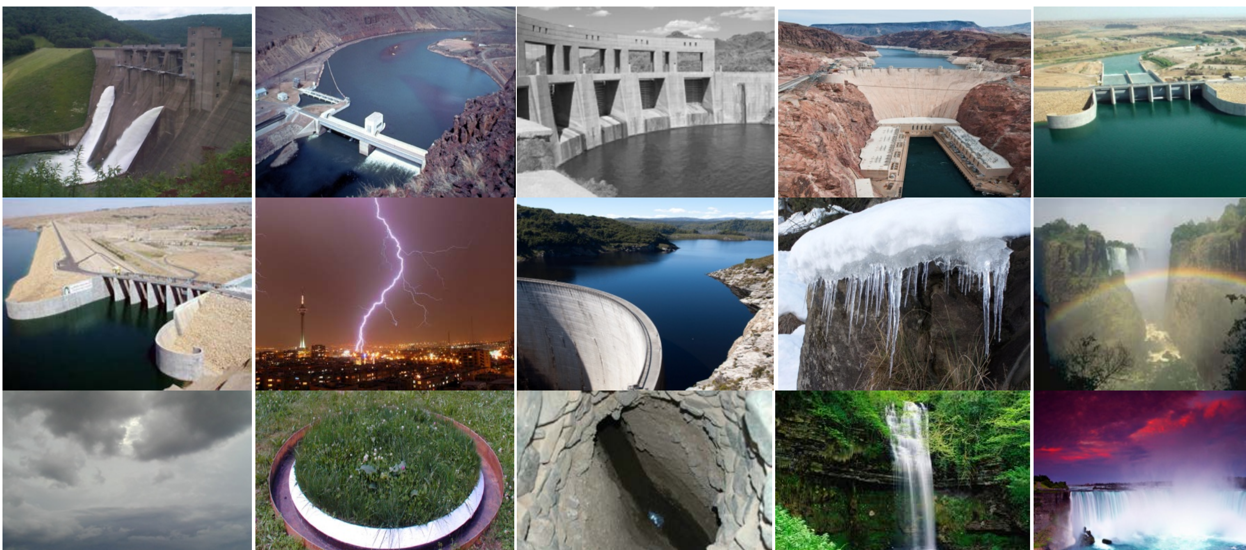
www.esciencecentral.org/ebooks

HANDBOOK OF HYDROLOGIC ENGINEERING PROBLEMS

Mohammad Valipour



Handbook of Hydrologic Engineering Problems

Edited by: Mohammad Valipour

Published Date: June 2014

Published by **OMICS Group eBooks**

731 Gull Ave, Foster City. CA 94404, USA

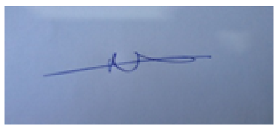
A free online edition of this book is available at www.esciencecentral.org/ebooks

Additional hard copies can be obtained from orders @ www.esciencecentral.org/ebooks

Preface

In near future, energy become a luxury item and water is considered as the most vital item in the world due to reduction of water resources in most regions. In this condition, role of water science researchers and hydraulic experts is more important than ever. If a hydrologic engineer student is not educated well, he/she will not solve problems of hydraulic sciences in future. Many engineer students learn all necessary lessons in the university, but they cannot to answer to the problems or to pass the exams because of forgetfulness or lack of enough exercise. This book contains one hundred essential problems related to hydraulic engineering with a small volume. Undoubtedly, many problems can be added to the book but the author tried to mention only more important problems and to prevent increasing volume of the book due to help to feature of portability of the book. To promotion of student skill, both SI and English systems have been used in the problems. All of the problems were solved completely. This book is useful for not only exercising and passing the university exams but also for use in actual project as a handbook. The handbook of hydraulic engineering problems is usable for agricultural, civil, and environmental students, teachers, experts, researchers, engineers, designers, and all enthusiastic readers in hydraulic, hydrodynamic, fluid mechanics, irrigation, drainage engineering, and water resources fields. The prerequisite to study of the book and to solve of the problems is each appropriate book about hydrologic science; however, the author recommends studying the References to better understanding the problems and presented solutions. It is an honor for the author to receive any review and suggestion improvement of book quality.

- **Mohammad Valipour**



About Author

Mohammad Valipour is a Ph.D. candidate in Agricultural Engineering-Irrigation and Drainage at Sari Agricultural Sciences and Natural Resources University, Sari, Iran. He completed his B.Sc. Agricultural Engineering-Irrigation at Razi University, Kermanshah, Iran in 2006 and M.Sc. in Agricultural Engineering-Irrigation and Drainage at University of Tehran, Tehran, Iran in 2008. Number of his publications is more than 50. His current research interests are surface and pressurized irrigation, drainage engineering, relationship between energy and environment, agricultural water management, mathematical and computer modeling and optimization, water resources, hydrology, hydrogeology, hydro climatology, hydrometeorology, hydro informatics, hydrodynamics, hydraulics, fluid mechanics, and heat transfer in soil media.

OMICS Group eBooks

Contents	Page #

Handbook of Hydrologic Engineering Problems

Mohammad Valipour*

Young Researchers and Elite Club, Kermanshah Branch, Islamic Azad University, Kermanshah, Iran

***Corresponding author:** Mohammad Valipour, Young Researchers and Elite Club, Kermanshah Branch, Islamic Azad University, Kermanshah, Iran; E-mail: vali-pour@hotmail.com

Problems

1. The volume of atmospheric water is 12900 km³. The evapotranspiration from land is 72000 km³/year and that from ocean is 505000 km³/year. Estimate the residence time of water molecules in the atmosphere (in days).

The residence time can be derived by dividing the volume of water by the flow rate

Total flow rate = 505000 + 72000 = 577000 km^3/s

The residence time = 12900/577000 = 0.0224 year = 8.2 days

2. A reservoir has the following inflows and outflows (in cubic meters) for the first three month of the year. If the storage at the beginning of January is 60 m³, determine the storage at the end of March.

Month	Jan	Feb	Mar
Inflow	4	6	9
Outflow	8	11	5

The storage change is

$\Delta S = I - O = (4+6+9) - (8+11+5) = -5\ m^3$

The storage is 60-5 = 55 m^3

3. Rain-gauge station D was inoperative for part of a month during which a storm occured. The storm rainfall recorded in the three surrounding stations A, B and C was 8.5, 6.7 and 9.0 cm, respectively. If the a.a.r for the stations are 75, 84, 70 and 90 cm, respectively, estimate the storm rainfall at station D.

By equating the ratios of storm rainfall to the a.a.r. at each station, the storm rainfall at station D (P_D) is estimated as

$$\frac{8.5}{75} = \frac{6.7}{84} = \frac{9.0}{70} = \frac{P_D}{90}$$

$$The\ average\ value\ of\ P_D = \frac{1}{3}\left[\frac{8.5}{75}\times 90 + \frac{6.7}{84}\times 90 + \frac{9.0}{70}\times 90\right] = 9.65\ cm$$

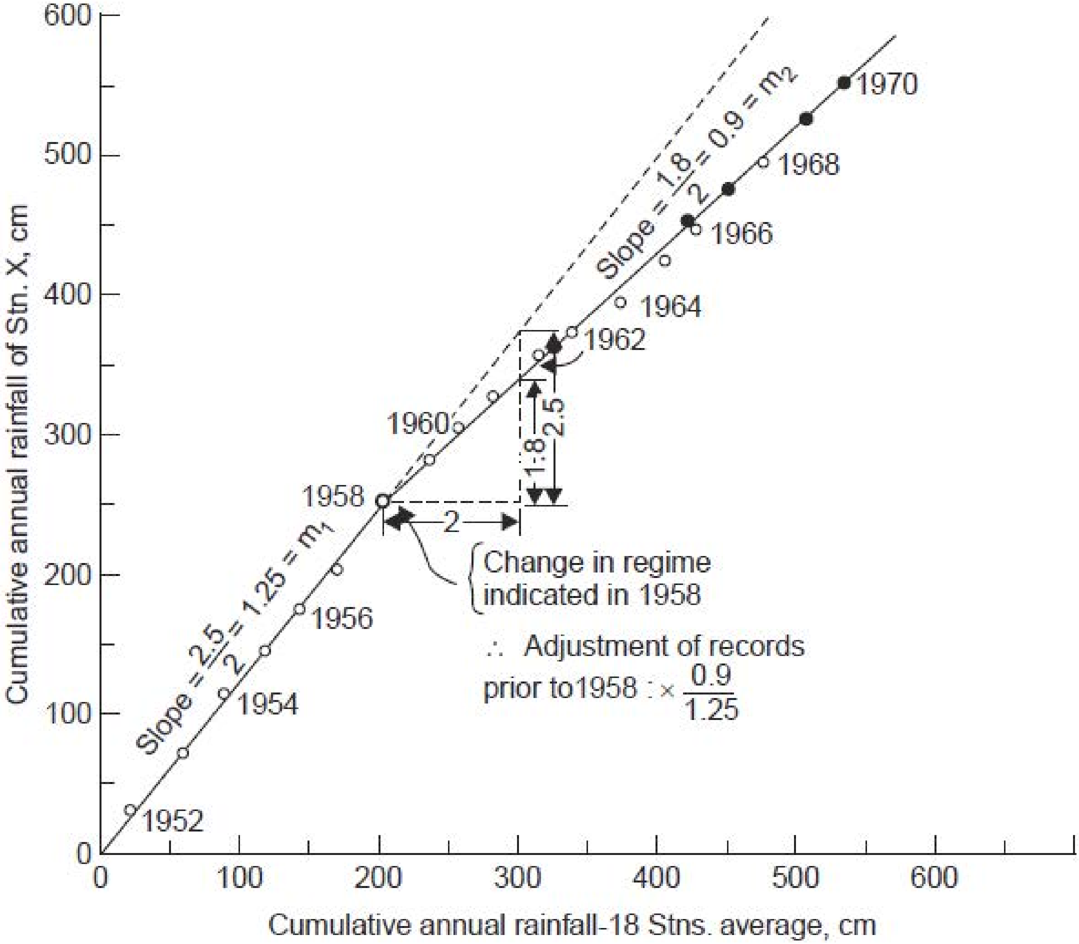
4. The annual rainfall at station X and the average annual rainfall at 18 surrounding stations are given below. Check the consistency of the record at station X and determine the year in which a change in regime has occurred. State how you are going to adjust the records for the change in regime. Determine the a.a.r. for the period 1952-1970 for the changed regime.

Annual rainfall (cm)		
year	Stn. X	18- atn average
1952	30.5	22.8
1953	38.9	35.0
1954	43.7	30.2
1955	32.2	27.4
1956	27.4	25.2
1957	32.0	28.2
1958	49.3	36.1
1959	28.4	18.4
1960	24.6	25.1
1961	21.8	23.6
1962	28.2	33.3
1963	17.3	23.4
1964	22.3	36.0
1965	28.4	31.2
1966	24.1	23.1
1967	26.9	23.4
1968	20.6	23.1
1969	29.5	33.2
1970	28.4	26.4

Cumulative annual rainfall (cm)		
year	Stn. X	18 atn average
1952	30.5	22.8
1953	69.4	57.8
1954	113.1	88.0
1955	145.3	115.4
1956	172.7	140.6
1957	204.7	168.8
1958	254.0	204.9
1959	282.4	233.3
1960	307.0	158.4
1961	328.8	282.0
1962	357.0	315.3
1963	374.3	338.7
1964	396.6	374.7

1965	425.0	405.9
1966	449.1	429.0
1967	476.0	452.4
1968	496.6	475.5
1969	526.1	508.7
1970	554.5	535.1

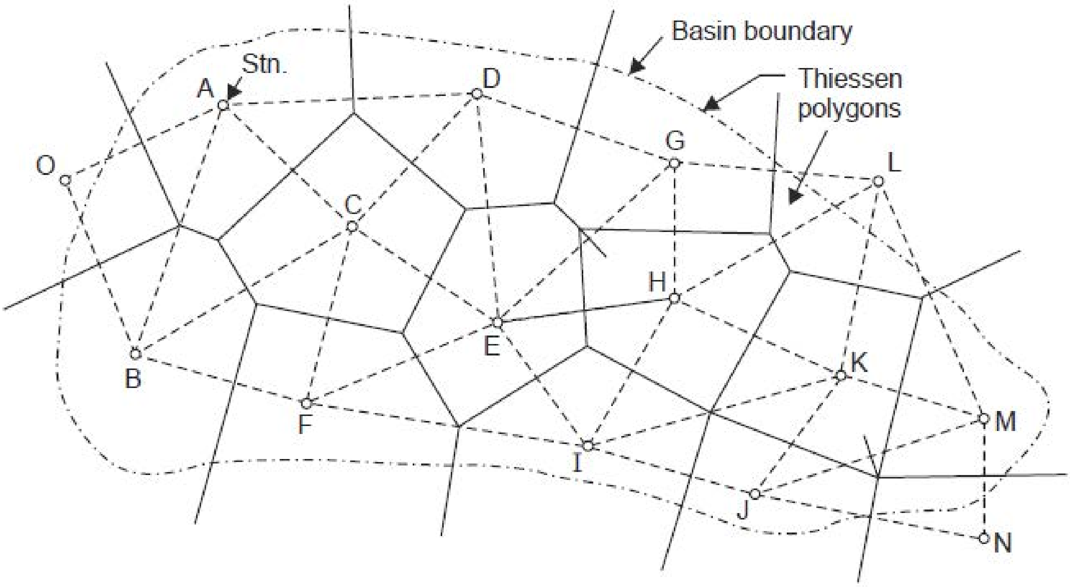
The above cumulative rainfalls are plotted as shown in the figure. It can be seen from the figure that there is a distinct change in slope in the year 1958, which indicates that a change in regime (exposure) has occurred in the year 1958. To make the records prior to 1958 comparable with those after change in regime has occurred, the earlier records have to be adjusted by multiplying by the ratio of slopes m_2/m_1 i.e., 0.9/1.25.



Cumulative rainfall 1958-1970= 554.5 – 204.7 = 349.8 cm
Cumulative rainfall 1952-1957
adjusted for changed environment= 204.7 ×(0.9/125)= 147.6 cm

Cumulative rainfall 1952-1970
(for the current environment)= 497.4 cm
a.a.r. adjusted for the current regime=497.4/19=26.2 cm

5. Point rainfalls due to a storm at several rain-gauge stations in a basin are shown in the figure. Determine the mean areal depth of rainfall over the basin by the three methods.



(i) Arithmetic average method

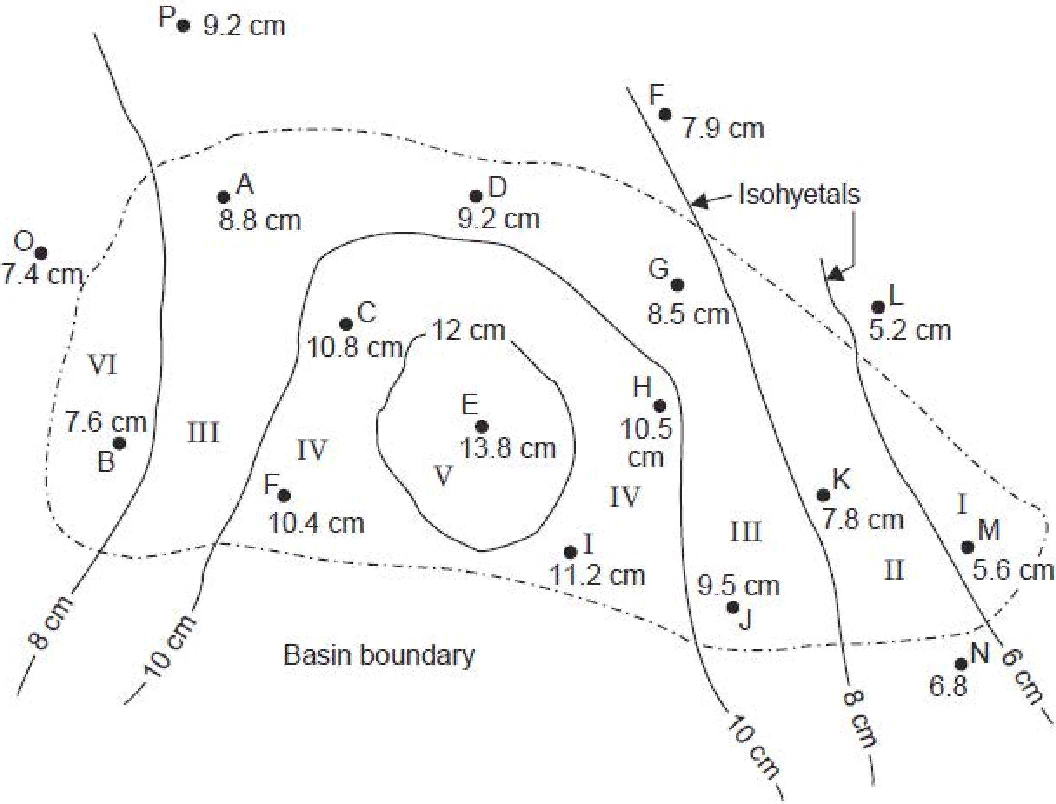
$$P_{ave} = \frac{\sum P_1}{n} = \frac{1331\,cm}{15} = 8.87\,cm$$

ΣP1 = sum of the 15 station rainfalls.

(ii) Thiessen polygon method—The Thiessen polygons are constructed as shown in the figure and the polygonal areas are planimetered and the mean areal depth of rainfall is worked out below:

Station	*Rainfall recorded, P_1 (cm)*	*Area of influential polygon, A_1 (km²)*	*Product (2) × (3) A_1P_1 (km²-cm)*	*Mean areal depth of rainfall*
1	*2*	*3*	*4*	*5*
A	8.8	570	5016	
B	7.6	920	6992	
C	10.8	720	7776	
D	9.2	620	5704	
E	13.8	520	7176	
F	10.4	550	5720	
G	8.5	400	3400	
H	10.5	650	6825	$P_{ave} = \frac{\Sigma A_1 P_1}{\Sigma A_1}$
I	11.2	500	5600	
J	9.5	350	3325	$= \frac{66714}{7180}$
K	7.8	520	4056	**= 9.30 cm**
L	5.2	250	1300	
M	5.6	350	1960	
N	6.8	100	680	
O	7.4	160	1184	
Total $n = 15$	1331 cm $= \Sigma P_1$	7180 km² $= \Sigma A_1$	66714 km²-cm $\Sigma A_1 P_1$	

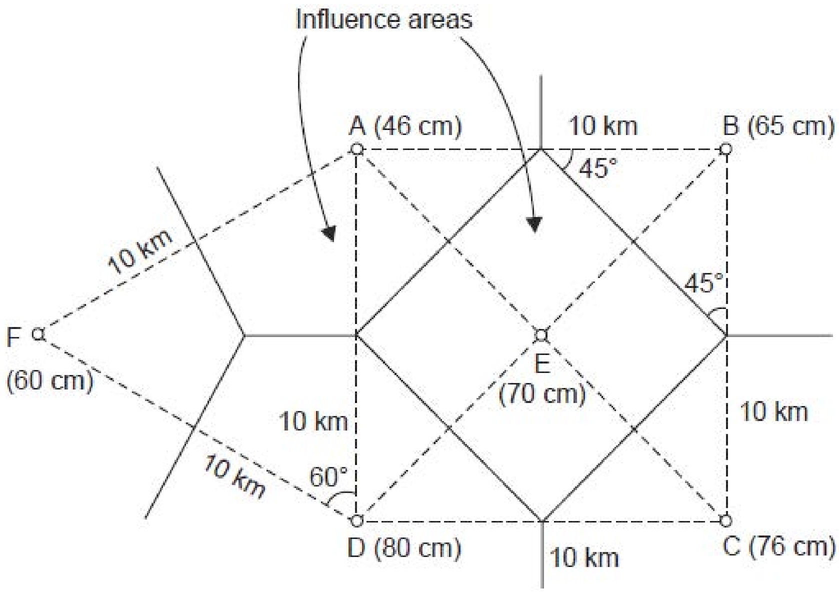
(iii) Isohyetal method—The isohyets are drawn as shown in the figure and the mean areal depth of rainfall is worked out below:



P 9.2 cm
F 7.9 cm
A 8.8 cm
D 9.2 cm
Isohyetals
O 7.4 cm
G 8.5 cm
L 5.2 cm
C 10.8 cm
12 cm
VI
E 13.8 cm
H 10.5 cm
7.6 cm
B
III
IV
F 10.4 cm
V
IV
K 7.8 cm
I
M 5.6 cm
III
I 11.2 cm
II
9.5 cm J
8 cm
10 cm
Basin boundary
N 6.8
6 cm
10 cm
8 cm

Zone	*Isohyets (cm)*	*Mean isohyetal value, $P_{1\text{-}2}$ (cm)*	*Area between isohyets, $A_{1\text{-}2}$ (km^2)*	*Product (3) × (4) (km^2-cm)*	*Mean areal depth of rainfall (cm)*
1	*2*	*3*	*4*	*5*	*6*
I	<6	5.4	410	2214	P_{ave} =
II	6-8	7	900	6300	$\dfrac{\Sigma A_{1-2}P_{1-2}}{\Sigma A_{1-2}}$
III	8-10	9	2850	25650	
IV	10-12	11	1750	19250	$= \dfrac{66754}{7180}$
V	>12	12.8	720	9220	
VI	<8	7.5	550	4120	= 930 cm
		Total	7180 km^2 = ΣA_{1-2}	66754 km^2-cm = $\Sigma A_{1-2}.P_{1-2}$	

6. The area shown in the figure is composed of a square plus an equilateral triangular plot of side 10 km. The annual precipitations at the rain-gauge stations located at the four corners and centre of the square plot and apex of the triangular plot are indicated in figure. Find the mean precipitation over the area by Thiessen polygon method, and compare with the arithmetic mean.



