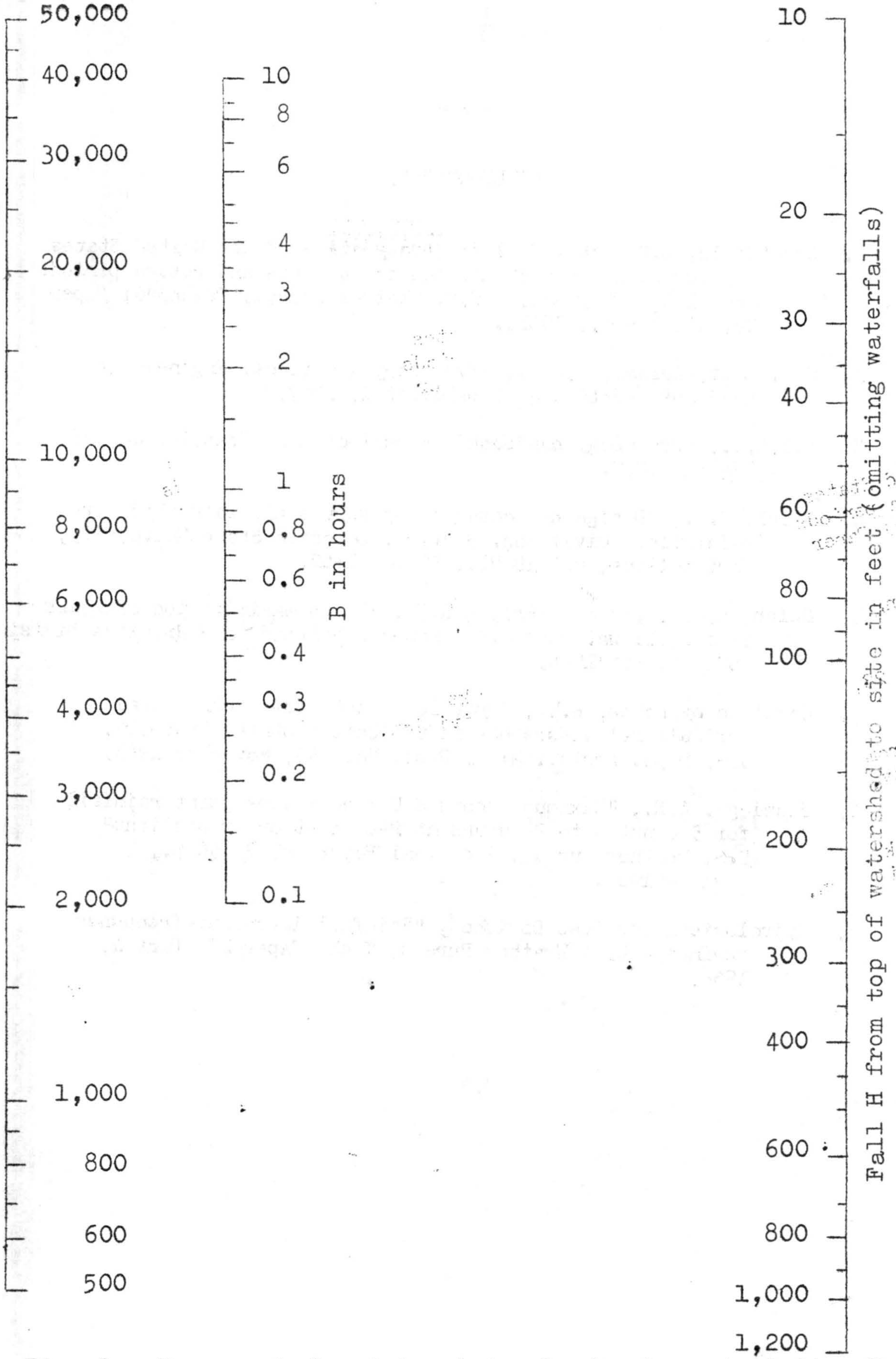
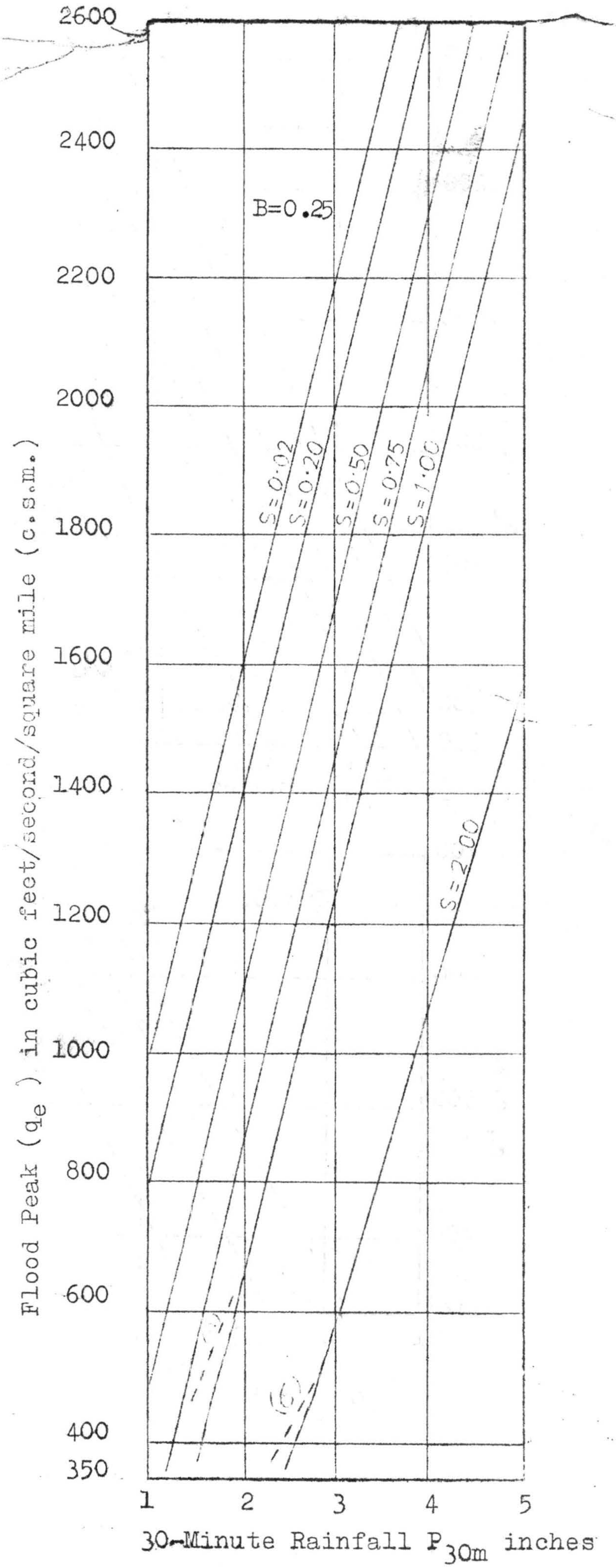


