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Estimation of maximum discharge for the design of hydraulic structures

BY

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ESTIMATION OF MAXIMUM DISCHARGE FOR THE DESIGN OF HYDRAULIC STRUCTURES

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1.0 An estimation of the maximum discharge is of fundamental importance in the design of any large hydraulic structure. The safe and economical design of the structures depends on the correct estimation of the maximum discharge and maximum and minimum water levels. An over estimation of maximum design discharge will increase the cost of the structure considerably due to larger waterway required. The Sukkur barrage designed for 1500,000 cfs had never experienced more than 1200,000 cfs in the last fifty years. The engineers realising that the waterway is more than needed, closed 10 days permanently to evolve a scheme for controlling excessive sediment entry into the right bank canals. If however the designed maximum discharge is understimated, then every time the flood discharge exceeds the design capacity, the structure can be in danger. Obviously the design engineer is mainly concerned with the determination of highest flood likely to occur for a given catchement and the frequency of normal and maximum floods.

1.1 ESTIMATION OF MAXIMUM FLOOD DISCHARGE FOR DESIGN OF A HYDRAULIC STRUCTURE

In Indo-Pakistan sub-continent the perennial rivers or streams have a base flow due to normal supply from springs and lakes and snowmelt on high mountains during the winter season. These and other perennial sources such as regeneration establish the sustained dry weather flow.

In Spring when the temperature begins to rise, the melting process of accumulated snow in the upper catchement is accelerated resulting in slow and progressive increase in flow over and above the sustained low-winter flow. The incidence of occasional rainfall, which apart from its own contribution in increasing the snowmelt due to relatively warmer rain water impinging on snow clad mountains, mainly cause freshets. The incidence of heavy intense and wide spread rainfall in the upper catchments during the monsoon season from July to September is generally the primary cause of high devastating floods. The average monsoon and annual rainfall for Pakistan (Figs. la & b) show that the excessive rainfall occurs in the hilly catchment and Sub-mountainous areas which mainly contribute to the runoff. The rainfall decreases as the rivers flow down into the alluvial plains and arid zones.

For hydraulic structures such as culverts, syphons, causeways and road bridges on flood and drainage channels and smaller streams and torrents with catchment areas, the perennial flow may not be due to snow melt but valley drainage and regeneration. The peak floods in such areas will normally depend on the drainage area, its shape and nature, the rainfall intensity and its extent.

1.2 FACTORS AFFECTING THE FLOOD MAGNITUDE

The factors which contribute to cause a flood of abnormal magnitude are so many and so varied and interdependent that it is physicallly impossible to deal with all of them. The main factors affecting the flood magnitude are the size of the catchment and the intensity of rainfall in the catchment. Considering the catchment as the only principal factor, the charcteristics which determine the magnitude of flood run-off are, the area, the shape and the slope of the catchment. To a great extent, the runoff also depends on the nature of soil in the catchment, its permeability, the vegetable cover, the initial state of wettness and

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the river valley storage. Taking rainfall as the factor contributing to the runoff, we have to consider the intensity of rainfall, its aeral distribution and duration in the catchment. Let us consider a hyprothetical stream of catchment shown in Fig. 2.

Assuming the rainfall to be continuous and nuiformly distributed over the entire catchment area upto a particular point B, where Q_{max} is to be estimated, if the duration of the rainfall is long, the discharge at the point B will continuously increase and will attain the peak value when the water from the farthest upstream point of the catchment reaches the point B. The time taken for the water from the farthest end to reach point B is known as the period of concentration. Obviously under supposed condition of uniform rainfall intensity over the catchment, the greater the size of the catchment area the greater will be the magnitude of the peak discharge. Thus for a storm of given rainfall intensity uniformly spread over the catchment area, the magnitude of Q_{max} should increase with the size of the catchment area. Moreover smaller the ratio of length for a given width of the catchment and greater the slope of catchment, the greater will be the peak discharge. Again a decrease in permeability of soil non-existence of vegetable cover, and a smaller valley storage will tend to increase the peak discharge and vice-versa. If a storm having as high an intensity as possible is uniformly distributed over the entire catchment area or over as great a part of it as possible, then if the duration of the storm tends to equal the concentration period, the discharge experienced at the point under consideration will tend to approach Q_{max} and vice-versa. Obviously some of the above factors make positive and others negative contribution to the realisation of Q_{max}. In fact it is the simultaneous chance occurrence of as large a number of factors controlling the entire phenomenon of rainfall runoff occurring in time, space and sequence in a way that all or as great a number of factors as possible make positive contribution that we get a record flood.

1.3 METHODS OF COMPUTATION OF Q_{max}

The method to be used preferably for estimation of Q_{max} for the design of a hydraulic structure depends on the importance of the structure. The availability of hydraulic the hydrologic and meteorological record of the data for the stream and its catchment characteristics may generally indicate the most appropriate method or methods for the computation of Q_{max} . The methods can be generally classified as under:

1) Estimation of Q_{max} from catchment area
2) Estimation of Q_{max} from catchment area and rainfall data.
3) Unit hydrograph method.
4) Estimation of Q_{max} from flood record by flood frequency analysis and probability method.
5) Knowing the highest flood level, computation of Q_{max} from flow formulae.

The above methods are discussed, giving the scope and limitation of each.

1.4 ESTIMATION OF Q_{max} FROM CATCHMENT AREA

A large number of empirical formulae based on analysis of records of maximum floods for rivers having catchment areas of particular characteristics and hydrometeorological conditions have been proposed and extensively used. In these formulae the maximum intensity of flood (Q_{max}) measured in cfs is related only to the catchment area M in squre miles. The proposed formulae have the general form:

$$Q_{max} = CM^{n}$$

where C is a coefficient depending on units and basin characteristics and n is an exponent generally varying in different formulae from 0.5 to 0.8. It has to be noted that the approach being purely empirical, only restricted and catious use of the formulae for relatively small catchment areas and small hydraulic structures such as culverts, syphons, aquaducts, etc. is justified. The coefficient recommended by the authorities proposing the formula vary with the type, size and hydrometeorological conditions of the catchment and the coefficient selected will be correctly applicable only if the catchment area under consideration has more or less similar characteristics and hydrometeorological conditions for which the formula is derived. Some of the more important catchment formulae are discussed below;

1.5.1 DICKEN'S FORMULA

From flood records of Northern India, Dickens(1) correlated the maximum flood intensity with catchment area by the formula in river basin having rainfall variation between 24" to 50" by:

$$Q_{max} = CM^{3/4} \quad \text{------------- } 1$$

where Q_{max} is the flood intensity in cubic feet per second and 'M' is the catchment area in square miles.

The recommended values of C generally vary between 500 to 825. It has been found that the smaller values are applicable to less rainfall areas and high values to vary high rainfall and runoff areas. For Western Ghats of India where the rainfall is very heavy, a value of C as high as 1600 is applicable. For central provinces in India a value between 1000- 1400 is used. The Highway code of practice (2) in Pakistan recommended C= 750 for mountainous areas and 500 for plains.

1.5.2 RYVES FORMULA

Ryves(3) working from Madras presidency streams proposed the formula

$$Q_{max} = CM^{2/3} \quad \text{------------}2$$

and recommended the following values of C

C = 450 for areas within 15 miles of coast

C = 563 for areas between 15 to 100 miles from the coast.

C = 673 for limited areas near the hill.

Pakistan Highway Corde gives C = 500 for mountainous areas and 350 for plains. The variations of the value of coefficient C are adjusted to take care of the changes in the catchment location and rainfall characteristics from place to place.

1.5.3 FANNING'S FORMULA

For American streams, Fannings(4) obtained the formula

$$Q_{max} = 200\, M^{5/6} \quad \text{----------}3$$

1.5.4 MYERS FORMULA

Myers(5) modified formulae for frequent and 100% maximum flood are:

$$Q_{max} = 5000\, pM^{1/2} \text{ (for frequent floods)...........4(a)}$$

and $Q_{max} = 10{,}000\ pM^{1/2}$ (for 100% maximum flood)........4(b)

where p is Myers rating coefficient showing position on percentage scale as given by

$$P = \frac{Q_{max}}{100\sqrt{M}} = \frac{q\sqrt{M}}{100} \quad ....4(C)$$

where Q_{max} is the maximum discharge

and $q = Q_{max}/M$ (discharge (cfs) per square mile of the catchment)

M = catchment area in square miles.

It has been recommended that for drainage area of 4 sq. miles and less only the first power of M should be used or

$Q_{max} = qM = 50\ pM$

or for rating of 100 per cent

$Q = qM = 500\ pM$

The values fo Meyers rating coefficients for different sites in U.S.A. are recorded by Davis in Hand Book of Hydraulics. The values of Myers rating coefficient are calculated by the author from known Q_{max} values and drainage areas for different observation sites on Rivers and streams of Indo-Pakistan Sub-continent and the data are recorded in Table 1.

1.5.5 INGLIS FAN-CATCHMENT FORMULA

Inglis(6) from study of record of catastrophic floods in rivers and streams of Northern, Central and Southern India proposed the formula;

$$Q_{max} = \frac{700\ M}{\sqrt{M+4}} \quad ....(5)$$

The data plotted by Inglis is reproduced in Fig. 3.

1.5.6 KUICHLING FORMULA

For frequent and rare floods in New York State, Kuichling(7) proposed the following to formulae:

$$Q = \left(\frac{44000}{M+170} + 20\right) M \text{ (frequent floods) ----- 6(a)}$$

$$Q = \left(\frac{127000}{M+370} + 7.4\right) M \text{ (rare floods) ------ 6(b)}$$

1.5.7 KARPOV KANWARSAIN CURVE

Karpov and Kanwarsain plotted Q_{max} ever recorded for thirty one rivers of Indo-Pakistan Sub-continent against the drainage area on a logarithmic paper. A curve enveloping all the points is shown in Fig. 4.

TABLE - I

Sr. No.	River	Gauging Site	M Drainage area sq. miles	Qmax cfs	Myers Rating %
1.	2.	3.	4.	5.	6.
1.	Ganga	Sera Bge	380000	2500,000	40.5
2.	Ganga	Hardwar	9500	840,000	86.3
3.	Godavari	Dowlalsh waram	130000	2000,000	55.4
4.	Jamuna	Allahabad	140000	1600,000	42.8
5.	Jamuna	Frozabad	4500	550,000	74.6
6.	Mahanadi	Naraj Weir	53000	1500,000	65.2
7.	Mahanadi	Sambaipur	32000	1300,000	72.6
8.	Sone	Dehri	25000	1200,000	76
9.	Tapti	Crossing of Hills	22000	1100,000	74.3
10.	Damodar	Raneegunj	7500	700,000	80.8
11.	Damodar	Barrakur	4300	270,000	41.2
12.	Narbada	Brooch	36000	110,000	5.8
13.	Poonch		1800	490,000	116
14.	Indus	Partab Bridge	57500	437,000	18.2
15.	Indus	Mandori	102000	823,000	25.8
16.	Indus	Kalabagh	103800	950,000	29.5
17.	Indus	Taunsa	136900	750,000	20.2
18.	Chenab	Mangla	12610	1100,000	98
19.	Chenab	Chiniot	26078	800,000	37
20.	Chenab	Marala	12610	718,000	64
21.	Chenab	Rivaz Bridg	17961	650,086	48.5
22.	Ravi	Meshopur	3000	630,000	114,9
23.	Ravi	Jasaar	3535	680,000	114
24.	Gilgit	Gilagit	4670	53,400	
25.	Siran	Thapla	1080	56,800	17.3
26.	Chitral	Chitral	4400	37,200	5.6
27.	Kabul	Nowshera	34200	223,000	12.1
28.	Swat	Kalam	780	18,400	6.6
29.	Swat	Chakdura	2230	35,400	7.5
30.	Haro	Khanpur	34200	223,000	11.1
31.	Soan	Rawalpindi	650	93,700	36.8
32.	Soan	Dhok Pathan	2500	178,000	35.6
33.	Sill	Chahan	93	20,000	20.8

THE CALCULATION OF Q_{max} FROM AREA AND RAINFALL DATA

1.5.8 CHAMIERS FORMULA

Chamier(8) proposed the following formula taking average rainfall of the area also into consideration

$$Q = i\ KM^{3/4}$$

where Q is the peak discharge in cfs, i is the average rate of rainfall in inches per hour for duration equal to time of concentration T_c which is the time taken by water to travel from the farthest point of the catchment to the poit of measurement.

K is a rainfall coefficient and M is the catchment area in square miles.

1.5.9 CRAIG'S FORMULA

Craig(9) has taken length and width of the catchment area into consideration and proposed from the analysis of stream data in India the following formula:

$$Q = 440\ WN \log_e (8\ L^2/W) \qquad \text{------- } 8$$

Where W is the average width of the catchment in miles L is the longest length of the catchment to the point under consideration in miles

and $N = KVi$

where v is the velocity of runoff (ft/sec), K is runoff coefficient and i is the average intensity of rainfall in inches per hour. In formula 8, N is a coefficient peculiar to a catchment and varies from 0.68 to 1.95.

1.6.0 GENERAL DISCUSSION OF THE FORMULAE

The formulae generally take only one variable, the catchment area into account, the other factors are suppressed. For assumed values of catchment areas between 10 and 100,00 square miles, the values of Q_{max} calculated by different formula are plotted logarithmically in Fig. 5.

It can be seen that for the same value catchment the calculated values of Q_{max} will differ very widely. The formulae, are in fact only applicable to the cathments whose characteristics excepting the area, are similar to catchments for which the formula is derived. For the maximum discharge intensity to occur the storm should cover an area equal to that of the catchment and the duration of the storm should be equal to the period of concentration. Only under such a condition the entire catchment will be contributing to produce the maximum flood intensity. However, when the catchment is very large, it cannot be covered entirely by a storm of uniform intensity. From meteorological conditions we know that the storms are of limited size. Actually during the storm the rainfall intensity at anyone point varies with time, generally rising to a maximum and then falling, the maximum intensity being attained when the centre of the storm is over the measuring point. The study of the distribution of the rainfall intensity in the storm area shows that there is a small area of maxim rainfall intensity and the rainfall gradually decreases as we move away from the centre of the storm. Obviously, these conditions for a storm of uniform intensity and long duration over the entire catchment can only be achieved for small catchment area. The committee on hydrology of the American Society of Civil Engineers(10) recommended that discharge drainage area formulae should not be used for catchment areas greater than 2000 square miles. It is stated again that these formulae give a rough indication and not a reliable result on which design of any large hydraulic structure can safely be based without complementary methods of computing Q_{max} described later.

1.7.0 THE RATIONAL FORMULA

The second category of formulae are those which involve the rainvall as a factor in addition to the catchment area. This is more rational approach and the general from of the rational formula is

$$Q = KiA \quad \text{------}9$$

where A is the drainage area in acres and Q is the peak discharge in cfs when the whole catchment is contributing, i is the estimated average rainfall intensity in inches per hour for a period equal to the concentration time T_c for the catchment. In the formula K is the coefficient depending on infiltration, retention and runoff characteristics of the basin. The runoff coefficient K can be expressed as

$$K = \frac{i-f}{i} \quad \text{-----}\ 9(a)$$

Where i has the meaning as defined above and f denotes all detention clements expressed in the same units as rainfall intensity. Obviously K should be less than unity. The greater the retention of the catchment, the lesser will be the value of K. It really indicates what fraction of rainfall would appear as runoff, to produce a flood peak. The values of coefficient K are excellently summarised in a book on Low Dams.(11) The values of K suggested for use by Richards are tabulated in Table -II.

Table -II

Type of Catchment		Values of K for	
		Large catchment area	Small and Steep Catchment areas
1)	Rocky and impermeable	0.8	1.0
2)	Slightly permeable, bare	0.6	0.8
3)	Slightly permeable partly cultivated or covered with vegetation	0.4	0.6
4)	Cultivated, permeable soil	0.3	0.4
5)	Sandy very permeable soil	0.2	0.3
6)	Heavy forest	0.1	0.2

Based on the rational approach for computing Q_{max}' using both the catchment area and its runoff characteristics and the rainfall intensity, two methods are available.

1) Richard's method
2) The Unit hydrograph method

The methods are briefly presented and their use is illustrated by examples.

1.7.1 RICHARD'S METHOD

Richard[12] proposed relations which include coefficients for some of the major factors. In a foregoing discussion it has been stated that the intensity of storm at any point in the catchment varies with time as well as area. Taking note of this fact, obviously the average intensity of storm, I at the point of maximum rainfall is an inverse function of the storm duration, that is if the storm lasts for longer duration, the rainfall intensity I will decrease. From a number of investigations it has been shown that rainfall can be expressed as a function of time by the relation

$$I = \frac{R}{T+C} \quad \text{-------- } 10$$

where I denotes the average rainfall intensity in inches per hour during the storm period at the point of maximum rainfall

T is the duration of the storm in hours

R is the rainfall coefficient

If unit of time is taken in hours

C is a constant which is found to vary between 1/4 to 1.

If F represents the total rainfall in inches, then

$$I = \frac{F}{T} = \frac{R}{T+C} \quad \text{------ } 11$$

Taking C = 1

$$I = \frac{R}{T+1}$$

The rainfall coefficient R can be expressed as

$$F = F\left(\frac{T+1}{t}\right) \text{....... } 11(a)$$

The coefficient R thus depends on the total rainfall in inches and its duration. The maximum values of R for British Isles are reported to be about 10, for U.S.A. about 20 and for India and Pakistan about 30. In a storm the average rainfall intensity i over an area A in relation to the maximum rainfall intensity I at a point occuring over a small area at the centre of the storm varies as an inverse function of the area of the storm

or $i = I \times f(A)$ ------- 11(b)

here i is the average rainfall intensity for the storm duration over the given area.

Richard considered a catchment of unit width and length L. He further assumed that a storm of uniform rainfall intensity i is spread equally over the entire catchment area and that the duration of the storm T is equal to the period of concentration (t), or (T=t) upto the point at which Q_{max} is to be found.

Now if the slope of the catchment is also taken to be uniform, applying Chezy's flow formula

$$v = c\sqrt{rS} \quad = c\sqrt{DS}$$

to the catchment runoff, Richard[12] by using the rational formula 9 obtained the following equation connecting the period of concentration t in hours for the basin with the factor representing the catchment characteristics

$$\frac{t^3}{t+1} = \frac{C'L^2}{K.S.R.f(A)} \qquad \text{-------}12$$

In this formula R is rainfall coefficient, K denotes the runoff coefficient and C' is a coefficient $9/4c^2$ where C is a coefficient in Chezy's flow formula and according to flow formulae itself a function of $\sqrt{r}$ and channel roughness C being a numerical constant to suit the units chosen. L and S represent the length and slope of the basin.

Using t, the time of concentration for the basin in place of the storm duration T, since t = T, the equation 11(b) can be written as:

$$i = \frac{R.f(A)}{t+1} \qquad \text{------}13$$

Q_{max} the maximum intensity of flood in cfs per 1000 acres is given by

$$Q_{max} = 1000\,Ki \qquad \text{-------}\ 14$$

In relation 13. R denotes the coefficient of rainfall and K is the runoff coefficient the other parameters have the same significance as explained above.

1.7.2 DETERMINATION OF COEFFICIENTS AND CONSTANTS

To find Q_{max} from equation 14, the average intensity of rainfall over the catchment in inches per hour, i, and K the runoff coefficient have to be determined. For determining i, the appropriate values of R, f(A) and t in formula 13 have also to be determined

$$\text{For } f(A),\ i/I = \frac{\text{average rainfall intensity}}{\text{maximum rainfall intensity}}$$

Curves have been drawn between, f (A) and the catchment area, A, in acres x 1000 in fig. 6(a) and against sq. miles x 1000 sq miles in Fig. 6(b). From these figures knowing catchment area, A in acres we can suitably fix f (A). Again knowing the length of the catchment L, and its general slope S, the values of t can be found from formula 12 provided the values of C, K and R are suitably selected.

Rainfall coefficient, R

The coefficient of rainfall R can be determined from rainfall records of the area. If the total rainfall is F inches and time T of the storm is equal to the concentration period of the basin t(or t=T) then R can be determined from formula 11.

Runoff Coefficient K

K is the coefficient of runoff and dentoes the percentage of rainfall that runs off as a surface flow, (I-K) being the water that is temporarily or permanently lost. The values of K for different types of catchment as summarised by Richard are given in Table II.

Coefficient C′

In the derivation of formula 12 the coefficient C is put as

$$C' = 9/4C^2$$

where C′ = is a constant to suit the units chosen and C is a coefficient in Chezy's flow formula

$v = C\sqrt{rS}$ where according to Basin $c = 137.6/ 1+N\sqrt{r}$

It has been shown by Richard that C can be correlated with KR by a curve reproduced in Fig. 7, and corresponding values of KR and C' are given in Table III

Table III

K.R.	C′
0.6	0.0365
1.2	0.0216
2.4	0.0137
4.8	0.0096
7.2	0.0079
9.6	0.0071
12.0	0.0065
14.4	0.0060
16.8	0.0056
19.2	0.0054

1.7. FACTORS FOR VARIATION OF RAINFALL INTENSITY DURING THE STORM PERIOD

It is well known that rainfall intensity is never constant within the storm period. It generally rises to a maximum, may momentaryily reach the peak or the rainfall may be steady for some time at the peak and then fall off. If the rainfall diagram is considered as trapezoid with rising, steady and falling periods a picture like the one in Fig. 8 will emerge.