The Thiessen polygon is constructed by drawing perpendicular bisectors to the lines joining the rain-gauge stations as shown in the figure. The weighted mean precipitation is computed in the following table:
Area of square plot = 10 × 10 = 100 km^2
Difference = 50 km^2
Area of each corner triangle in the square plot = 56/4 = 12.5 km^2

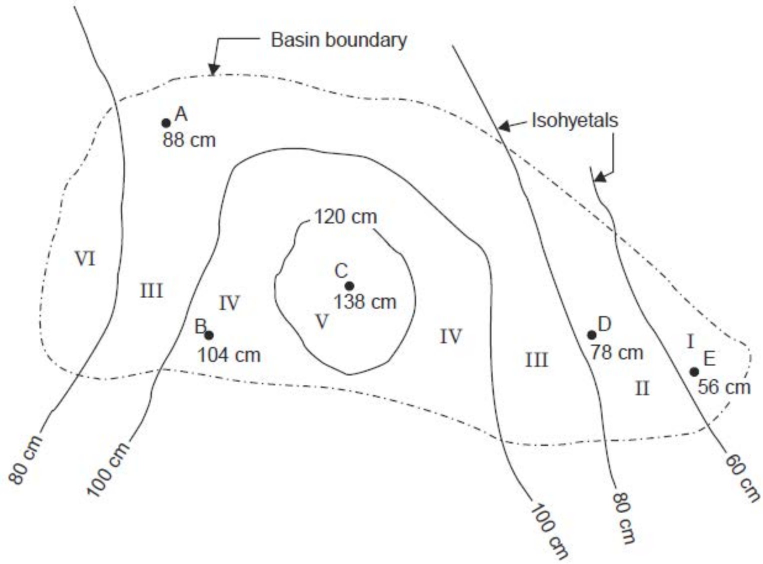
$$\frac{1}{3}\, area\ of\ the\ equilateral\ triangular\ plot = \frac{1}{3}\left(\frac{1}{2}\times 10\times 10\sin 60\right) = 14.4\ km^2$$

Station	Area, A (km^2)	Precipitation P (cm)	$A \times P$ (km^2-cm)	P_{ave} (cm)
A	(12.5 + 14.4) = 26.9	46	1238	
B	12.5	65	813	
C	12.5	76	950	$= \frac{\Sigma A.P}{\Sigma A}$
D	(12.5 + 14.4) = 26.9	80	2152	$= \frac{9517}{143.2}$ = **66.3 cm**
E	50	70	3500	
F	14.4	60	864	
n = 6	ΣA = 143.2 = $100 + 25\sqrt{3}$ as a check	ΣP = 397	$\Sigma A.P.$ = 9517	

$$Arithmetic\ mean = \frac{\sum P}{n} = \frac{397}{6} = 66.17\ cm$$

which compares fairly with the weighted mean.

7. For the basin shown in the figure, the normal annual rainfall depths recorded and the isohyetals are given. Determine the optimum number of rain-gauge stations to be established in the basin if it is desired to limit the error in the mean value of rainfall to 10%. Indicate how you are going to distribute the additional rain-gauge stations required, if any. What is the percentage accuracy of the existing network in the estimation of the average depth of rainfall over the basin?



Station	*Normal annual rainfall, x (cm)*	*Difference* $(x - \bar{x})$	*(Difference)*^{2} $(x - \bar{x})^2$	*Statistical parameters* $\bar{x}$, σ, C_v
A	88	– 4.8	23.0	$\bar{x} = \frac{\Sigma x}{n} = \frac{464}{5}$
B	104	11.2	125.4	= 92.8 cm
C	138	45.2	2040.0	$\sigma = \sqrt{\frac{\Sigma(x-\bar{x})^2}{n-1}}$
D	78	– 14.8	219.0	$= \sqrt{\frac{3767.4}{5-1}} = 30.7$
E	56	– 36.8	1360.0	
$n = 5$	$\Sigma x = 464$		$\Sigma(x - \bar{x})^2 = 3767.4$	$C_v = \frac{\sigma}{\bar{x}} = \frac{30.7}{92.8} \times 100$
				= 33.1%

The optimum number of rain-gauge stations to limit the error in the mean value of rainfall to p = 10%.

$$N = \left(\frac{C_v}{p}\right)^2 = \left(\frac{33.1}{10}\right)^2 = 11$$

Additional rain-gauge stations to be established = N – n = 11 – 5 = 6
The additional six raingauge stations have to be distributed in proportion to the areas between the isohyetals as shown below:

zone	I	II	III	IV	V	VI	Total
Area (Km^2)	410	900	2850	1750	720	550	7180
Area as decimal	0.06	0.12	0.40	0.24	0.10	0.08	1.00
N x area in decimal(N= 11)	0.66	1.32	4.4	2.64	1.1	0.88	
Rounded as	1	1	4	3	1	1	11
Rain-guages existing	1	1	1	1	1	-	5
Additional rain guages	-	-	-	2	-	1	6

These additional rain-gauges have to be spatially distributed between the different isohyetals after considering the relative distances between rain-gauge stations, their accessibility, personnel required for making observations, discharge sites, etc.
The percentage error p in the estimation of average depth of rainfall in the existing network,

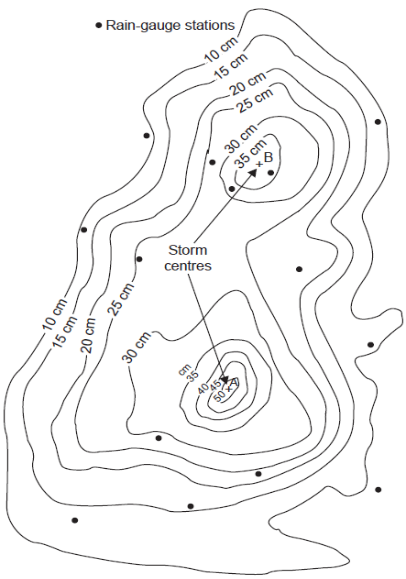
$$p = \frac{C_v}{N}, \ putting\ N = n$$

$$p = \frac{33.1}{\sqrt{5}} = 14.8\%$$

Or, the percentage accuracy = 85.2%

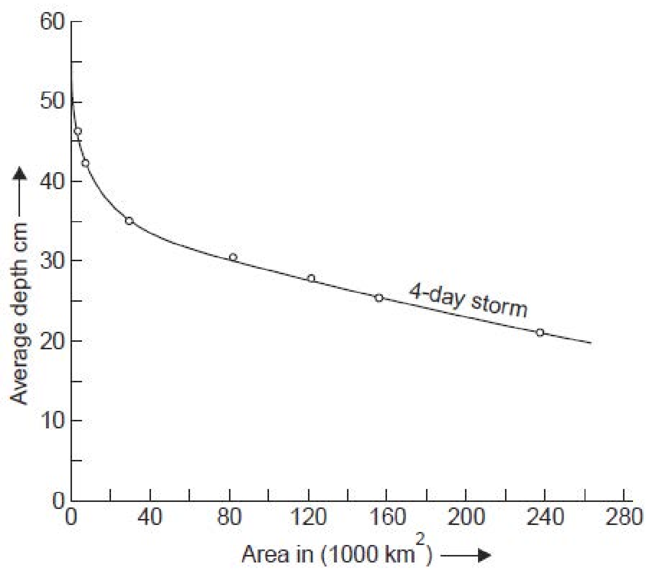
8. An isohyetal pattern of critical consecutive 4-day storm is shown in the figure. Prepare the DAD curve.

Computations to draw the DAD curves for a 4-day storm are made in the table.

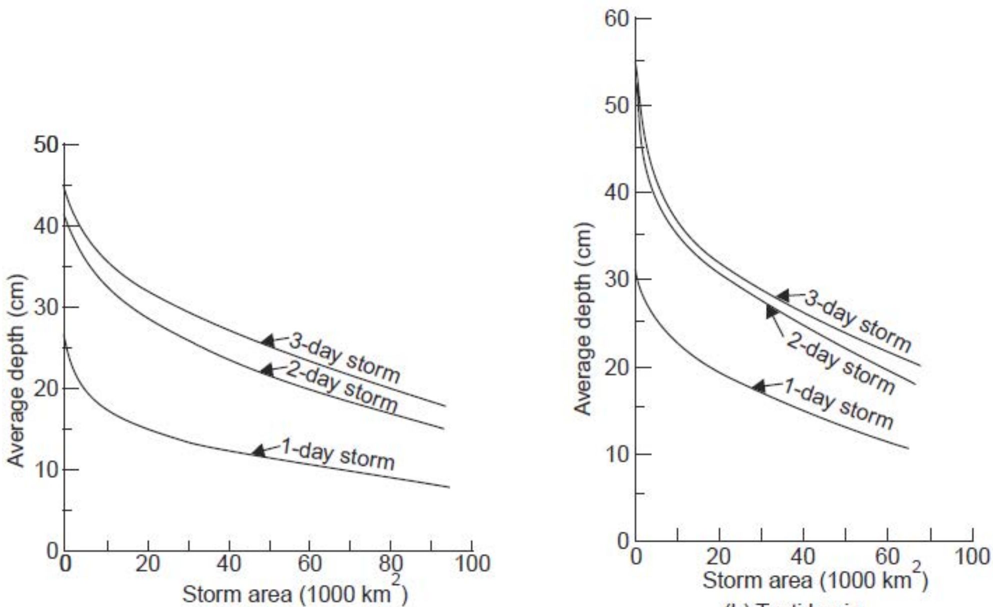


Storm centre	Encom passing isohyets (cm)	Area enclosed (km^2) (1000)	Isohyetal range (cm)	Average isohyetal value (cm)	Area between isohyets (km^2) (1000)	Incremental volume (cm. km^2) (1000)	Total volume (cm.km^2) (1000)	Average depth (8)/ (3) (cm)
1	2	3	4	5	6	7	8	9
A	50	0.5	>50	Say,55	0.5	27.5	27.5	55
	40	4	40-50	45	3.5	157.5	185.0	46.25
	35	7	35-40	37.5	3	112.5	297.5	42.5
	30	29	30-35	32.5	22	715.0	1012.5	34.91
	35	2	>35	Say,37.5	2	75.0	75.0	37.5
	30	9.5	30-35	32.5	7.5	244.0	319.0	33.6
	25	82	25-30	27.5	43.5	1196.2	2527.8	30.8
		122	20-25	22.5	40	900	3427.8	28.1
	15	156	15-20	17.5	34	595	4022.8	25.8
		236	10-15	12.5	80	1000	5022.8	21.3

Plot 'col. (9) vs. col. (3)' to get the DAD curve for the maximum 4-day critical storm, as shown in the figure.



Isohyetal patterns are drawn for the maximum 1-day, 2-day, 3-day and 4-day (consecutive) critical rainstorms that occurred during 13 to 16th July 1944 in the Narmada and Tapti catchments and the DAD curves are prepared as shown in the figure. The characteristics of heavy rainstorms that have occurred during the period 1930–68 in the Narmada and Tapti basins are given below:



(a) Narmada basin (b) Tapti basin

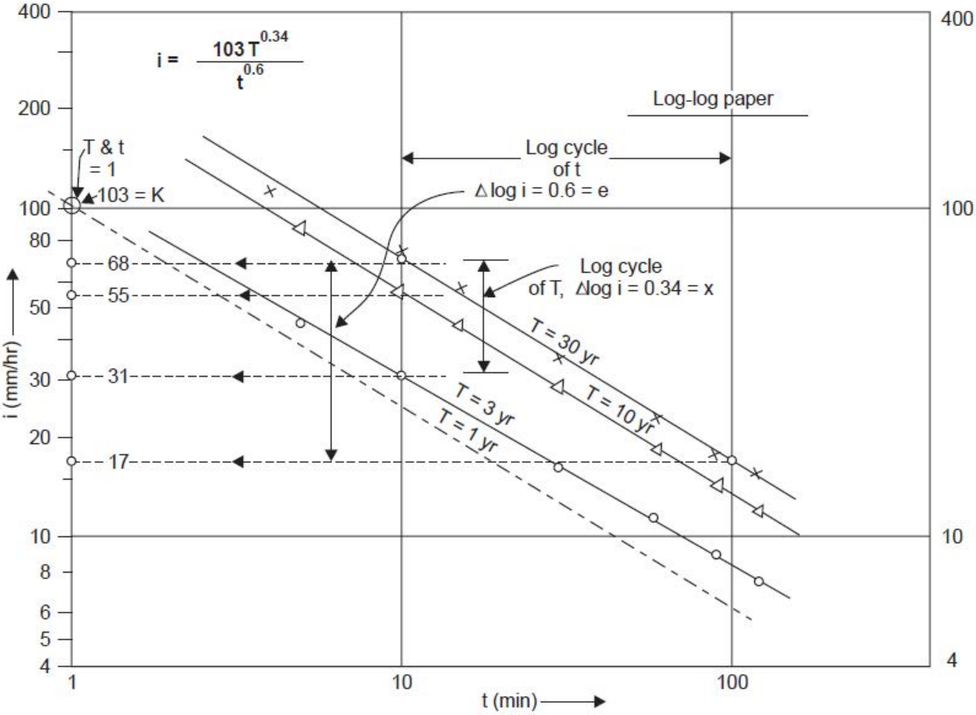
year	River basin	Maximum depth of rainfall (cm)			
		1-day	2-day	3-day	4-day
13-16 july 1944	Narmada	8.3	14.6	18.8	22.9
	Tapti	6.3	9.9	11.2	15.2
4-6 august 1968	Narmada	7.6	14.5	17.4	
	Tapti	11.1	19.0	21.1	
8-9 september 1961	Narmada	8.8	11.9		
	Tapti	4.7	7.5		
21-24 september1945	Narmada	4.1	7.4	10.4	12.9
	Tapti	10.9	14.7	18.0	20.0
17 august 1944	Narmada	3.8			
	Tapti	10.4			

9. In a Certain water shed, the rainfall mass curves were available for 30 (n) consecutive years. The most severe storms for each year were picked up and arranged in the descending order (rank m). The mass curve for storms for three years are given below. Establish a relation of the form $i = kT^x / t^e$, by plotting on log-log graph paper.

Time (min) accumulated depth (mm)	5	10	15	30	60	90	120
For m = 1	9	12	14	17	22	25	30
For m = 3	7	9	11	14	17	21	23
For m = 10	4	5	6	8	11	13	14

Time t (min)	5	10	15	30	60	90	120	T-yr = $\frac{n+1}{m}$
Intensity i (mm/hr)								
for $m = 1$	$\frac{9}{5} \times 60$ = 108	$\frac{12}{10} \times 60$ = 72	56	34	22	16.6	15	$\frac{30+1}{1} \simeq 30$ yr
for $m = 3$	$\frac{7}{5} \times 60$ = 84	$\frac{9}{10} \times 60$ = 54	44	28	14	14	11.5	$\frac{30+1}{3} \simeq 10$ yr
for $m = 10$	$\frac{4}{5} \times 60$ = 48	$\frac{5}{10} \times 60$ = 30	24	16	11	8.7	7	$\frac{30+1}{10} \simeq 3$ yr

The intensity-duration curves (lines) are plotted on log-log paper, which yield straight lines nearby parallel. A straight line for T = 1 – yr is drawn parallel to the line T = 10-yr at a distance equal to that between T = 30–yr and T = 3-yr. From the graph at T = 1- yr and t = 1 min, k = 103.



The slope of the lines, say for T = 30-yr is equal to the change in log i per log cycle of t, i.e., for t = 10 min and 100 min, slope = log 68 – log 17 = 1.8325 – 1.2304 = 0.6021 ≅0.6 = e.

At t = 10 min, the change in log i per log cycle of T, i.e., between T = 3–yr and 30–yr lines (on the same vertical), log 68 – log 31 = 1.8325 – 1.4914 = 0.3411 ≅0.34 = x.

Hence, the intensity-duration relationship for the watershed can be established as

$$i = \frac{104T^{0.34}}{t^{0.6}}$$

For illustration, for the most severe storm (m = 1, T = 30–yr), at t = 60 min, i.e., after 1 hr of commencement of storm,

$$i = \frac{103(30)^{0.34}}{(60)^{0.6}} = 28\, mm/hr$$

which is very near to the observed value of 22 mm/hr.

10. A small water shed consists of 2 km² of forest area (c = 0.1), 1.2 km² of cultivated area (c = 0.2) and 1 km² under grass cover (c = 0.35). A water course falls by 20 m in a length of 2 km. The IDF relation for the area may be taken as

$i=80T^{0.2}/(t+12)^{0.5}$

Estimate the peak rate of runoff for a 25 yr frequency.

Time of concentration (in hr)

$$t_c = 0.06628\,L^{0.77}S^{-0.385} = 0.06628 \times 2^{0.77}\left(\frac{20}{2\times 1000}\right)^{-0.385} = 40\ \text{min}$$

i = i_c when t = t_c in the given IDF relation

$$i_c = \frac{80\times 25^{0.2}}{(40+12)^{0.5}} = 21.1\,cm/hr$$

Q_{peak} = 2.78 C i_c A, rational formula, CA = ΣC_iA_i= 2.78 × 21.1 × (0.1 × 2 + 0.2 × 1.2 + 0.35 × 1) = 46.4 cumec

11. The annual rainfall at a place for a period of 10 years from 1961 to 1970 are respectively 30.3, 41.0, 33.5, 34.0, 33.3, 36.2, 33.6, 30.2, 35.5, 36.3. Determine the mean and median values of annual rainfall for the place.

$$(i)\ Mean\ \bar{x} = \frac{\sum x}{n} = \frac{(30.3+41.0+33.5+34.0+33.3+36.2+33.6+30.2+35.5+36.3)}{10} = 34.39\ cm$$

(ii) Median: Arrange the samples in the ascending order 30.2, 30.3 33.3, 33.5, 33.6, 34.0,35.5, 36.2, 36.3, 41.0

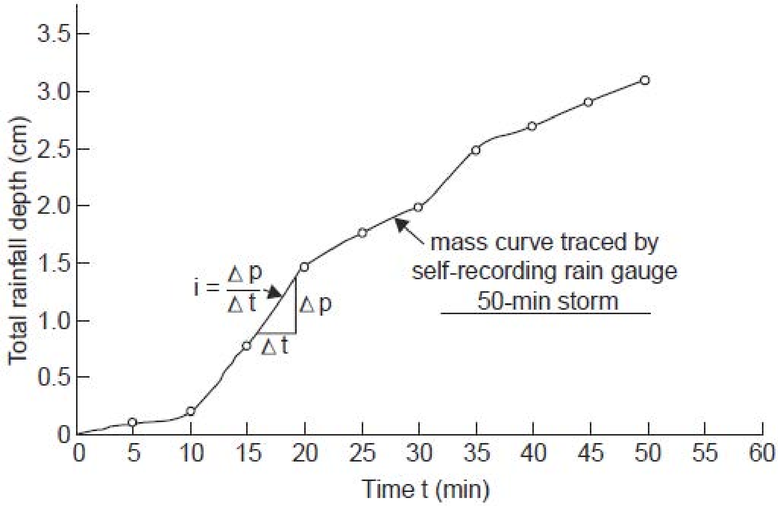
No. of items = 10, i.e., even

$$Median = \frac{33.6+34.0}{2} = 33.8\ cm$$

12. The following are the rain gauge observations during a storm. Construct: (a) mass curve of precipitation, (b) hyetograph, (c) maximum intensity-duration curve and develop a formula, and (d) maximum depth-duration curve.

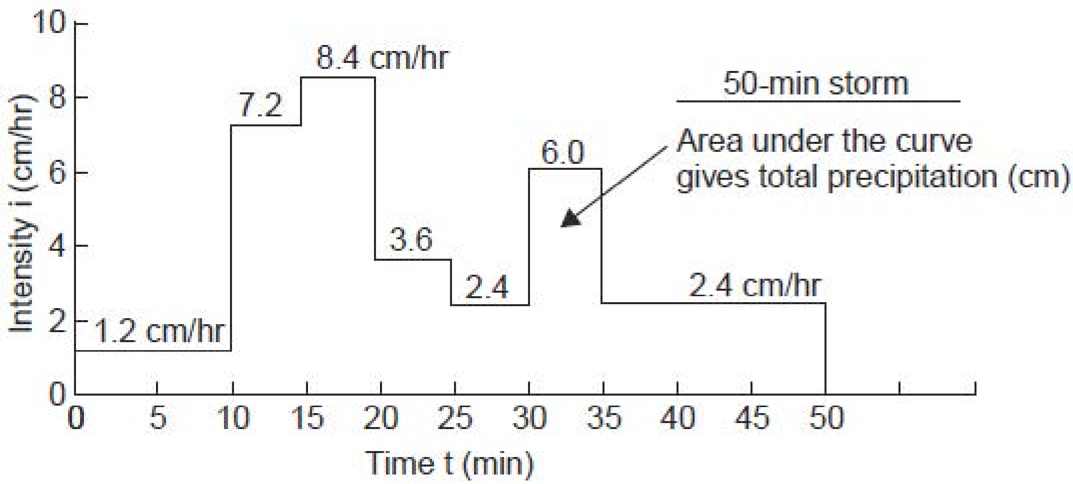
Time since commencement of storm (min)	Accumulated rainfall (cm)
5	0.1
10	0.2
15	0.8
20	1.5
25	1.8
30	2.0
35	2.5
40	2.7
45	2.9
50	3.1

(a) Mass curve of precipitation. The plot of 'accumulated rainfall (cm) vs. time (min)' gives the 'mass curve of rainfall' figure.