Fig. 1. Nomograph for determining basin characterisitc, B.



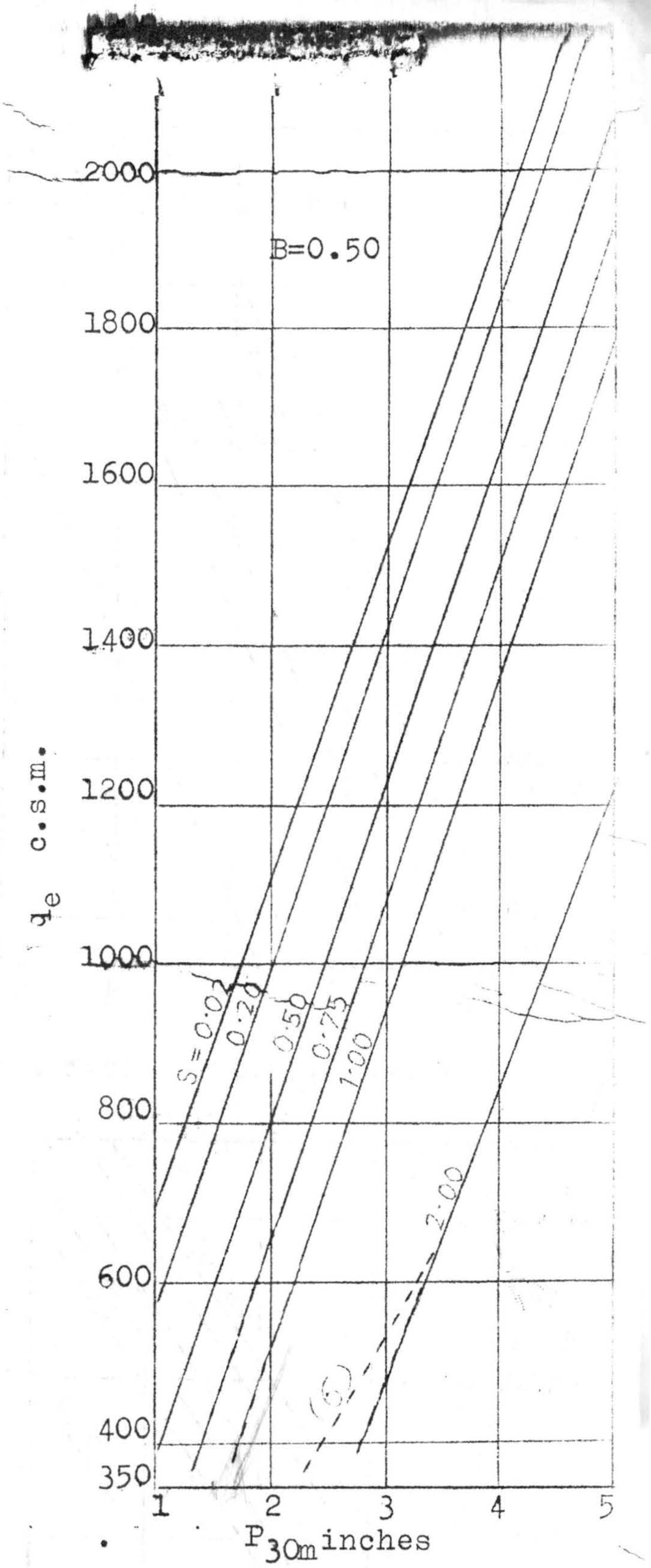
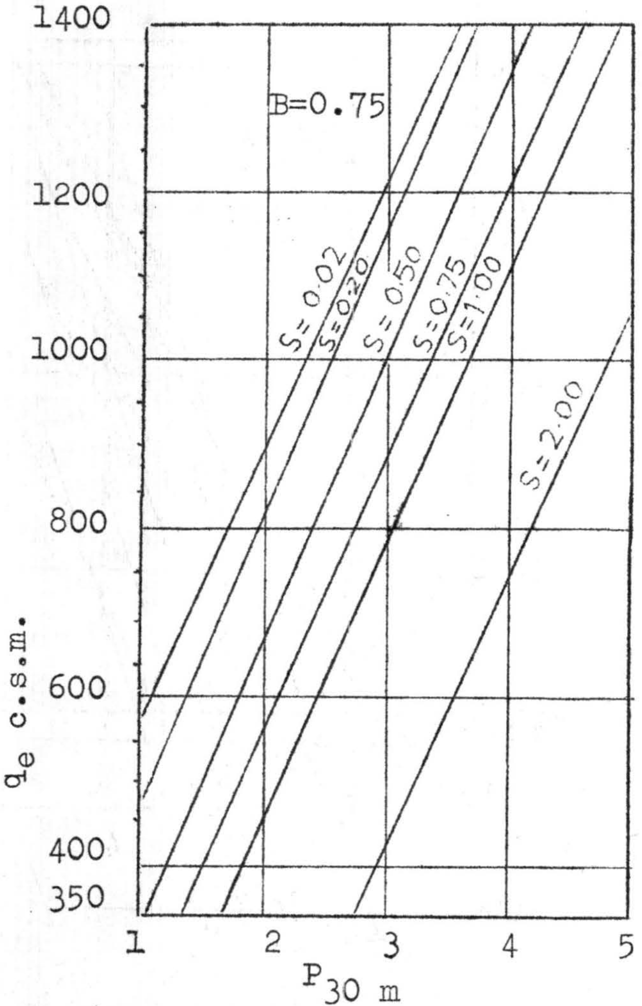


Fig. 2. Design charts for estimating normal flood peaks.