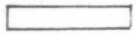
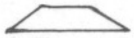
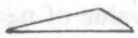
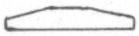
By varying the ratio of T_1:T_2:T_3 the various shapes of rainfall diagrams can be approximately represented. Richard by assuming different ratios of T_1, T_2, T_3 suggested a correction factor n_5 to be applied to equation 12. The equation can be written, when T = t, as

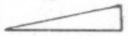
$$\frac{t^3}{t+1} = n_5 \frac{C'L^2}{K.S.R.f(A)} \quad \text{--------15}$$

The values of n_5 for some important cases as computed by Richard are given in Table IV.

Table IV

Values of Ratio i : I and of coefficient n_5 for various diagram

No.	Rainfall diagram	Ratio to T of				
		T1	T2	T3	i:I	n_5
1.		0.00	1.00	0.00	1.00	1.000
2.		0.33	0.33	0.33	0.66	1.102
3.		0.50	0.00	0.50	0.50	1.115
4.		0.00	0.33	0.66	0.66	0.763
5.		0.00	0.00	1.00	0.50	0.720
6.		0.66	0.33	0.00	0.66	1.624
7.		1.00	0.00	0.00	0.50	1.777
8.		0.66	0.00	0.33	0.50	1.295
9.		0.33	0.00	0.66	0.50	0.965
10.		0.33	0.33	0.33	0.833	1.035

For detailed study of the topic the reader should refer to original text(12) In fact, n_5 is a coefficient denoting the variation of flood intensity according to the time distribution of the rainfall. It is minimum when the rainfall time distribution diagrams is  when $n_5 = 0.720$. The coefficient n_5 is maximum, ($n_5 = 1.777$) when the diagram is  like.

From the field data the catchment area A in acres, length of catchment L in miles, slope of catchment S, are known. The values of K.R. f(A) and n_5 are suitably selected as explained above. The value of t is determined from formula 15 using the tables for $t^3/(t + I)$ and t values in appendix I. Knowing the value of t, the rainfall intensity can be determined from equation 13. Finally the maximum discharge is calculated using the equation 14.

Example

For Kurram River, determine from Richards formula the maximum design discharge for the data given below. Catchment area = A = 1775000 acres (2800 sq. miles)

Length L = 100 miles

Slope S = .001

Coefficient of runoff K = 0.6

Maximum rainfall is 15.3" in 6 hours.

Maximum rainfall intensity i = 15.3/6 = 2.56" per hour

From the ratios of average to maximum rainfall intensity (i=I f(A)) expressed as a function of catchment area in thousand acres Fig.(6) we have f(A) = 0.6. The rainfall intensity I and R, the coefficient of rainfall, are related by

$$i = \frac{R.f(A)}{T + I}$$

T = 6 hours, f(A) = 0.6 and i = 2.56", we have

R = 29.9. (say 30)
KR = 30 x 0.6 = 18

The formula connecting t = the period of concentration, the rainfall coefficient R and other parameters defining the characteristics of the catchment such as A. L. and S is:

$$\frac{t^3}{t+1} = n_5 \frac{C'L^2}{K.S.R.f(A)}$$

where n_5 depends on the rainfall and time distribution diagram.

For a normal maximum flood where the distribution diagram is of the type recommended value of n_5 = 0.72 and for a type of distribution diagram, the value of n_5= 1.777. We can compute t for the two cases

n_5 = .72

$$\frac{t^3}{t+1} = \frac{0.720 \times .005 \times (100)^2}{0.6 \times .001 \times 30 \times 0.6} = \frac{36}{.0108} = 3333$$

or t = 58.3 inches per hour (from appendix I)

$$\text{and } i = \frac{R\,f(A)}{t + I} = \frac{30 \times 0.6}{58.3 + 1} = \frac{18.0}{59.3}$$

= 0.303 inches per hour

and Q = 1000 KiA (for A in 1000 acres)

= 1000 x .6 x 0.30

= 180 cusecs per 1000 acres of catchment

or Q_{max} = 180 x 1775
= 319500 cfs (say 320,000)

If n_5 = 1.777 then a similar calculation will vield

Q_{max} = 202350 cfs

1.8.0 THE UNIT HYDROGRAPH METHOD

The Unit Hydrograph method of flood prediction was first developed by L.K.Sherman(13). It postulates that for a given drainage basin the net rainfall intensity of uniform or of specific areal distribution, the sucfce runoff measured at a given point in a stream for storms of different intesities occurring in a unit of time (1 hour, 6 hours, 24 hours etc) will produce hydrograph of nearly equal time base, but the ordinates of the hydrograph representing direct runoff at any given time will be in same propotion as the rainfall intensities. If at time t, in Fig. 9(a)

$i_2 = ni_1$, then $Q_2 = nQ_1$

Thus if a unit hydrograph represents one inch of runoff for a selected unit duration say 6 hour unit, then a two inch excess rainfall in the same unit duration will give twice as much runoff as in the unit hydrograph.

The concept underlying the unit hydrograph principle is that for storms of similar rainfall characteristics a unit-hydrograph completely represents the total effect of all the physical characteristics of the basin, such as drainage area, shape, channel and land slope etc.

In case of consecutive or isolated storms, the principle of super position is applicable to unit graph. The total hydrograph for net runoff can be obtained by adding the net runoff of separate unit hydrograph of each storm. The principle of super position of unit graph is illustrated in Fig. 9(b)

Inspite of the fact that the basic postulate of unit graph namely that all storms of unit duration but variable magnitude for a given catchment produces hydrographs of nearly equal base is open to question, the method is very realistic and helpful because if we can draw a unit graph for a given catchment corresponding to given storm we can compute the Q_{max} which the catchment is capable of producing but can also draw a flood hydrograph if maximum possible storm is realistically selected keeping in view the risks involved for the safety of the hydraulic structure against failure. In fact risks involved due to discharge exceeding the design for safety of a dam, a spillway, a diversion weir or a flood levee vary widely. For each case it is essential to select the design storm of a particular frequency and magnitude while applying it to the given catchment unit hydrograph for estimating the designed Q_{max}.

1.8.1 TERMS USED IN DEVELOPMENT OF UNIT HYDROGRAPH

In dealing with the unit hydrograph method certain terms generally used such as net rainfall, base flow, time of concentration and time lag have to be defined and properly understood. These are defined and methods of computing each from available dydrometeorological data are very briefly discussed. Detailed treatment of the subject is not within the scope of this paper.

1.8.2 NET RAINFALL, EFFECTIVE RAINFALL OR RAINFALL EXCESS

It is the volume of rainfall water that appears as runoff.It is equal to the total rainfall minus the losses due to absorption and retention.

The rainfall excess or the net rainfall can be determined either by:

1) The Φ index method, or

2) The trial and error method.

THE Φ INDEX METHOD

Since the net rainfall is that part of the rainfall which appears as a runoff without being lost by infiltration and or held by local depressions etc., the index is defined as the average rainfall intensity above which the volume of rainfall is equal to that of the runoff resulting from the given storm. The procedure for computation of net rainfall is illustrated with reference to plot of rainfall intensity against time in Fig. 10.

Suppose that a total rainfall of a 6 hour storm equal to 3.8 inches with hourly distribution of 0.4, 0.8, 1.25, 0.6, 0.5 inches respectively, produces a runoff equivalent to 2.05 inches, then in Fig. 10 Φ index will be line which divides the total area in such a way that the shaded area contributed 2.05 inches of runoff. In this case the index Φ =0.3 inches per hour represents the rainfall which did not appear as runoff, it therefore represents all the losses including infiltration, surface retention and evaporation. This average rate of retention makes the runoff rate equal to the net rainfall shown shaded in Fig. 10.

THE TRIAL AND ERROR METHOD

The trial and error method for calculating excess or net rainfall for data in Fig.10 is illustrated in Table V. In this method retention rates is successively assumed till the computed excess or net rainfall is equal to the storm runoff.

Table V

Time in hours	Rainfall inches (basin average	Assumed retention rate inches	Rainfall excess inches/ hour	Assumed retention rate inches/hr	Rainfall excess inches/ hour	Assumed retention rate inches	Rainfall excess inches/ hour
0							
1	0.4	0.5	-	0.4	-	0.3	0.1
2	0.8	0.5	0.3	0.4	0.4	0.3	0.5
3	1.25	0.5	0.75	0.4	0.85	0.3	0.95
4	0.6	0.5	0.1	0.4	0.20	0.3	0.3
5	0.5	0.5	0	0.4	0.10	0.3	0.2
6	0.25	0.5	-	0.4	-	0.3	-
Total			1.15		1.55		2.05

For the assumed average retention rate of 0.3 inches, the rainfall excess of 2.05 inches is equal to the actual runoff rate.

In the above method a constant average retention rate has been assumed. Actually the capacity rate of retention progressively decreases for a long duration storm till it attains a constant minimum value. Again the retention rate will be more for the first storm over a dry basin. There are more elaborate methods of computing excess rainfall by using varying capacity rate methods. These involve rather complicated computations which very often cannot be used in view of the meagre information about the relevant characteristics of the basin. These methods are therefore not discussed here.

1.8.3 BASE FLOW

The discharge in the channel from other sources than the rain storm such as contribution due to regeneration from ground water table and contribution from perennial sources and other delaying sources is known as the base flow.

Computation of Base Flow

For calculating direct runoff, the base flow has to be calculated. In a hydrograph the surface runoff and the base flow are separated by a line AN (Fig.11) where N represents the point on the recession limb of the hydrograph where surface runoff has completely ceased.

The point A being the point of sudden rise of discharge on the hydrogrpah. The point N can be determined by either of the following methods.

When for a long period of time, a continuous record of discharge is available the selected portions of recession curves (a-a), (b-b) ----- (e-e) of the hydrographs Fig. 12(a) are examined more thoroughly and log plots of recession lines (a-a) to (e-e) are made in Fig. 12 (b) to draw the depletion line x-x which again is plotted linearly in Fig. 12(c). The depletion curve transferred to the flow hydrograph fixes the point N at the inter-section of the two curves indicated in Fig.11.

The alternate method of separating base flow consists in fixing the position of N by determining the greatest point of curvature on the recession limb of the hydrograph. This can be done by plotting ratio of Q/Q (+6) against selected unit time interval (6 hours interval in this case) as illustrated with reference to Fig.11 in Table Q/Q (+6) is calculated from the recession limb of bar hydrograph in Fig. 11 for 6 hours time interval periods in the second and third day.

Table VI

Date	Hour	Q cfs in thousand	Q(+6) cfs in thousands	Q / Q (+6)
19	6	20.0	14.0	1.43
	12	14.0	10.0	1.40
	18	10.0	7.5	1.33
	24	7.5	6.0	1.25
20	6	6.0	5.25	1.14
	12	5.25	5.0	1.05
	18	5.0	4.8	1.02
	24	4.8	4.8	1.0

These are plotted as inset in Fig.11. The point of change of slope indicates the point of maximum curvature N of the dydrograph.

When prior to the beginning of flow contribution from a rain storm, the flow in the stream is due to regeneration from ground water flow from the two banks of the river valley then there will be a recession of flow represented by AA in the hydrograph (Fig.13). Water flow consists in extending the groundwater depletion curve AA to point B under crest of the hydrograph. The strainght line BC is then drawn to intersect the recession lindes of the hydrograph N days after the peak. The value of N can be calculated (in days) by the formula

$$N = M^{0.2}$$

where M is drainage area in square miles. The value of N can be used for all storms.

For large floods, when the base flow is only a small fraction of the hydrograph, great refinements in the estimation of the base flow may not be necessary. In case of streams fed by snowmelt prior to monsoon rainfall, the base flow from other sources than the storm contribution can be found by drawing a straight line or a smooth curve drawn tangent to low points at either end of hydrograph, for a small flood when the base flow may be considerable part of the total runoff the more refined methods discussed above may be adopted.

1.9.1 METHOD OF ANALYSIS OF RAINFALL-RUNOFF RECORD

The unit hydrograph is the hydrograph of a unit volume of surface runoff produced by a storm of uniform intensity of unit duration. The unit surface volume is 1 inch rain fall depth over the entire drainage basin and unit duration is arbitararily selected. it may be one hour, two hours, 6 hours or a day. Generally the unit period should not be greater than 1/4 of the time from the start of the runoff to the peak.

For deriving a unit hydrograph the following steps are taken:

1) Prepare a map of the basin showing sites of the guging stations of stream and of rainfall recording sites.

2) Collect hydro-meteorological data and select a storm over the basin well isolated from other period. Also select some other rainfall periods producing 1" or 2" rainfall to work out the volume of run-off for each period.

3) Prepare mass rainfall curve from rainfall data for the selected periods and plot discharge hydrograph for each selected period.

4) Estimate the base flow (see article 1.8.3) and subtract it from the total hydrograph runoff to get surface runoff.

5) Measure volume under hydrograph of surface runoff by planimeter or computations. Compute rainfall excess (see article 1.8.2) and plot the data in the form of a hydrograph to obtain unit hydrograph.

6) Divide the ordinates of the hydrograph for each unit storm by the volume under the repective surface runoff Hydrograph expressed in inches of runoff from the drainage area.

As an example to illustrate the construction of unit Hydrograph the data of storm on August 8th to 11th in 1973 at Marala on river Chenab is used. Tree hourly discharge data for period August 8 to 11th was collected and plotted in Fig. 15, the rainfall data is also plotted as inset. The direct run off was computed by subtracting the corresponding base flow from the total discharge. Direct run off for the storm period is 5257000 cft. The runoff in inches, the drainage area M= 12610 sq. miles

$$\text{Direct runoff} = \frac{5257000}{8 \times 12610 \times 2609} = 1.937 \text{ inches}$$

The unit hydrograph ordinates were obtained by dividing direct runoff by runoff in inches 1.937 and are recorded in Table VIII and plotted in Fig.15.

Table VIII

Marala at River Chenab

Date	Hours	Total Q	Base Flow	Direct R.O.	U.G. ord.	Hours after start
8.8.73	1500	160000	160000	0	0	0
	1800	180000	157000	23000	11874	3
	2100	197000	155000	42000	21683	6
	2400	297000	154000	143000	73528	9
9.8.73	300	360000	153000	207000	106866	12
	600	398000	151000	241000	124419	15
	900	458000	15000	308000	159008	18
	1200	588000	150000	438000	226123	21
	1500	672000	148000	524000	270521	24
	1800	710000	147000	563000	290655	27
	2100	762000	146000	616000	318017	30
	2400	744000	144000	600000	309757	33
10.8.73	300	664000	143000	521000	268972	36
	600	493000	142000	351000	181208	39
	900	391000	140000	251000	129582	42
	1200	336000	139000	197000	101703	45
	1500	235000	137000	98000	505594	48
	1800	190000	136000	35000	18069	51
	2100	160000	135000	25000	12906	54
	2400	158000	13300	25000	12906	57
11.8.73	300	151000	132000	19000	9809	60
	600	142000	132000	10000	5163	63
	900	142000	132000	10000	5163	66
	1200	142000	132000	10000	5163	69
	1500	132000	132000	0	0	72
	1800	132000	132000	0	0	75
				5257000		

Direct Runoff in inches per day per sq. mile = $\frac{5257000}{8 \times 12610 \times 2609} = 1.937$ inches

1.10.0 DESIGN DISCHARGE DETERMINATION BY SYNTHETHIC UNIT HYDROGRAPH

Synthetic unit hydrograph method is used when no runoff record is available. In this method the hydrograph is synthesised by using the data of other basins of different sizes and characteristics.

Since the unit hydrograph is based on the main concept that for storms of similar rainfall characteristics unit hydrograph is completely determined by the basin characteristics, naturally if the basin characteristics be expressed numerically and their influence on the hydrograph determined analytically, it should be possible to construct the unit hydrograph from only the basin parameters. Generally the basin area M, the stream length L, average slope of the stream in the basin and slope can be readily used in quantitative relationship. The assumption of similar rainfall characteristics is not strictly true because each storm can have different duration time intensity pattern, and areal distribution. By considering the basins sizes less than 2000 sq. mile and taking the storm duration about one quarter of the basin time lag, variations can be kept within acceptable limits. Evidently inspite of the fact that synthetic unit hydrograph gives only an approximation of actual, it is a practical and useful method.

Out of many methods available for constructing synthetic unit hydrograph only the following three methods will be presented:

1) Snyder's method

2) Linslay Kohler and Paulhus method

3) Soil Conservation Services method (S.C.S.method).

The main parameters considered sufficient to define the unit hydrograph and which have to be computed are:

1) Basin time lag= T_p. It is lag time from mid point of rainfall excess to the peak of unit graph.

2) Peak discharge = q_p. It is the peak discharge for 1".0 of rainfall excess

3) Total base time of the unit hydrograph = T_b.

Fig 16 represents the general symbole used in the method of constructing synthetic Unit Hydrograph

1.10.1 SYNDERS METHOD

Synder(14) in 1938 proposed the synthetic hydrograph method correlating T_P, g_p and T_b as marked in Fig. 16 with certain basin characteristics.

The formulae obtained are:

$T_p = C_t\,(L.L_c)^{0.3}$ ------------ 16.

$Q_p = 640.\ C_p\ M/T_p$ ------------ 17

where L and L_c are the distances in miles between the given station to the upstream end of the basin, and to centroid of the basin respectively.

M = drainage area in sq. miles

C_T and C_p are constants varying between 0.54 to 0.69 and 1.8 to 2.2 respectively. Snyder defined the standard duration of rainfall excess $Tr = T_p/5.5$ and proposed the following formula for total base time of the hydrograph as

$T_r = 3\,(1 + T_p/24)$ --------- 18

If rainfall duration $T_r > Tp/5.5$, say t_{ro}

then

$T_{pr} = T_p + (T_{ro} - T_r)/4$ ----------- 18(a)

In this case T_{pr} is used instead of T_p in formula 16 for computing T_p and q_p

The limitations of this method are:

1) The selection of most suitable value of C_T and C_p depend on the best judgement.

2) Since in Equation 18, T_b can not be less than 3 days the application of this formula to smaller basins becomes questionable.

3) Basin slope, or stream slope is not taken into consideration. Taylor and Schwartz however in 1952 accounted for this by correlating slope S with constant C_T by putting

$$C_T = 0.6/S^{0.5}$$

1.10.2 LINSLAY KOHLER AND PAULHUS METHOD

The following set of formulae were propsed:

$$T_p = C_T \,[\,L.L_c/S_o^{0.5}]^{0.38} \qquad \text{---------- 19}$$

where S_o is the average or weighted slope of the main stream of the basin in feet per mile. In this formula

$C_T = 1.2$ for mountaneous area

$C_T = 0.72$ for foot hill area

and $C_T = 0.35$ for valley drainage

$$(q_p = 640.C_p\, M)/T_p \qquad \text{---------- 20}$$

where $C_p = 0.6$ as an average value

The weighted average slope So can be calculated by taking slopes in different reaches of the stream. If S_1, S_2, ...S_n are the slopes in n reaches of the stream, the weighted slope So is given by

$$1/S_o^{1/2} = 1/n\,[1/S_1^{1/2} + 1/S_2^{1/2} + ... 1/S_n^{1/2}] \text{ .... 20A}$$

1.10.3 UNITED STATES SOIL CONSERVATION SERVICE METHOD

The hydrograph in Fig. 16 for all practical purposes can be approximated to a triangle in Fig. 17. The equation developed by US BR are:

$$T_p = D/2 + 0.6\, T_c \qquad \text{------ 21}$$

where T_c is the time of concentration in hours t_p is time to peak and D is rainfall excess period in storm. T_c can also be obtained when length of stream L and slope or drop H are known by using the Nomogram for small catchments (SCS Guide), Fig. 18. Solution for T_p can be made by using Eq. 22 and 21.

$$T = T_c = [11.9L^3/H]\ 0.385 \qquad \text{------- 22}$$

where $T = T_c$ is concentration time in Hours

L = Length of longest water course in miles

and H = Elevation difference in feet

Using T_c from Nomogram in Fig. 18 to obtain Tp, q_p is given by

$$q_p = \frac{KMQ}{T_p} \qquad \text{------ 23}$$

where Q = total runoff in inches for unit hydrograph Q = 1 inch

M is catch ment area in sq. miles

and K is a constant depending on the shape of the hydrograph and is given by

$$K = 1290.6/(1+H)$$

where $H = T_r/T_p$

As a general rule S.C.S. have recommended adoption of H = 1,67 for ungaged basins

For H = 1.67 and K 484

$T_b = 2.67\ T_p$

The equation for general Peak discharge q_p is

$q_p = 484\ MQ/T_P$

Using $L = 0.6\ T_c$

$$q_p = \frac{484 \times Q}{D/2 + 0.6\ T_c} \qquad \text{---- 24}$$

The method of computation of Q_{max} by using C.C.S. method is illustrated by an example.

1.10.4 COMPUTE MAXIMUM PROBABLE DESIGN FLOOD DISCHARGE FOR GAMBILA RIVER AT SITE FOR A HIGHWAY BRIDGE

The following data are available. The area falls in a zone of 3.95" to 8.2" average rainfall during June to September. The size of drainage area upto the point of interest is 1354 sq. miles. Taking a factor of 4 for hydrological uncertainities

The maximum probable precipitation in 6 hours is = 3.95" x 4 = 15.8 (say 16") this can be taken as six hours 10 square mile probable maximum precipitation. The method as recommended by S.C.S. is now used to illustrate the computation procedure. Using curves between depth area duration relationship for different zones East of 1050 Maridearns in USA for zone 1. Reproduced as Fig.19. The hour 10 square mile rainfall is adjusted to values for the given drainage area and longer duration.