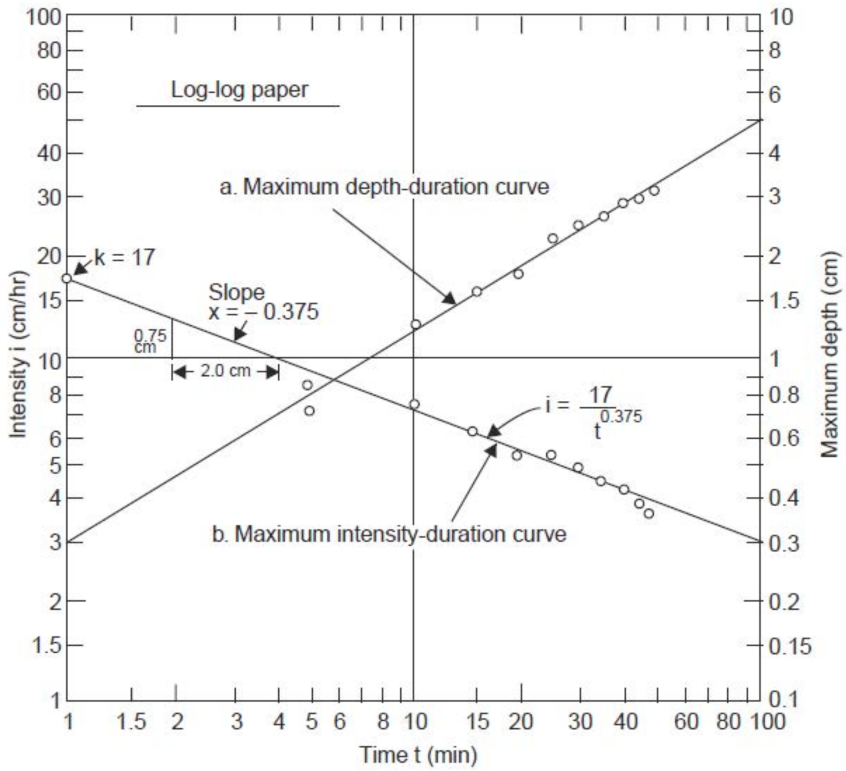


Time, t *(min)*	*Accumulated rainfall* *(cm)*	*Δ P in time* Δt *= 5 min* *(cm)*	*Intensity,* $i = \frac{\Delta P}{\Delta t} \times 60$ *(cm/hr)*
5	0.1	0.1	1.2
10	0.2	0.1	1.2
15	0.8	0.6	7.2
20	1.5	0.7	8.4
25	1.8	0.3	3.6
30	2.0	0.2	2.4
35	2.5	0.5	6.0
40	2.7	0.2	2.4
45	2.9	0.2	2.4
50	3.1	0.2	2.4

(b) Hyetograph. The intensity of rainfall at successive 5 min interval is calculated and a bar-graph of 'i (cm/hr) vs. t (min)' is constructed; this depicts the variation of the intensity of rainfall with respect to time and is called the 'hyetograph.



(c) Maximum depth–duration curve. By inspection of time (t) and accumulated rainfall (cm) the maximum rainfall depths during 5, 10, 15, 20, 25, 30, 35, 40, 45 and 50 min durations are 0.7, 1.3, 1.6, 1.8, 2.3, 2.5, 2.7, 2.9, 3.0 and 3.1 cm respectively. The plot of the maximum rainfall depths against different durations on a log-log paper gives the maximum depth-duration curve, which is a straight line



(d) Maximum intensity-duration curve. Corresponding to the maximum depths obtained in (c) above, the corresponding maximum intensities can be obtained (ΔP/Δt)× 60, i.e., 8.4, 7.8, 6.4, 5.4, 5.52, 5.0, 4.63, 4.35, 4.0 and 3.72 cm/hr, respectively. The plot of the maximum intensities against the different duration on a log-log paper gives the maximum intensity-duration curve which is a straight line.

The equation for the maximum itensity duration curve is of the form

$i = kt^x$

Slope of the straight line plot,

$$-x = \frac{dy}{dx} = \frac{0.75}{2.00} = 9.375$$

k = 17 cm/hr when t = 1 min

Hence, the formula becomes

$$i = \frac{17}{t^{0.375}}$$

which can now be verified as

t = 10 min, i = 7.2 cm/hr

t = 40 min, i = 4.25 cm/hr

which agree with the observed data

13. The annual rainfall at a place for a period of 21 years is given below. Draw the rainfall frequency curve and determine:
(a) the rainfall of 5-year and 20-year recurrence, interval
(b) the rainfall which occurs 50% of the times
(c) the rainfall of probability of 0.75
(d) the probability of occurrence of rainfall of 75 cm and its recurrence interval.

year	Rainfall (cm)	year	Rainfall (cm)
1950	50	1961	56
1951	60	1962	52
1952	40	1963	42
1953	27	1964	38
1954	30	1965	27
1955	38	1966	40
1956	70	1967	100
1957	60	1968	90
1958	35	1969	44
1959	55	1970	33
1960	40		

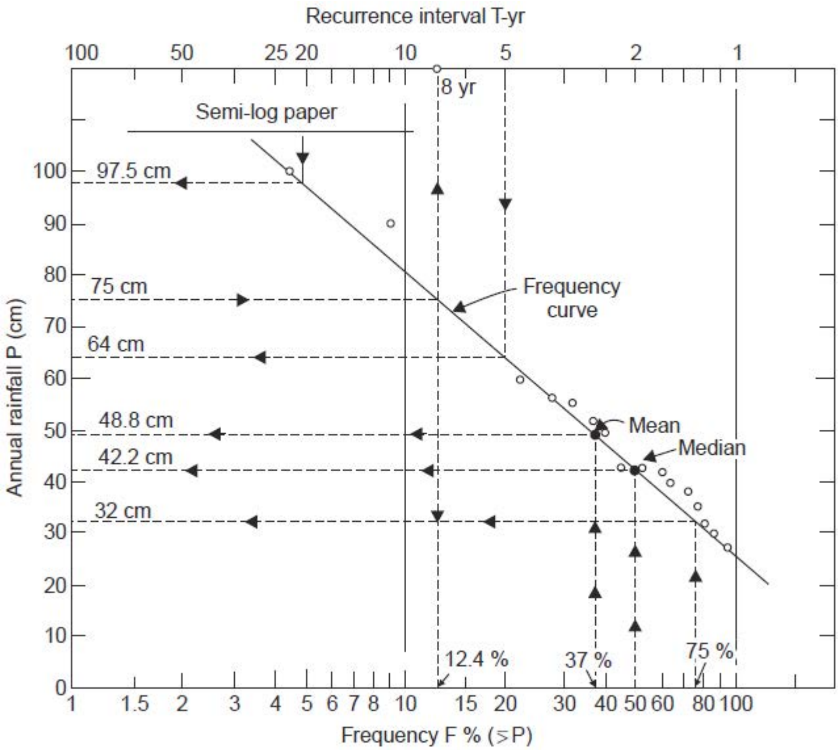
Arrange the yearly rainfall in the descending order of magnitude as given below. If a particular rainfall occurs in more than one year, m = no. of times exceeded + no. of times equaled.

Year	*Rainfall* *P (cm)*	*Rank (m)* *(no. of times ≥ P)*	*Frequency* $F = \frac{m}{n+1} \times 100\%$
1967	100	1	4.6
1968	90	2	9.1
1956	70	3	13.6
1951, 1957	60	5	22.7
1961	56	6	27.3
1959	55	7	31.8
1962	52	8	36.4
1950	50	9	40.9
1969	43	10	45.5
1963	42	11	50.0
1952, 1960, 1966	40	14	63.7
1955, 1964	38	16	72.8
1958	35	17	77.3
1970	33	18	81.8
1954	30	19	86.4
1953, 1965	27	21	95.5
	Total $\Sigma x = 1026$	$n = 21$	

Draw the graph of 'P vs. F' on a semi-log paper which gives the rainfall frequency curve. From the frequency-curve, the required values can be obtained as

$$(a)\,T = 5 - yr,\ F = \frac{1}{T} \times 100 = \frac{100}{5} = 20\%\ for\ which\ P = 64\ cm$$

$$T = 20 - year,\ F = \frac{1}{20} \times 100 = 5\%\ for\ which\ P = 97.5\ cm$$



(b) For F = 50%, P = 42.2 cm which is the median value, and the mean value

$$\bar{x} = \frac{\sum x}{n} = \frac{1026}{21} = 48.8\ cm$$

which has a frequency of 37%.

(c) For a probability of 0.75 F = 75% for which P = 32 cm

$$(d)\ For\ P = 75\ cm,\ F = 12.4\%,\ T = \frac{1}{F} \times 100 = \frac{100}{12.4} = 8\ yr$$

and its probability of occurrence = 0.124

14. The following are the monthly pan evaporation data (Jan.-Dec.) at Krishnarajasagara in a certain year in cm.
16.7, 14.3, 17.8, 25.0, 28.6, 21.4
16.7, 16.7, 16.7, 21.4, 16.7, 16.7
The water spread area in a lake nearby in the beginning of January in that year was 2.80 km² and at the end of December it was measured as 2.55 km². Calculate the loss of water due to evaporation in that year. Assume a pan coefficient of 0.7.

Mean water spread area of lake

$$A_{ave} = \frac{1}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right) = \frac{1}{3}\left(2.80 + 2.55 + \sqrt{2.80 \times 2.55}\right) = 2.673\, km^2$$

Annual loss of water due to evaporation (adding up the monthly values)= 228.7 cm
Annual volume of water lost due to evaporation= (2.673 × 106) ×(228.7/100)× 0.7= 4.29 × 10^6 m^3

15. Compute the daily evaporation from a Class A pan if the amounts of water added to bring the level to the fixed point are as follows:

Day	1	2	3	4	5	6	7
Rainfall (mm)	14	6	12	8	0	5	6
Water added (mm): removed	-5	3	0	0	7	4	3

What is the evaporation loss of water in this week from a lake (surface area = 640 ha) in the vicinity, assuming a pan coefficient of 0.75?

Pan evaporation, Ep, mm = Rainfall + water added or − water removed

day	1	2	3	4	5	6	7
Ep:	14-5	6+3	12	8	7	5+4	6+3
(mm):	=9	=9				=9	=9

$$Pan\ evaporation\ in\ the\ week = \sum_{1}^{7} E_p = 63\ mm$$

Pan coefficient 0.75 = E_L/E_p
Lake evaporation during the week E_L= 63 × 0.75 = 47.25 mm

$$Water\ lost\ from\ the\ lake = A \times E_L = 640 \times \frac{47.25}{1000} = 30.24\ ha - m$$

16. The total observed runoff volume during a storm of 6-hr duration with a uniform intensity of 15 mm/hr is 21.6 Mm³. If the area of the basin is 300 km², find the average infiltration rate and the runoff coefficient for the basin.

(i) Infiltration loss F_p = Rainfall (P) – Runoff (R)= 15 × 6 –(21.6/300)× 1000=18 mm

$$f_{ave} = \frac{F_p}{t} = \frac{18}{6} = 3\ mm/hr$$

(ii) Yield = C A P

$$21.6\times10^{6}\ m^{3}=C\left(300\times10^{6}\right)\frac{90}{1000}$$

C=0.8

17. Determine the evapotranspiration and irrigation requirement for wheat, if the water application efficiency is 65% and the consumptive use coefficient for the growing season is 0.8 from the following data:

Month	Mean monthly temp (°C)	Monthly percentage of sunshine (hours)	Effective rainfall (cm)
November	18	7.20	2.6
December	15	7.15	2.8
January	13.5	7.30	3.5
february	14.5	7.10	2.0

Month	*Mean monthly temp. (°C)* t	*Monthly % of sunshine (hours)* p	*Effective rainfall (cm)* P_e	*Monthly consumptive use factor* $f=\frac{p(4.6t+81.3)}{100}$
Nov.	18	7.20	2.6	11.82
Dec.	15	7.15	2.8	10.74
Jan.	13.5	7.30	3.5	10.48
Feb.	14.5	7.10	2.0	10.50
			$\Sigma P_e = 10.9$	$\Sigma f = 43.54$

Seasonal consumptive use, U = K Σ f= 0.8 × 43.54= 34.83 cm

Field irrigation requirement,

$$F.I.R.=\frac{U-\sum P_e}{\eta_i}=\frac{34.83-10.90}{0.65}=36.9\ cm$$

18. Assuming a growing season of 4 months December-March for wheat, determine the consumptive use of wheat in the month of January if the pan evaporation for the month is 9.5 cm. Take the consumptive use coefficient at 40%, stage growth of the crop as 0.52.

$E_t = kE_p$

The crop season is December to March i.e., 120 days. By middle of Jaunary the number of days of growth is 47, i.e., 47/120=40% stage growth of the crop has reached and k for this stage is 0.52 and E_p for the month of January is 9.5 cm.

E_t = 0.52 × 9.5 = 4.94 cm

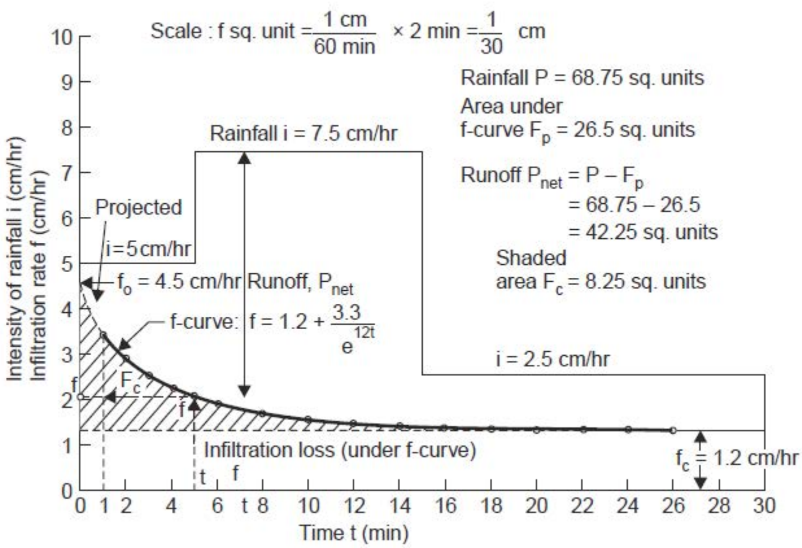
The daily consumptive use for the month of January= (4.94×10)/31=1.6 mm/day

19. For a given basin, the following are the infiltration capacity rates at various time intervals after the beginning of the storm. Make a plot of the f-curve and establish an equation of the form developed by Horton. Also determine the total rain and the excess rain (runoff).

Time (min)	Precipitation rate (cm/hr)	Infiltration capacity (cm/hr)
1	5.0	3.9
2	5.0	3.4
3	5.0	3.1
4	5.0	2.7
5	5.0	2.5
6	7.5	2.3
8	7.5	2.0
10	7.5	1.8
12	7.5	1.54
14	7.5	1.43
16	2.5	1.36
18	2.5	1.31
20	2.5	1.28
22	2.5	1.25
24	2.5	1.23
26	2.5	1.22
28	2.5	1.20
30	2.5	1.20

The precipitation and infiltration rates versus time are plotted as shown in the figure. In the Hortons equation, the Horton's constant

$$k = \frac{f_0 - f_c}{F_c}$$



From the figure, shaded area

$$F_c = 8.25\left(\frac{1\,cm}{60\text{ min}} \times 2\text{ min}\right) = 0.275\,cm$$

$$k = \frac{(4.5-1.2)\,cm/hr}{0.275\,cm} = 12\,hr^{-1}$$

The Hortons equation is

$f = f_c + (f_0 - f_c)e^{-kt} = 1.2 + (4.5 - 1.2)e^{-12t}$

is the equation for the infiltration capacity curve (f-curve) for the basin, where f is in cm/hr and t in hr.

$$f = 1.2 + \frac{3.3}{e^{12\times(1/6)}} = 1.7\,cm/hr, \text{ which is very near compared}$$

to the observed value of 1.8 cm/hr.

$$\text{Total rain P} = 68.75 \text{ sq. units} = 68.75 \times \frac{1}{30} = 2.29\,cm$$

Excess rain $P_{net} = P - F_p = 68.75 - 26.5 = 42.25$ sq. units= 1.41 cm

$$\text{Total infiltration } F_p = 26.5 \times \frac{1}{30} = 0.88\,cm$$

The total infiltration loss F_p can also be determined by intergrating the Hortons equation for the duration of the storm.

$$F_p = \int_0^t f\,dt = \int_0^{\frac{30}{60}} \left(1.2 + \frac{3.3}{e^{12t}}\right) dt = 1.2t + \frac{3.3}{12}\left(1 - \frac{1}{e^6}\right) = 0.6 + \frac{3.3}{12}\left(1 - \frac{1}{408}\right) = 0.88\,cm$$

$P_{net} = P - F_p = 2.29 - 0.88 = 1.41$ cm

which compares with the value obtained earlier.

$$\text{Ave. infiltration loss } f_{ave} = \frac{F_p}{t} = \frac{0.88\text{ cm}}{0.5} = 1.76\,cm/hr$$

To determine the Horton's constant by drawing a semi-log plot of t vs. $(f - f_c)$:

The Horton's equation is

$f = f_c + (f_0 - f_c)e^{-kt}$

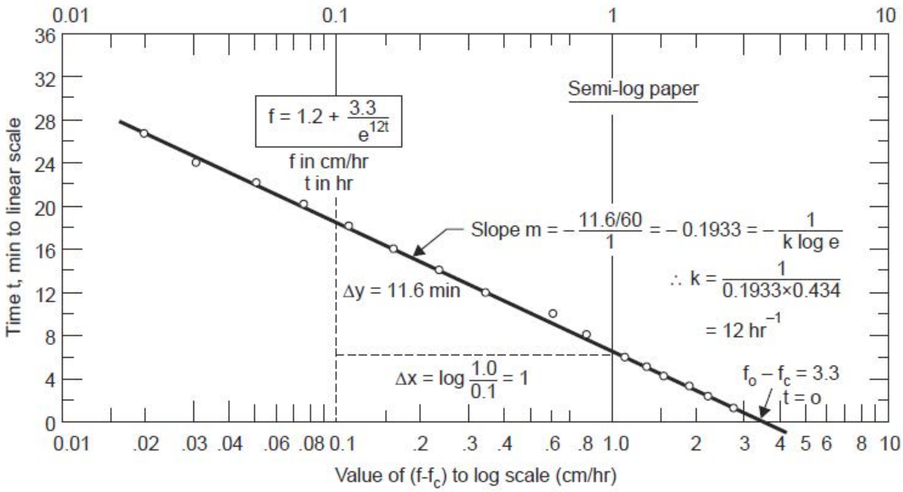
$\log (f - f_c) = \log (f_0 - f_c) - kt \log e$

Solving for t,

$$t = \frac{\log\left(f_0 - f_c\right)}{k \log e} - \frac{\log\left(f - f_c\right)}{k \log e}$$

which is in the form of a straight line y = mx + c in which y = t, x = $\log (f - f_c)$, m = $-1/k \log e$. Hence, from a plot of t vs. $(f - f_c)$ on a semi-log paper (t to linear scale), the constants in the Horton's equation can be determined.

From the given data, f_c = 1.2 cm/hr and the values of $(f - f_c)$ for different time intervals from the beginning are: 2.7, 2.2, 1.9, 1.5, 1.3, 1.1, 0.8, 0.6, 0.46, 0.32, 0.22, 0.16, 0.12, 0.05, 0.04, 0.02, 0.0 cm/hr, respectively; (note: 3.9 – 1.2 = 2.7 cm/hr and like that for other readings). These values are plotted against time on a semi-log paper as shown in the figure.



From the figure, m = – 0.1933 = –1/k log e

$$k = \frac{1}{0.1933 \times 0.434} = 12\ hr^{-1}$$

Also from the graph, when t = 0,

$f - f_c = 3.3 = f_0 - f_c$, (since $f = f_0$ when t = 0)

$f_0 = 3.3 + 1.2 = 4.5$ cm/hr

Hence, the Hortons equation is of the form

$f = 1.2 + (4.5 - 1.2)e^{-12t}$

$$Total\ rain\ P = 5 \times \frac{5}{60} + 7.5 \times \frac{10}{60} + 2.5 \times \frac{15}{60} = 2.29\ cm$$

Infiltration loss F_p = 0.88 cm

Excess rain (runoff), $P_{net} = P - F_p = 2.29 - 0.88 = 1.41$ cm

which compares with the value obtained earlier.