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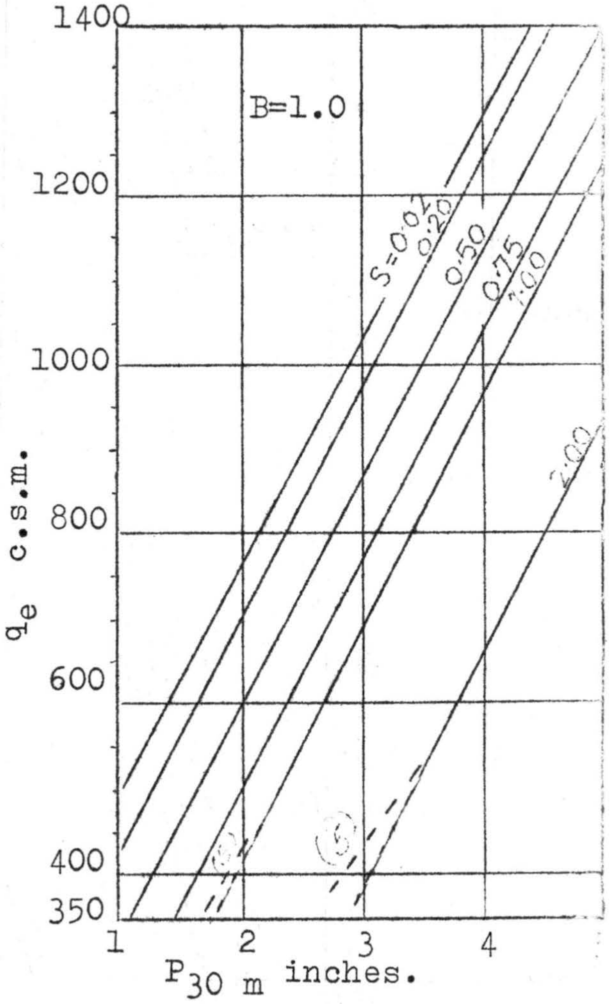
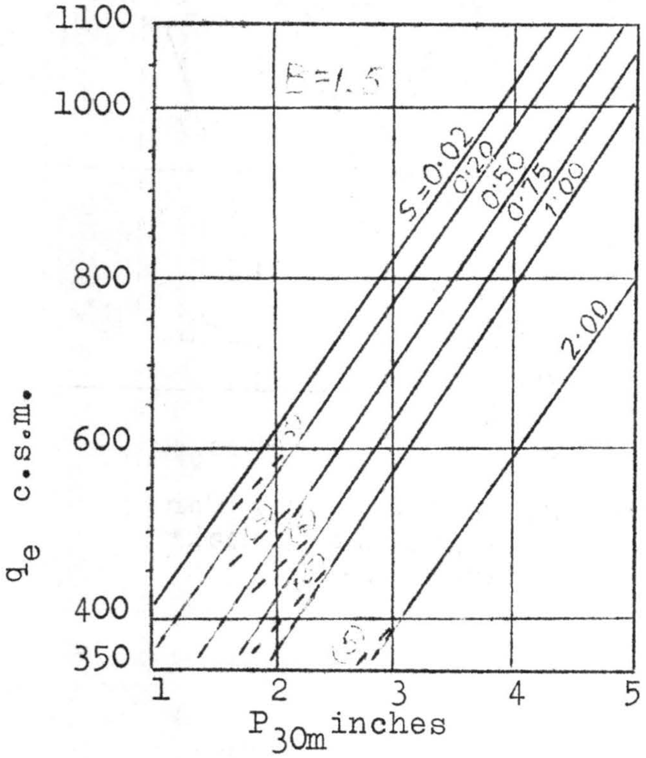