2) Hourly Probable Precipitation in Maximum First 6 hours period

Using lower curve for zone A (Fig.19) Table IX-B is prepared

Hourly PMP, 6 Hour Period

Table - IX -A

Duration in hours	Percent of 10 sq. mile* 6 hour value	Total PMP in inches
1	2	3
0-6	47	7.52
0-12	60	9.48
0-24	70	11.06
0-48	77	12.17

(col 3) = 16 x (col2)/100

PMP = Probable Maximum Precipitation

Table - IX-B

Time hours No 1	Percent 6 hour rain 2	Accumulative rain inches 3*	Incremental rain** inches 4
1	30	2.26	2.26
2	48	3.60	1.34
3	64	4.81	1.21
4	78	5.87	1.06
5	89	6.70	0.83
6	100	7.52	0.82

*Col 3= 7.52 x (col2)/100, Col.4 =** (col 3)n - (col 3)n-1 = enter under col.4

3) PMP design storm arrangement and design rainfall for PMP

Arrange the incremental rainfall during the maximum 6 hour period in the descending order of magnitude obtained to the following hourly magnitude squence as through 6 hours: 6, 4, 3, 1, 2, 5

4) Estimation of Direct Runoff

For this basin with the hills in their catchment having harbareous growth, the soil cover complex is 88, say 90 be adopted.

Using runoff equation

$$Q = \left[\frac{P - 0.25}{P + 0.85}\right]^2 \qquad \text{-------25}$$

and the U.S soil conservation service curves connecting rainfall P in inches and direct runoff in inches (reproduced in Fig.19) (a-u) the direct runoff is computed and recorded in Table IX-D below:

Table - IX-C

Time ending hours	Incremental rain in inches arranged in descending order as above	Accumulative rain in inches *	Accumulative design rain fall by reduction of 10% rainfall value	Incremental Design rainfall Col. 4 [n-(n-1)]
n	2	3	4	5
1	0.82	0.82	0.74	0.74
2	1.06	1.88	1.69	0.95
3	1.21	3.09	2.78	1.09
4	2.26	5.35	4.82	2.04
5	1.34	6.69	6.02	1.20
6	0.83	7.52	6.77	1.75
12	1.96	9.48	8.53	1.76
24	1.58	11.06	9.95	1.42
48	1.11	12.17	10.95	1.00

*Col. 3 = Σ n values of col. 2

Table- IX-D

Time Design	Incremental rainfall raised by 10% *	Accumulative design rainfall	Direct Runoff (inches)		Incremental Intention loss (Col.2-(Col.5)
			Accumula-tive +	Incremental	
	2	3	4	5	6
1	.74	.74	0.18	0.18	0.56
2	.95	1.74	0.9	0.72	0.23
3	1.19	2.93	1.93	1.03	0.06
4	2.04	4.97	3.8	1.87	0.17
5	1.20	6.17	5.05	1.25	0.05
6	1.75	7.92	6.75	1.70	0.05
12	1.76	9.68	8.40	1.65	-
24	1.42	11.10	10.3	1.2	-
48	1.00	12.10	11.0	0.7	-

* From Table IX-C Col.5

\+ For soil cover complex No.90, read Fig/19 for Col.3 enter in Col.4.

5) Time of Concentration

Length of basin L = 100 miles

Drop H = 500 ft

$$T_c = \left[\frac{11.9 \times L^3}{H}\right]^{0.385} = \left[\frac{11.9 \times (100)^3}{500}\right]^{0.385} = 48.9 \text{ hours (say 49 hours)}$$

6) Hydrograph

Determine T_p, T_b, q_p for the given value of T_c = 49 hours

A = 1354 sq. miles

q = 1 inch

from the formulae

$T_p = D/L + 0.6\ T_c$

$T_b = 2.67\ T_p$

$q_p = 484\ AQ/T_p$

Table - IX-E

Hours	T_p	T_b	q_p
1	2	3	4
1	29.9	79.8	21918
6	32.4	86.5	20226
12	35.4	94.5	18512
24	41.4	110.5	15829
48	53.4	142.6	4596

7) Plotting Table

For each incremental runoff in Table IX-D, Col.5, the triangular hydrograph is computed. The peaks are obtained by multiplying the q_p (for 1 inch runoff) by the incremental runoff.

The maximum probable flood hydrograph is obtained by plotting incremental triangular hydrograph from Table X-F on plain graph paper (Fig. 20). For each chosen ordinate on Times selected add the ordinates of triangular Hydrographs Ordinates should be added only at the times of beginning peak and end of each incremental hydrograph.

The hydrograph is plotted in Fig. 20 showing maximum discharge as 184000 cfs.

Table - IX-F

Time duration	Incremental runoff* Inches	q_p for 1 inch	q_p for Incremental runoff col 4=col 1x col3)**	Incremental Hydrograph Begin Time(H)	Peak time ((hrs)	End Time (Hrs)
1	2	3	4	5	6	7
1	.18	21918	3945	0	29.9	79.8
2	.72	21918	15781	1	30.9	80.8
3	1.03	21918	22576	2	32.9	82.8
4	1.87	21918	40987	3	32.9	83.8
5	1.25	21916	37398	4	33.9	84.8
6	1.70	21918	37261	5	34.9	85.8
12	1.65	20226	33373	6	38.9	92.5
24	1.2	18512	22214	12	45.9	106.5
48	.7	15829	11080	24	58.9	120.5

* from Col 5, Table IX-D

** col 3 X col. 2 = col 4

11.0 ESTIMATION OF Q_{max} FROM FLOOD RECORDS BY FLOOD FREQUENCY ANALYSIS AND PROBABILITY METHOD

The methods to be discussed under this head depend on the availability of extensive flood record. This approach is generally known as the flood frequency analysis probability studies. The methods primarily consist of statistical analysis of the available flow record to determine the frequency of the occurrence of a particular discharge at the site in any given interval of time (say 10,100 or 1000 years). Frequency analysis will in fact determine the N-year flood or a flood which over a long period is likely ot be equalled or exceeded once in N-year.

It has to be borne in mind that the results of analysis by probability or frequency analysis methods depend on the data punt in. If the data were for a considerably more period or say a little less than what has been used the result might have been different. This is again due to the fact that the analysis is based on data alone which may not include all the physical factors which contribute to make a maximum flood. Fully realizing the inadequacy of this method it may still be preferable to use the method by application of a factor of safety to the maximum recorded flood.

Many probability methods have been used to estimate the frequency and magnitude of Q_{max}. More common of these methods are:

1) Hazen's method

2) Fuller's Method

and 3) Gumbel's Method

Each method is discussed below and its use is illustrated by a solved example.

11.1 HAZEN'S[16] METHOD

If the magnitude of maximum annual flood is logarithmically plotted as ordinate and the frequency of each flood as abscissa, the flood being expressed as percentage of time, it is found that the line slightly bends off giving a skew curve.

The general slope of the curve is given by the coefficient of variation C_v

$$C_v = \sqrt{\frac{\Sigma d^2}{N-1}} \quad \text{------} \quad 21$$

and deviation from the straight line by the coefficient of skew C_s

$$C_s = \frac{\Sigma d^3}{(N-1)(C_v)^3} \quad \text{--------} 21(a)$$

Where $$d = \frac{\text{recorded value of Q}}{\text{mean value of Q}} - 1 \quad \text{-------} 21(b)$$

N = number of years for which observations are analysed. To allow for shortness of record, the coefficient of skew C_s is modified to allow for the period of record. This is done by multiplying C_s by a factor

(1+ 8.5/n) for type III and by a factor 1+ 6/n for type I of Pearsonian probability curves.

The procedure of analysis is to arrange the annual maximum discharge choronologically, and then in descending order of magnitude and fainally to express them as a ratio of the mean flood. The variation is determined by substracting unity from this ratio. Determining squares and cubes of the variation, C_V and C_S are computed from the above-mentioned formulae.

From statistical tables for type III and type I curves, the skew curve factor corresponding to C_V = 100 are obtained for various frequencies and are entered against the frequency. These values are multiplied by actual Cv computed from the data, and the probable flood in terms of the mean flood is obtained by adding the skew curve factors to or subtracting them from 1.00. Actual flood values are obtained by multiplying the ratios Q/Q' with mean annual flood.

Example

For the annual flood discharge data of river Ravi at Shahdara from 1921 to 1954 is given in Table-x. Calculate discharge by Bezin's method for different frequencies of occurrence expressed as percentage of Time [IRI.AR(1955)].

The annual maximum floods are arranged chronologically in Table X column (3). The floods are rearranged in descending order of magnitude in column (4) and expressed as a ratio of mean flood Q/Q' in column (5). Frequency % time F as computed from formula given below is given in column (6)

$$F = \frac{2n-1}{2N} \times 100 \qquad \text{------ } 22.$$

where n is serial number and N is the total number of observations. The variation 'd' in mean flood is entered in column (7) and squares and cubes of variations in columns (8) and (9), respectively. The coefficient of variation, Cv, and skew, Cs are then computed by the equation given in Equations 21 and 21(a). To allow for the shortness of record, the coefficient of skew so computed is adjusted by multiplying by the factor

$$\left(1 + \frac{8.5}{N}\right)$$

The skew curve factor from statistical tables corresponding to Cv = 1 & C_S = 2.61 are obtained for various frequencies. These are entered in column 2 of Table XI and multiplied by the actual C_V = 0.6 are entered in column 3. The probable flood Q_p in terms of mean flood Q', or Q_p/Q' for each frequency is computed by adding one to skew curve factor given in Column 3. The probable flood values Qp for different frequencies are obtained by multiplying the ratio Qp/Q, by the mean annual flood value Q'. These are recorded in coulumn 5 of Table XI. According to the computation the 100 year flood = 312000 cfs.

TABLE No. X

ANNUAL MAXIMA FLOODS ARRANGED CHRONOLOGICALLY RAVI AT SHAHDARA

Serial No.	Year	Maximum flood	Q in Decending Order	Q/Q1	Frequency % time F	Variation d	d2	d3
1	2	3	4	5	6	7	8	9
1	1921-22	63.6	244.0	2.977	1.471	1.977	3.908529	7.727162
2	1922-23	63.0	225.0	2.745	4.412	1.745	3.045025	5.313569
3	1923-24	53.2	193.0	2.355	7.355	1.355	1.836025	2.487814
4	1924-25	84.8	129.8	1.584	10.294	0.584	0.341056	0.199177
5	1925-26	63.7	122.5	1.495	13.235	0.495	0.245025	0.121287
6	1926-27	60.1	106.9	1.304	16.176	0.304	0.092416	0.028094
7	1927-28	80.4	100.0	1.231	19.118	0.231	0.053361	0.012326
8	1928-29	54.1	86.4	1.054	22.059	0.054	0.002916	0.000157
9	1929-30	60.0	86.2	1.052	25.00	0.053	0.002704	0.000141
10	1930-31	80.0	84.8	1.035	27.941	0.035	0.001225	0.000043
11	1931-32	70.2	82.7	1.009	30.882	0.009	0.000081	0.000001
12	1932-33	67.3	80.2	0.976	33.824	-.024	0.000576	0.000014
13	1933-34	39.3	79.9	0.938	36.767	-.062	0.003844	-.000238
14	1934-35	54.6	70.2	0.875	39.706	-.143	0.020440	-.002924
15	1935-36	106.9	67.3	0.821	42.647	-.179	0.032041	-.005735
16	1936-37	62.0	63.7	0.77	45.528	-.223	0.049729	-.011090
17	1937-38	51.6	63.6	0.776	48.529	-.224	0.050176	-.011239
18.	1938-39	50.4	63.0	0.769	51.471	-.231	0.053361	-.012326
19	1939-40	55.5	62.0	0.756	54.412	-.244	0.059536	-.014527
20	1940-41	39.4	60.5	0.738	57.353	-.262	0.068644	-.017985
21	1941-42	129.8	60.1	0.733	60.294	-.267	0.071289	-.019034
22	1942-43	100.9	60.0	0.732	63.235	-.268	0.071824	-.019249
23	1943-44	46.2	55.9	0.682	66.176	-.318	0.101124	-.032157
24	1944-45	60.5	55.5	0.677	69.118	-.323	0.1043229	-.033698
25	1945-46	76.9	54.6	0.666	72.059	-.334	0.111556	-.037260
26	1946-47	225.0	54.2	0.661	75.000	-.339	0.114921	-.038958
27	1947-48	86.2	54.1	0.660	77.941	-.340	0.115600	-.039304
28	1948-49	51.8	51.8	0.632	80.882	-.368	0.135424	-.049836
29	1949-50	103.0	51.6	0.630	83.824	-.370	0.136900	-.050653
30	1950-51	44.0	50.4	0.615	86.765	-.385	0.148225	-.057067
31	1951-52	55.9	46.2	0.564	89.706	-.436	0.190096	-.08
32	1952-53	82.7	44.0	0.537	92.647	-.463	0.214369	-.09
33	1953-54	122.5	39.4	0.481	95.588	-.519	0.269361	-.14
34	1954-55	244.0	39.3	0.480	98.529	-.520	0.270400	-.14
		Total	2786.5				11.922137	14.97

V=Q/Q′+1 F=(2n-1)/2Nx100 n=Serial No. N=No. of terms. Mean=Q′=2786.5/34=81.96

Co-efficient of Variation $=C_v=\left(\frac{\Sigma V^2}{N-1}\right)^{\frac{1}{2}}=\frac{11.922137}{33}=(36127687)^{\frac{1}{2}}=.60$

Co-efficient of Skew $=C_s=\frac{V^3}{(N-1)(C_v)^3}=\frac{14.973936}{33\times 0.21715}=2.089595$

Adjusted Co-efficient of Skew $=\left(\frac{1+8.5}{N}\right)\ C_3=\left(\frac{1+8.5}{N}\right)\ (2.089595=2.61)$

Table XI

Frequency % time	Skew curve factor $C_s = 2.61$			Q_p Probable flow in 10^3 cfs
	$C_v = 1$	$C_v = 0.6$	Ratio Q_p/Q'	
1	2	3	4	5
99	-.97	-.582	.418	34.25
95	-.89	-.534	.466	39.19
80	-.71	-.426	.574	47.04
50	-.32	-.192	.808	66.2
20	-.49	.294	1.294	106.05
5	2.09	1.254	2.254	184.73
1	4.68	2.08	3.808	312.10
1	10.15	6.090	7.090	581.09

11.2 FULLER'S METHOD

Based on frequencies of flood, Fuller(17) developed the following formulae for determining maximum probable flood Q_p for a given data of N years

$$Q/Q_m = 1. + 0.8 \log_{10} N \qquad \text{-----23}$$

and $$Q_p = Q\,(1 + 2M^{-0.3}) \qquad \text{----23(a)}$$

where Q_p is the probable maximum flood in cfs likely to occur in N years. and Q_m is the average of the yearly maximum flows in cfs from data for N years

M = is the area of catchment in square miles
N = is the number of years for which the data is analysed.

Example

Taking the discharge data for Kalabagh barrage site for the period 1928 to 1970 given in Table XII. determine Q_p. The catchment area M= 103800 square miles.

From the data Q_m = 534000 cfs. Knowing Q_m from the data, using Equation 23.

$Q = Q_m\,(1+0.8 \log_{10} N)$

If N = 43, Q = 534000 (1+0.8 x 1.63) = 53400 x 2.304 = 1230336 or 1230000 cfs.

Substituting the value of q in Eq.23 (a) we have

$Q = Q_m\,(1+2M^{-0.3})$

M = 103800 square miles

$Q_p = Q\,(1+2 \times .0313)$

= 1230000 x 1.0626 = 1307000 cfs

Table XII

ANNUAL MAXIMUM RECORD FLOODS IN RIVER INDUS AT KALABAGH

Year	Annual Q_{max} x 1000 C_s
1928	357
1929	819
1930	484
1931	339
1932	570
1933	520
1934	591
1935	535
1936	460
1937	473
1938	502
1939	540
1940	491
1941	468
1942	950
1943	562
1944	691
1945	708
1946	511
1947	527
1948	772
1949	788
1950	745
1951	488
1952	466
1953	630
1954	382
1955	445
1956	453
1957	443
1958	689
1959	587
1960	524
1961	434
1962	342
1963	432
1964	553
1965	443
1966	497
1967	444
1968	467
1969	470
1970	405

$$Q_{mean} = \frac{22957000}{43} = 534000 \text{ cfs}$$

11.3 GUMBLE'S METHOD

Guble's(18) method is based on the premises that if a sample consists of highest values of series, e.g. the highest annual flood recorded on any day, the probability, P, of occurrence of a value equal to or less than X is given by:

$$P = e^{-e^{-y}} \quad \text{--- 24}$$

where e is the base of Naperian logarithm and Y is a dimensionless variable given by:

$$y = a\,(x - x_f) \quad \text{--------25}$$

and according to Gumbel, x_f should be computed from

$$x_f = \bar{x} - 0.456\,\sigma_x \quad \text{------ 26}$$

where $\bar{x}$ = the mean value

and σ_x = standard deviation of the x values

$$\sigma_x = \sqrt{\frac{\Sigma\,(x - x)^2}{N}} \quad \text{------- 27}$$

where x is any individual value, $\bar{x}$ is the mean of all the values, x-$\bar{x}$ is the difference or deviation of any value from the mean value and N is the total number of the values of which $\bar{x}$ represents the mean.

The factor, a in Eq. 25 is given by the equation

$$a = \frac{1}{0.7797\,\sigma x} \quad \text{----- 28}$$

A special plotting paper has been used for this method. The ordinate representing flood flow and the abscissa scale for y between the limits -2 and -7 are linear. This gives the scales for plotting the function given by Eq. 25 which is a straight line. In order to interpret the results in terms of frequency or return period on auxiliary abscina scale in terms of probability can be worked out by using Eq. 24 and the scale for the return period T may be computed from:

$$T = 1/(1-P) \quad \text{------29}$$

If we assume some values of the probability p, then we can determine y from Eq. 24 and knowing y, x can be calculated from Equation 25, if we know the values of 'a' and x_f for the given data which are determinate, we get from Equation

$$X = \frac{y}{a} + x_f$$

$$= .7797 \times \sigma_x . y + \bar{x} - 0.45\,\sigma_x$$

$$= \sigma_x\,(0.7797\,y - 0.45) + \bar{X} \quad \text{---- 30}$$

This equation can be used to determine x corresponding to different values of p and hence of y, as follows;

$$P = a^{-e^{-y}}$$

Taking logarithm again with respect to base e

$$-y = \log_e(-\log_e p)$$

or $$y = -\log_e(-\log_e p) \qquad \text{------- } 31$$

For assumed values of P equal to 0.1, 0.5, 0.8, 0.9, 0.95, 0.98, 0.99 0.995, 0.998, the corresponding values of y are worked out from Eq. 31 and are recorded in Table XIII (a).

The calculations for x and the standard drivation σx for the annual maximum discharge data for a given site can be worked out with the help of Eq.25. The values of x can be calculated for the values of P as selected above. The calculations are given in Table XIV.

Gumbel's Correction

If there are N items of data in a series and the serial number of a particular event in order of magnitude beginning with the lowest value is m, then if m' is the serial Number of an item in a series arranged in reverse order then we can compute m/N. Gumbel has suggested a correction factor C which is a function of the relative serial number m/N and the nature of the distribution and he has given a curve shown in Fig.21 which gives values of c applicable to the distribution of largest values (the annual floods). The plotting position is given by

$$T = \frac{t}{m' + c^{-1}} \quad 1)$$

Not much difference is made by the use of this correction but the above mentioned data of annual maximum discharges for Attock site on Indus has been analysed by making use of this correction.

The main calculations and the results are given in Table XIX (a, b) and the plottings are given in Fig. 22 which depict the plot of observed and theoretical discharges corresponding to the various return periods.