20. For a small catchment, the infiltration rate at the beginning of rain was observed to be 90 mm/hr and decreased exponentially to a constant rate of 8 mm/hr after 2.5 hr. The total infiltration during 2.5 hr was 50 mm. Develop the Horton's equation for the infiltration rate at any time t<2.5 hr.

$$k = \frac{f_0 - f_c}{F_c} = \frac{90 - 8}{50 - 8 \times 2.5} = 2.73\ hr^{-1}$$

$f = f_c + (f_0 - f_c)e^{-kt} = 8 + (90 - 8)e^{-2.73\,t}$

21. A 24-hour storm occurred over a catchment of 1.8 km² area and the total rainfall observed was 10 cm. An infiltration capacity curve prepared had the initial infiltration capacity of 1 cm/hr and attained a constant value of 0.3 cm/hr after 15 hours of rainfall with a Horton's constant k = 5 hr⁻¹. An IMD pan installed in the catchment indicated a decrease of 0.6 cm in the water level (after allowing for rainfall) during 24 hours of its operation. Other losses were found to be negligible. Determine the runoff from the catchment. Assume a pan coefficient of 0.7.

$$F_p = \int_0^{24}\left[f_c + \left(f_0 - f_c\right)e^{-kt}\right]dt = \int_0^{24}\left[0.3 + \left(1.0 - 0.3\right)e^{-5t}\right]dt = 0.3t + \frac{0.7}{-5e^{5t}}\Big]_0^{24}$$

$$= \left[0.3 \times 24 - \frac{0.7}{5e^{5\times 24}}\right] - \left[0 - \frac{0.7}{5e^{0}}\right] = 7.2 + \frac{0.7}{5}\left(1 - \frac{1}{e^{120}}\right) = 7.34\,cm$$

Runoff = P – F_p – E = 10 – 7.34 – (0.60 × 0.7)= 2.24 cm
Volume of runoff from the catchment= (2.24/100)(1.8 × 10^6)=40320 m³

22. In a double ring infiltrometer test, a constant depth of 100 mm was restored at every time interval the level dropped as given below:

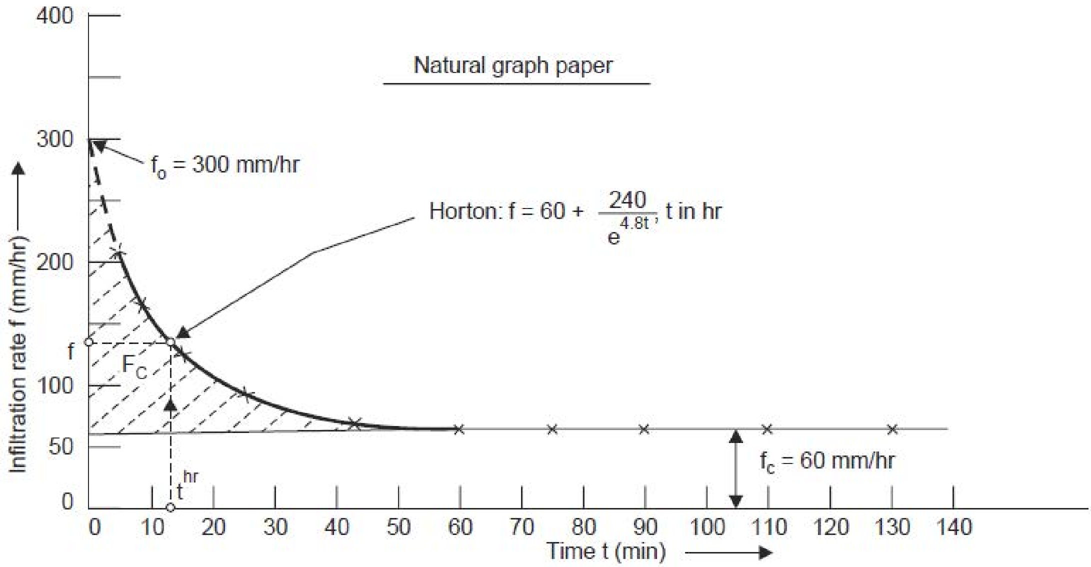
Time (min)	0	5	10	15	25	45	60	75	90	110	130
Depth of water (mm)	100	83	87	90	85	78	85	85	85	80	80

(i) Establish the infiltration equation of the form developed by Horton.

(ii) Obtain the equation for cumulative infiltration of the form (a) $F = at^n$ (b) $F = at^n + b$.

Time t (min)	*Depth to water Surface (mm)*		*Depth of infiltration* *d (mm)*	*Infiltration rate* $f = \frac{d}{\Delta t} \times 60$ *(mm/hr)*	*Cumulative infiltration* $F = \Sigma d$ *mm*	$f_c = 60$ *mm/hr* $f - f_c$ *mm/hr*
	Before filling	*After filling*				
0	100	—	0	f_0	0	—
5	83	100	17	$\frac{17}{5} \times 60 = 204$	17	144
10	87	100	13	$\frac{13}{10-5} \times 60 = 156$	30	96
15	90	100	10	$\frac{10}{15-10} \times 60 = 120$	40	60
25	85	100	15	90	55	30
45	78	100	22	66	77	6
60	85	100	15	$60 = f_c$	92	0
75	85	100	15	60	107	0
90	85	100	15	60	122	0
110	80	100	20	60	142	0
130	80	100	20	60	162	0

(i) (a) Plot on natural graph paper, t vs. f,



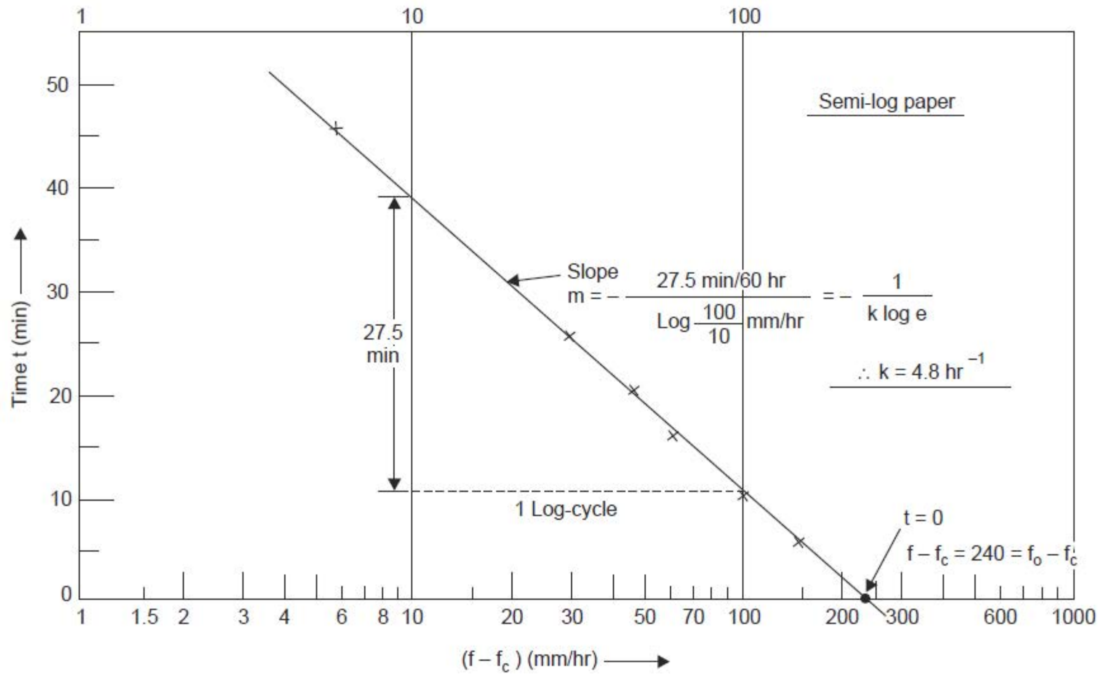
Horton's equation $f = f_c + (f_0 - f_c)\, e^{-kt}$

f_0 = 300 mm/hr, f_c = 60 mm/hr

F_c = shaded area= 6 sq. units × (50/60) × 10 min= 50 mm

$$k = \frac{f_0 - f_c}{F_c} = \frac{300 - 60}{50} = 4.8\, hr^{-1}$$

(b) Plot on semi-log paper'‘t vs. log $(f - f_c)$’,



$f = f_c + (f_0 - f_c)e^{-kt}$

$\log (f - f_c) = \log (f_0 - f_c) - kt \log e$

$$t = \frac{\log(f_0 - f_c)}{k \log e} - \frac{\log(f - f_c)}{k \log e}$$

i.e., of the form, y = c + mx

$$Slope\ m = -\frac{1}{k \log e} = -\frac{27.5/60}{1} \rightarrow k = 4.8\ hr^{-1}$$

at t = 0, $f - f_c = 240 = f_0 - f_c$

$f_0 = 240 + 60 = 300$ mm/hr

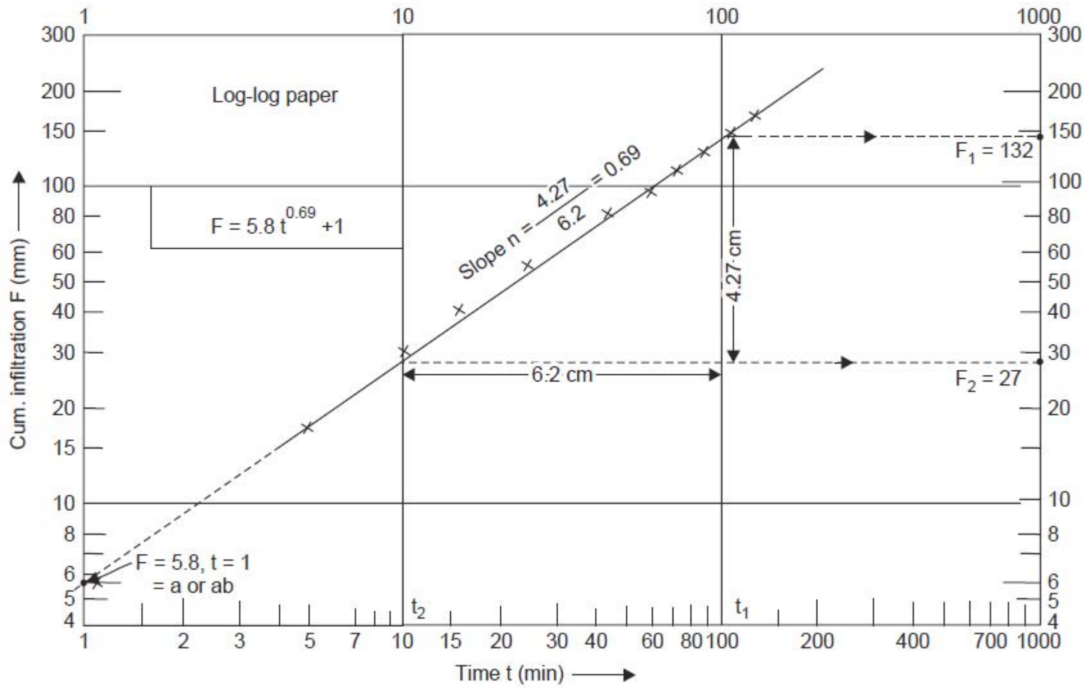
(ii) Cumulative infiltration curve

(a) $F = at^n$, kostiakov

Plot 't vs. F' on log-log paper

log F = log a + n log t

i.e., y = c + mx form, yields a straight line,
when t = 1, a = F = 5.8



$$also, \log \frac{F_1}{F_2} = n \log \frac{t_1}{t_2}$$

$$\frac{F_1}{F_2} = \left(\frac{t_1}{t_2}\right)^n$$

$$\frac{132}{27} = \left(\frac{100}{10}\right)^n$$

n = 0.69, also from the plot,

$$f = \frac{dF}{dt} = 5.8 \times 0.69 t^{-0.31}$$

(b) $F = at^n + b$

log F = log ab + n log t, yields straight line plot,

$$\frac{F_1}{F_2} = \left(\frac{t_1}{t_2}\right)^n$$

n = 0.69 = slope from the plot

when t = 1, F = ab = 5.8, from the plot,

$$Try\; b = 1, a = \frac{5.8}{1} = 5.8$$

say t = 25 min, F = 5.8 $(25)^{0.69}$ + 1 = 55 mm

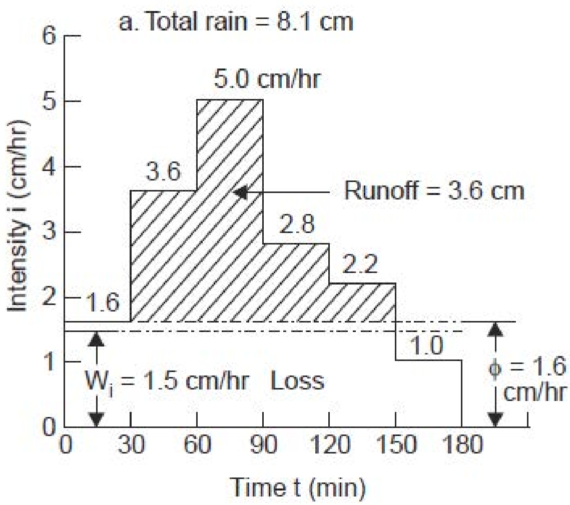
$$also\; f = \frac{dF}{dt} = 5.8 \times 0.69 t^{-0.32}$$

$$i.e., f = \frac{4}{t^{0.32}}, \text{at t} = 25 \text{ min, f} = 1.48 \text{ mm/ min}$$

which are very near the observed values; otherwise a second trial value of b is necessary.

23. The rates of rainfall for the successive 30 min period of a 3-hour storm are: 1.6, 3.6, 5.0, 2.8, 2.2, 1.0 cm/hr. The corresponding surface runoff is estimated to be 3.6 cm. Establish the φ-index. Also determine the W-index.

Construct the hyetograph as shown in the figure.



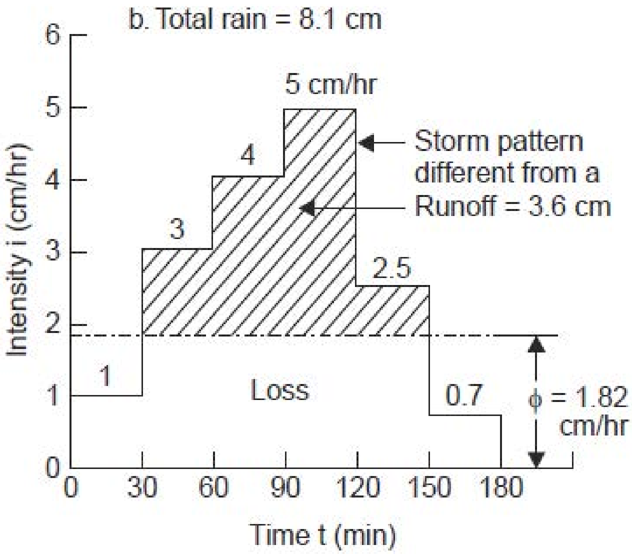
Σ(i – φ)t = P_{net}, and thus it follows

$$\left[(3.6-\phi)+(5.0-\phi)+(2.8-\phi)+(2.2-\phi)\right]\frac{30}{60} = 3.6 \rightarrow \phi = 1.6\, cm/hr$$

$$P = (1.6+3.6+5.0+2.8+2.2+1.0)\frac{30}{60} = 8.1\, cm$$

$$W-index = \frac{P-Q}{t_R} = \frac{8.1-3.6}{3} = 1.5\, cm/hr$$

Suppose the same 3-hour storm had a different pattern as shown in the figure producing the same total rainfall of 8.1 cm. To obtain the same runoff of 3.6 cm (shaded area), the φ-index can be worked out as 1.82 cm/hr. Hence, it may be seen that a single determination of φ-index is of limited value and many such determinations have to be made and averaged, before the index is used. The determination of φ-index for a catchment is a trial and error procedure.



24. Hourly rainfalls of 2.5, 6, and 3 cm occur over a 20-ha area consisting 4 ha of φ = 5 cm/hr, 10 ha of φ = 3 cm/hr, and 6 ha of φ = 1 cm/hr. Derive hourly values of net rain.

$$1st\ hour\ (P = 2.5\ cm)\ P_{net-mean} = \frac{4(0)+10(0)+6(2.5-1)}{20} = 0.45\ cm$$

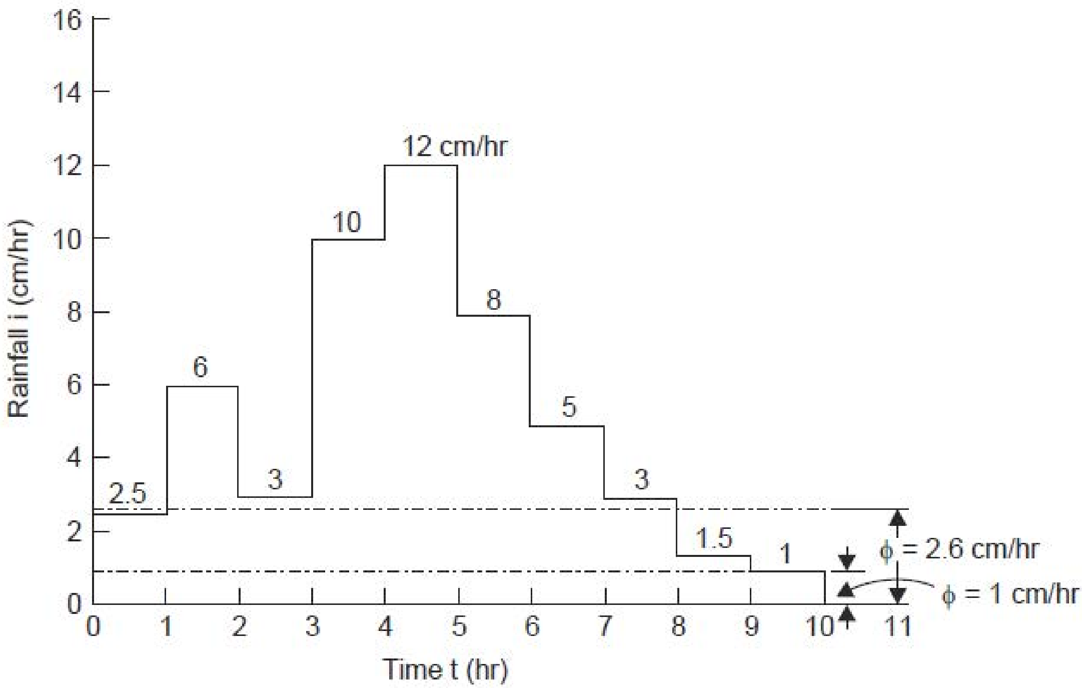
$$2nd\ hour\ (P = 6\ cm)\ P_{net-mean} = \frac{4(6-5)+10(6-3)+6(6-1)}{20} = 3.20\ cm$$

$$3rd\ hour\ (P = 3\ cm)\ P_{net-mean} = \frac{4(0)+10(0)+6(3-1)}{20} = 0.60\ cm$$

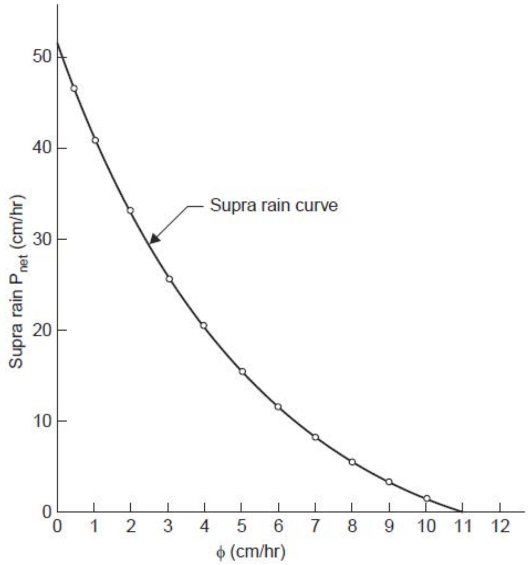
Total net rain for the 3-hour storm = 4.25 cm

25. The successive hourly rains of a 10-hour storm are: 2.5, 6.3, 10, 12, 8, 5, 3, 1.5, 1 cm. Using the supra-rain-curve technique, determine the total net rain and its time distribution for a 20-hr area consisting of 4 ha of φ = 5 cm/hr, 10 ha of φ = 3 cm/hr and 6 ha of φ = 1 cm/hr.