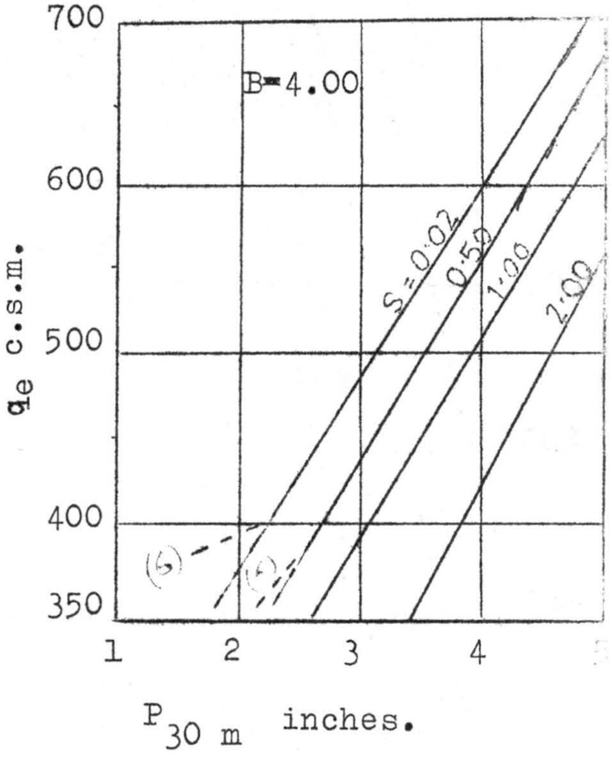
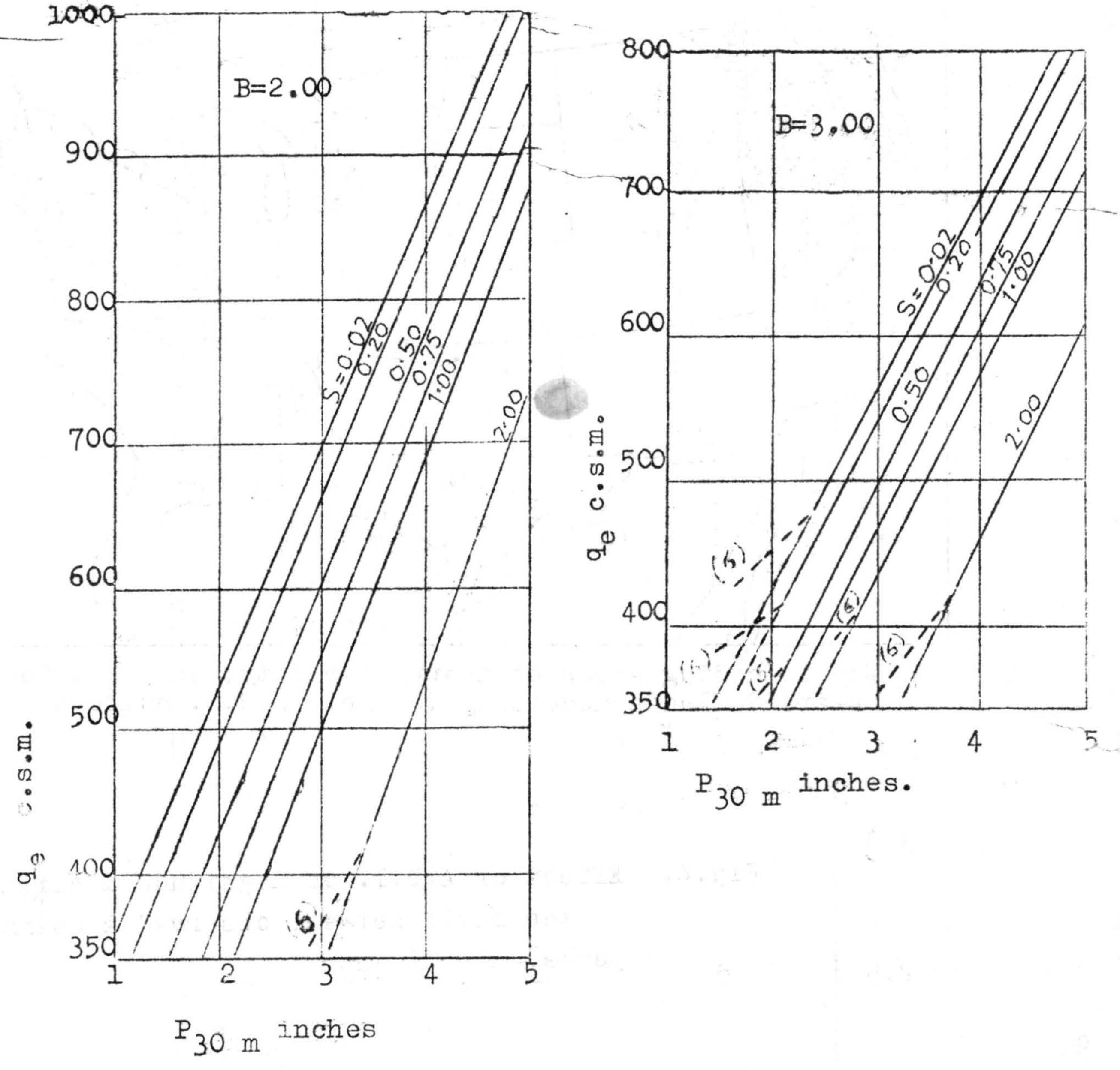


Fig.2. (Continued)



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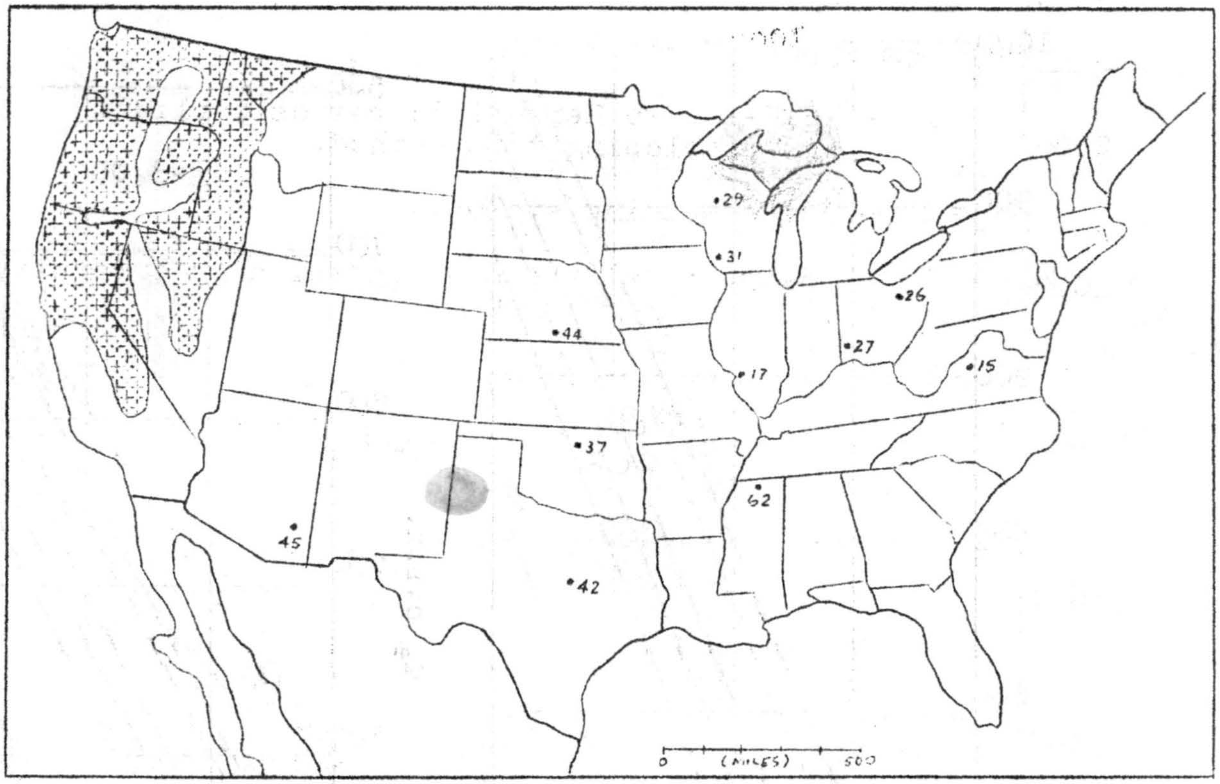


Fig. 3. Map : shading areas of inapplicability; and locating observed floods according to their A.R.S. numbers.