TABLE - XIII (a)

Values of y for different values of P

P	$\log_{10} p$	$\log_e P$	$\log_{10}(-\log_e P)$	$Y = -\log_e(-\log_e P)$
0.1	-1.00000	-2.30258	0.36221	-0.83402
0.5	-0.30103	-0.69315	-0.15918	-0.36652
0.8	-0.09691	-0.22314	-0.65142	1.49995
0.9	-0.04576	-0.10537	-0.97732	2.25036
0.95	-0.02228	-0.05130	-1.28988	2.97005
0.98	-0.0087	-0.02019	-1.69490	3.90264
0.99	-0.00436	-0.01004	-1.99830	4.60123
0.995	-0.00218	-0.00502	-1.29930	5.29432
0.998	-0.00087	-0.00200	-2.69897	6.211459

TABLE - XIV

P	I-P	T=I/(I-P)	(.7797y-.45)	(.7797y-.45)	xx 0(.7797y-.45) σx + x̄
0.1	.9	1.11	-1.10029	-84.61	402.69
0.5	.5	2	-0.16422	-12.63	474.67
8.8	.2	5	0.71951	+55.33	542.63
0.9	.1	10	1.30461	100.32	587.62
0.95	.05	20	1.86575	143.48	630.78
0.98	.02	50	2.59289	199.39	686.69
0.99	.01	100	3.13759	241.28	728.58
0.995	.005	200	3.67798	282.84	770.14
0.998	.002	500	4.39552	338.02	825.32

x = 487.3
σx = 76.90

TABLE XIV (a)

INDUS

YEARLY MAXIMUM DISCHARGE (CFS) AT ATTOCK

Year	Discharge
1922	538000
1923	462000
1924	644000
1925	474000
1926	427000
1927	450000
1928	486000
1929	760000
1930	567000
1931	427000
1932	577000
1933	484800
1934	510100
1935	488200
1936	412800
1637	429200
1938	452400
1939	489300
1940	431400
1941	423400
1942	561000
1943	482400
1944	511200
1945	514500
1946	515000
1947	369000
1948	506600
1949	459600
1950	477600
1951	406200
1952	456000
1953	583000
1954	377600
1955	453600
1956	462000
1957	457200
1958	633000
1959	462000
1960	544000
1961	427000

TABLE NO. XIV (b) *Paper No. 493*

FREQUENCY ANALYSIS FOR THE INDUS RIVER AT ATTOCK BY THE CUMBEL METHOD

Annual flood in decending order (1000 Cs.)	Serial No. m'	Serial number in reverse order	m/N	C from curve	T=N/(m'+c-1)	P=1-1/t	$Y=-\log_e(-\log_e P)$	$x=(.7797y-.45)\sigma_x+\bar{x}$
760.0	1	40	1.00	1.0	40.00	.97500	3.67577	673.09030
644.0	2	39	.98	.99	20.10	.95025	2.97496	631.07070
633.0	3	38	.95	.97	13.47	.92576	2.56215	606.31890
583.0	4	37	.93	.96	10.10	.9100	2.26083	588.25201
567.0	5	36	.90	.94	8.10	.8765	2.02636	574.19318
567.0	6	35	.88	.93	6.75	.8519	1.83080	562.47206
561.0	7	34	.85	.92	5.78	.8270	1.66101	552.28742
544.0	8	33	.83	.91	5.06	.8024	1.51344	545.43931
538.0	9	32	.80	.89	4.50	.7778	1.38120	535.51015
514.0	10	31	.78	.88	4.05	.7531	1.26034	528.26386
511.0	11	30	.75	.87	3.68	.7283	1.14871	521.57049
510.0	12	29	.73	.86	3.37	.7033	1.04429	515.30929
506.6	13	28	.70	.84	3.12	.6795	0.95087	509.70789
489.3	14	27	.68	.83	2.89	.6540	0.85651	504.05036
488.2	15	26	.65	.81	2.70	.6296	0.77074	490.90806
486.0	16	25	.63	.80	2.53	.6047	0.68711	493.89341
484.8	17	24	.60	.78	2.38	.5798	0.60682	489.07947
498.4	18	23	.58	.77	2.25	.5556	0.53150	484.56313
477.6	19	22	.55	.75	2.13	.5305	0.45586	480.02757
474.0	20	21	.53	.73	2.03	.5074	0.38792	475.95417
462.0	21	20	.50	.72	1.93	.4819	0.31469	471.48628
462.0	22	19	.48	.71	1.84	.4565	0.24311	467.27139
462.0	23	18	.45	.69	0.76	.4318	0.17458	463.16263
459.6	24	17	.43	.68	1.69	.4083	0.11011	459.29686
2457.2	25	16	.40	.66	1.62	.3827	0.04027	455.10966
456.0	26	15	.38	.65	1.56	.3590	0.02415	451.28667
453.6	27	14	.35	.63	1.50	.3333	0.09429	447.04131
452.4	28	13	.33	.61	1.45	.3103	0.15722	443.26860
450.0	29	12	.30	.58	1.40	.2851	0.22535	439.18367
431.4	30	11	.28	.57	1.35	.2593	0.30003	434.70578
429.2	29	10	.25	.55	1.31	.2366	0.36558	430.77542
427.0	31	9	.23	.53	1.27	.2126	0.43726	426.47748
427.0	33	8	.20	.51	1.23	.1870	0.31679	421.70891
427.0	34	7	.18	.48	1.19	.15966	0.60684	416.30976
423.4	35	6	.15	.46	1.16	.1379	0.68357	411.70884
415.0	36	5	.13	.46	1.13	.1150	0.77127	406.45042
412.8	37	4	.10	.41	1.10	.0909	0.87466	400.25151
406.2	38	3	.08	.38	1.07	.0654	0.00317	392.54613
377.6	39	2	.05	.33	1.04	.0385	1.18115	381.87471
369.0	40	1	.03	.30	1.02	.0196	1.36907	370.60733

12.0 PROBABILITY STUDY OF FLOOD DATA

The probable frequency of a flood of a given magnitude can be determined by the application of laws of probability by either of the following methods:

a) Basic Stage Method;

b) Yearly Flood Method;

The first method involves the analysis of all, discharges greater than a given base discharge which is generally the lowest of the maximum annual discharges.

The second method involves the analysis of only the maximum yearly floods.

The results obtained by both these methods are not materially different.

In the yearly flood method the annual maximum flood discharges are arranged in the descending order of magnitude and the number of times, n, that each flood was equalled or exceeded is noted. According to the law of probability the percentage of years in which a flood equal or greater than a given discharge Q will occur is obtained from the equation:

$$P = \frac{100\,n}{N}$$

where P is the percentage of future years in which a flood of a equal or greater than a discharge, Q, will occur.

N is the total number of years of record. The peak discharges are plotted against the values of p on probability paper and the probability curve is drawn through these points

Example:

From flood data of Indus at Kalabagh (Table XII) determine Q_{max} which can occur once in 100, 50 and 20 years.

In Table XV, the discharges are arranged in descending order of magnitude in Col. 1. and n are recorded in Col.2, P is computed from : P = 100 n/N and recorded in Col. 3.

The recurrence interval in years are computed by the ration (N+1)/n in which N is the number of years in the record and n is the order of magnitude as assigned. Using this recurring intervals are plotted in the figure giving the probability curve for the site in Fig. 23.

From this curve the magnitudes of floods occurring once in 20, 50 and 100 years can be read and the results are about 75,0000; 1100,000 and 1250,000 cfs.

TABLE XV

PROBABILITY ANALYSIS OF DISCHARGES FOR KALABAGH BARRAGE SITE ON FROM TABLE XII.

Col. arranged in descending order of Magnitude	No. of Times Peak was excluded	Percent-age of Year P 100n/N	Discharge arranged in descend-ing order	No. of Times Peak was excluded	Percent-age of Years D3
950	1	2.3	484	26	60.5
819	2	4.45	473	27	26.8
772	3	6.98	470	28	65.1
745	4	9.3	468	29	67.4
708	5	11.6	467	30	69.8
699	6	13.95	466	31	72.1
695	7	16.38	460	32	74.4
691	8	18.6	449	33	76.7
689	9	20.9	448	34	79.1
591	10	23.3	444	35	81.4
587	11	25.6	443	36	83.7
570	12	27.9	434	37	86.0
570	13	30.2	432	38	88.4
562	14	32.6	.422	39	90.7
553	15	34.9			
540	16	37.2	405	40	93.0
535	17	39.5	357	41	95.4
527	18	41.9	342	42	97.7
524	19	44.2	339	N = 43	100
523	20	46.5			
520	21	48.8			
511	22	51.2			
502	23	53.5		P = 100 n/N	
497	24	55.8			
491	25	58.1		I = 100/P = N/n	

13.0 COMPUTATION OF MAXIMUM FLOOD DISCHARGE FROM FLOW DATA OF A STREAM

If a hydraulic structure such as a bridge is to be constructed at site (x-x) on a stream (Fig 24 (a)) then the hydraulic data needed will be:

1) At least a cross section at the proposed site for the hydraulic structure and two other sections one upstream and another on the downstream side.
2) Estimation of slope S between 1, 2 and 3 sites. Determine mean value of S.
3) Work out cross sectional area A and wetted perimeter P +. Compute R= A/p for section for widths a-b, b-c, c-d for assumed flow stage near the Maximum Flood Mark.
4) Assume Manning N-values for main channel b-c and spill areas a-b and c-d
5) For different assumed stages at site x-x work out the areas of cross section for a-b, b-c and c-d at section x-x. Check these values at sections 1 and 3 also. Find wetted perimeter P calculate R= A/p using Mannings fromula

$$Q = A\frac{1.49}{N} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$= K_1 S^{\frac{1}{2}}$$

$$\text{where } K_1 = \frac{1.49}{N} AR^{\frac{2}{3}}$$

From the field data, tabulate the results in the following form for selected flow stage near the Maximum Flood Mark. For estimation of Q_{max}, mean values of N, A, P, R for a-b, b-c, c-d, at a section x-x can be used. The computations are illustrated by an example.

Example

From a cross section of stream (Fig. 24 b) at a site of proposed bridge, determine Q_{max} with the data givn in Table XVI with reference to cross section at x-x in Fig XVI, S= 0.0008

Table XVI

		N	1.49/N	A	P	R	$K=(1.49/N)\ AR^{2/3}$	$Q_x=(KQ/K_1)Q$
	1	2	3	4	5	6	7	8
Q_R	a-b	.035	42.6	255	185	3.3	24100	681
Q_1	b-c	.025	59.6	1750	300	5.3	317000	8966
Q_2	c-d	.030	49.6	375	165	4.5	50500	1431

$K_1 = \Sigma K = 381700$

$Q = K_1 S^{1/2}$

and $Q_{max} = \Sigma Q_x$

From the table with

$K_1 = 381700$

$Q = 381700 \times (.00008)^{1/2}$

$=10796$ cfs.

From col. 8 of Table

$$Q_{max} = \Sigma Q_x = 11078 \text{ cfs}$$

Computations can be repeated by assuming HFL a foot or two higher or lower.

SUMMARY

In this paper the subject of "Estimation of Maximum Design Discharge for a Hydraulic Structure" has been dealt with in comprehensive manner. The following methods with all details have been presented;

1) Q_{max} from catchment area.
2) Q_{max} from catchment area and rainfall data.
3) Unit Hydrograph methods.
4) Estimation of Q_{max} from flood record by flood frequency and probability methods.
5) Computation of Q_{max} from flow formula and HFL.

The above methods are discussed in great detail pointing to the scope and limitations in each case. Each method is illustrated where necessary by a solved example.

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13.	Sherman L.K.	Stream Flow from Rainfall by Unit Hydrograph Method, Engineering News Record, Vol. 108, April, 1950.
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15.	Bureau of Reclamation, U.S.A.	Design of Small Dams,
16.	Hazen A.	Flood Flows - Jhon Willey & Sons. N.Y. 1930.
17.	Fuller H.A.	Trans. ASCE Vol. 77, 1914.
18.	Gumbel E.J.	Floods Estimated reg: Probability Method - Engineering News Record, June 14, 1945.