For φ = 1 cm/hr, P_{net} (supra-rain) from the hyetograph—the figure is 41 cm. Similarly, for φ = 0.5, 2, 3, 4, 5, 6, 7, 8, 9 and 10 cm, P_{net} (supra-rain) values are 47, 33.5, 26, 21, 16, 12, 9, 6, 4 and 2 cm, respectively. With these values, the supra-rain-curve is plotted as shown in the figure. The supra-rain for the 20-ha area can be obtained by weighing for the sub-areas as follows:



Sub area A1 (ha)	Φ-index (cm/hr)	Sub-areal supra rain P_{net-1} (cm)	A_1/A (decimal)	Product 3 x 4 (cm)
1	2	3	4	5
4	5	16	0.2	3.2
10	3	26	0.5	13.0
6	1	41	0.3	12.3
A = 20 ha			Total net rain over basin	= 28.5 cm

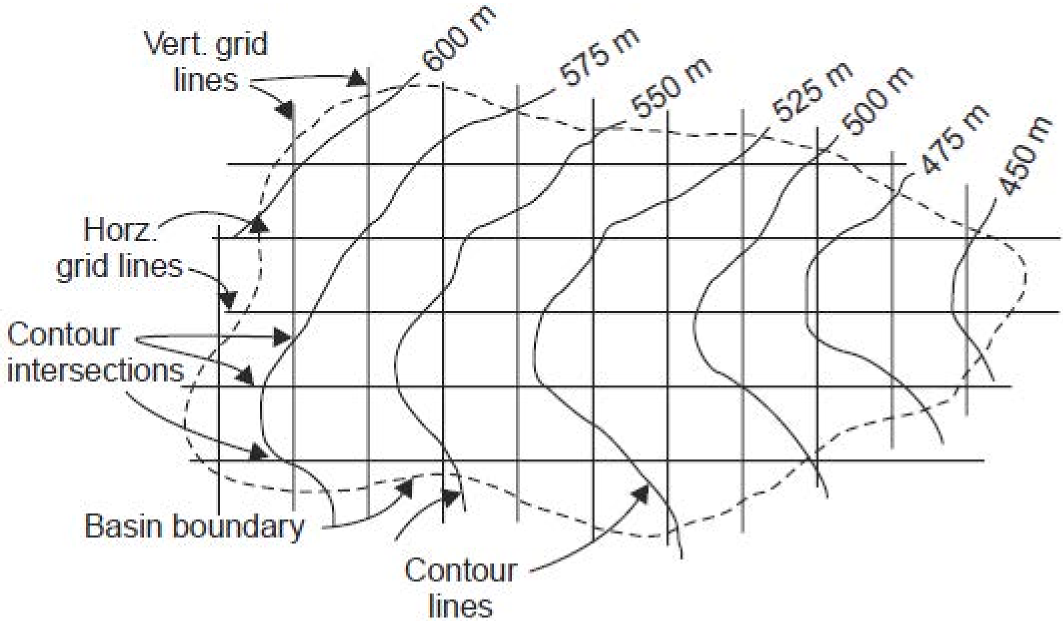


Corresponding to this supra-rain of 28.5 cm, the mean effective φ-index for the entire 20 ha, from the figure, is 2.6 cm/hr. Application of φ = 2.6 cm/hr to the values of hourly rainfalls of the 10-hr storm. Fig. 3.13 gives the values of hourly net rain as 0, 3.4, 0.4, 7.4, 9.4, 5.4, 0.4, 0 and 0 cm, respectively, giving a total of 28.8 cm.

The hourly net rains are obtained in the table, which also gives a total net rain of 28.80 cm, though the hourly net rains are slightly different from those obtained from the supra-rain-curve technique.

hour	Rainfall(cm)	Rainfall excess, P_{net} from sub areas			Weighted P_{net} from sub-areas			P_{net} over a basin (cm)
		A1 Φ = 5cm/hr	A2 Φ = 3cm/hr	A3 Φ = 1cm/hr	3 x 0.2 (cm)	4 x 0.5 (cm)	5 x 0.3 (cm)	6 + 7 + 8
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
1	2.5	-	-	1.5	-	-	0.45	0.45
2	6	1	3	5	0.2	1.5	1.50	3.20
3	3	-	-	2	-	-	0.6	0.60
4	10	5	7	9	1.0	3.5	2.7	7.20
5	12	7	9	11	1.4	4.5	3.3	9.20
6	8	3	5	7	0.6	2.5	2.1	5.20
7	5	-	2	4	-	1.0	1.2	2.20
8	3	-	-	2	-	-	0.6	0.60
9	1.5	-	-	0.5	-	-	0.15	0.15
10	1	-	-	-	-	-	-	0

26. The contour map of a basin is subdivided into a number of square grids of equal size by drawing horizontal and vertical lines as shown in the figure. The contour interval is 25 m.



The number of contour intersections by vertical lines is 75 and by horizontal lines 126. The total length of the vertical grid segments (after multiplying by the scale) is 53260 m and of the horizontal grid segments 55250 m. Determine the mean slope of the basin.

Slope in the vertical direction

$$S_v = \frac{N_c \times C.I.}{\sum Y} = \frac{75 \times 25}{53260} = 0.0352\, m/m$$

Slope in the horizontal direction

$$S_x = \frac{N_c \times C.I.}{\sum X} = \frac{126 \times 25}{55250} = 0.0570\, m/m$$

Mean slope of the basin

$$S = \frac{S_v + S_x}{2} = \frac{0.0352 + 0.0570}{2} = 0.0461\, m/m$$

Also, from the Hortons equation,

$$S = \frac{1.5(C.I.)N_c}{\sum L} = \frac{1.5 \times 25(75+126)}{(53260+55250)} = 0.0695$$

27. A basin has an area of 26560 km^2, perimeter 965 km and length of the thalweg

230 km. Determine: (i) form factor, (ii) compactness coefficient, (iii) elongation ratio, and (iv) circularity ratio.

$$(i)\ Form\ factor,\ F_f = \frac{A}{L_b^2} = \frac{26560}{230^2} = 0.502$$

An inverted factor will give 2

(ii) Compactness Coefficient C_c

Radius R of an equivalent circular area is given by

$26560 = \pi R^2 \rightarrow R = 91.9$ km

$$C_c = \frac{P_b}{2\pi R} = \frac{965}{2\pi(91.9)} = 1.67$$

$$(iii)\ Elongation\ ratio,\ E_r = \frac{2R}{L_b} = \frac{2(91.9)}{230} = 0.8$$

(iv) Circularity ratio C_r

Radius R′ of a circle of an equivalent perimeter as the basin is given by

$2\pi R' = 965 \rightarrow R' = 153.5$ km

$$C_r = \frac{A}{\pi R'^2} = \frac{26560}{\pi(153.5)^2} = 0.358$$

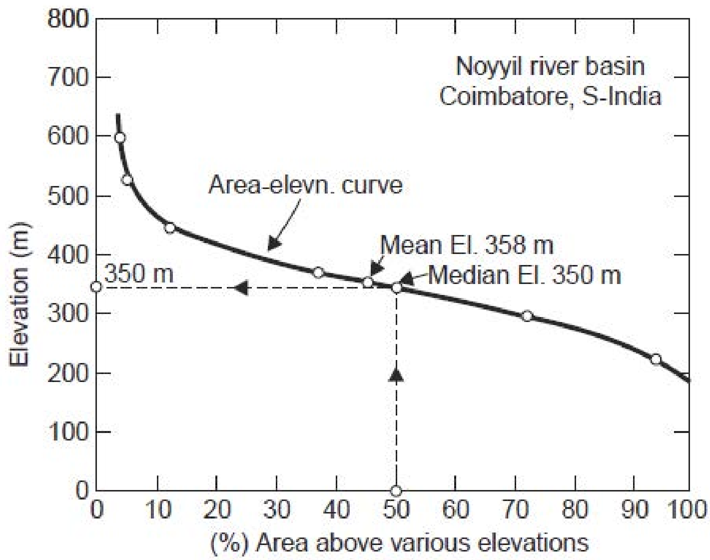
28. The areas between different contour elevations for the Noyyil River basin, Coimbatore (south India) are given below. Determine the mean and the median elevation for the basin.

Contour elevations (m)	Area between contours (km^2)
< 225	181
225-300	723
300-375	1144
375-450	814
450-525	216
525-600	46
>600	140

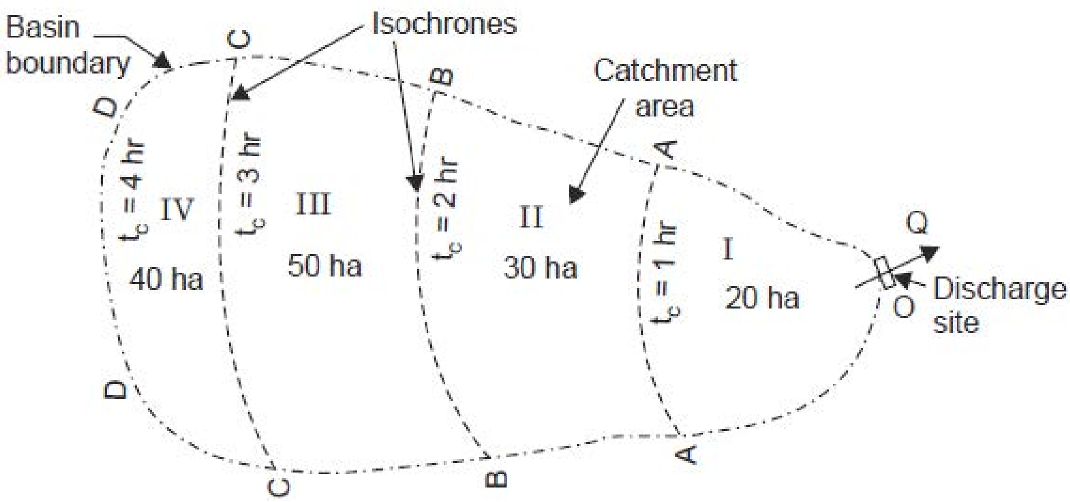
Contour elevation (m)	*Mean elevation between contours, z_1 (m)*	*Area between contours, a_1 (km^2)*	*Product $a_1 z_1$ (2) × (3) (km^2–m)*	*Mean elevation (m)*
1	*2*	*3*	*4*	*5*
<225	200	181	36200	
225-300	262.5	723	190000	$z_b = \frac{\Sigma a_1 z_1}{\Sigma a_1}$
300-375	337.5	1144	386000	
375-450	412.5	814	335500	$= \frac{1169940}{3264}$
450-525	487.5	216	105400	**= 358 m**
525-600	562.5	46	25840	
>600	650	140	91000	
		3264 $= \Sigma a_1 = A$	1169940 $= \Sigma a_1 z_1$	

Contour elevations (m)	Area between contours, a1 (km^2)	Percentage of total area (%)	% of total area over given lower limit
<225	181	5.5	100.0
225-300	723	22.1	94.5
300-375	1144	35.1	72.4
375-450	814	25.0	37.3
450-525	216	6.6	12.3
525-600	46	1.4	5.7
<600	140	4.3	4.3
	A= 3264		

The hypsometric curve is obtained by plotting the contour elevation (lower limit) against the corresponding percent of total area; the median elevation for 50% of total area is read from the curve as 350 m, while the mean elevation is 358 m.



29. A 4-hour rain of average intensity 1 cm/hr falls over the fern leaf type catchment as shown in the figure. The time of concentration from the lines AA, BB, CC and DD are 1, 2, 3 and 4 hours, respectively, to the site 0 where the discharge measurements are made. The values of the runoff coefficient C are 0.5, 0.6, and 0.7 for the 1st, 2nd and 3rd hours of rainfail respectively and attains a constant value of 0.8 after 3 hours. Determine the discharge at site 0.



The discharge computations are made in the table.

Sub-area (zone) contributing runoff (ha)	Time from beginning of storm (hr)							
	1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
I	20	20	20	20				
II		30	30	30	30			
III			50	50	50	50		
IV				40	40	40	40	—
Discharge at 0 from sub-areas $Q = \Sigma CAP$	$0.5\,(20 \times 10^4)\,\frac{1}{100}$	$0.6\,(20 \times 10^4)\,\frac{1}{100}$	$0.7\,(20 \times 10^4)\,\frac{1}{100}$	$0.8\,(20 \times 10^4)\,\frac{1}{100}$	—			
		$0.5\,(30 \times 10^4)\,\frac{1}{100}$	$0.6\,(30 \times 10^4)\,\frac{1}{100}$	$0.7\,(30 \times 10^4)\,\frac{1}{100}$	$0.8\,(30 \times 10^4)\,\frac{1}{100}$	—		
			$0.5\,(50 \times 10^4)\,\frac{1}{100}$	$0.6\,(50 \times 10^4)\,\frac{1}{100}$	$0.7\,(50 \times 10^4)\,\frac{1}{100}$	$0.8\,(50 \times 10^4)\,\frac{1}{100}$	—	
				$0.5\,(40 \times 10^4)\,\frac{1}{100}$	$0.6\,(40 \times 10^4)\,\frac{1}{100}$	$0.7\,(40 \times 10^4)\,\frac{1}{100}$	$0.8\,(40 \times 10^4)\,\frac{1}{100}$	—
Discharge at 0 Q (m^3/hr)	1000	2700	5700	8700	8300	6800	3200	—

30. The following data are collected for a proposed tank in the Deccan plains of south India:
Catchment area = 1200 ha
a. a. r. = 90 cm
Intensity of rainfall of duration and frequency 35 years = 5 cm/hr
Average runoff coefficient for the whole catchment = 20%
Tank gets filled = 1 ½ times in a year
Difference between the maximum water level (MWL) and full tank level (FTL) = 0.6 cm
Determine
(a) the yield of the catchment and the capacity of the tank
(b) the area of rice crop that can be irrigated from the tank
(c) the duties of water assumed and the discharge at the head to the distributor
(d) the length of clear overfall weir near one flank.

A.A.R. is available only in 50% of the years. To ensure filler of the tank in deficient years dependable rainfall ≈ 75% of a. a. r.=0.75 × 90 = 67.5 cm or 0.675 m

$$R = \frac{(P-17.8)P}{254} = \frac{(67.5-17.8)67.5}{254} = 13.2\,cm$$

Since the runoff coefficient C = 20% (given)

R = CP = 0.20 × 67.5 = 13.5 cm

which compares well with the value obtained above by applying the empirical formula for the region.

Yield from the catchment = CAP = 0.2 × 1200 × 0.675 = 162 ha-m

Since the tank gets filled 1.5 times in a year,

$$Capacity\ of\ the\ bank = \frac{162}{1.5} = 108\,ha-m$$

(b) Assuming loss of water due to evaporation and seepage as 10% in the tank and 20% in the distributary
Water available at the field outlet = 162 (1 – 0.3) = 113.4 ha-m
For rice crop assuming U = 88 cm, crop period B = 120 days

$$Field\ irrigation\ requirement, \Delta = \frac{U}{\eta_{irrgn}} = \frac{88}{0.7} = 126\ cm$$

$$Area\ of\ rice\ crop\ that\ can\ be\ irrigated = \frac{113.4\ h-m}{1.26\ m} = 90\ ha$$

$$(c)\ Tank\ duty = \frac{1-0.3}{1.26} = 0.555\ ha/ha-m\ of\ annual\ storage\ (i.e.,\ yield)$$

$$For\ 1\ Mm^3 : \frac{(1-0.3)10^6\ m^3}{1.26\times 10^4\ m^2/ha} = 55.5\ ha/Mm^3\ of\ annual\ storage$$

$$D = \frac{8.64B}{\Delta} = \frac{8.64\times 120\ days}{1.26\ m} = 823\ ha/cumec$$

$$Discharge\ at\ field\ outlet = \frac{90}{823} = 0.1093\ cumec$$

Discharge at the head of the distributary, i.e., tank outlet= 0.1093/0.80 = 0.137 cumec = 137 lps

(d) Length of the clear overfall weir (L):

Using the rational formula for the maximum rate of runoff

$$Q = CiA = 0.2\frac{5}{100(60\times 60)}\left(1200\times 10^4\right) = 33.3\ cumec$$

Weir formula is $Q = CLH^{3/2}$
Head over the weir H = MWL – FTL = 0.6 m, L = length of the weir
Assuming a weir coefficient C of 1.84, the weir formula becomes
$33.3 = 1.84\ L\ (0.6)^{3/2}$
L = 39 m

31. A small watershed consists of 1.5 km² of cultivated area (c = 0.2), 2.5 km² under forest (c = 0.1) and 1 km² under grass cover (c = 0.35). There is a fall of 20 m in a watercourse of length 2 km. The I–D–F relation for the area is given by $I=(80T^{0.2})/(t+12)^{0.5}$. Estimate the peak rate of runoff for a 25-year frequency.

Time of concentration (Kirpich's formula–modified)

$$t_c \approx 0.02L^{0.8}S^{-0.4} = 0.02(2000)^{0.8}\left(\frac{20}{2000}\right)^{-0.4} = 55\ \min = t$$

$$I = \frac{80\times 25^{0.2}}{(55+12)^{0.5}} = 18.6\ cm/hr$$

Q = CIA = 2.78 I (Σ Ci Ai) = 2.78 × 18.6 (1.5 × 0.2 + 2.5 × 0.1 + 1 × 0.35) = 46.5 Cumec

32. The mean daily streamflow data from a drainage basin is given below. It is known that the recession limb of the discharge hydrograph has components of channel storage, interflow and base flow. Find the values of the recession coefficients for each of the three components.

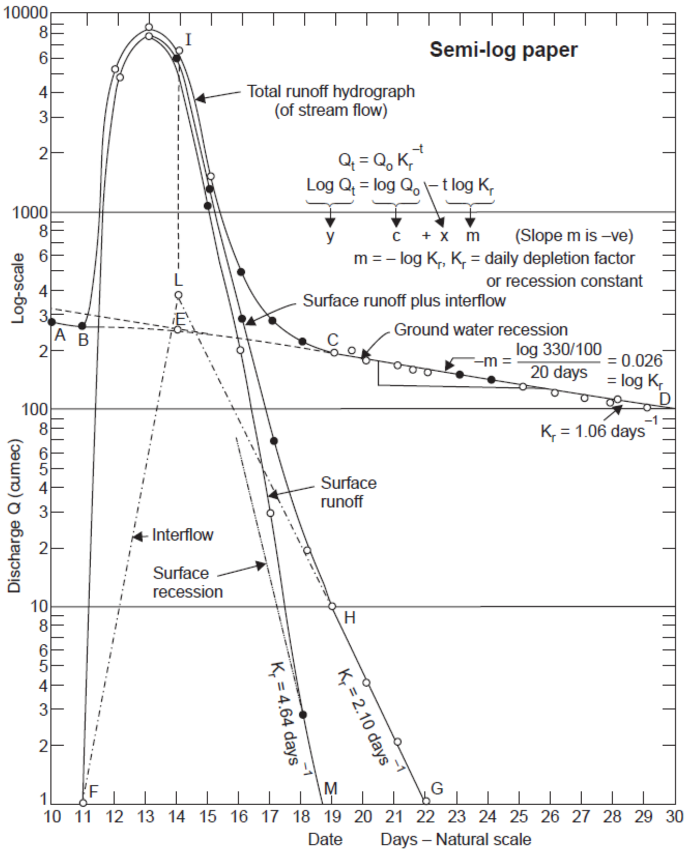
date	Mean daily discharge (cumec)	date	Mean daily discharge (cumec)
1978, oct 4	278	1978, oct 14	179
5	265	15	167
6	5350	16	157
7	8150	17	147
8	6580	18	139
9	1540	19	131
10	505	20	123
11	280	21	117
12	219	22	111
13	195	23	105
		24	100

Also determine

(a) ground water storage on October 14, 1978.

(b) ground water storage and stream flow on October 30, 1978, assuming no rainfall during the period.

The discharge hydrograph is drawn on a semi-log paper and the flow components are separated by the method proposed by Bernes as shown in the figure. The recession coefficients (Kr) for the three components of base flow (ground water contribution), interflow and channel storage are computed as 1.059, 2.104 and 4.645, respectively.



(a) Ground water storage (S0) on October 14, 1978 when the ground water depletion starts.

$$S_0 = \frac{Q_0}{\log_e k_r} = \frac{179 \times 86400\, m^3 / day}{\log_e 1.06 / day} = 2.75 \times 10^8\, m^3$$

(b) Stream flow on October 30, 1978, i.e., after 16 days

$Q_t = Q_0 K_r^{-t}$

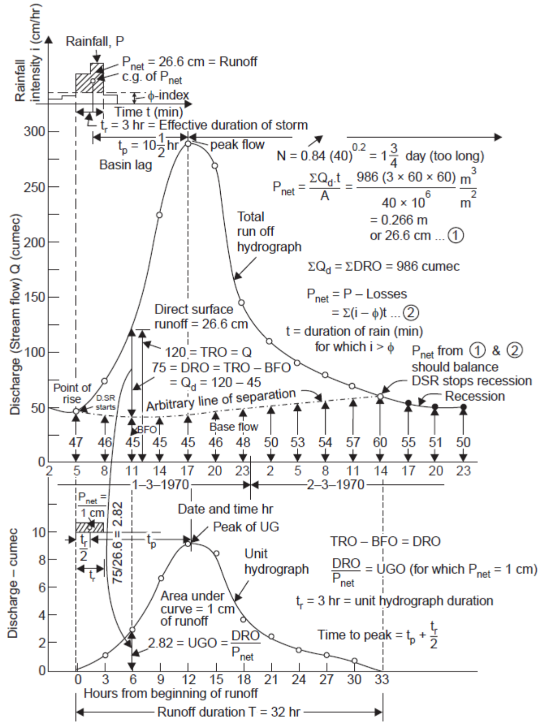
$Q_{16\ days} = 179\ (1.06)^{-16} = 71.6$ cumec

Ground water storage (S_t) on October 30, 1978 can be determined from

$$\frac{Q_0}{Q_t} = \frac{S_0}{S_t}$$

$$S_t = S_0 \times \frac{Q_t}{Q_0}$$

$$S_{16\,days} = \left(2.75 \times 10^8\right) \frac{71.6}{179} = 1.10 \times 10^8\, m^3$$



33. The runoff data at a stream gauging station for a flood are given below. The drainage area is 40 km2. The duration of rainfall is 3 hours. Derive the 3-hour unit hydrograph for the basin and plot the same.

date	Time (hr)	Discharge (cumec)	Remarks
1-3-1970	2	50	
	5	47	
	8	75	
	11	120	
	14	225	
	17	290	← peak
	20	270	
	23	145	
2-3-1970	2	110	
	5	90	
	8	80	
	11	70	
	14	60	
	17	55	
	20	51	
	23	50	

State the peak of the unit hydrograph you derive.