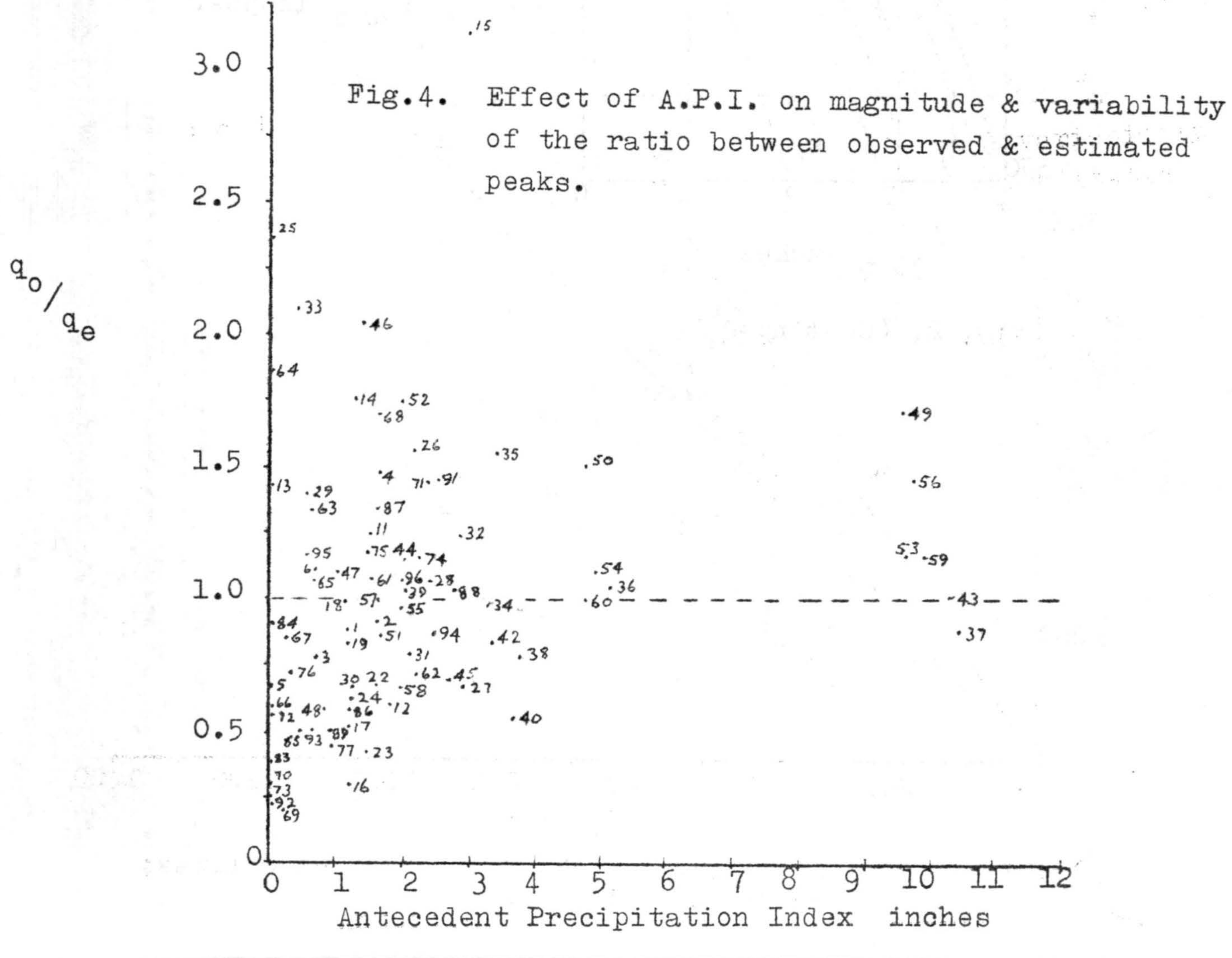


Fig.4. Effect of A.P.I. on magnitude & variability of the ratio between observed & estimated peaks.