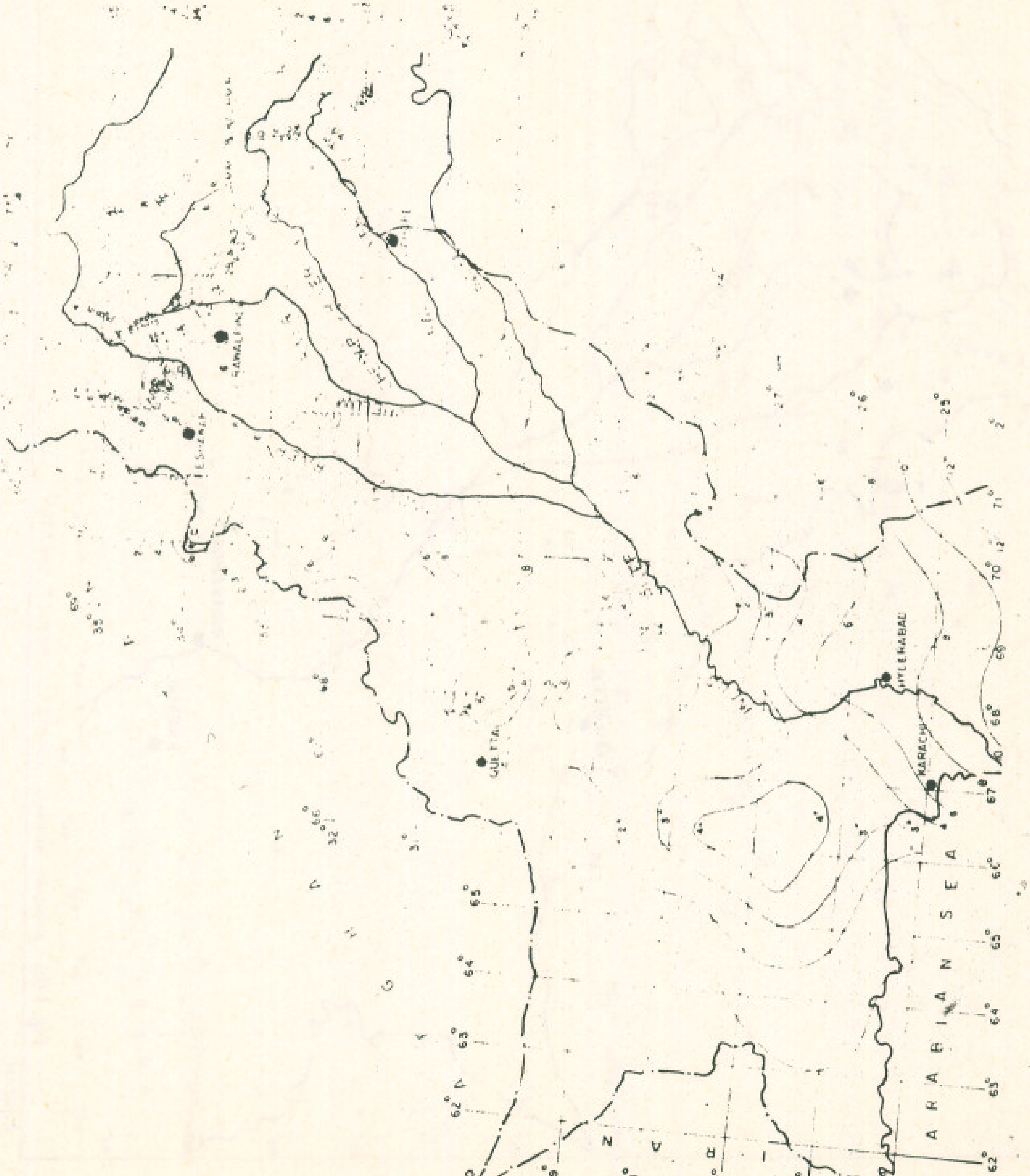


Fig. 1 (a) — MONSOON NORMAL ISOHYETAL MAP OF PAKISTAN

A. Nizami.

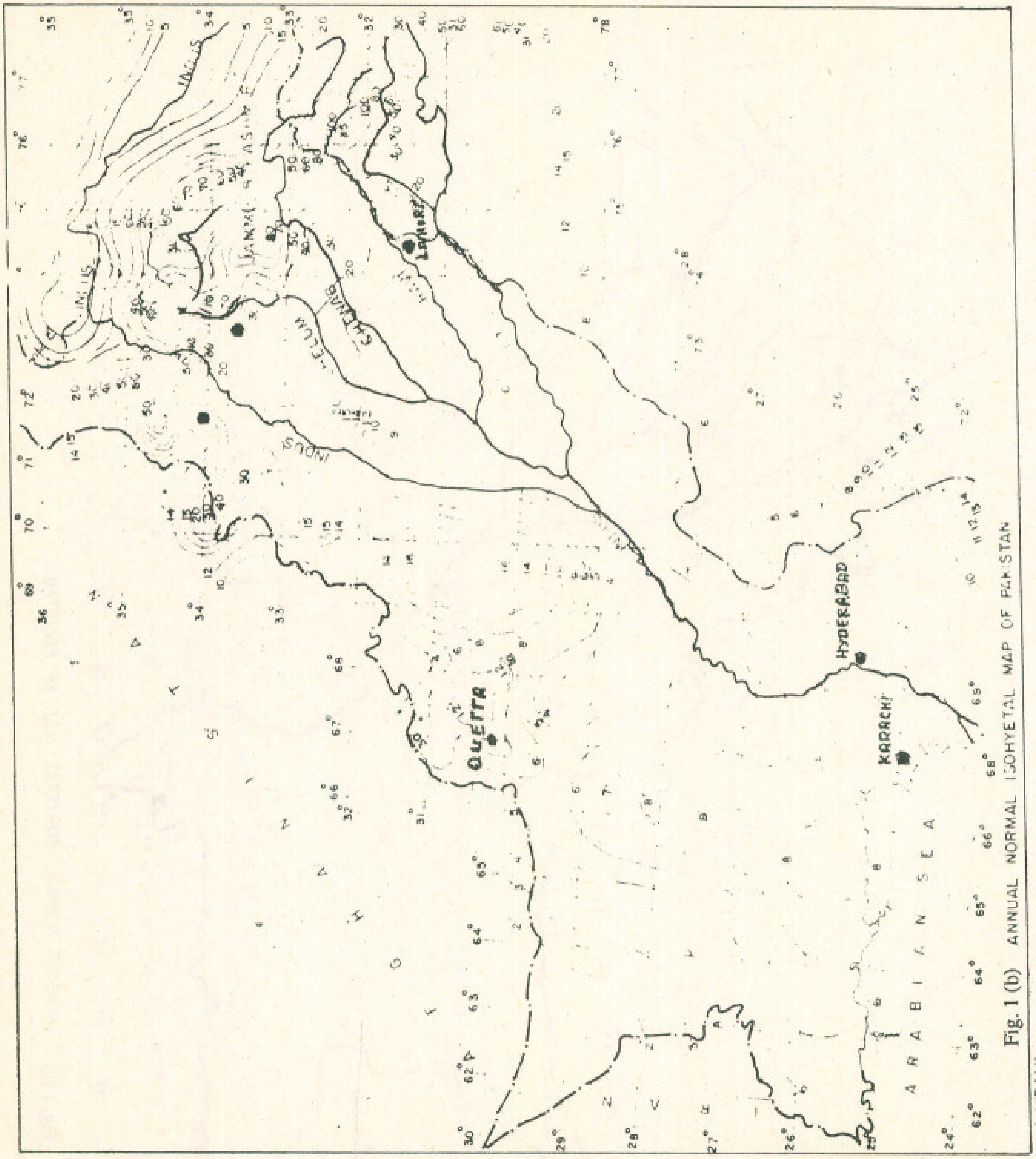


Fig. 1 (b) ANNUAL NORMAL ISOHYETAL MAP OF PAKISTAN

A. NIZAMI.

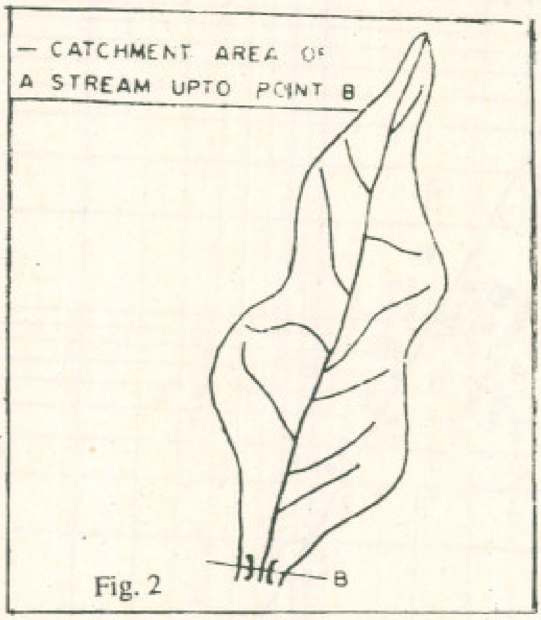


Fig. 2

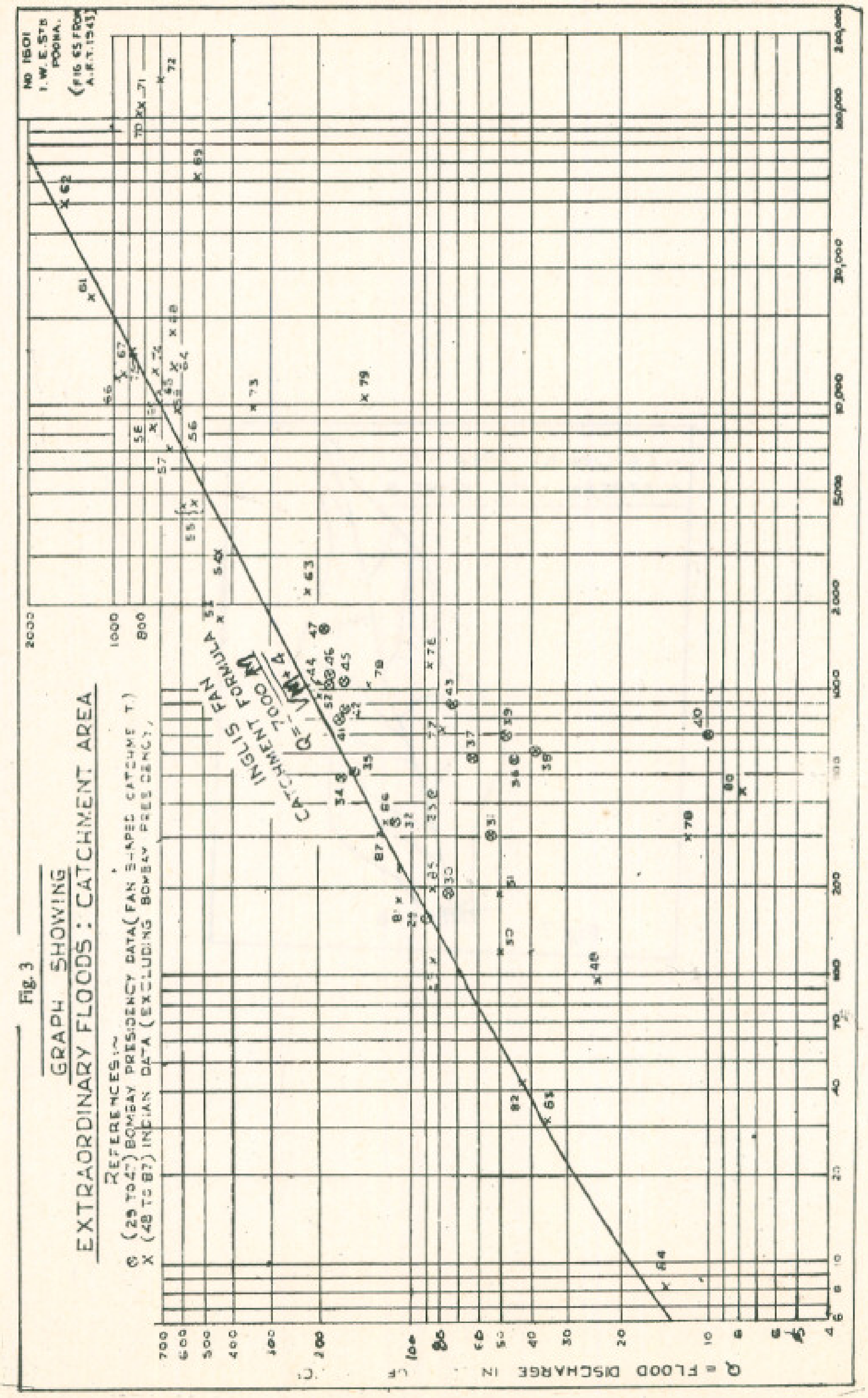
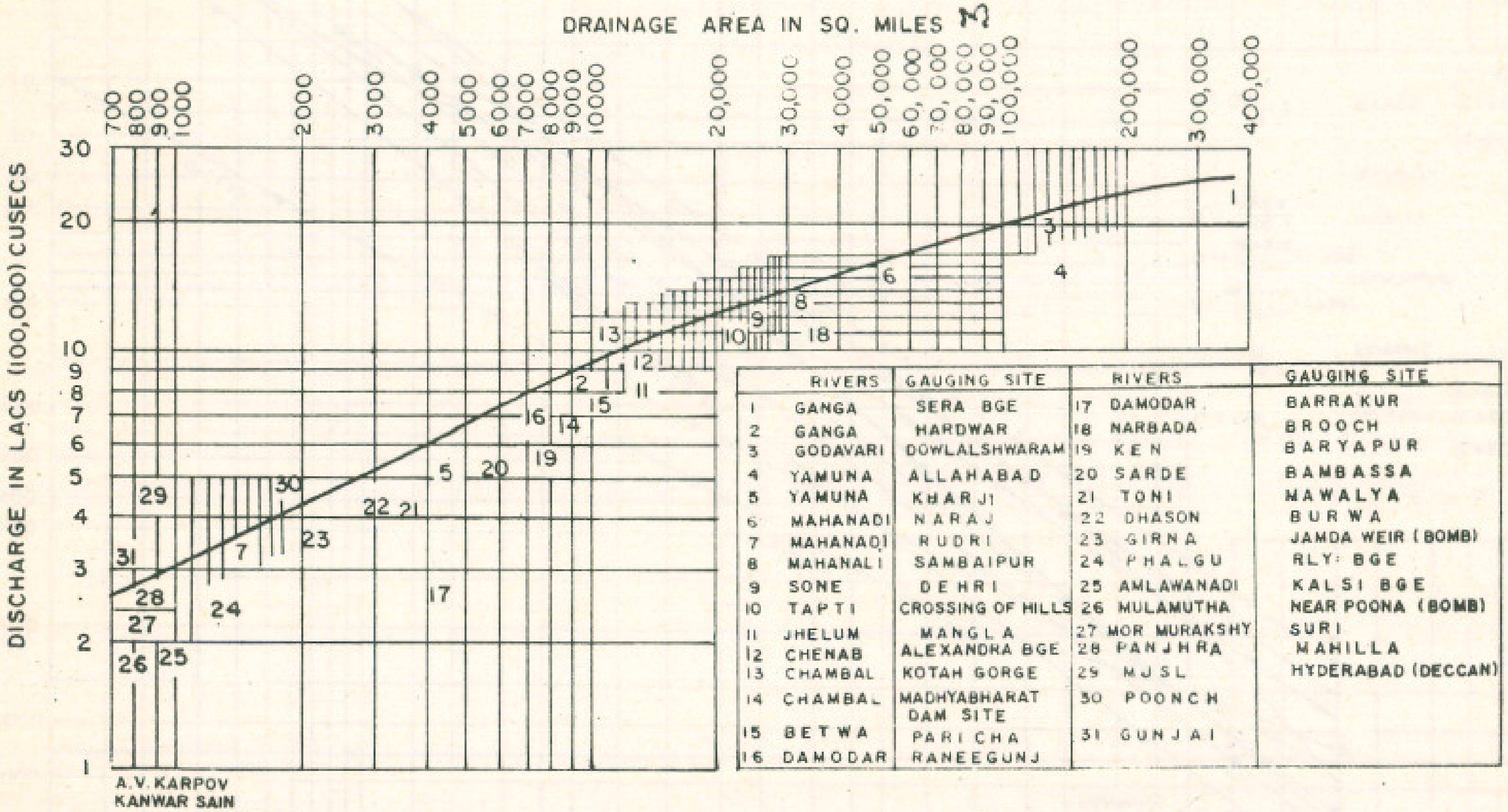


Fig. 3
GRAPH SHOWING
EXTRAORDINARY FLOODS : CATCHMENT AREA
REFERENCES:-
⊗ (29 TO 47) BOMBAY PRESIDENCY DATA (FAN SHAPED CATCHMENT)
X (48 TO 87) INDIAN DATA (EXCLUDING BOMBAY PRESIDENCY)
INGLIS FORMULA FAN CATCHMENT
$Q = \frac{7000\,M}{\sqrt{M+4}}$
Q = FLOOD DISCHARGE IN
NO 1601 I.W.E. STB POONA
(FIG 65 FROM A.R.T. 1943)



	RIVERS	GAUGING SITE		RIVERS	GAUGING SITE
1	GANGA	SERA BGE	17	DAMODAR	BARRAKUR
2	GANGA	HARDWAR	18	NARBADA	BROOCH
3	GODAVARI	DOWLALSHWARAM	19	KEN	BARYAPUR
4	YAMUNA	ALLAHABAD	20	SARDE	BAMBASSA
5	YAMUNA	KHARJI	21	TONI	MAWALYA
6	MAHANADI	NARAJ	22	DHASON	BURWA
7	MAHANADI	RUDRI	23	GIRNA	JAMDA WEIR (BOMB)
8	MAHANALI	SAMBAIPUR	24	PHALGU	RLY: BGE
9	SONE	DEHRI	25	AMLAWANADI	KALSI BGE
10	TAPTI	CROSSING OF HILLS	26	MULAMUTHA	NEAR POONA (BOMB)
11	JHELUM	MANGLA	27	MOR MURAKSHY	SURI
12	CHENAB	ALEXANDRA BGE	28	PANJHRA	MAHILLA
13	CHAMBAL	KOTAH GORGE	29	MUSL	HYDERABAD (DECCAN)
14	CHAMBAL	MADHYABHARAT DAM SITE	30	POONCH	
15	BETWA	PARICHA	31	GUNJAI	
16	DAMODAR	RANEEGUNJ			

Fig. 4 — ENVELOPE CURVE-NORTHERN AND CENTRAL INDIA RIVERS

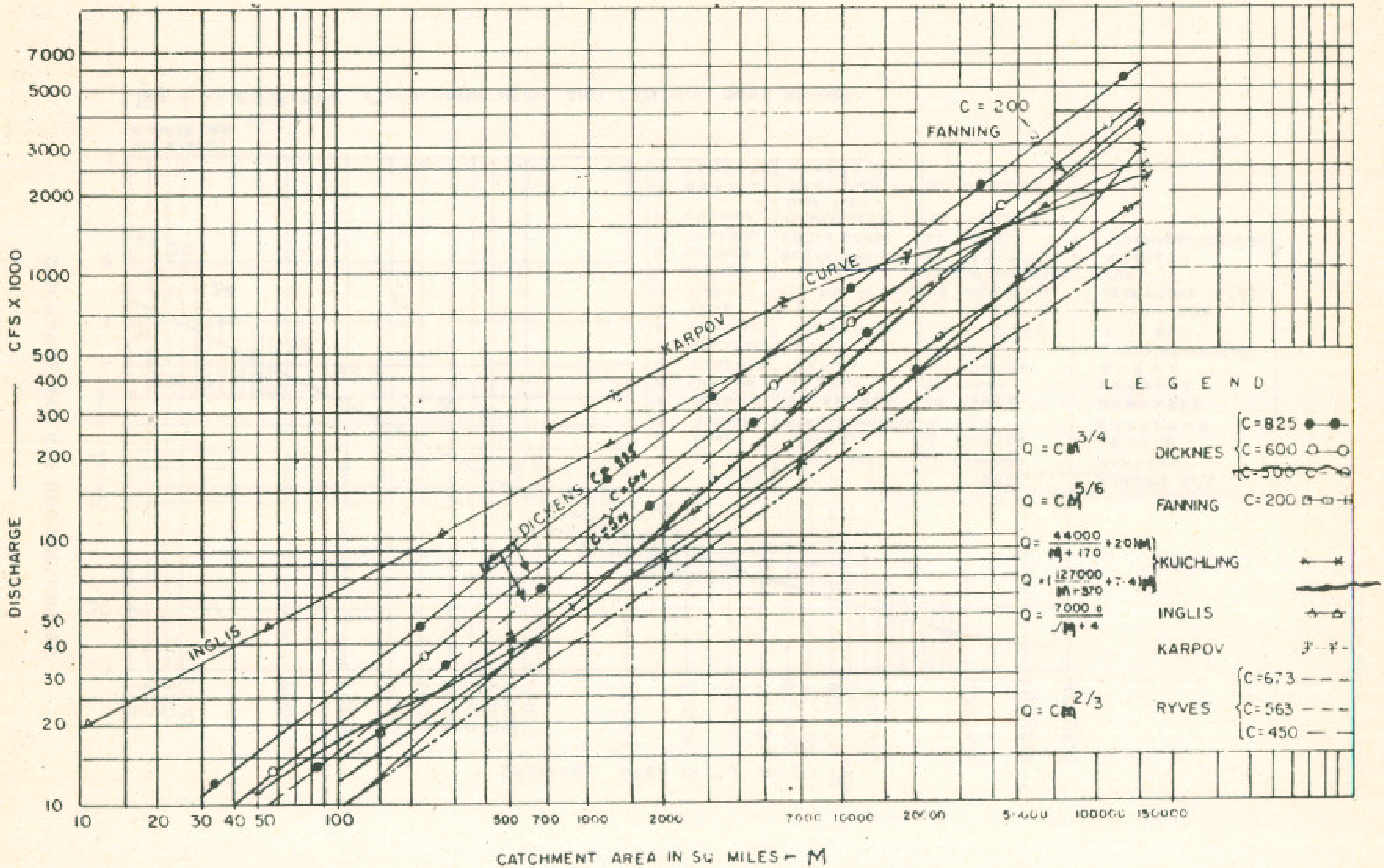

DISCHARGE —— CFS X 1000
CATCHMENT AREA IN SQ MILES – M
INGLIS
DICKENS
KARPOV
CURVE
C = 200
FANNING
L E G E N D
Q = CM^3/4
DICKNES
C=825
C=600
C=500
Q = CM^5/6
FANNING
C= 200
Q = 44000/(M + 170) + 20 M
Q = (127000/(M + 370) + 7·4) M
KUICHLING
Q = 7000 a/√M + 4
INGLIS
KARPOV
Q = CM^2/3
RYVES
C=673
C=563
C=450

Fig. 5

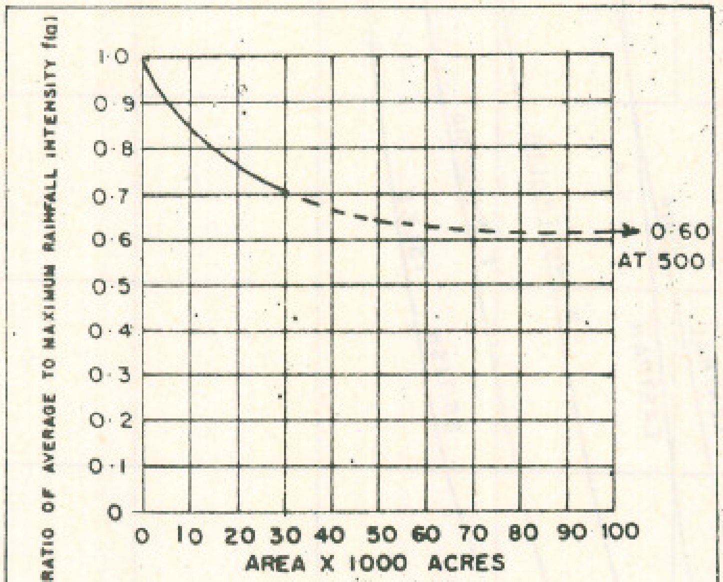

RATIO OF AVERAGE TO MAXIMUM RAINFALL INTENSITY f(a)
1·0
0·9
0·8
0·7
0·6
0·5
0·4
0·3
0·2
0·1
0
0 10 20 30 40 50 60 70 80 90 100
AREA X 1000 ACRES
0·60
AT 500

Fig. 6 (a)

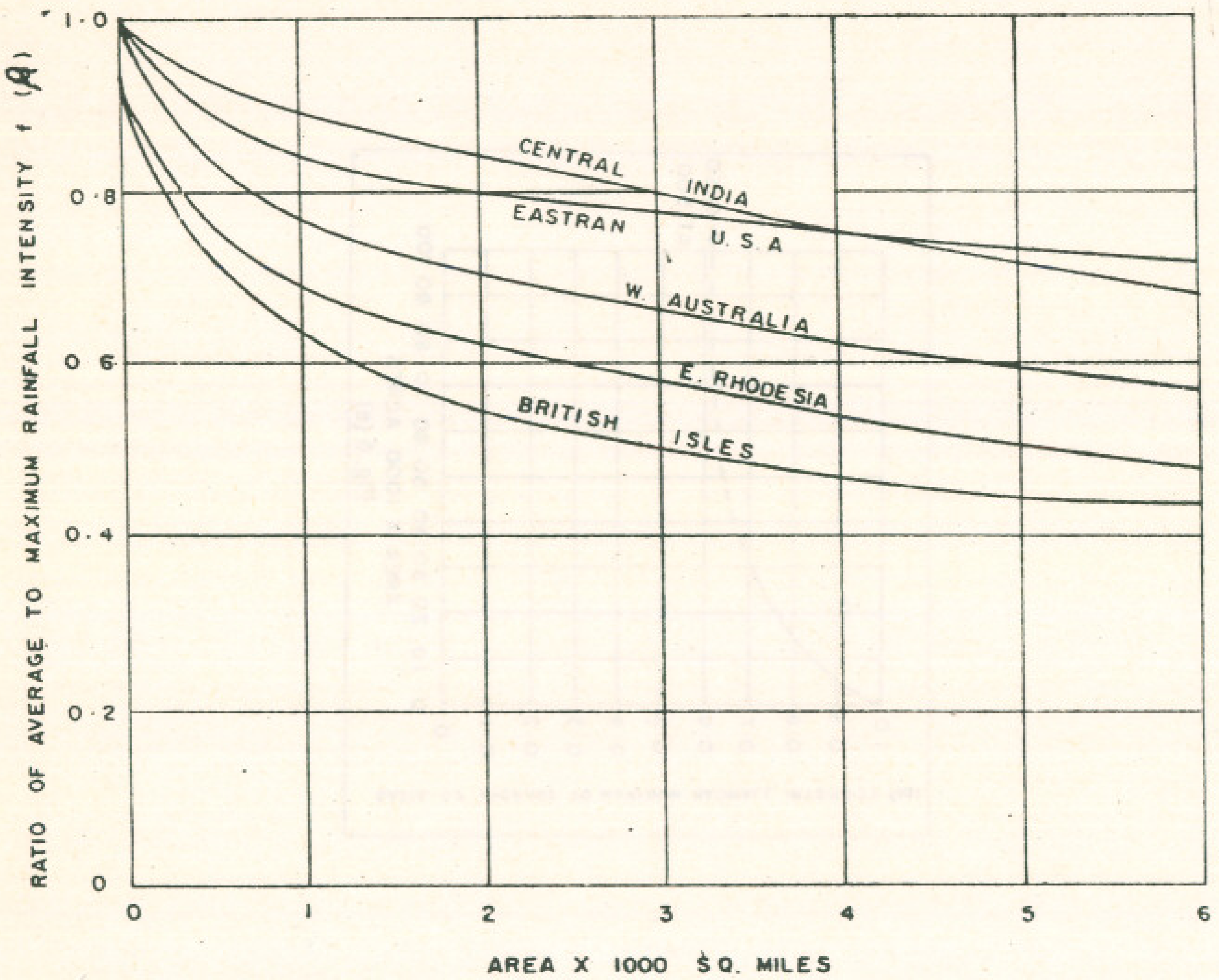


Fig. 6 (b) — RATIO OF AVERAGE TO MAXIMUM RAINFALL INTENSITY VS AREA OF CATCHMEN

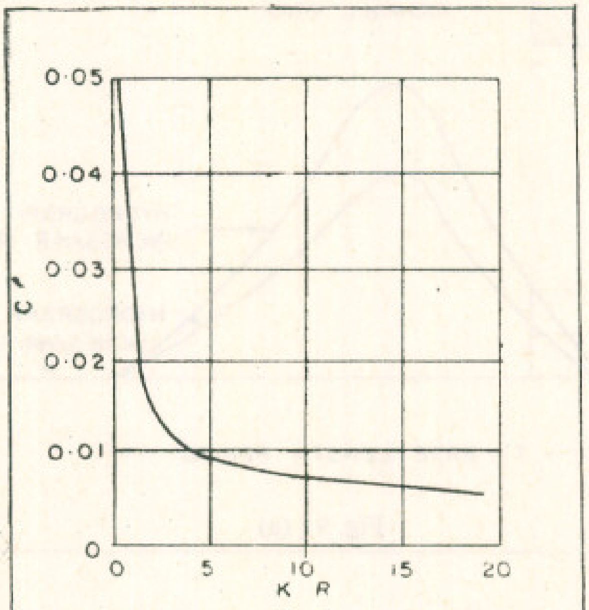


Fig. 7

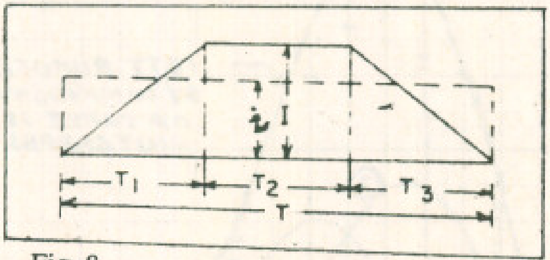


Fig. 8

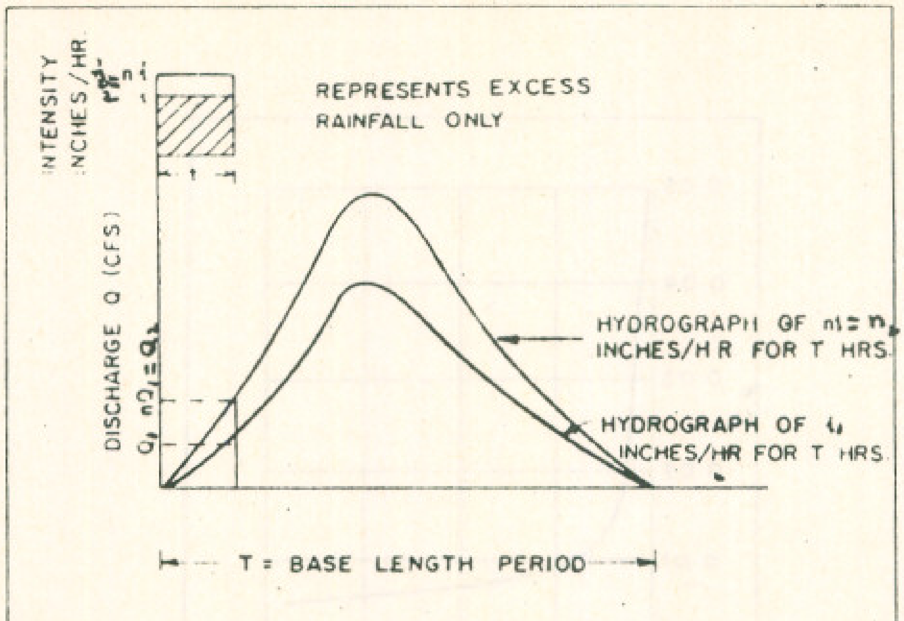


Fig. 9 ((a)