Date	Time (hr)	TRO[1] (cumec)	BFO[2] (cumec)	DRO[2] (3)–(4) (cumec)	UGO[4] (5) ÷ P_{net} (cumec)	Time[5] from beginning of surface run off (hr)
1	2	3	4	5	6	7
1-3-1970	2	50	50	—	—	—
	5	47	47	0	0	0
	8	75	46	29	1.09	3
	11	120	45	75	2.82	6
	14	225	45	180	6.77	9
	17	290	45	245	9.23	12
	20	270	46	224	8.44	15
	23	145	48	97	3.65	18
2-3-1970	2	110	50	60	2.26	21
	5	90	53	37	1.39	24
	8	80	54	26	0.98	27
	11	70	57	13	0.49	30
	14	60	60	0	0	33
	17	55	55	—		
	20	51	51	—		
	23	50	50	—		
				Σ *DRO* = 986 cumec		

[1]*TRO*—Total runoff ordinate = gauged discharge of stream

[2]BFO—Base flow ordiante read from graph separation line shown in Fig. 5.10 (*a*). $N = 0.83\ A^{0.2} = 0.89(40)^{0.2}$ = 1.73 days = 1.73 × 24 = 41.4 hr from peak, which is seen not applicable here; hence an arbitrary separation line is sketched.

[3]*DRO*—Direct runoff ordinate = *TRO*—*B.F.O.*

[4]*UGO*—Unit hydrograph ordinate

$$= \frac{DRO}{P_{net}};\ P_{net} = \frac{\Sigma DRO\,.\,t}{A} = \frac{986(3\times60\times60)\text{m}^3}{40\times10^6\ \text{m}^2}$$

$$= 0.266\ \text{m} = 26.6\ \text{cm}$$

[5]Time from beginning of direct surface runoff is at 5 hr on 1-3-1970, which is reckoned 0 hr for unit hydrograph. The time base for unit hydrograph is 33 hours.

The 3-hour unit hydrogryph is plotted in Fig. 5.10 (*b*) to a different vertical scale and its peak is 9.23 cumec.

34. The stream flows due to three successive storms of 2.9, 4.9 and 3.9 cm of 6 hours duration each on a basin are given below. The area of the basin is 118.8 km². Assuming a constant base flow of 20 cumec, derive a 6-hour unit hydrograph for the basin. An average storm loss of 0.15 cm/hr can be assumed.

Time (hr)	0	3	6	9	12	15	18	21	24	27	30	33
Flow (cumec)	20	50	92	140	199	202	204	144	84.5	45.5	29	20

Let the 6-hour unit hydrograph ordinates be u_0, u_1, u_2, u_3, u_4, …., u_7 at 0, 3, 6, 12, ….,21 hours, respectively. The direct runoff ordinates due to the three successive storms (of 6 hours duration each) are obtained by deducting the base of flow of 20 cumec from the streamflows at the corresponding time intervals as shown in the table. The net storm rains are obtained by deducting the average storm loss as

0-6 hr: x = 2.9 – 0.15 × 6 = 2 cm

6-12 hr: y = 4.9 – 0.15 × 6 = 4 cm

12-18 hr: z = 3.9 – 0.15 × 6 = 3 cm

Time (hr)	UGO*	DRO due to**			Equation	Solution
		1st storm UGO × x	2nd storm UGO × y	3rd storm UGO × z	Total DRO = TRO— BFO	6-hr UGO
0	$u_0 = 0$	0	—	—	$0 = 20 - 20$	$u_0 = 0$
3	u_1	$2u_1$	—	—	$2u_1 = 50 - 20$	$u_1 = 15$
6	u_2	$2u_2$	0	—	$2u_2 = 92 - 20$	$u_2 = 36$
9	u_3	$2u_3$	$4u_1$	—	$2u_3 + 4u_1 = 140 - 20$	$u_3 = 30$
12	u_4	$2u_4$	$4u_2$	0	$2u_4 + 4u_2 + 0 = 199 - 20$	$u_4 = 17.5$
15	u_5	$2u_5$	$4u_3$	$3u_1$	$2u_5 + 4u_3 + 3u_1 = 202 - 20$	$u_5 = 8.5$
18	u_6	$2u_6$	$4u_4$	$3u_2$	$2u_6 + 4u_4 + 2u_2 = 204 - 20$	$u_6 = 3$
21	u_7	$2u_7$	$4u_5$	$3n_3$	$2u_7 + 4u_5 + 3u_3 = 144 - 20$	$u_7 = 0$
24			$4u_6$	$3u_4$	$4u_6 + 3u_4 = 84.5 - 20$	$\Sigma u = 110$
27			$4u_7$	$3u_5$	$4u_7 + 3u_5 = 45.5 - 20 \quad \therefore u_5 = 8.5$	check for
30				$3u_6$	$3u_6 = 29 - 20 \quad \therefore u_6 = 3$	UGO derived
33				$3u_7$	$3u_7 = 20 - 20 \quad \therefore u_7 = 0$	above

*Except the first column, all other columns are in cumec.

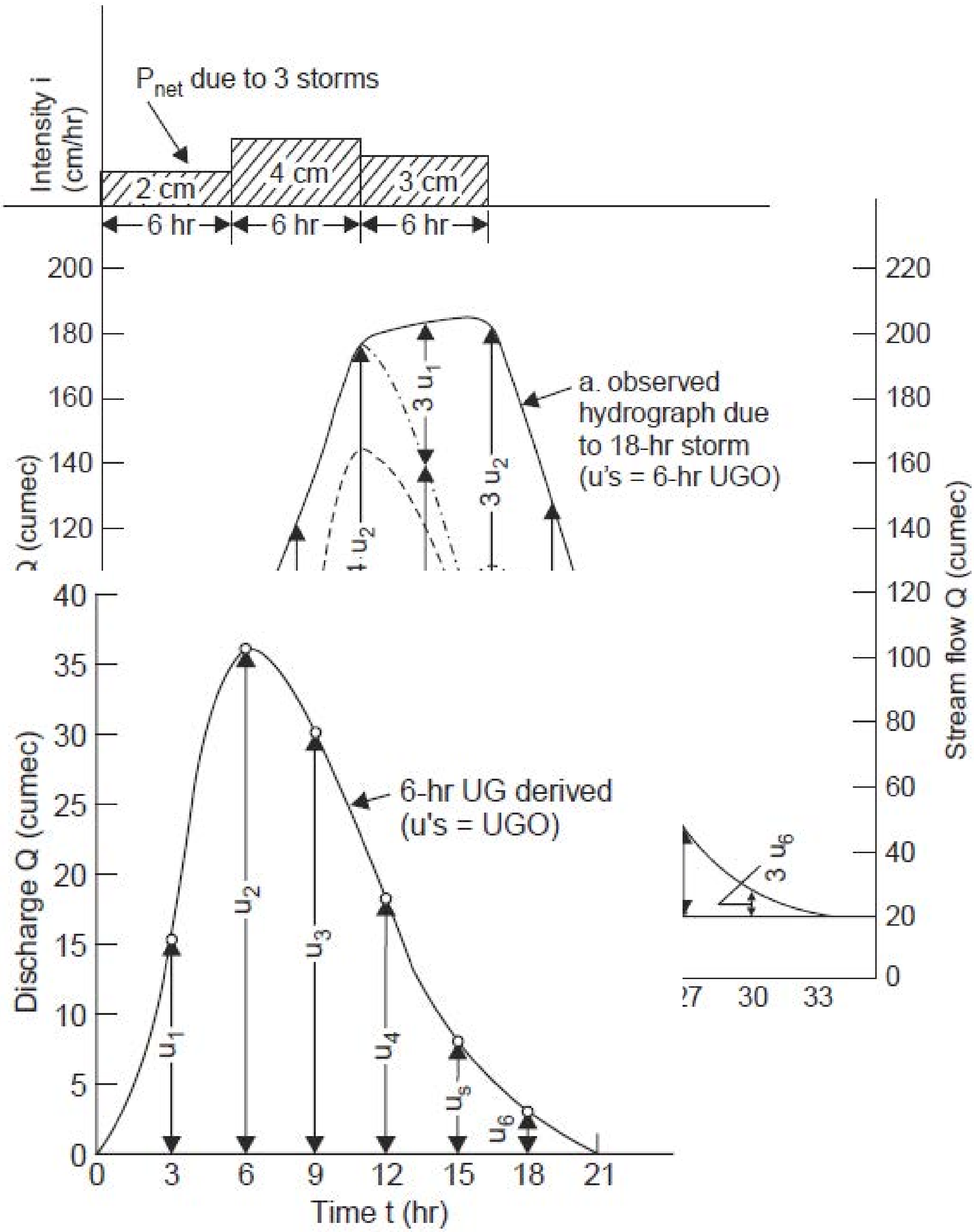
**$x = 2$ cm, $y = 4$ cm, $z = 3$ cm

The equations can be easily arrived by entering in a tabular column and successively solving them. The 6-hr unit hydrograph ordinates are obtained in the last column; of course the ordinates are at 3-hr intervals since the streamflows are recorded at 3-hr intervals. The last four equations in the table serve to check some of the UGO's derived. Another check for the UGO's derived is that the area under the UG should give a runoff volume equivalent to 1 cm, i.e.,

$$\frac{\sum ut}{A} = 1\,cm, \textit{in consistent units}$$

Σu = sum of the UGO's = 110 cumec

$$\frac{110\left(3 \times 60 \times 60\right)}{118.8 \times 10^{6}} = 0.01\,m$$

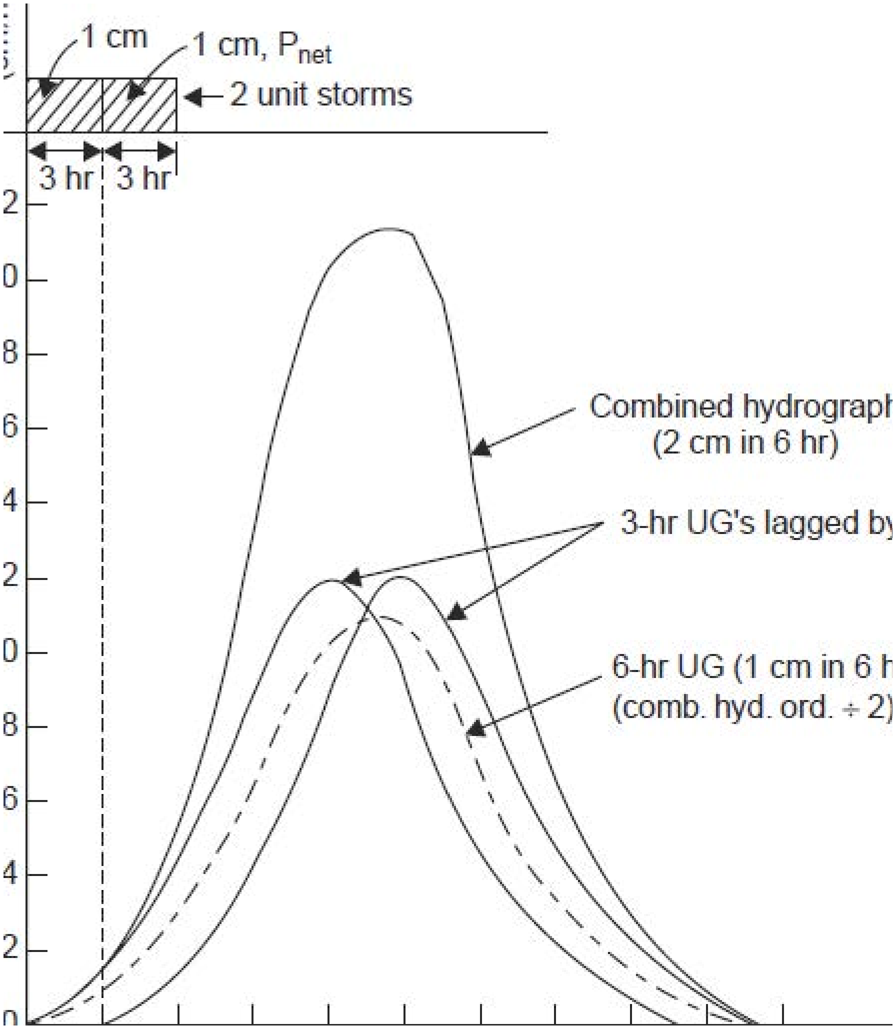


35. The following are the ordinates of a 3-hour unit hydrgraph. Derive the ordinates

of a 6-hour unit hydrograph and plot the same.

Time (hr)	3-hr UGO	Time (hr)	3-hr UGO (cumec)
0	0	15	9.4
3	1.5	18	4.6
6	4.5	21	2.3
9	8.6	24	0.8
12	12.0		

Time (hr)	3-hr UGO (cumec)	3-hr UGO (logged) (cumec)	Total 2 + 3 (cumec)	6-hr UGO 4 + 2 (cumec)
1	2	3	4	5
0	0		0	0
3	1.5	0	1.5	0.7
6	4.5	1.5	6.0	3.0
9	8.6	4.5	13.1	6.5
12	12.0	8.6	20.6	10.3
15	9.4	12.0	21.4	10.7
18	4.6	9.4	14.0	7.0
21	2.3	4.6	6.9	3.4
24	0.8	2.3	3.1	1.5
27		0.8	0.8	0.4



36. The ordinates of a 4-hour unit hydrograph for a particular basin are given below. Derive the ordinates of (i) the S-curve hydrograph, and (ii) the 2-hour unit hydrograph, and plot them, area of the basin is 630 km².

Time (hr)	Discharge (cumec)	Time (hr)	Discharge (cumec)
0	0	14	70
2	25	16	30
4	100	18	20
6	160	20	6
8	190	22	1.5
10	170	24	0
12	110		

Time (hr)	4-hr UGO (cumec)	S-curve additions (cumec) (unit storms after every 4 hr = t_r)					S-curve ordinates (cumec) (2) + (3)	lagged S-curve (cumec)	S-curve difference (cumec) (4) – (5)	2-hr UGO (6) × 4/3 (cumec)
1	2	3					4	5	6	7
0	0	—	—	—	—	—	0	—	0	0
2	25	—	—	—	—	—	25	0	25	50
4	100	0	—	—	—	—	100	25	75	150
6	160	25	—	—	—	—	185	100	85	170
8	190	100	0	—	—	—	290	185	105	210
10	170	160	25	—	—	—	335	290	65	130
12	110	190	100	0	—	—	400	355	45	90
14	70	170	160	25	—	—	425	400	25	50
16	30	110	190	100	0	—	430	425	5	10*
18	20	70	170	160	25	—	445	430	15	30
20	6	30	110	190	100	0	436	445	– 9	– 18*
22	1.5	20	70	170	160	25	446.5	436	10.5	21
24	0	6	30	110	190	100	436	446.6	– 10.5	– 21*

*Slight adjustment is required to the tail of the 2-hour unit hydrograph.

Col (5): lagged S-curve is the same as col (4) but lagged by $t_r' = 2$ hr.

Col (7): col (6) × $\frac{t_r}{t_r'}$, $t_r = 4$ hr. $t_r' = 2$ hr.

Col (3): No. of unit storms in succession = $T/t_r = 24/4 = 6$, to produce a constant outflow.

$Q_e = \frac{2.78\,A}{t_r} = \frac{2.78 \times 630}{4} = 437$ cumec, which agrees very well with the tabulated *S*-curve terminal value of 436.

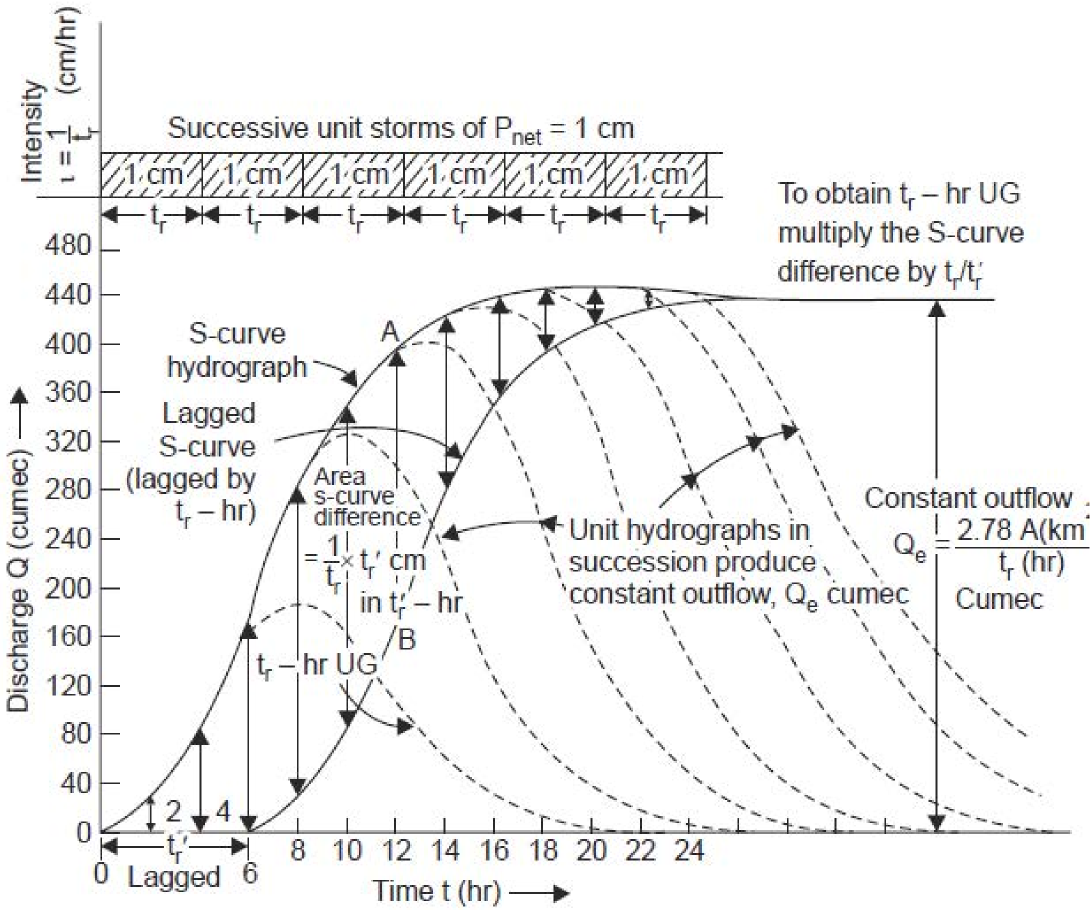
37. The ordinates of a 4-hour unit hydrograph for a particular basin are given below. Determine the ordinates of the S-curve hydrograph and therefrom the ordinates of the 6-hour unit hydrograph.

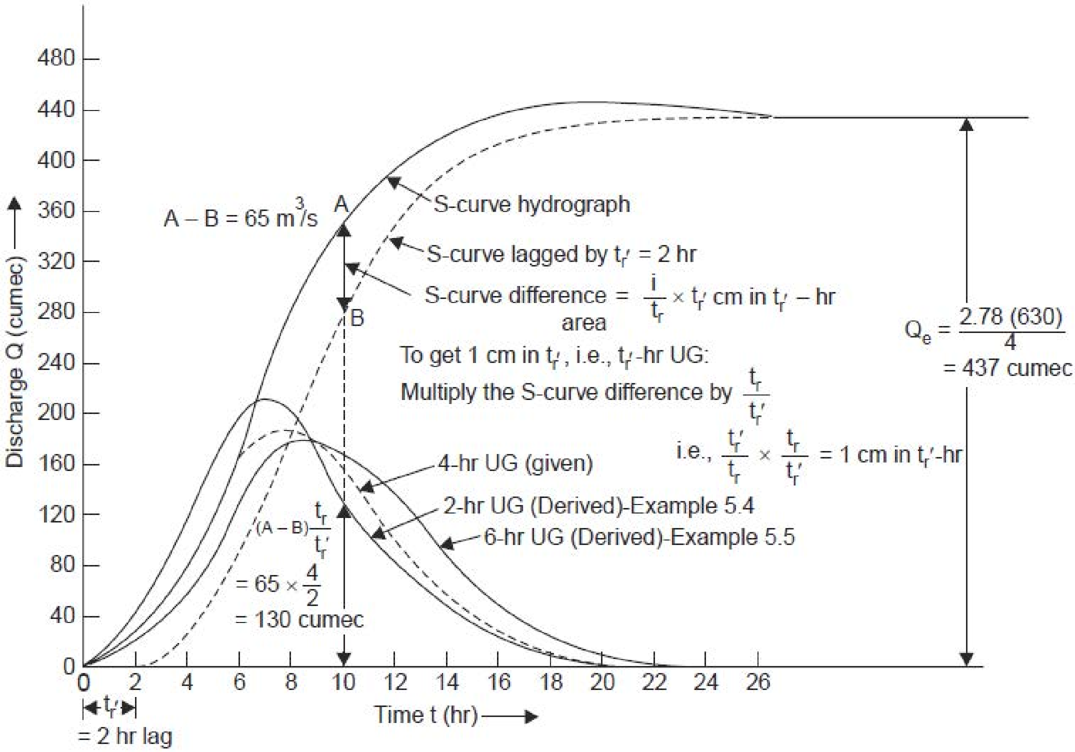
Time (hr)	4-hr UGO (cumec)	Time (hr)	4-hr UGO (cumec)
0	0	12	110
2	25	14	70
4	100	16	30
6	160	18	20
8	190	20	6
10	170	22	1.5
		24	0

Time (hr)	*4-hour UGO (cumec)*	*S-curve[1] additions (cumec)*	*S-curve ordinates (cumec) (2) + (3)*	*lagged[2] C-curve (cumec)*	*S-curve difference (cumec) (4) – (5)*	*6-hr UGO (cumec) (6) × 4/6*
1	*2*	*3*	*4*	*5*	*6*	*7*
0	0 →	—	0	—	0	0
2	25 →	—	25	—	25	16.7
4	100 + →	0	100	—	100	66.7
6	160 + →	25	185	0	185	123.3
8	190	100	290	25	265	176.7
10	170	185	355	100	255	170.0
12	110	290	400	185	215	143.3
14	70	355	425	290	135	90.0
16	30	400	430	355	75	50.0
18	20	425	445	400	45	30.0
20	6	430	436	425	11	7.3
22	1.5	445	446.6	430	16.5	11.0
24	0	436	436	445	– 9	– 6.0

1—Start the operation shown with 0 cumec after $t_r = 4$ hr.