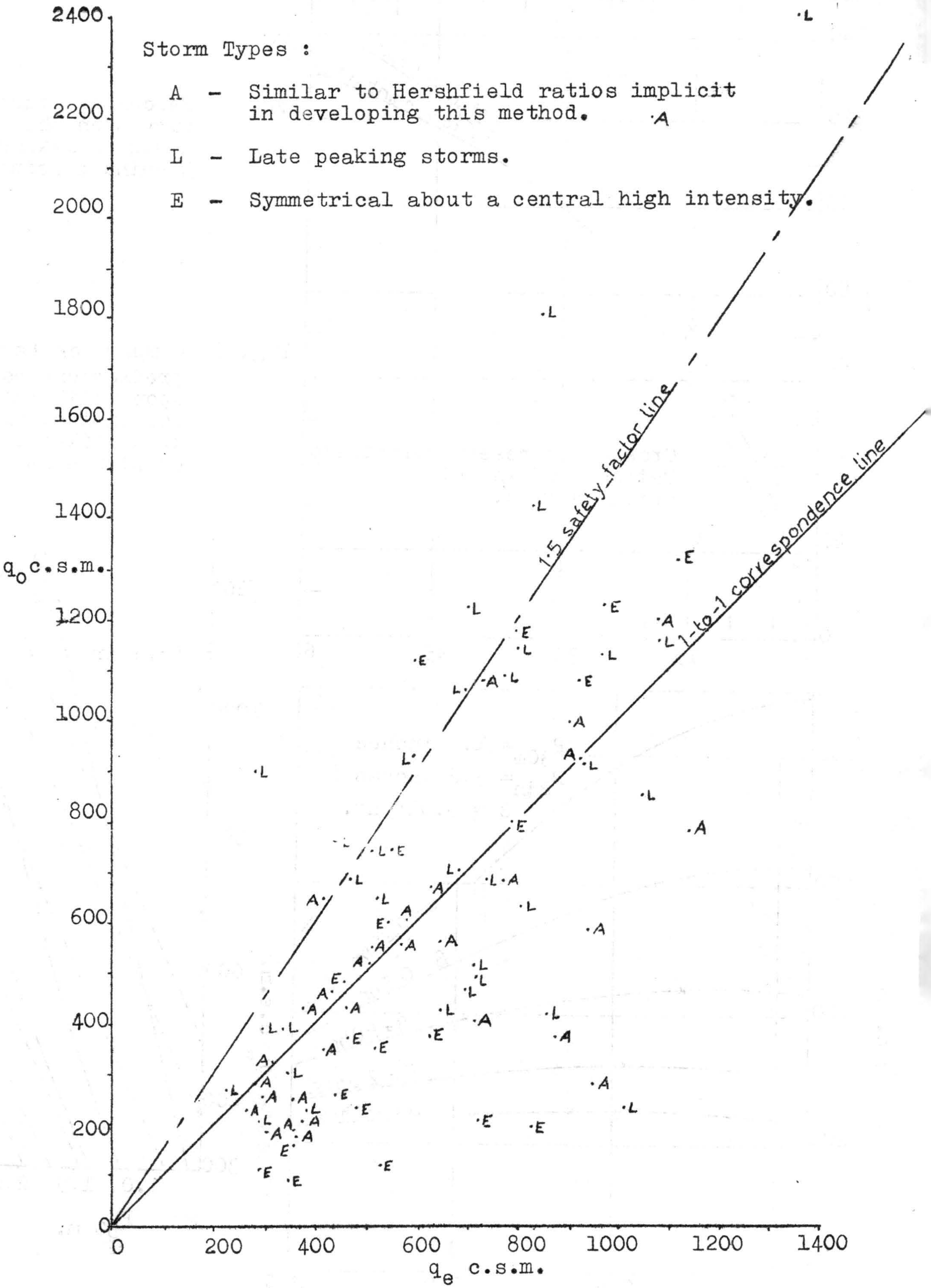


Fig. 5. Scatter of observed peaks about predicted lines; with storm types indicated.

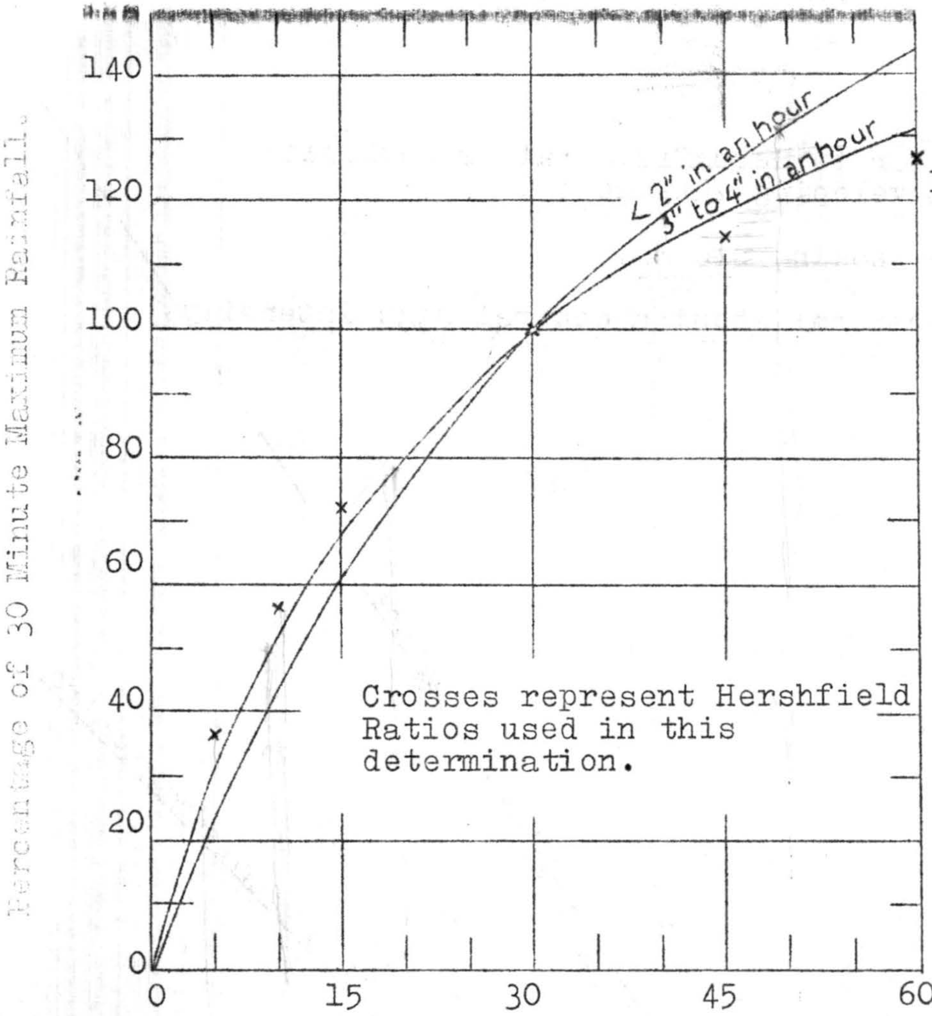
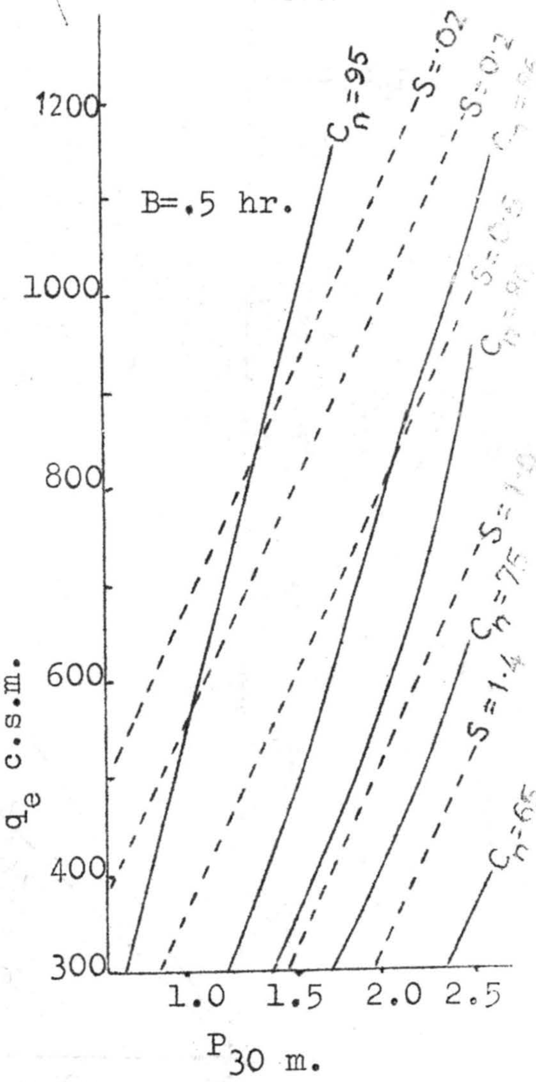