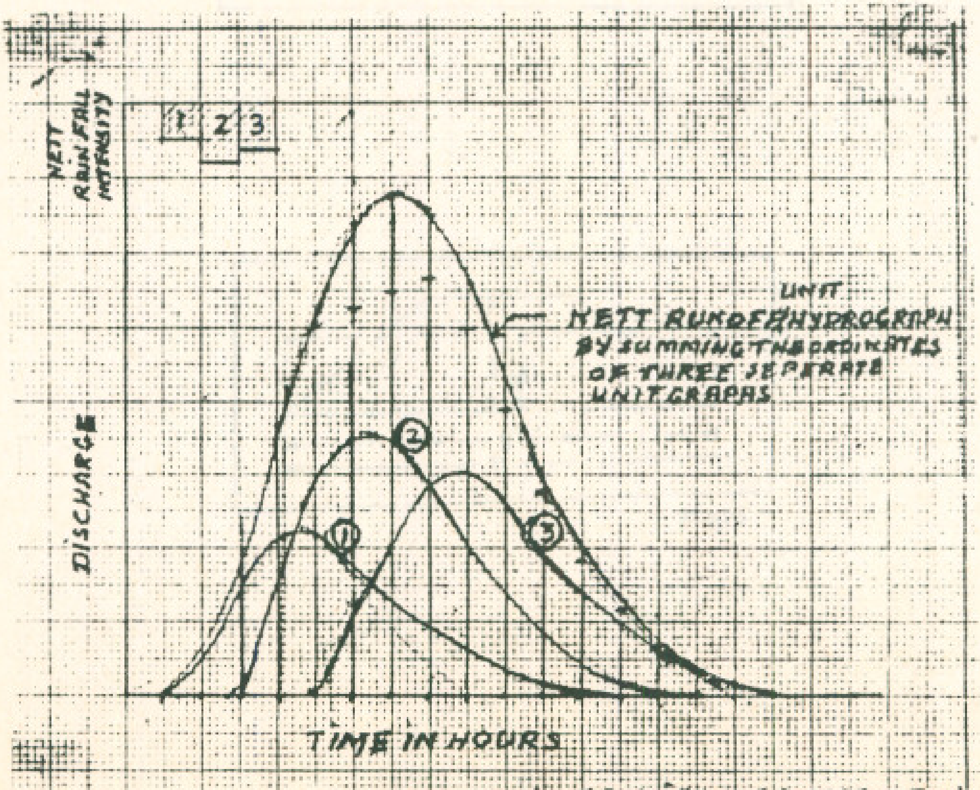


Fig. 9(b) UNITGRAPH — PRINCIPAL OF SUPERPOSITION

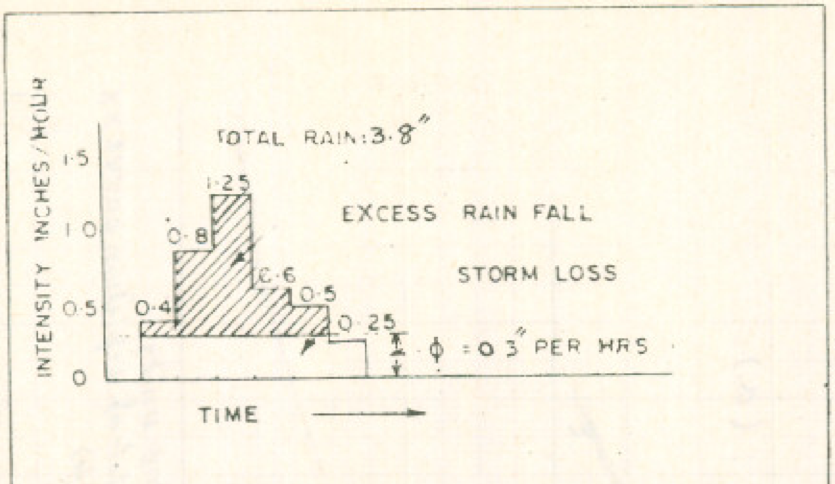


Fig. 10 INDEX METHOD OF FINDING EXCESS RAINFALL

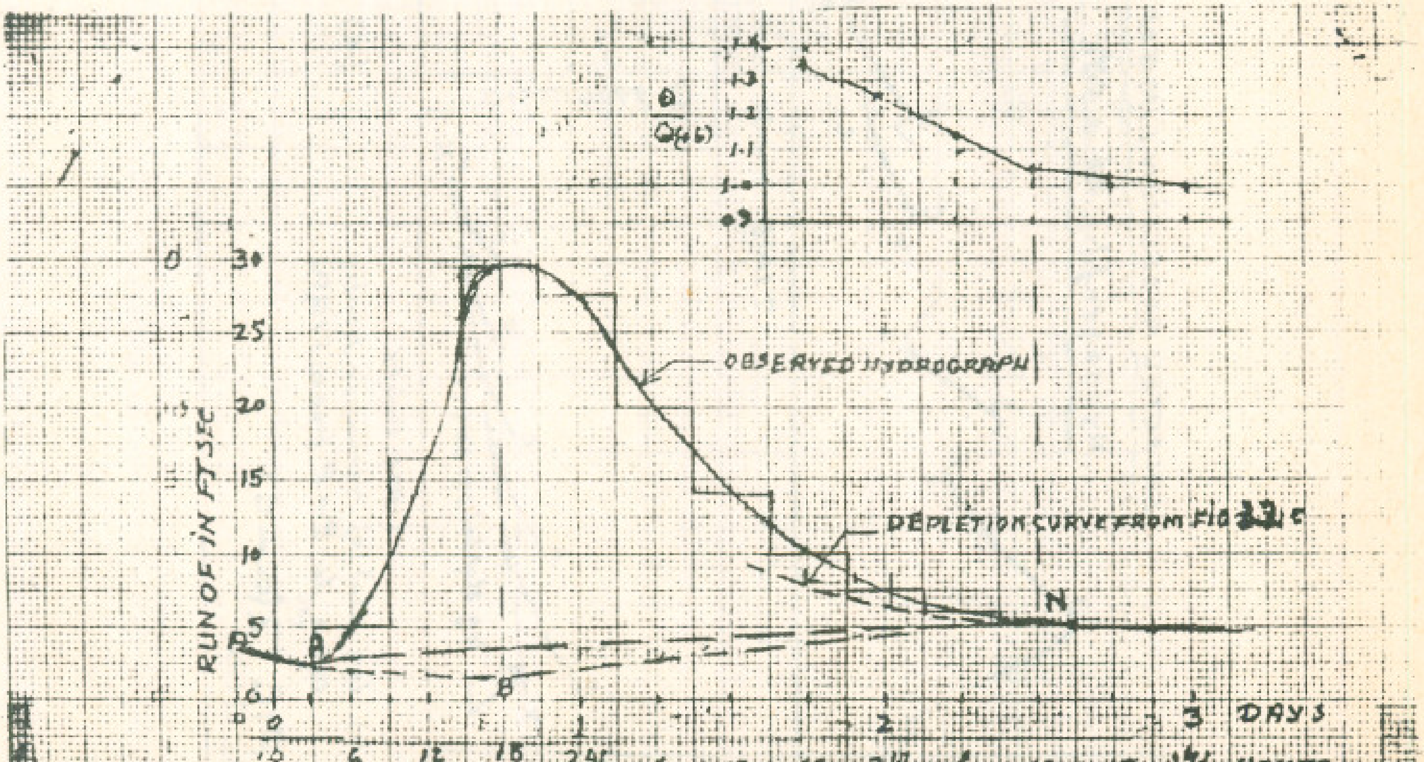


Fig. 11

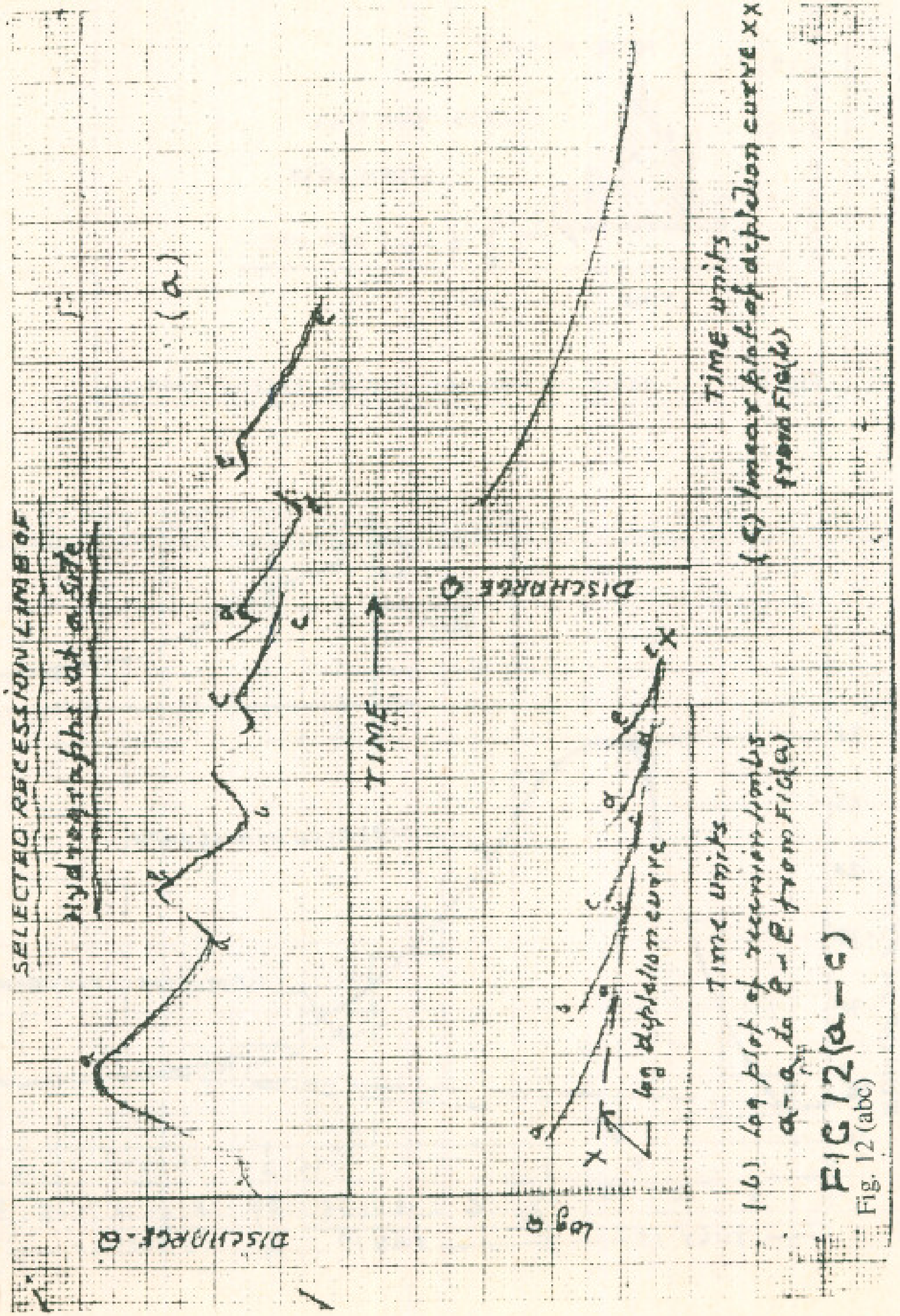

SELECTED RECESSION LIMB OF
hydrographs at a site
(a)
DISCHARGE Q
TIME
DISCHARGE Q
TIME units
(c) linear plot of depletion curve XX
from Fig(b)
log Q
log depletion curve
Time units
(b) log plot of recession limbs
a-a to e-e from Fig(a)
FIG 12(a-c)

Fig. 12 (abc)

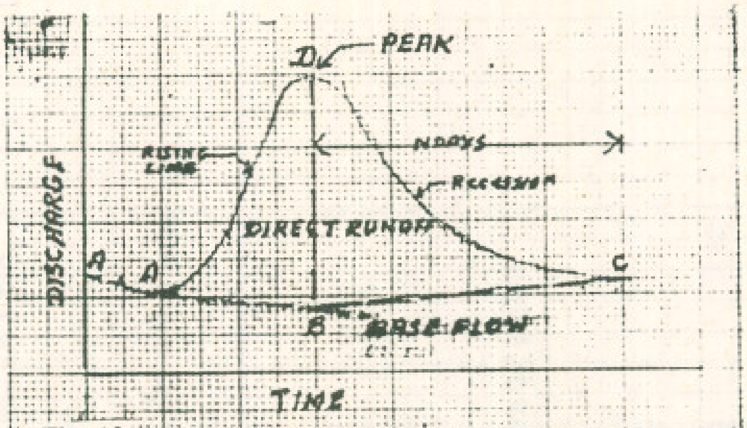


Fig. 13. METHOD OF SEPERATION OF DIRECT and GROUND water or BASEFLOW

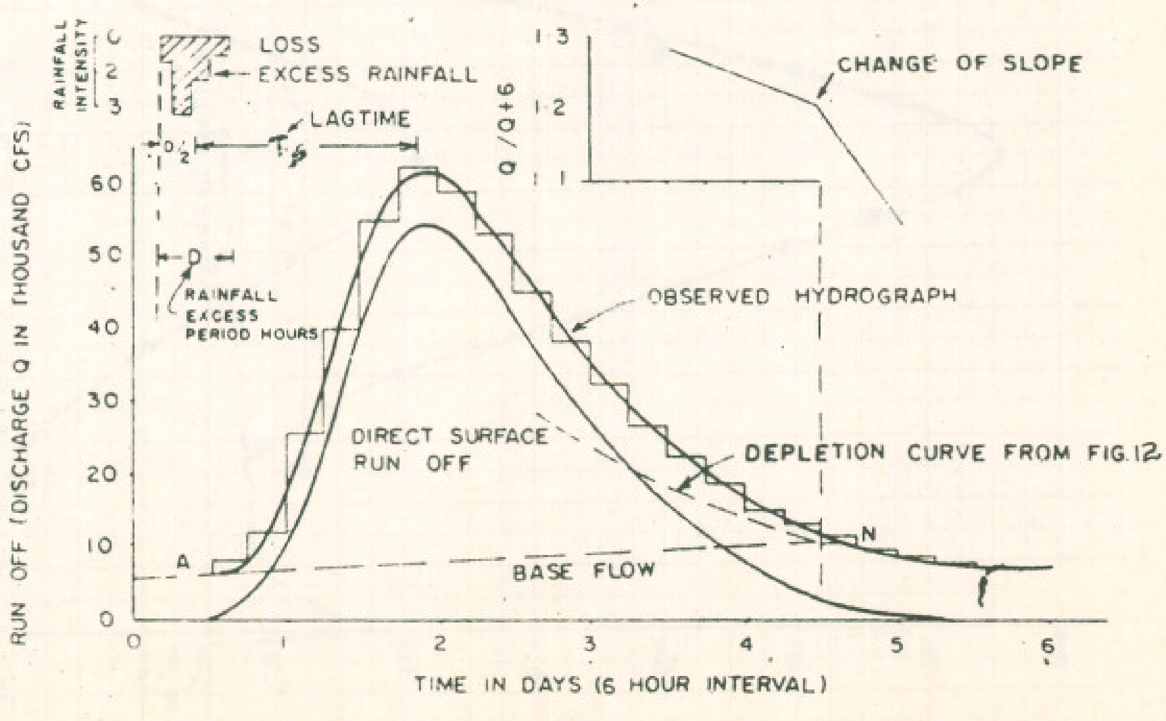


Fig. 14

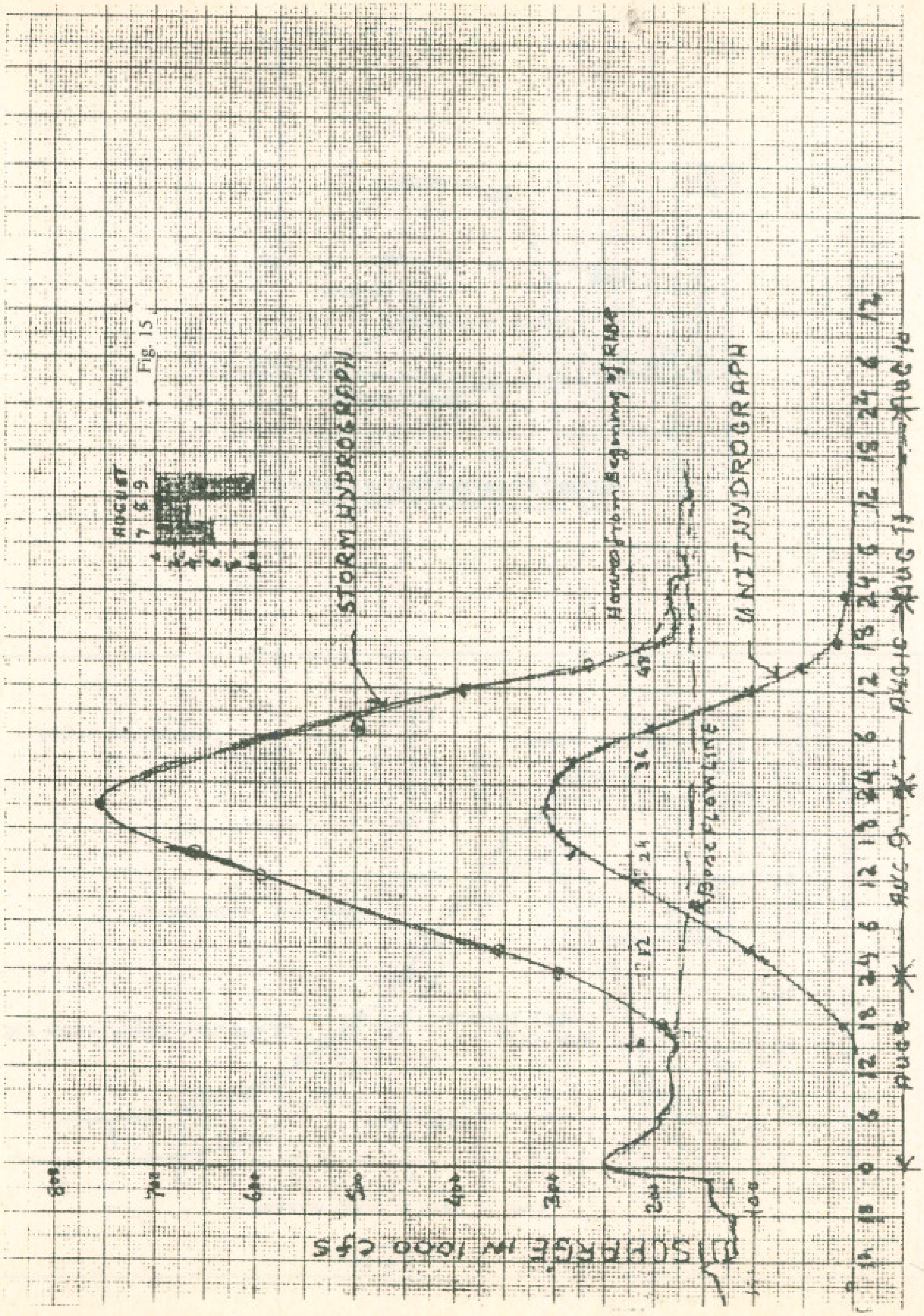

STORM HYDROGRAPH
UNIT HYDROGRAPH
BASE FLOW LINE
AUGUST
DISCHARGE IN 1000 cfs
AUG 9
AUG 10
AUG 11
AUG 12

Fig. 15

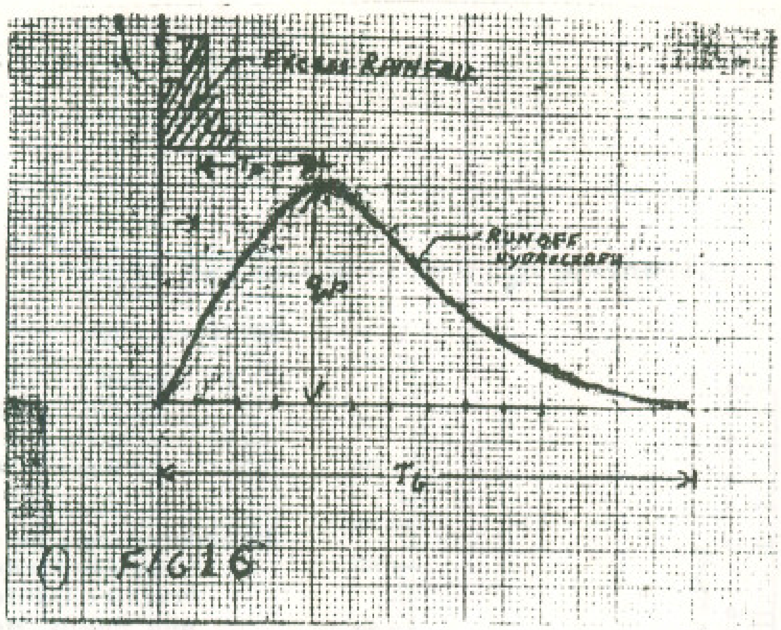


Fig. 16

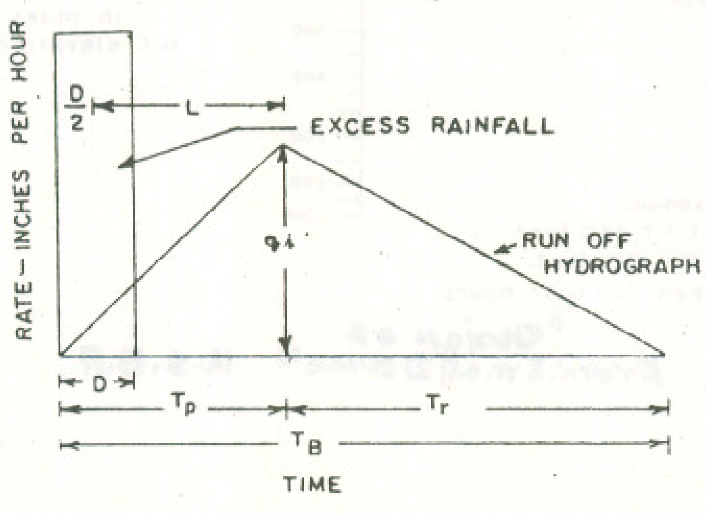
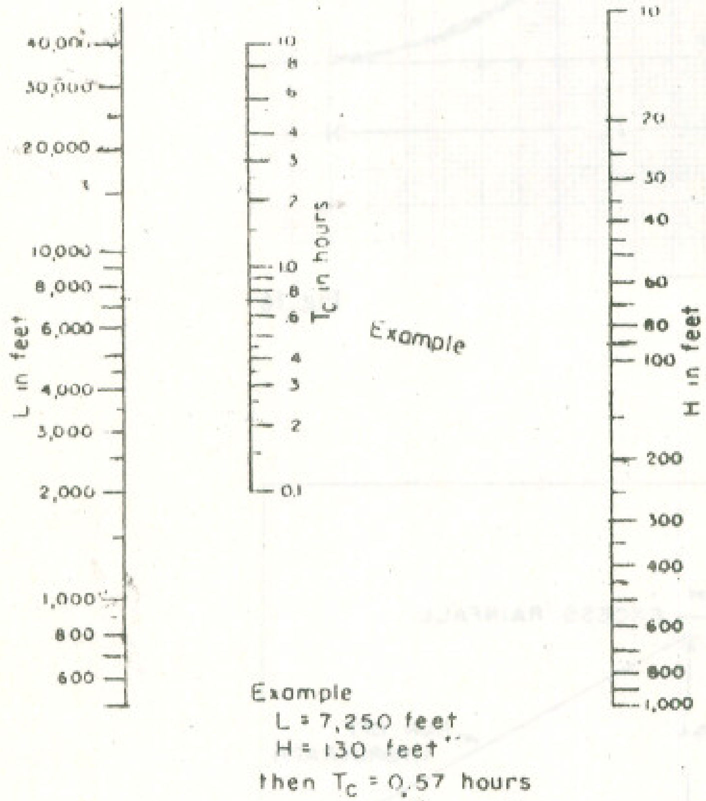


Fig. 17 — TRIANGULAR HYDROGRAPH

C. ESTIMATING T_c FROM LENGTHS AND SLOPE



(a) Nomograp...