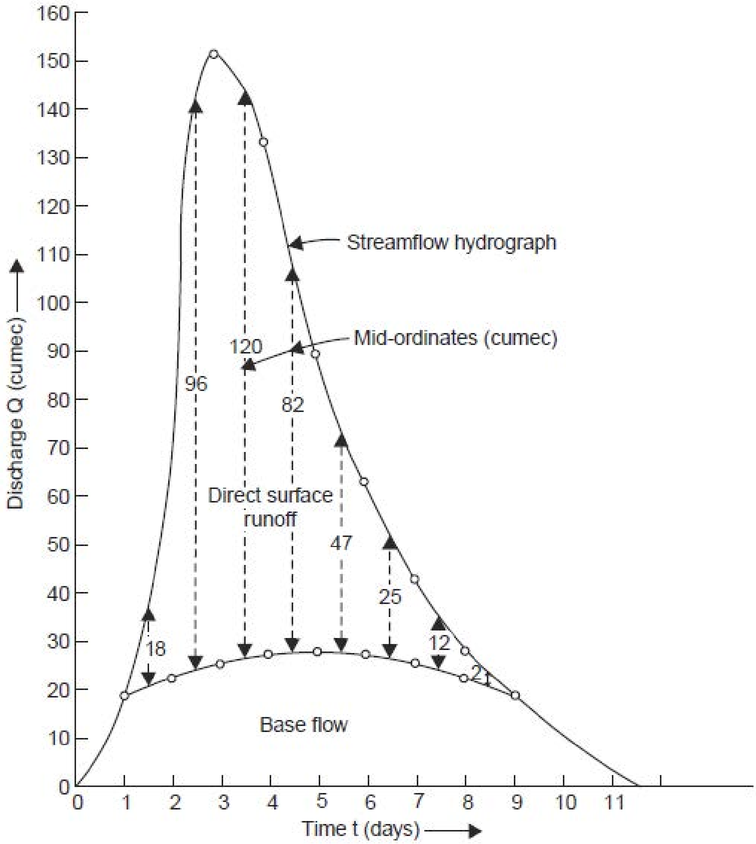
2—Lag the S–curve ordinates by $t_r' = 6$ hr.


Successive unit storms of P_{net} = 1 cm
1 cm
Intensity $\iota = \frac{1}{t_r}$ (cm/hr)
To obtain t_r – hr UG multiply the S-curve difference by t_r/t_r'
S-curve hydrograph
Lagged S-curve (lagged by t_r – hr)
Area s-curve difference $= \frac{1}{t_r} \times t_r'$ cm in t_r' – hr
A
B
t_r – hr UG
Unit hydrographs in succession produce constant outflow, Q_e cumec
Constant outflow
$Q_e = \frac{2.78\ A(km}{t_r\ (hr)}$
Cumec
Discharge Q (cumec)
Time t (hr)
Lagged
t_r'



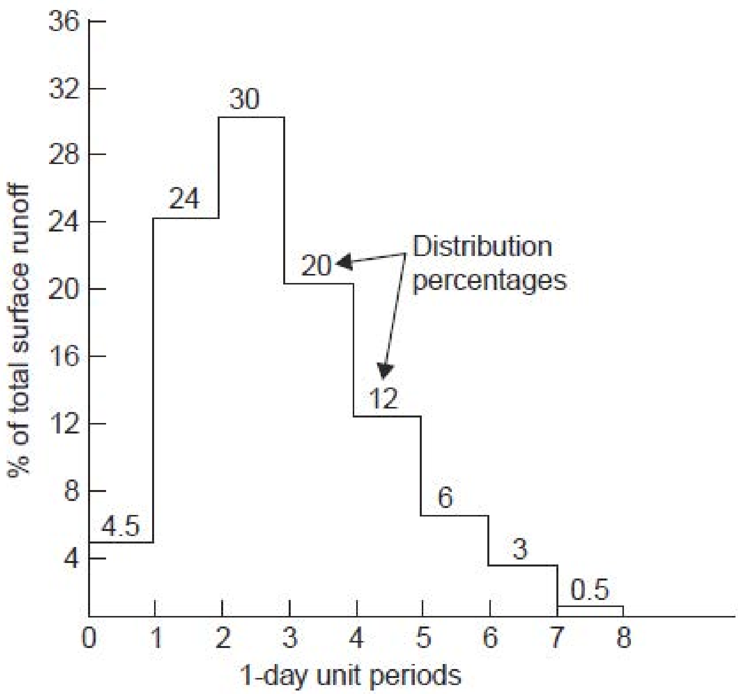
38. Analysis of the runoff records for a one day unit storm over a basin yields the following data:
Total stream flow at concentration point on successive days are 19.6, 62.4, 151.3, 133.0, 89.5, 63.1, 43.5, 28.6, and 19.6 cumec.
Estimated base flow during the corresponding period on successive days are 19.6, 22.4, 25.3, 28.0, 28.0, 27.5, 25.6, 22.5 and 19.6 cumec.
Determine the distribution graph percentages.
On the same basin (area = 2850 km²) there was rainfall of 7 cm/day on July 15 and 10 cm/day on July 18 of a certain year. Assuming an average storm loss of 2 cm/day, estimate the value of peak surface runoff in cumec and the date of its occurrence.

The total runoff hydrograph and estimated base flow are drawn in the figure and the direct runoff ordiantes on successive mid-days are determined as DRO = TRO – BFO and the percentages of direct runoff on successive days computed in the table Column (3) gives the distribution percentages, and the derived distribution graph for 1-day unit storms is shown in the figure.



Discharge Q (cumec)
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
Streamflow hydrograph
Mid-ordinates (cumec)
120
96
82
Direct surface runoff
47
25
12
18
2
Base flow
0
1
2
3
4
5
6
7
8
9
10
11
Time t (days)

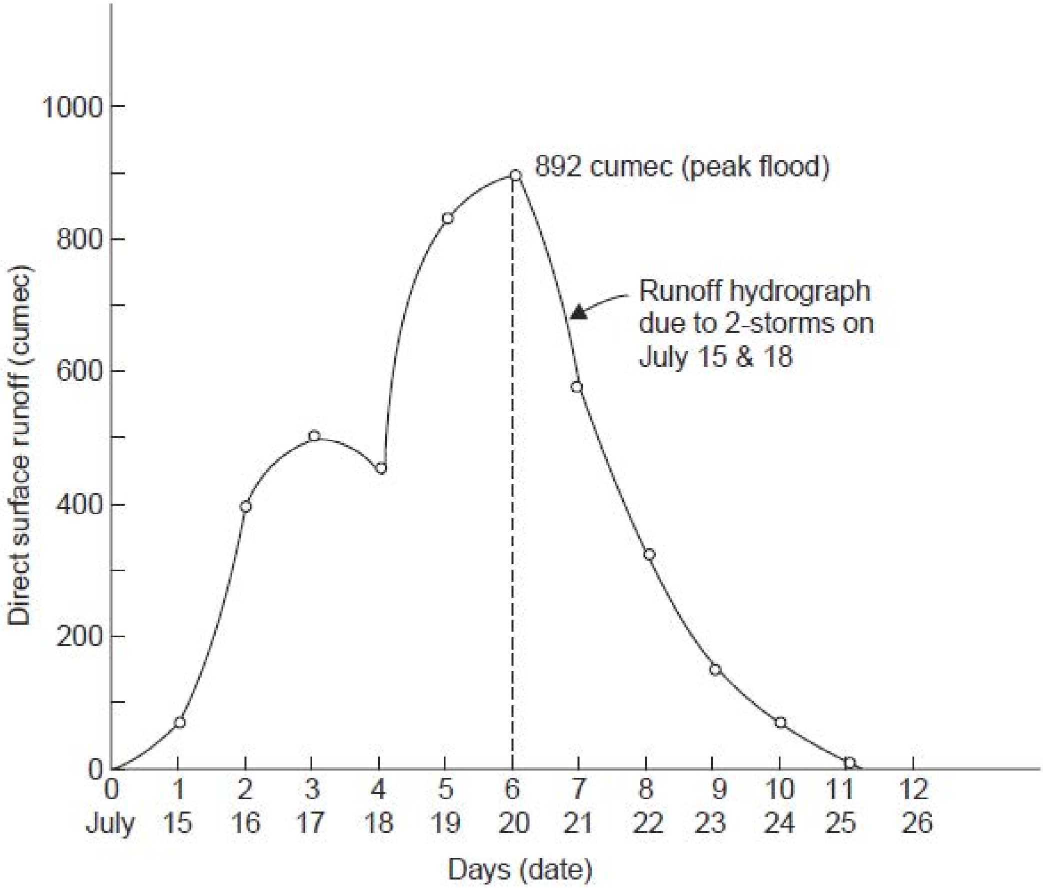
Day since beginning of direct runoff	*DRO on mid-day (cumec)*	*Percentage of ΣDRO*	*Remarks*
1	*2*	*3*	*4*
1	18	4.5	$= \frac{18}{402} \times 100$
2	96	24	
3	120	30	
4	82	20	
5	47	12	
6	25	6	
7	12	3	
8	2	0.5	
8 equal time (*i.e.*, a day) intervals	Σ *DRO* = 402	Total = 100.0	



Applying the distribution percentages computed in col. (3) above the direct surface discharge on successive days due to the two storms (lagged by 3 days) is computed in the table.

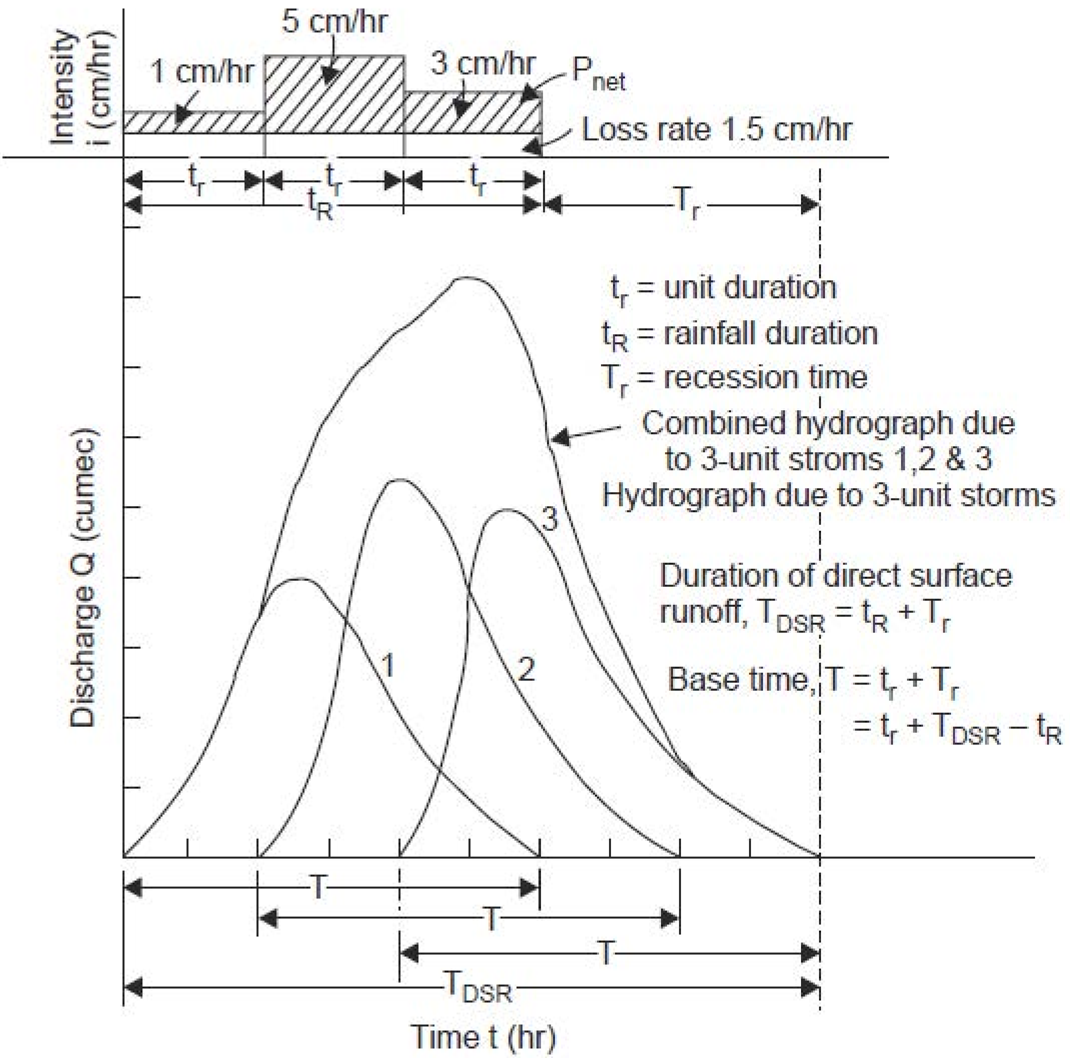
		Unit periods (day)										
		1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*
Total rainfall (cm) →		7			10							
Loss of rain (cm) →		2			2							
Net rain (cm) →		5			8	(= 13 cm)						
Unit distribution					Distribution (cm/day)							
Periods	Percentages											
1	4.5	0.225			0.36							
2	24		1.20			1.92						
3	30			1.50			2.40					
4	20				1.00			1.60				
5	12					0.60			0.96			
6	6						0.30			0.48		
7	3							0.15			0.24	
8	0.5								0.025			0.04
Total (cm/day):		0.225	1.20	1.50	1.36	2.52	2.70	1.75	0.985	0.48	0.24	0.04 = 13 cm (check)
× 330 = cumec:		74	396	495	450	833	892	578	325	158	79	13 (0.225 × 330) = 74 cumec)
Date: July		15	16	17	18	19	20	21	22	23	24	25

The peak surface runoff is 892 cumec and occurs on July 20 of the year. The flood hydrograph is shown in the figure.



39. Analysis of rainfall and runoff records for a certain storm over a basin (of area 3210 km²) gave the following data:
Rainfall for successive 2 hr periods: 2.5, 6.5 and 4.5 cm/hr.
An average loss of 1.5 cm/hr can be assumed.
Direct surface discharge at the concentration point for successive 2-hr periods: 446, 4015, 1382, 25000, 20520, 10260, 4900 and 1338 cumec.
Derive the unit hydrograph in the form of distribution percentages on the basis 2-hr unit periods.

The rainfall may be considered for three unit periods of 2 hr each, then from the figure,



$T_{DSR} = t_R + T_r$

$T = t_r + T_r$

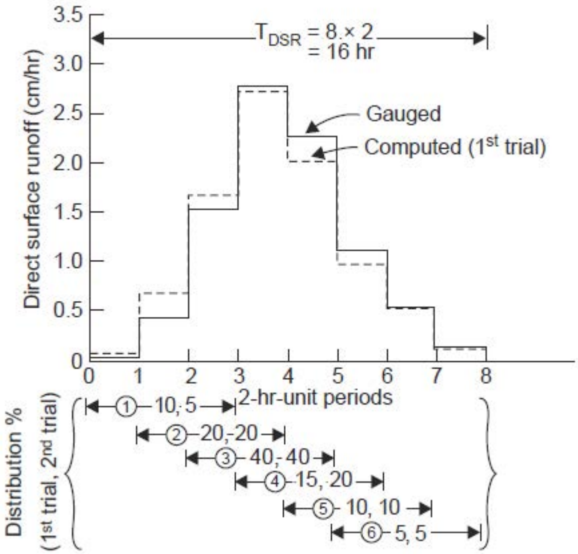
$T = t_r + T_{DSR} - t_R = 2 + 8 \times 2 - 3 \times 2 = 12$ hr

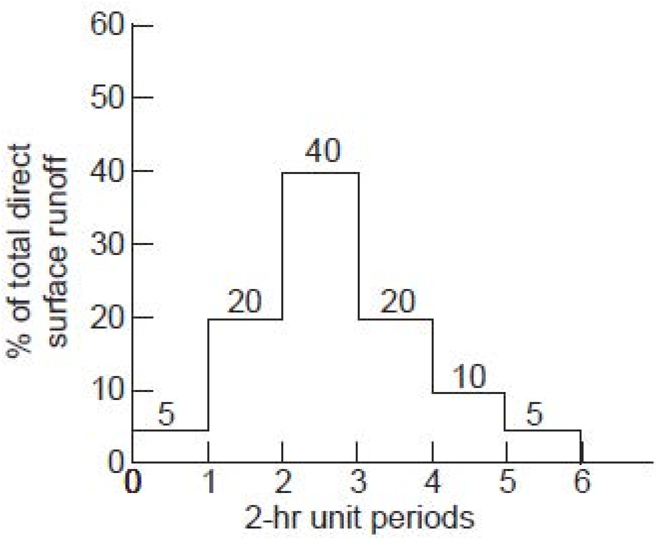
The base width is 12 hr or 6 unit periods. As a first trial, try a set of six distribution percentages of 10, 20, 40, 15, 10, 5 which total 100%. The direct surface discharge can be converted into cm/hr as

		2-hr-unit periods								
		1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	
Rainfall rate (cm/hr) →		2.5	6.5	4.5						
Loss rate (cm/hr) →		1.5	1.5	1.5						
Net rain (cm/hr) →		1	5	3 (= 9 × 2 cm)						
Unit distribution				Distribution (cm/hr)						
Periods	Percentage									
1	10 (5)	(0.05)	(0.25)	(0.15)						
		0.10	0.50	0.30						
2	20		0.20	1.00	0.60					
3	40			0.40	2.00	1.20				
4	15 (20)				(0.20)	(1.00)	(0.60)			
					0.15	0.75	0.45			
5	10					0.10	0.50	0.30		
6	5						0.05	0.25	0.15	
Total 100 (100) cm/hr		0.10	0.70	1.70	2.75	2.05	1.00	0.55	0.15	(= 9 × 2 cm check)
		(0.05)	(0.45)	(1.55)	(2.80)	(2.30)	(1.15)			
cumec		446	4015	1382	25000	20520	10260	4900	1338	(446/8920 = 0.05 cm/hr)

Note Figures in brackets indicate the adjusted values in the second trial.

and the direct surface runoff for successive 2-hr periods are 0.05, 0.45, 1.55, 2.80, 2.30, 1.15, 0.55, and 0.15 cm/hr. The first trial hydrograph computed in the table is shown by dashed lines in the figure for comparison and selection of the distribution percentages for the second trial. The first percentage affects the first 3 unit periods, the second percentage affects the 2nd, 3rd and 4th unit periods and like that. Since the first trial hydrograph gives higher values (than gauged) for the first three unit periods, a lower percentage of 5 (instead of 10%) is tried. Similarly, the other percentages are adjusted till the computed discharge values agree with the gauged values. Thus, the second trial distribution percentages are 5, 20, 40, 20, 10, 5 which total 100 and are final and the distribution graph thus derived is shown in the figure. In most cases, more trials are required to obtain the desired degree of accuracy.





40. The following are the ordinates of the 9-hour unit hydrograph for the entire catchment of the river Damodar up to Tenughat dam site:

Time (hr)	0	9	18	27	36	45	54	63	72	81	90
Discharge (cumec)	0	69	1000	210	118	74	46	26	13	4	0

and the catchment characteristics are

A = 4480 km², L = 318 km, L_{ca} = 198 km

Derive a 3-hour unit hydrograph for the catchment area of river Damodar up to the head of Tenughat reservoir, given the catchment characteristics as:

A = 3780 km², L = 284 km, L_{ca} = 184 km

Use Snyder's approach with necessary modifications for the shape of the hydrograph.