Fig. 6. Percentage mass curves on the basis of maximum 30-minute rainfall.

Fig. 8. Comparison between predictions based upon infiltration capacities, and upon soil-cover complex numbers.



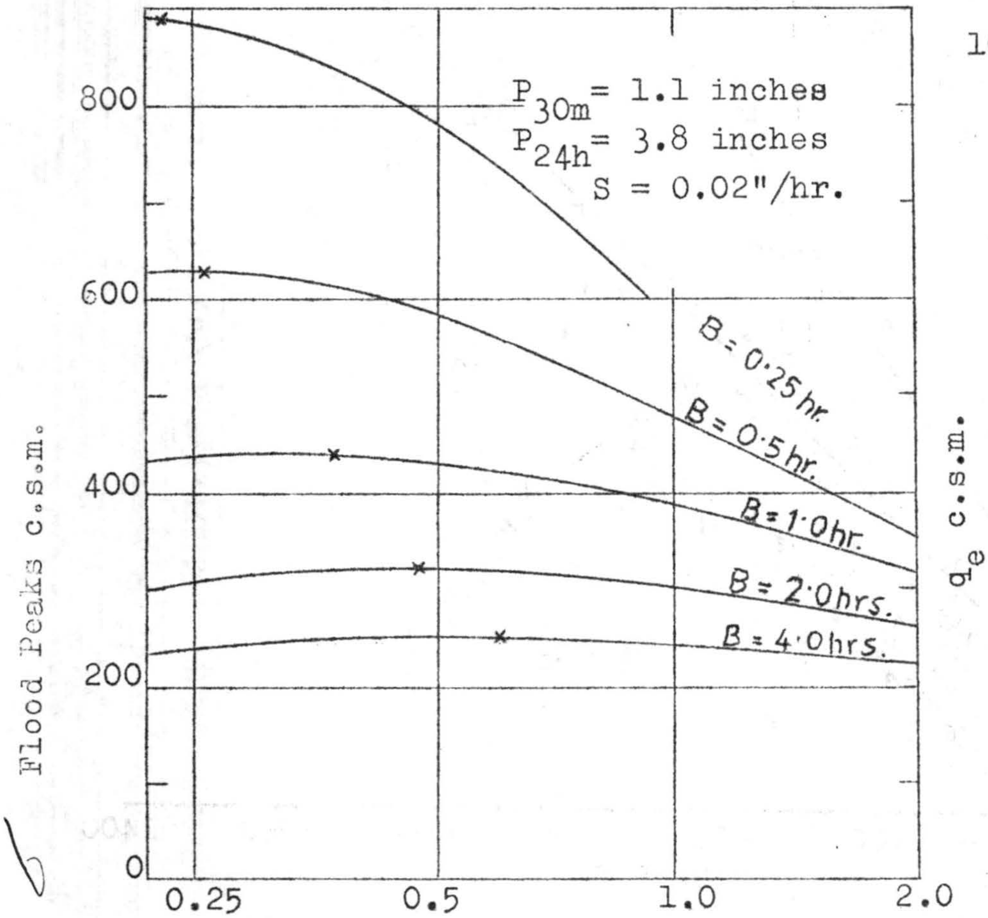


Fig. 7. Variation of flood peaks with D and B.

TABLE 1. VALUES OF f_1 (INFILTRATION CAPACITY SHOWN BY STANDARD CURVES AT TIME 1 HOUR) FOR BARE SOILS

Infiltration Characteristic.	Ranges in f_1 (in inches per hour)
High	0.50 to 1.00
Intermediate	0.10 to 0.50
Low	0.01 to 0.10

TABLE 2. COVER FACTOR, F.

Cover		Range in value of cover factor F.
Type	Condition	
Permanent (forest and grass)	Good	3.0 to 7.5
	Medium	2.0 to 3.0
	Poor	1.2 to 1.4
Close growing crops	Good	2.5 to 3.0
	Medium	1.6 to 2.0
	Poor	1.1 to 1.3
Row crops	Good	1.3 to 1.5
	Medium	1.1 to 1.3
	Poor	1.0 to 1.1

TABLE 3. SOME F-VALUES USED WITH OBSERVED DATA.