L ... length ... longest ... course ... feet

H = differen... in elevat... between ... tlet point

(b.) Solution may be made by e... from California Culvert Practice, California Highw. and Public Works, Septe. 1942.

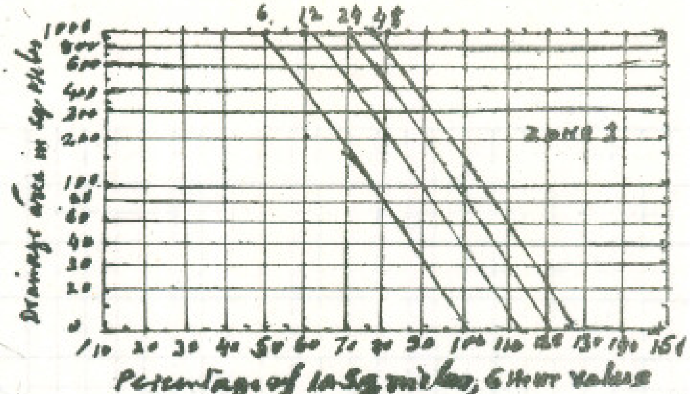
$$T = \left(\frac{11.9\,L^3}{H}\right)^{0.385}$$

T = T_c in hours

L = length of lo... st water ...urse in miles

H = elevation difference in feet

Fig. 18 "Design of Farm: Small Dams" U.S.B.R



Depth area duration relationship percentage to be applied to 10 sq miles, 6 hour probable maximum precipitation value

Fig. 19

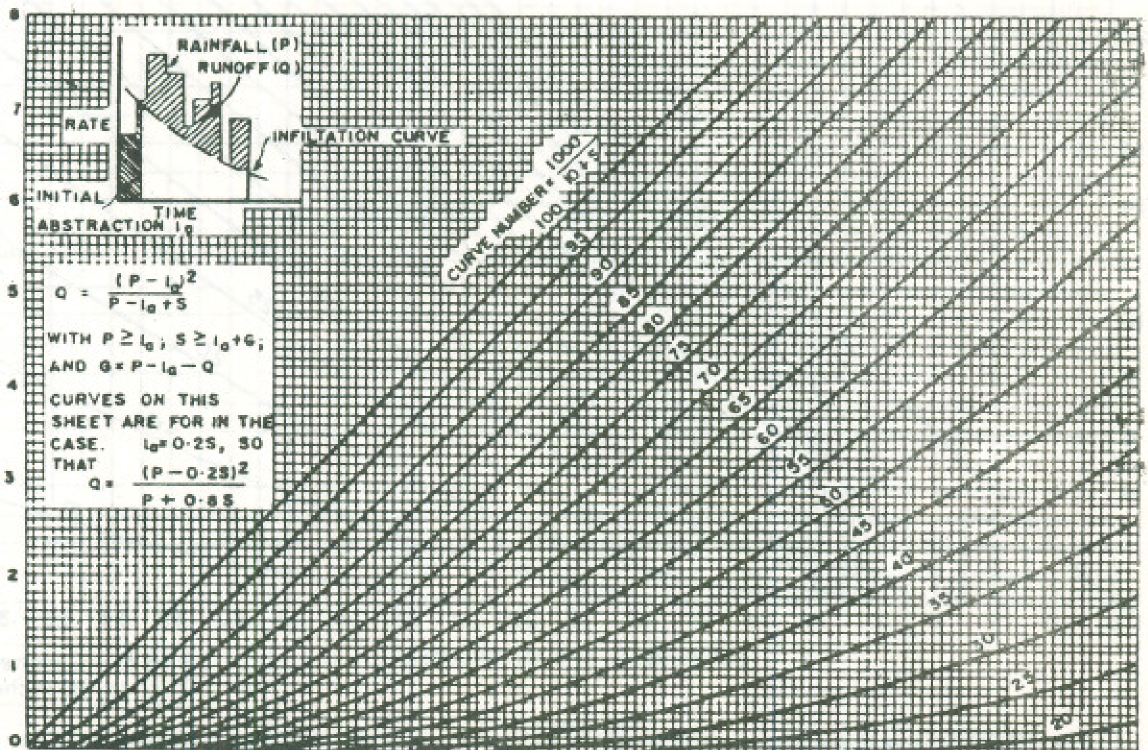


Fig. 19 (a) SOLUTION OF RUNOFF EQUATION, $Q = \frac{(P - 0{\cdot}2S)^2}{P + 0{\cdot}8S}$

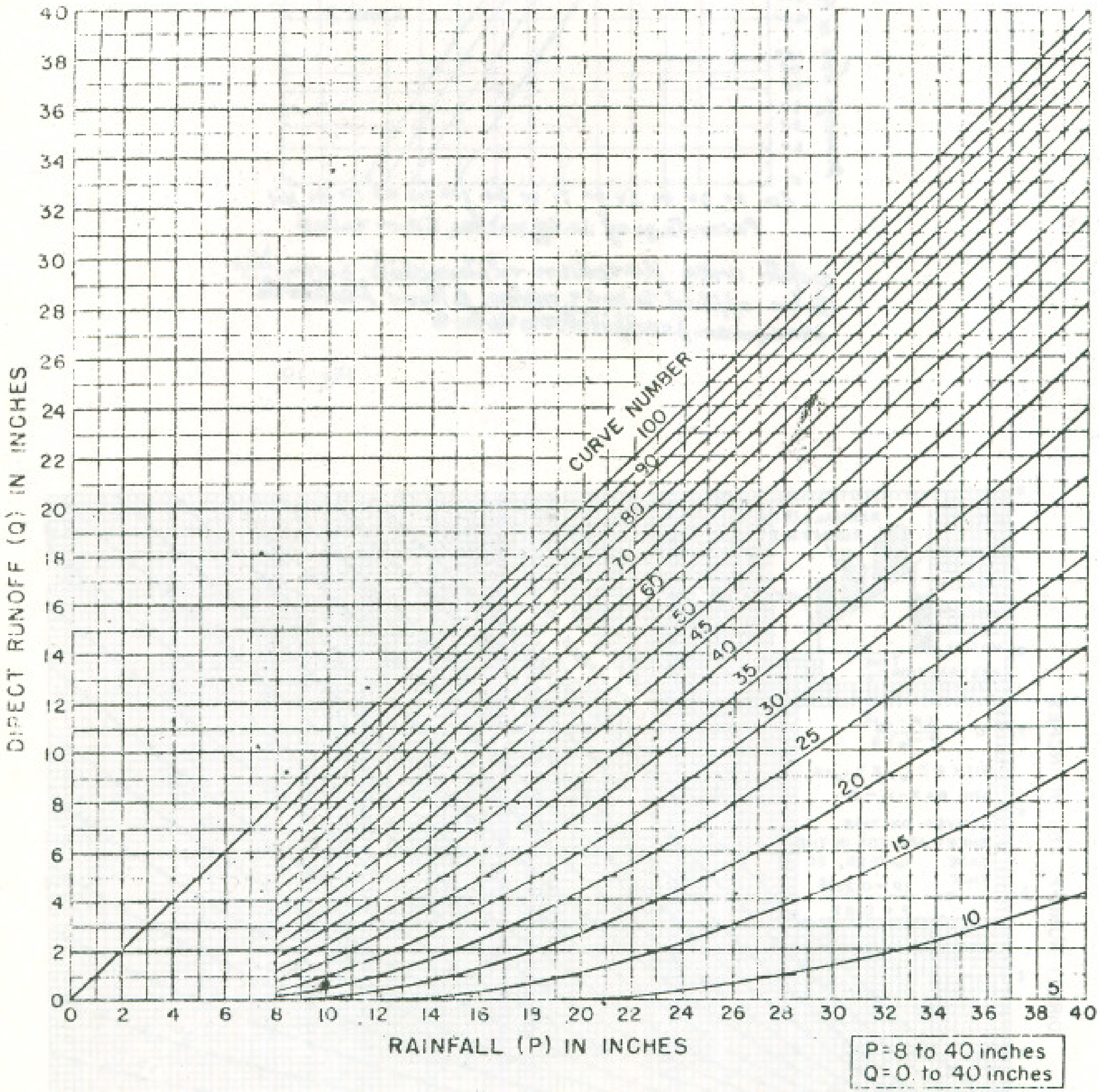


Fig. 19 (b) Solution of runoff equation, $Q = \frac{(P - 0.2S)^2}{P + 0.8S}$ (Sheet 2 of 2.) (U.S. Soil Conservation Service.)

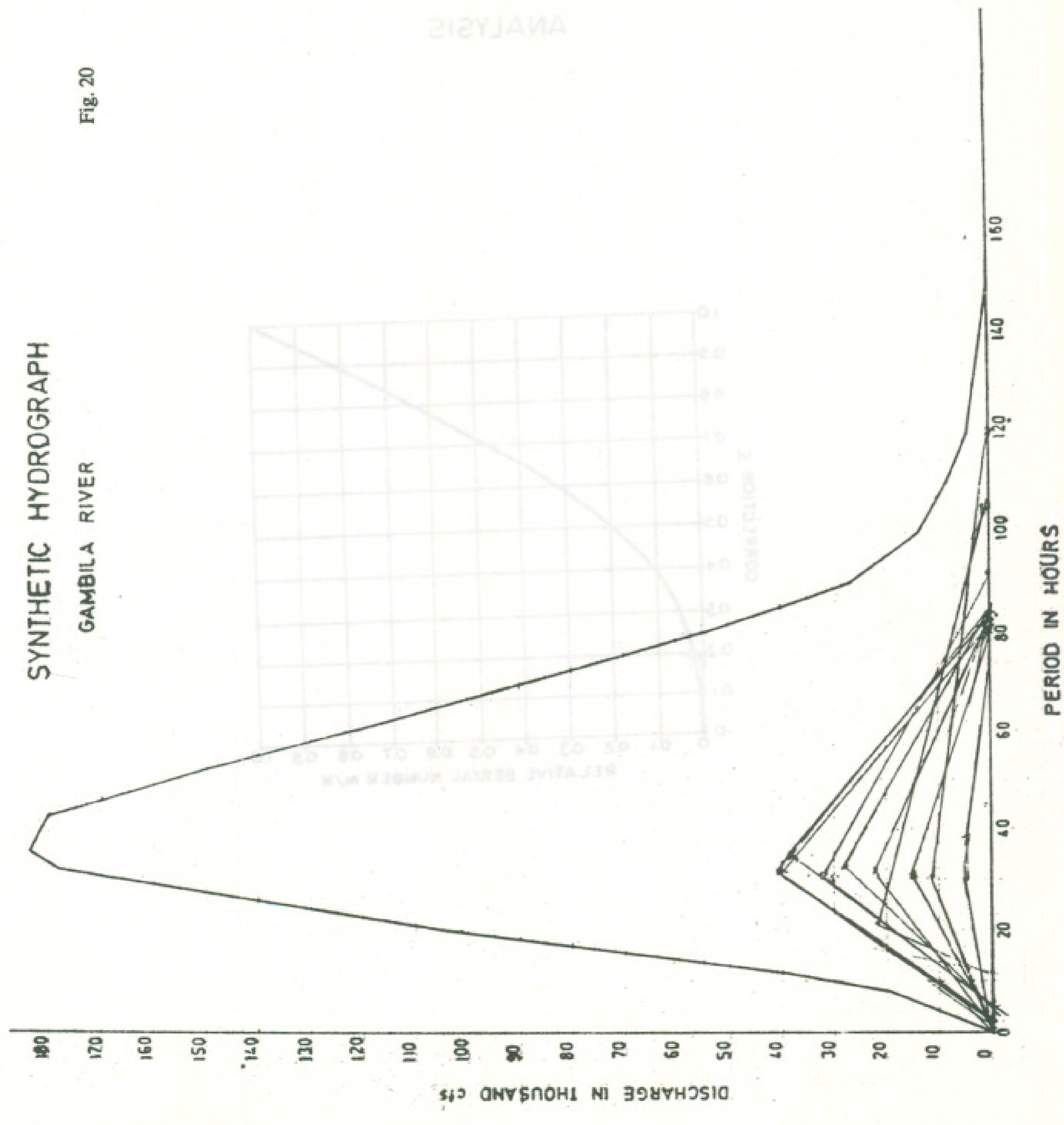

SYNTHETIC HYDROGRAPH
GAMBILA RIVER
DISCHARGE IN THOUSAND cfs
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
PERIOD IN HOURS
0
20
40
60
80
100
120
140
160

Fig. 20

CURVE OF CORRECTION 'C' IN GUMBEL'S METHOD OF FREQUENCY ANALYSIS

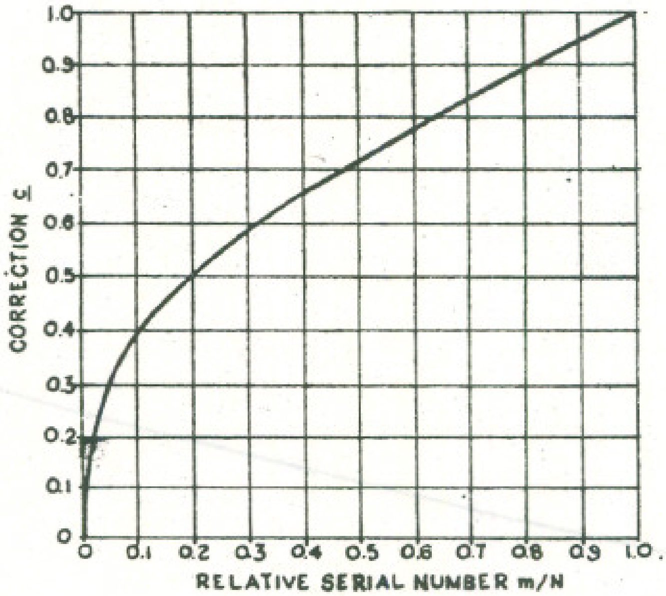
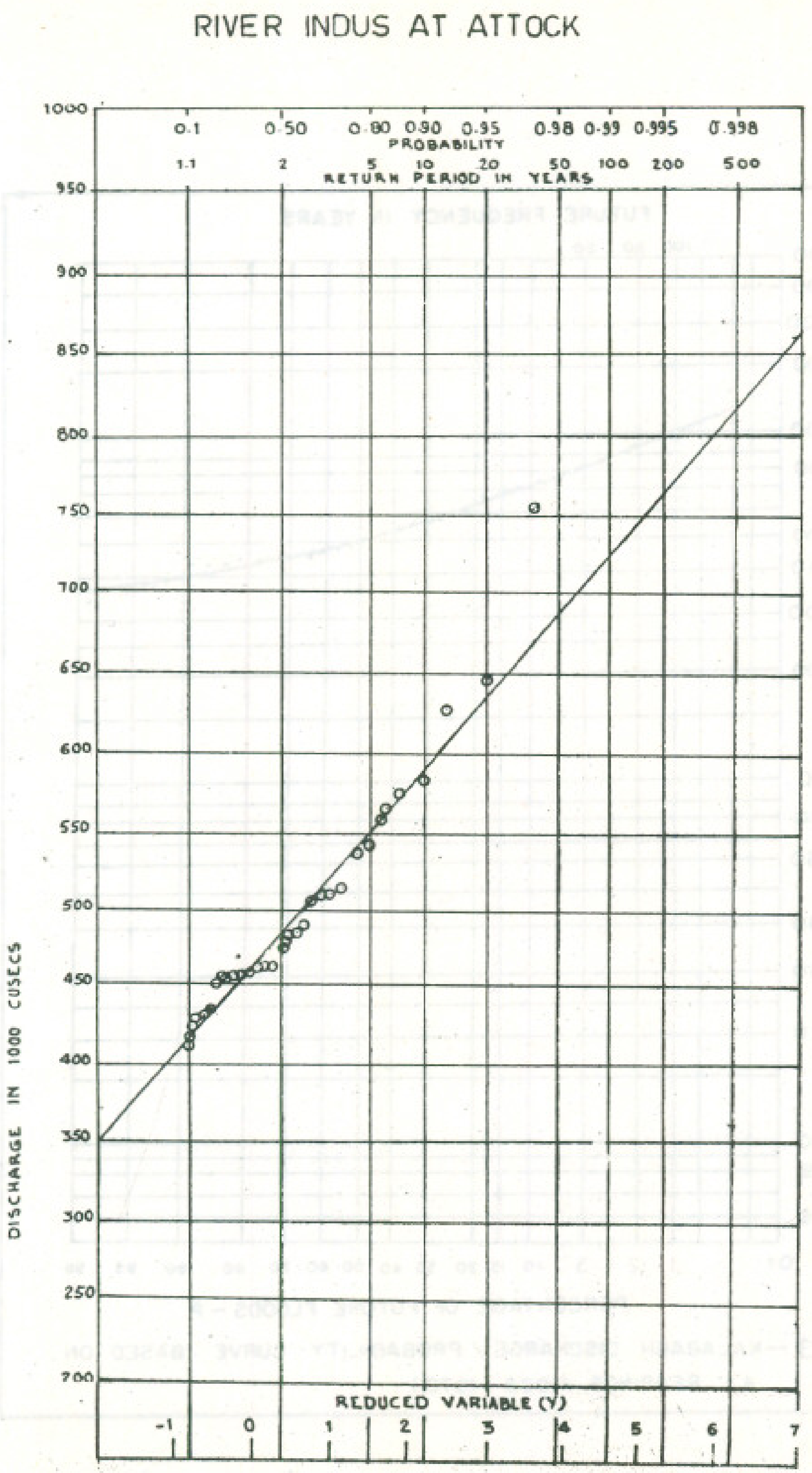


Fig. 21



FREQUENCY CURVE BY GUMBEL'S METHOD

Fig. 22

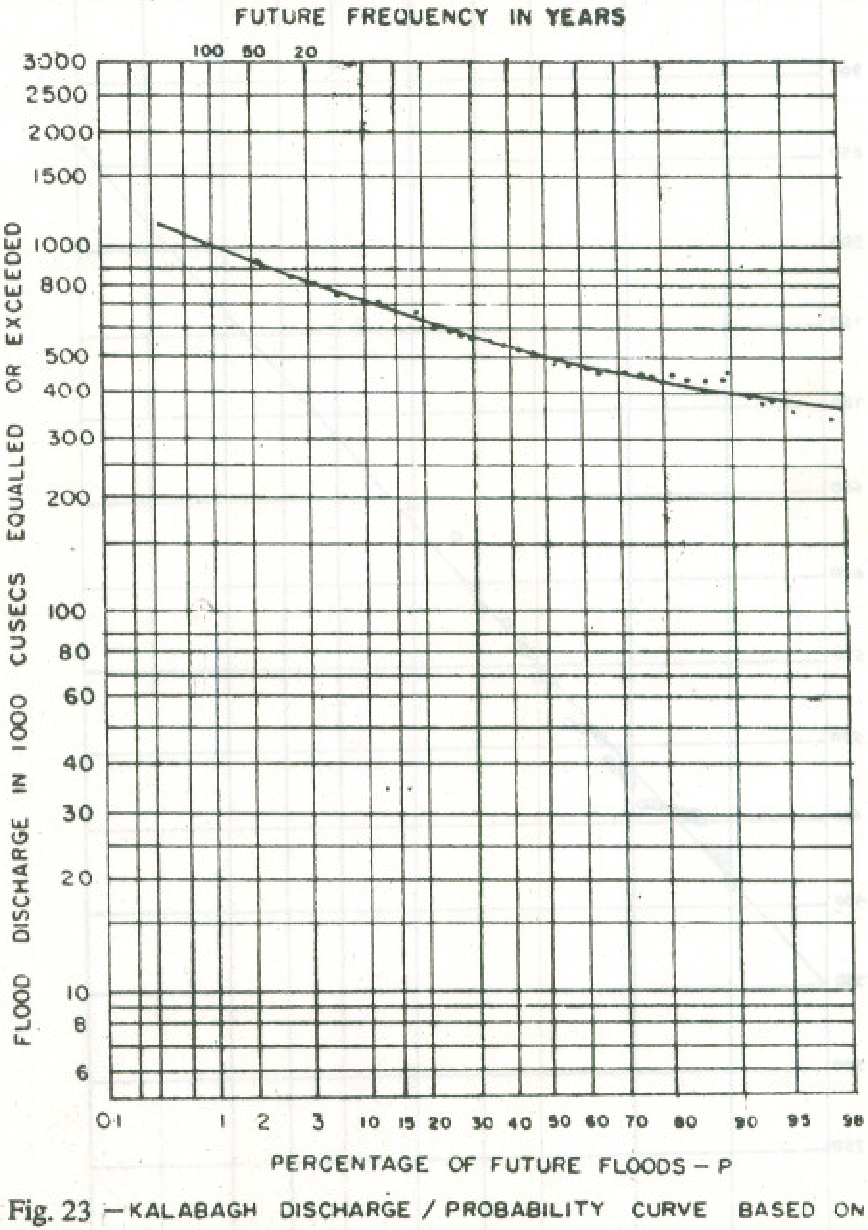


Fig. 23 — KALABAGH DISCHARGE / PROBABILITY CURVE BASED ON 43 READINGS (1928 – 1970)

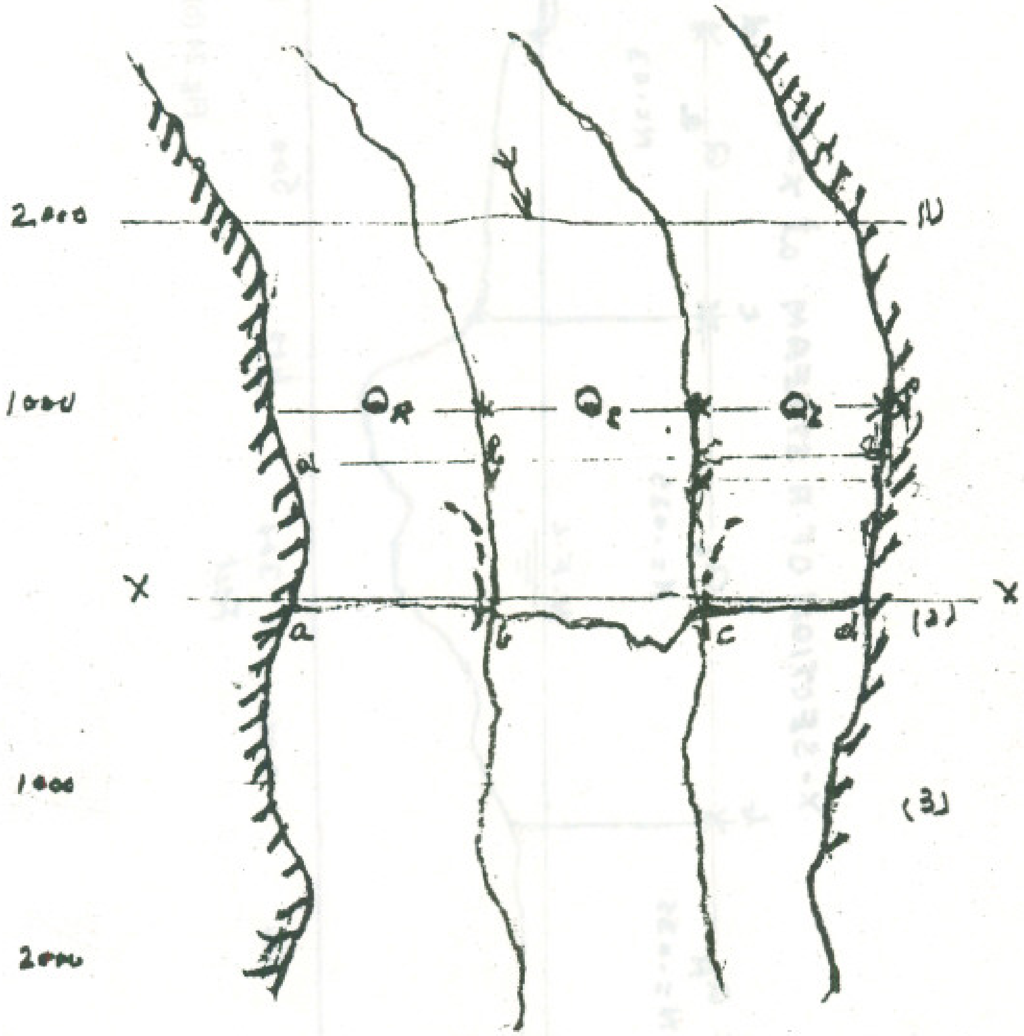


Fig. 24 (a)

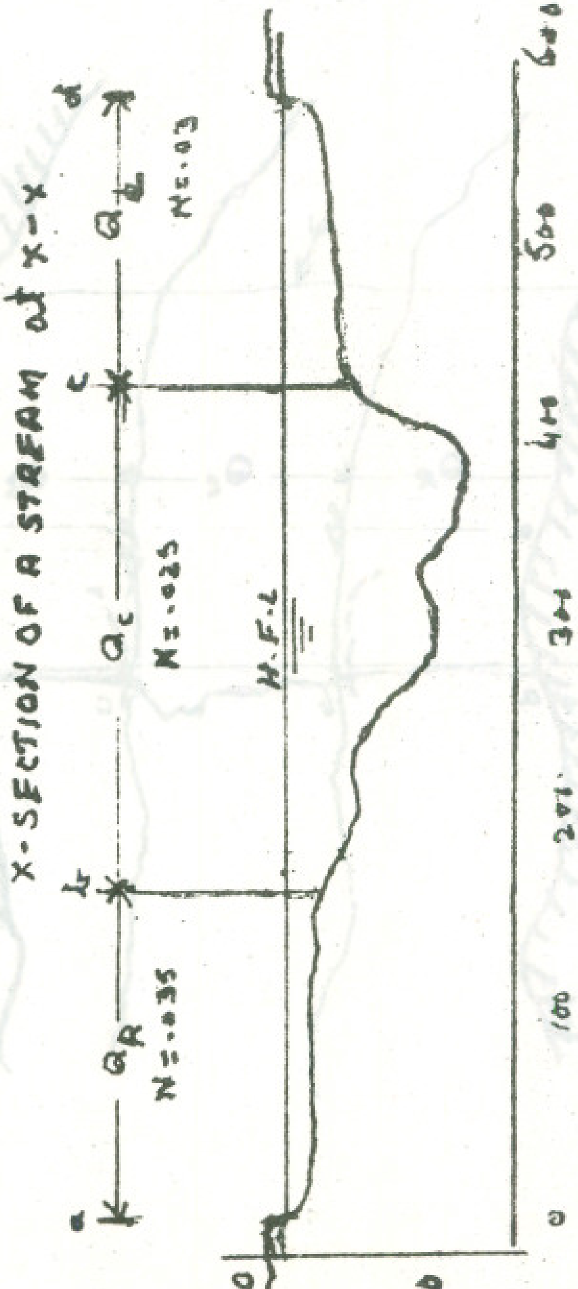

X-SECTION OF A STREAM at X-X
a
b
c
d
Q_R
N=.035
Q_c
N=.025
Q_L
N=.03
H.F.L
910
900
0
100
200
300
400
500
600
Feet

Fig. 24 (b)

APPENDIX I

Table showing values of

$$\frac{t^3}{t+1} : t$$

Values of $\frac{t^3}{t+1}$ for values of t from 0·30 to 10·00 by hundredths.

t	0	1	2	3	4	5	6	7	8	9	Diff.
0·3	·021	·023	·025	·027	·029	·032	·034	·037	·040	·043	·003
0·4	·045	·049	·052	·055	·059	·063	·067	·071	·075	·079	·004
0·5	·083	·088	·092	·097	·102	·107	·112	·118	·123	·129	·005
0·6	·135	·141	·147	·153	·160	·166	·173	·180	·187	·194	·007
0·7	·202	·210	·218	·225	·233	·241	·250	·259	·267	·273	·008
0·8	·284	·293	·303	·312	·322	·332	·342	·352	·362	·373	·010
0·9	·384	·394	·405	·426	·427	·439	·451	·463	·475	·487	·011
1·0	·500	·512	·525	·538	·551	·564	·578	·591	·605	·619	·013
1·1	·634	·648	·662	·677	·692	·707	·723	·738	·754	·769	·015
1·2	·785	·801	·818	·834	·851	·868	·885	·902	·920	·937	·017
1·3	·955	·973	·991	1·01	1·03	1·05	1·07	1·09	1·11	1·12	·019
1·4	1·14	1·16	1·18	1·20	1·22	1·24	1·26	1·28	1·31	1·33	·02
1·5	1·35	1·37	1·39	1·41	1·43	1·45	1·48	1·50	1·53	1·55	·02
1·6	1·57	1·59	1·62	1·64	1·67	1·69	1·72	1·74	1·77	1·79	·02
1·7	1·82	1·84	1·87	1·89	1·92	1·94	1·97	2·00	2·03	2·05	·02
1·8	2·08	2·11	2·14	2·16	2·19	2·22	2·25	2·27	2·30	2·33	·03
1·9	2·36	2·39	2·42	2·45	2·48	2·51	2·54	2·57	2·60	2·63	·03
2·0	2·67	2·70	2·73	2·76	2·79	2·82	2·86	2·89	2·92	2·95	·03
2·1	2·99	3·02	3·05	3·08	3·12	3·15	3·19	3·22	3·26	3·29	·03
2·2	3·33	3·36	3·40	3·43	3·47	3·50	3·54	3·57	3·61	3·65	·04
2·3	3·69	3·72	3·76	3·79	3·83	3·87	3·91	3·95	3·99	4·03	·04
2·4	4·07	4·10	4·14	4·18	4·22	4·26	4·30	4·34	4·38	4·42	·04
2·5	4·46	4·50	4·54	4·58	4·63	4·67	4·71	4·75	4·79	4·83	·04
2·6	4·88	4·92	4·97	5·01	5·06	5·10	5·14	5·18	5·23	5·27	·04
2·7	5·32	5·36	5·41	5·45	5·50	5·54	5·59	5·63	5·68	5·72	·04
2·8	5·77	5·82	5·87	5·92	5·97	6·01	6·06	6·11	6·16	6·20	·05
2·9	6·25	6·30	6·35	6·40	6·45	6·50	6·55	6·60	6·65	6·70	·05
3·0	6·75	6·80	6·85	6·90	6·95	7·00	7·05	7·10	7·16	7·21	·05
3·1	7·26	7·32	7·37	7·42	7·48	7·53	7·59	7·64	7·69	7·74	·05
3·2	7·80	7·85	7·91	7·96	8·02	8·07	8·13	8·19	8·25	8·30	·05
3·3	8·36	8·41	8·47	8·53	8·59	8·64	8·70	8·76	8·82	8·87	·05
3·4	8·93	8·99	9·05	9·11	9·17	9·23	9·29	9·34	9·40	9·46	·06
3·5	9·52	9·58	9·65	9·72	9·77	9·83	9·89	9·94	10·01	10·08	·06
3·6	10·14	10·21	10·27	10·33	10·39	10·45	10·52	10·58	10·65	10·71	·06
3·7	10·78	10·84	10·91	10·97	11·04	11·10	11·17	11·23	11·30	11·36	·06
3·8	11·43	11·49	11·56	11·63	11·70	11·77	11·84	11·90	11·97	12·03	·07
3·9	12·10	12·17	12·24	12·31	12·38	12·45	12·52	12·59	12·66	12·73	·07
4·0	12·80	12·87	12·94	13·01	13·08	13·15	13·22	13·29	13·37	13·44	·07
4·1	13·51	13·58	13·66	13·73	13·80	13·87	13·95	14·02	14·10	14·17	·07
4·2	14·25	14·32	14·40	14·47	14·55	14·62	14·70	14·77	14·85	14·92	·07
4·3	15·00	15·07	15·15	15·22	15·30	15·38	15·46	15·54	15·62	15·70	·07
4·4	15·78	15·86	15·94	16·02	16·10	16·18	16·26	16·35	16·42	16·50	·08
4·5	16·58	16·66	16·74	16·82	16·90	16·98	17·06	17·14	17·22	17·30	·08
4·6	17·38	17·46	17·54	17·62	17·70	17·78	17·86	17·94	18·03	18·12	·08
4·7	18·21	18·29	18·38	18·46	18·55	18·63	18·72	18·80	18·89	18·97	·08
4·8	19·06	19·14	19·23	19·31	19·40	19·49	19·58	19·67	19·76	19·85	·08
4·9	19·93	20·02	20·11	20·20	20·29	20·38	20·47	20·56	20·65	20·74	·09
5·0	20·83	20·92	21·01	21·10	21·19	21·28	21·37	21·46	21·55	21·64	·09
5·1	21·74	21·83	21·93	22·02	22·11	22·21	22·31	22·40	22·50	22·59	·09
5·2	22·68	22·77	22·87	22·96	23·06	23·15	23·25	23·34	23·44	23·53	·09
5·3	23·63	23·72	23·82	23·91	24·01	24·10	24·20	24·29	24·39	24·49	·09
5·4	24·60	24·70	24·80	24·90	25·00	25·10	25·20	25·30	25·40	25·50	·10
5·5	25·60	25·70	25·80	25·90	26·00	26·10	26·20	26·30	26·40	26·50	·10
5·6	26·61	26·71	26·81	26·91	27·01	27·11	27·21	27·31	27·42	27·53	·10
5·7	27·64	27·74	27·85	27·95	28·06	28·16	28·27	28·37	28·48	28·58	·10
5·8	28·69	28·79	28·90	29·00	29·11	29·22	29·32	29·42	29·53	29·64	·10
5·9	29·75	29·86	29·97	30·08	30·19	30·30	30·41	30·52	30·63	30·74	·11

t	0	1	2	3	4	5	6	7	8	9	Diff.
6·0	30·85	30·96	31·07	31·18	31·29	31·40	31·52	31·63	31·74	31·85	·11
6·1	31·97	32·08	32·20	32·31	32·42	32·53	32·65	32·76	32·87	32·98	·11
6·2	33·10	33·21	33·33	33·44	33·56	33·67	33·79	33·90	34·02	34·13	·11
6·3	34·25	34·36	34·48	34·60	34·71	34·83	34·95	35·06	35·18	35·30	·11
6·4	35·42	35·54	35·66	35·78	35·90	36·02	36·14	36·26	36·38	36·50	·12
6·5	36·62	36·74	36·86	36·98	37·10	37·22	37·34	37·46	37·58	37·70	·12
6·6	37·83	37·95	38·07	38·19	38·32	38·44	38·56	38·68	38·80	38·93	·12
6·7	39·06	39·18	39·31	39·43	39·56	39·68	39·80	39·93	40·06	40·18	·12
6·8	40·31	40·43	40·56	40·69	40·81	40·94	41·07	41·19	41·32	41·45	·12
6·9	41·58	41·70	41·83	41·96	42·09	42·22	42·35	42·48	42·61	42·74	·12
7·0	42·87	43·00	43·13	43·26	43·39	43·52	43·65	43·78	43·91	44·04	·13
7·1	44·18	44·31	44·44	44·58	44·71	44·85	44·98	45·12	45·25	45·39	·13
7·2	45·52	45·65	45·79	45·92	46·06	46·19	46·33	46·46	46·60	46·73	·13
7·3	46·87	47·00	47·14	47·28	47·41	47·55	47·69	47·83	47·96	48·10	·13
7·4	48·24	48·37	48·51	48·65	48·79	48·93	49·07	49·21	49·35	49·49	·14
7·5	49·63	49·77	49·91	50·05	50·19	50·33	50·47	50·61	50·75	50·89	·14
7·6	51·04	51·18	51·32	51·47	51·61	51·76	51·90	52·05	52·19	52·34	·14
7·7	52·48	52·62	52·76	52·91	53·05	53·20	53·34	53·49	53·63	53·78	·14
7·8	53·92	54·06	54·21	54·35	54·50	54·65	54·80	54·95	55·10	55·25	·14
7·9	55·40	55·54	55·69	55·84	55·99	56·14	56·29	56·44	56·59	56·74	·14
8·0	56·89	57·04	57·19	57·34	57·49	57·64	57·79	57·94	58·09	58·24	·15
8·1	58·40	58·55	58·70	58·86	59·01	59·16	59·32	59·47	59·62	59·77	·15
8·2	59·93	60·08	60·24	60·39	60·55	60·70	60·86	61·01	61·17	61·32	·15
8·3	61·48	61·64	61·80	61·95	62·11	62·26	62·42	62·58	62·73	62·89	·16
8·4	63·05	63·20	63·36	63·52	63·68	63·84	64·00	64·16	64·32	64·48	·16
8·5	64·64	64·80	64·96	65·12	65·28	65·44	65·60	65·76	65·92	66·08	·16
8·6	66·25	66·41	66·58	66·74	66·90	67·07	67·23	67·40	67·56	67·72	·16
8·7	67·89	68·05	68·22	68·38	68·55	68·71	68·88	69·04	69·21	69·37	·16
8·8	69·54	69·70	69·87	70·04	70·20	70·37	70·54	70·70	70·87	71·04	·16
8·9	71·21	71·37	71·54	71·71	71·88	72·05	72·22	72·39	72·56	72·73	·16
9·0	72·90	73·07	73·24	73·41	73·58	73·75	73·92	74·09	74·26	74·43	·17
9·1	74·61	74·78	74·95	75·13	75·30	75·47	75·65	75·82	75·99	76·17	·17
9·2	76·34	76·51	76·69	76·86	77·04	77·21	77·39	77·56	77·74	77·91	·17
9·3	78·09	78·26	78·44	78·62	78·79	78·97	79·15	79·32	79·50	79·68	·17
9·4	79·86	80·03	80·21	80·39	80·57	80·75	80·93	81·11	81·29	81·47	·18
9·5	81·65	81·83	82·02	82·19	82·37	82·55	82·73	82·91	83·09	83·27	·18
9·6	83·46	83·64	83·82	84·01	84·19	84·37	84·56	84·74	84·92	85·10	·18
9·7	85·29	85·47	85·66	85·84	86·03	86·21	86·40	86·58	86·77	86·96	·18
9·8	87·15	87·34	87·52	87·71	87·89	88·08	88·26	88·45	88·64	88·83	·19
9·9	89·02	89·20	89·39	89·58	89·77	89·96	90·15	90·34	90·53	90·72	·19
10·0	90·91										

$$\frac{t^2}{t+1}$$

for values of t from 10 to 30 by tenths.

t	0	1	2	3	4	5	6	7	8	9	Diff.
10	90·9	[illegible]	94·7	96·7	98·7	100·7	102·7	104·7	106·7	108·5	2·0
11	110·9	113·0	115·1	117·3	119·5	121·7	123·9	126·1	128·3	130·6	2·2
12	132·9	135·2	137·5	139·9	142·3	144·6	147·0	149·5	152·0	154·4	2·3
13	156·9	159·4	162·0	164·5	167·1	169·7	172·3	174·9	177·6	180·1	2·5
14	182·7	185·5	188·3	191·0	193·8	196·6	199·5	202·3	205·2	208·0	2·8
15	210·9	213·8	216·7	219·7	222·7	225·7	228·7	231·8	234·8	237·8	3·0
16	240·9	244·0	247·2	250·3	253·4	256·6	259·9	263·1	266·4	269·6	3·2
17	272·9	276·2	279·5	282·9	286·3	289·7	293·1	296·5	300·0	303·4	3·4
18	306·9	310·4	314·0	317·5	321·1	324·7	328·3	331·9	335·6	339·3	3·6
19	343·0	346·7	350·4	354·2	357·9	361·6	365·4	369·3	373·2	377·1	3·8
20	381·0	384·9	388·8	392·7	396·7	400·7	404·7	408·7	412·8	416·9	4·0
21	421·0	425·6	429·2	433·3	437·5	441·7	445·9	450·1	454·4	458·7	4·2
22	463·0	467·3	471·6	475·9	480·3	484·7	489·1	493·5	498·0	502·5	4·4
23	507·0	511·5	516·0	520·5	525·1	529·7	534·3	538·9	543·6	548·3	4·6
24	553·0	557·7	562·4	567·1	571·9	576·7	581·5	586·3	591·2	596·1	4·8
25	601·0	605·9	610·8	615·7	620·7	625·7	630·7	635·7	640·8	645·9	5·0
26	651·0	656·1	661·2	666·3	671·5	676·7	681·9	687·1	692·4	697·7	5·2
27	703·0	708·3	713·6	718·9	724·3	729·7	735·1	740·5	746·0	751·5	5·4
28	757·0	762·5	768·0	773·5	779·1	784·7	790·3	795·9	801·6	807·3	5·6
29	813·0	818·7	824·4	830·1	835·9	841·7	847·5	853·3	859·2	865·1	5·8
30	871·0										

For values of t from 30 to 100 by whole numbers.

t	0	1	2	3	4	5	6	7	8	9
30	871	931	933	1,057	1,123	1,101	1,261	1,333	1,407	1,483
40	1,561	1,641	1,723	1,807	1,893	1,981	2,071	2,161	2,257	2,353
50	2,451	2,551	2,653	2,757	2,863	2,971	3,081	3,193	3,307	3,423
60	3,541	3,661	3,783	3,907	4,033	4,161	4,291	4,423	4,557	4,693
70	4,831	4,971	5,113	5,257	5,403	5,551	5,701	5,853	6,007	6,163
80	6,321	6,481	6,643	6,807	6,973	7,141	7,311	7,483	7,657	7,833
90	8,011	8,191	8,373	8,557	8,743	8,931	9,121	9,313	9,506	9,703
100	9,901	10,101								

For values of over 10,000, the function $\frac{t^2}{t+1}$ may be taken as approximately equal to t^2.

APPENDIXTABLE FROM REF: 12