Description of Cover Condition	F-value ascribed in this study.
Farmsteads and roads	1.0
Newly planted corn, or small grains	1.0
Idle land (mostly grass and weeds)	1.1
Wheat stubble, or oat stubble, or fallow	1.1
Winter wheat in spring	1.2
9" Corn plus weeds	1.2
Cotton in early fruiting stage, or bloom stage	1.2
Winter wheat in the summer	1.3
48" Corn plus 30" weeds	1.3
Rotation pasture	1.4
Row sudan 10" high	1.4
Brush timber	1.4
Conservation cotton lands	1.4
Conservation croplands; corn or sorghum	1.5
Small grains fairly soon after planting	1.7
Small grains later	1.8
Broadcast sorghum	1.8
Bermuda grass and weeds (fair cover)	1.8
Bermuda grass and weeds (good cover)	2.0
18" Alfalfa; or broadcast sweetclover	2.0
Pastured woodlands; or Hay fields	2.0
Permanent pasture, or woodland	2.5
Conservation small grains	2.6
Meadow	3.0
Conservation grassland, or conservation pasture	3.0
Conservation (or dense growth) meadow	3.5
Conservation woodland	4.0
Clover	5.2

TABLE 4. WATERSHED DATA.

1	2	3	4	5	6	7	8	9
A.R.S. Watershed Number	Location	Date of Event	Watershed shape Symbol	Basin characteristic	Soil & Cover characteristics.			Event No.
					f_1	F	S	
15.1	Staunton, Virginia	13 April 49	X	0.50	.30	2.15	1.08	1
17.4	Edwardsville Ill.	27 May 38	V	0.53	.05	1.50	0.07	2
		21 June 42				1.50	0.07	3
		31 March 52				1.50	0.07	4
		31 March 52				1.50	0.07	5
		2 July 52				1.50	0.07	6
26.30	Coshocton Ohio	23 Sept. 45	V	0.27	.20	2.30	0.46	91
		16 June 46			.20	2.10	0.42	11
		16 Aug. 47			.20	1.90	0.38	12
		1 Sept. 50			.20	2.30	0.46	13
		12 June 57			.20	2.40	0.48	14
		28 June 57			.20	2.40	0.48	15
26.32		12 June 57	VY	0.14	.20	2.16	0.43	16
26.33		12 June 57	Y	0.24	.35	2.68	0.93	17
26.34		12 June 57	V	0.75	.35	2.75	0.96	18
26.35		12 June 57	V	[illegible]00	.35	2.89	1.01	19
27.1	Hamilton Ohio	7 July 43	VX	0.28	.30	2.04	0.61	22
29.1	Colby Wisc.	4 June 58	X	0.63	.03	2.06	0.06	23
31.1	Fennimore Wisc.	12 Aug. 43		0.45	.25	1.85	0.46	24
		11 July 44				1.85	0.46	25
		28 June 45				1.79	0.45	26
		24 June 49				1.77	0.44	27
		15 July 50				1.81	0.45	28
		5 Aug. 51				1.88	0.47	29
31.4		12 Aug. 43	VY	0.26	.25	1.75	0.44	30
		11 July 44				1.77	0.44	92
		28 June 45				1.70	0.42	31
		24 June 49				1.67	0.42	32
		15 July 50				1.81	0.45	93
		15 July 50				1.81	0.45	94
		6 Aug. 51				1.86	0.46	95
37.3	Stillwater Okl.	18 April 57	U	0.58	.05	1.20	0.06	33
		27 June 57				1.30	0.07	34
		2 Oct. 59				1.40	0.07	35
		2 Oct. 59				1.40	0.07	36
		2. Oct. 59				1.40	0.07	96
42.2	Riesel (Waco) Tex.	24 April 57	U	0.80	.05	1.66	0.08	37
		13 May 57				1.67	0.08	38
42.3		10 June 41	U	1.20	.05	1.49	0.08	39
		15 June 42			.05	1.54	0.08	40
		15 July 50				1.86	0.09	42
		24 April 57				1.93	0.10	43
		3 May 57				1.92	0.10	44
42.4		23 June 59	Y	2.5	.05	1.62	0.08	45

1	2	3	4	5	6	7	8	9
A.R.S. Watershed Number	Location	Date of Event	Watershed shape Symbol	Basin characteristic	Soil & Cover characteristic			Event No.
					f_1	F	S	
42.6	Riesel (Waco) Texas.	10 June 41	V	0.50	.05	1.32	0.07	46
		26 March 46				1.32	0.07	47
		27 April 49				1.48	0.07	48
		24 April 57				1.33	0.07	49
		13 May 57				1.42	0.07	50
		4 June 57				1.30	0.07	51
		23 June 59				1.33	0.07	52
42.7		24 April 57	U	0.28	.05	1.82	0.09	53
		13 May 57				1.90	0.10	54
		23 June 59				1.95	0.10	55
42.11		24 April 57	Y	0.47	.05	2.05	0.10	56
		4 June 57				1.97	0.10	57
		23 June 59				2.04	0.10	58
42.12		24 April 57	Y	0.38	.05	2.20	0.11	59
		13 May 57				2.30	0.11	60
		4 June 57				2.50	0.12	61
		23 June 59				2.28	0.11	62
44.1	Hastings Neb.	20 June 39	V	0.74	.20	1.70	0.34	63
		10 July 51				1.70	0.34	64
		7 June 53				1.30	0.26	65
		22 April 57				1.19	0.24	66
		1 May 57				1.35	0.27	67
		15 June 57				1.54	0.31	68
		12 June 57				1.30	0.26	69
44.2		12 June 58	V	0.63	.20	2.10	0.42	70
		3 July 59				2.34	0.47	71
44.3		10 July 51	XYZ	2.40	.20	1.63	0.33	72
		12 June 58				1.67	0.33	73
		3 July 59				2.18	0.44	74
44.4		15 June 57	YZ	3.25	.20	1.65	0.34	75
45.1	Safford Ariz.	26 July 57	XU	1.00	.40	1.03	0.41	76
		3 Aug. 59				1.03	0.41	77
45.3		28 July 58	X	1.20	.50	1.04	0.52	83
		16 Aug. 58				1.04	0.52	84
45.4		30 Aug. 57	UX	0.90	.50	1.04	0.52	85
62.1	Oxford Miss.	9 Sept. 59	VU	1.20	.30	1.91	0.57	86
62.2		10 June 59	V	1.00	.30	1.93	0.58	87
		11 June 59				1.93	0.58	88
62.6		4 June 57	V	0.40	.30	1.83	0.55	89