The 9-hr UG is plotted in Fig. 5.30 and from that t_p = 13.5 hr

$$t_r = 9\,hr, \frac{t_p}{5.5} = \frac{13.5}{5.5} = 2.46\,hr \neq t_r\ of\ 9\,hr$$

tr′ = 9 hr, t_{pr} = 13.5 hr and t_p has to be determined

$$t_{pr} = t_p + \frac{t_r^{'} - t_r}{4}$$

$$13.5 = t_p + \frac{9 - t_p / 5.5}{4} \rightarrow t_p = 11.8\,hr$$

$t_p = C_t\,(LL_{ca})^{0.3}$

$11.8 = C_t\,(318 \times 198)^{0.3}$

$C_t = 0.43$

$$Peak\ flow, Q_p = C_p \frac{A}{t_{pr}}$$

$$1000 = C_p \frac{4480}{`3.5} \rightarrow C_p = 3$$

The constants of C_t = 0.43 and C_p = 3 can now be applied for the catchment area up to the head of the Tenughat reservoir, which is meteorologically and hydrologically similar.

$t_p = C_t\,(LL_{ca})^{0.3} = 0.43\,(284 \times 184)^{0.3} = 11.24$ hr

$$\frac{t_p}{5.5} = \frac{11.24}{5.5} = 2.04\,hr \neq t_r\ of\ 3\,hr\ (duration\ of\ the\ required\ UG)$$

t_r' = 3 hr, t_r = 2.04 hr and t_{pr} has to be determined.

$$t_{pr} = t_p + \frac{t_r^{'} - t_r}{4} = 11.24 + \frac{3 - 2.04}{4} = 11.5\,hr$$

$$Peak\ flow, Q_p = C_p \frac{A}{t_{pr}} = 3 \times \frac{3780}{11.5} = 987\ cumec$$

Time to peak from the beginning of rising limb

$$t_{peak} = t_{pr} + \frac{t_r'}{2} = 11.5 + \frac{3}{2} = 13\ hr$$

$$Time\ base\ (Snyder's)\ T\ (days) = 3 + 3\left(\frac{t_{pr}}{24}\right) = 3 + 3\left(\frac{11.5}{24}\right) = 4.44\ days$$

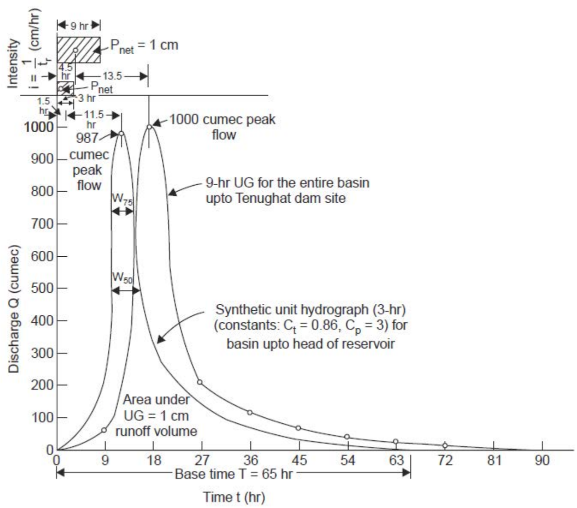
This is too long a runoff duration and hence to be modified as
$T(hr) = 5 \times t_{peak} = 5 \times 13 = 65$ hr
To obtain the widths of the 3-hr UG at 50% and 75% of the peak ordinate :

$$q_p = \frac{Q_p}{A} = \frac{987}{3780} = 0.261\ cumec / km^2$$

$$W_{50} = \frac{5.6}{(q_p)^{1.08}} = \frac{5.6}{(0.261)^{1.08}} = 23.8\ hr$$

$$W_{75} = \frac{3.21}{(q_p)^{1.08}} = \frac{3.21}{(0.261)^{1.08}} = 13.6\ hr$$

These widths also seem to be too long and a 3-hr UG can now be sketched using the parameters Q_p = 987 cumec, t_{peak} = 13 hr and T = 65 hr such that the area under the UG is equal to a runoff volume of 1 cm, as shown in the figure.



41. Construct a 4-hr UH for a drainage basin of 200 km2 and lag time 10 hr by the SCS method, given (pk = peak):

$\frac{t}{t_{pk}}$:	0.5	1	2	3	4	5
$\frac{Q}{Q_p}$:	0.4	1	0.32	0.075	0.018	0.004

$$t_{pk} = \frac{t_r}{2} + t_p = \frac{4}{2} + 10 = 12\,hr$$

$$(i)\,Q_p = \frac{5.36A}{t_{pk}} = \frac{5.36 \times 200}{12} = 89.33\,cumec, \text{ which occurs at } \frac{t}{t_{pk}} = 1 \text{ or } t = t_{pk} = 12\,hr$$

$$(ii)\,At\,\frac{t}{t_{pk}} = 0.5 \text{ or } t = 0.5 \times 12 = 6\,hr, \frac{Q}{Q_p} = 0.4 \text{ or } Q = 0.4 \times 89.33 = 35.732\,cumec$$

$$(iii)\,At\,\frac{t}{t_{pk}} = 2 \text{ or } t = 2 \times 12 = 24\,hr, \frac{Q}{Q_p} = 0.32 \text{ or } Q = 0.32 \times 89.33 = 28.6\,cumec$$

$$(iv)\,At\,\frac{t}{t_{pk}} = 3 \text{ or } t = 3 \times 12 = 36\,hr, \frac{Q}{Q_p} = 0.075 \text{ or } Q = 0.075 \times 89.33 = 6.7\,cumec$$

Time base T = 5 t_{pk} = 5 × 12 = 60 hr; $W_{75} = W_{50}/1.75$
With this, a 4-hr UH can be sketched.

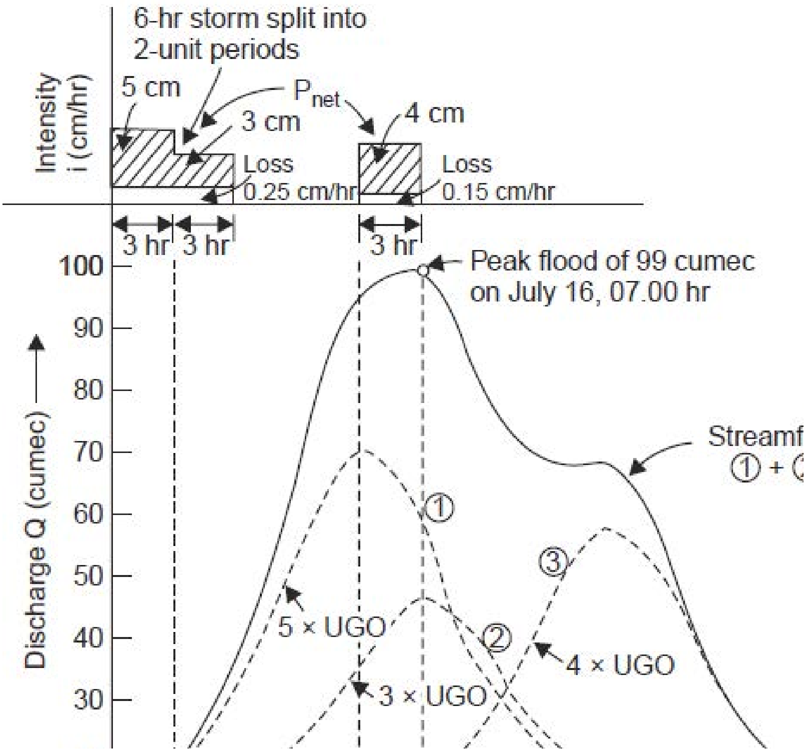
42. The 3-hr unit hydrograph ordinates for a basin are given below. There was a storm, which commenced on July 15 at 16.00 hr and continued up to 22.00 hr, which was followed by another storm on July 16 at 4.00 hr which lasted up to 7.00 hr. It was noted from the mass curves of self-recording raingauge that the amount of rainfall on July 15 was 5.75 cm from 16.00 to 19.00 hr and 3.75 cm from 19.00 to 22.00 hr, and on July 16, 4.45 cm from 4.00 to 7.00 hr. Assuming an average loss of 0.25 cm/hr and 0.15 cm/hr for the two storms, respectively, and a constant base flow of 10 cumec, determine the stream flow hydrograph and state the time of occurrence of peak flood.

Time (hr)	0	3	6	9	12	15	18	21	24	27
UGO (cumec)	0	1.5	4.5	8.6	12.0	9.4	4.6	2.3	0.8	0

Since the duration of the UG is 3 hr, the 6-hr storm (16.00 to 22.00 hr) can be considered as 2-unit storm producing a net rain of 5.75 – 0.25 × 3 = 5 cm in the first 3-hr period and a net rain of 3.75 – 0.25 × 3 = 3 cm in the next 3-hr period. The unit hydrograph ordinates are multiplied by the net rain of each period lagged by 3 hr. Similarly, another unit storm lagged by 12 hr (4.00 to 7.00 hr next day) produces a net rain of 4.45 – 0.15 × 3 = 4 cm which is multiplied by the UGO and written in col (5) (lagged by 12 hr from the beginning), the table. The rainfall excesses due to the three storms are added up to get the total direct surface discharge ordinates. To this, the base flow ordinates (BFO = 10 cumec, constant) are added to get the total discharge ordinates (stream flow).

The flood hydrograph due to the 3 unit storms on the basin is obtained by plotting col (8) vs. col. (1).

Time (hr)	UGO*	DRO due to rainfall excess			total	BFO	TRO	Remarks
		I	II	III	DRO			
		UGO X 5 cm	UGO x 3cm	UGO x 4cm	3 + 4 + 5	constant	6 + 7	
1	2	3	4	5	6	7	8	
0	0	0	-	-	0	10	10.0	July 15, 16 hr commencement of food
3	1.5	7.5	0	-	7.5	10	17.5	
6	4.5	22.5	4.5	-	27.0	10	37.0	
9	8.6	43.0	13.5	-	56.5	10	66.5	
12	12.0	60.0	25.8	0	85.8	10	95.8	
15	9.4	47.0	36.0	6	89.0	10	99.0	Peak flood on july 16, 07.00 hr
18	4.6	23.0	28.2	18	69.2	10	79.2	
21	2.3	11.5	13.8	34.4	59.7	10	69.7	
24	0.8	4.0	6.9	48	58.9	10	68.9	
27	0	0	2.4	37.6	40.0	10	50.0	
30			0	18.4	18.4	10	28.4	
33				9.2	9.2	10	19.2	
36				3.2	3.2	10	13.2	
39				0	0	10	10.0	Flood subsides on july 17 , 07.00 hr



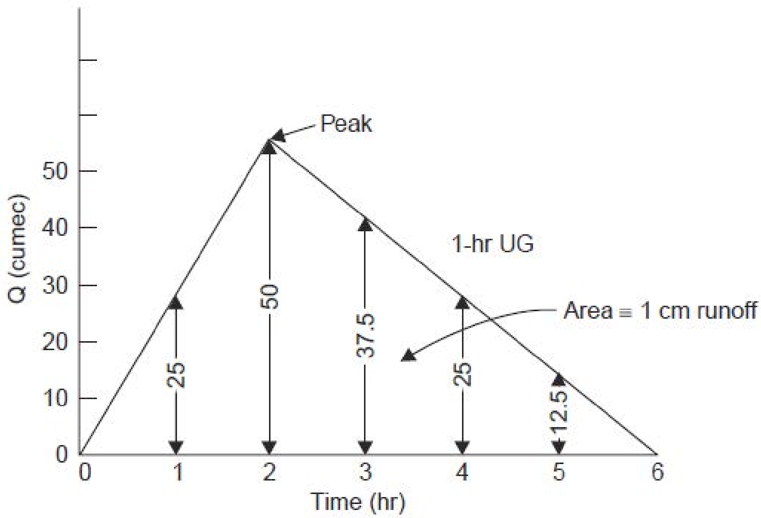
43. The design storm of water shed has the depths of rainfall of 4.9 and 3.9 cm for the consecutive 1-hr periods. The 1-hr UG can be approximated by a triangle of base 6

hr with a peak of 50 cumec occurring after 2 hr from the beginning. Compute the flood hydrograph assuming an average loss rate of 9 mm/hr and constant base flow of 10 cumec. What are the areas of water shed and its coefficient of runoff?

(i) The flood hydrograph due to the two consecutive hourly storms is computed in the table and the figure.

Time hr	UGO* cumec	DRO due to rain-fall excess cumec		Total cumec	BF cumec	TRO cumec	Remarks
		4.9-0.9 =4cm	3.9-0.9= 3cm				
1	0	0	-	0	10	10	
2	25	100	0	100	10	110	
3	50	200	75	275	10	285	
4	37.5	150	150	300	10	310	←peak flood+
5	25	100	112.5	212.5	10	222.5	
6	12.5	50	75	125	10	135	
7	0	0	37.5	37.5	10	47.5	
8	-	-	0	0	10	10	

*ordinates by proportion in the triangular UG. + Peak flood of 310 cumec, after 4hr from the commencement of the storm.



(ii) Area of water shed—To produce 1-cm net rain over the entire water shed (A km^2). Volume of water over basin = Area of UG (triangle)

$$\left(A\times10^{6}\right)\frac{1}{100}=\frac{1}{2}\left(6\times60\times60\right)50$$

from which, A = 54 km^2

$$(iii)\ \textit{Coefficient of runoff}\ C=\frac{R}{P}=\frac{(4.9-0.9)+(3.9-0.9)}{4.9+3.9}=0.795$$

44. Storm rainfalls of 3.2, 8.2 and 5.2 cm occur during three successive hours over an area of 45 km². The storm loss rate is 1.2 cm/hr. The distribution percentages of successive hours are 5, 20, 40, 20, 10 and 5. Determine the streamflows for successive hours assuming a constant base flow of 10 cumec. State the peak flow and when it is expected; the precipitation started at 04.00 hr, on June 4, 1982.

The computation of stream flow hydrograph from the distribution percentages due to net rainfall in three successive hours (i.e., from a complex storm) over an area of 45 km² is made in the table.

ne -)	*Distribution percentages*	*Rainfall excess (cm)*	*DRO due to rainfall excess (cm)*			*Total DRO*		*BF (cumec)*	*Stream flow (cumec)*	*Remarks*
		P-loss = P_{net}	*2*	*7*	*4*	*(cm)*	*(cumec)*			
	5	3.2 – 1.2 = 2	0.10	—	—	0.10	12.5*	10	22.5 ←	June 4, 1982 04.00 hr
	20	8.2 – 1.2 = 7	0.40	0.35	—	0.75	93.75	10	103.75	Commencement of flood
	40	5.3 – 1.2 = 4	0.80	1.40	0.20	2.40	300	10	310	
	20		0.40	2.80	0.80	4.00	500	10	510 ←	Peak flood at 07.00 hr on June 4, 1982
	10		0.20	1.40	1.60	3.20	400	10	410	
	5		0.10	0.70	0.80	1.60	200	10	210	
	—		—	0.35	0.40	0.75	93.75	10	103.75	
	—		—	—	0.20	0.20	25	10	35	
al	100	13	2.00	7.00	4.00	13.00	1625			

$$* \frac{0.10}{100} \frac{(45 \times 10^6)}{1 \times 60 \times 60} = 12.5 \text{ cumec.}$$

45. The successive three-hourly ordinates of a 6-hr UG for a particular basin are 0, 15, 36, 30, 17.5, 8.5, 3, 0 cumec, respectively. The flood peak observed due to a 6-hr storm was 150 cumec. Assuming a costant base flow of 6 cumec and an average storm loss of 6 mm/hr, determine the depth of storm rainfall and the streamflow at successive 3 hr interval.

DRO peak = Flood peak – BF= 150 – 6 = 144 cumec

$$P_{net} = \frac{DRO_{peak}}{UG_{peak}} = \frac{144}{36} = 4\,cm$$

Depth of storm rainfall,

$P = P_{net} + \text{losses} = 4 + 0.6 \times 6 = 7.6$ cm.

DRO = UGO × P_{net}; DRO + BF = TRO

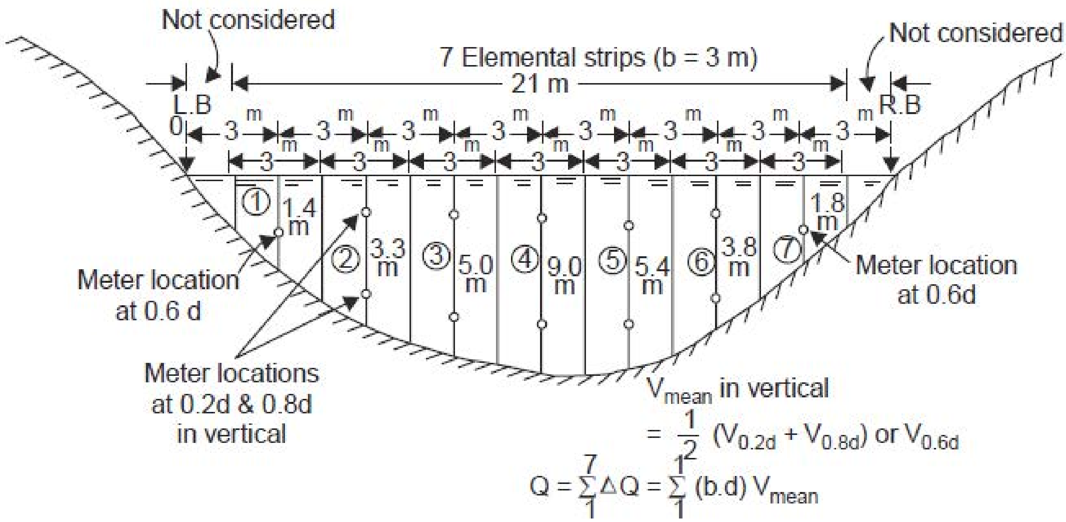
Hence, multiplying the given UGO by 4 cm and adding 6 cumec, the stream flow ordinates at successive 3-hr intervals are: 6, 66, 150, 126, 76, 40, 18, 6 cumec, respectively.

46. The following data were collected for a stream at a gauging station. Compute the discharge.

Distance from one end of water surface ((m)	Depth, d (m)	Immersion of current meter below water surface					
		At 0.6d		At 0.2d		At 0.8d	
		rev	sec	Rev.	Sec.	Rev.	Sec.
3	1.4	12	50				
6	3.3			38	52	23	55
9	5.0			40	58	30	54
12	9.0			48	60	34	58
15	5.4			34	52	30	50
18	3.8			35	52	30	54
21	1.8	18	50				

Rating equation of current meter: v = 0.3 N + 0.05, N = rps, v = velocity, (m/sec), Rev.-Revolutions, Sec-time in seconds.

The discharge in each strip, $\Delta Q = (bd)\, V$, where V is the average velocity in each strip. In the first and the last strips (near the banks) where the depth is shallow, $V = v_{0.6d}$, and in the other five intermediate strips (with deep water), $V=(v_{0.2d}+v_{0.8d})/2$. Width of each strip, b = 3 m, mean depth of strip = d, and the total discharge, $Q = \Sigma\, \Delta Q = 20.6$ cumec, as computed in the table.



Distance from one end of water surface (m)	Depth, d (m)	Immersion of current meter below water surface					Average velocity in strip V (m/sec)	Discharge in strip ΔQ = (bd) V b = 3m
		depth = xd (x = 0.6, 0.2, 0.8) (m)	Rev. R	time (sec.)	N = R/t (rps)	v = 0.3 N + 0.05 (m/sec)		
3	1.4	0.84	12	50	0.24	0.122	0.122	0.51
6	3.3	0.66	38	52	0.73	0.269	0.223*	2.16
		2.64	23	55	0.42	0.176		
9	5.0	1.00	40	58	0.69	0.257	0.236	3.54
		4.00	30	54	0.56	0.218		
12	9.0	1.80	48	60	0.80	0.290	0.259	7.00
		7.20	34	58	0.59	0.227		
15	5.4	1.08	34	52	0.65	0.245	0.238	3.85
		4.32	30	50	0.60	0.230		
18	3.8	0.76	35	52	0.67	0.251	0.234	2.68
		3.04	30	54	0.56	0.218		
21	1.8	1.08	18	50	0.36	0.158	0.158	0.86
							Total Q = 20.60 cumec	

* $\frac{0.269 + 0.176}{2} = 0.223.$

47. The stream discharges for various stages at a particular section were observed to be as follows. Obtain an equation for the stage-discharge relationship and determine the discharge for a stage of 4.9 m and 12 m.

Stage (m)	1.81	1.81	2.00	2.90	3.70	4.50
Discharge (cumec)	1.00	1.50	2.55	5.60	11.70	20.20
Stage (m)	5.40	6.10	7.30	7.70	8.10	
Discharge (cuemc)	32.50	44.50	70.0	80.0	90.0	

The relation between the stage (h) and discharge (Q) of the stream can be assumed of the form

$Q = K (h - a)^n$

where K, a and n are the constants. Plot Q vs. (h – a) on a lag-lag paper assuming a value for the constant a = 0.6 m (say); the curve obtained is concave downwards. Now assume a value a = 1.2 m (say) and the curve obtained is concave upward. Now try an intermediate value a = 0.9 m, which plots a straight line and represents the stage discharge relationship. The slope of this straight line gives the value of the exponent n = 2.2, and from the graph for h – a = 1, Q = 1.2 = K. Now the constants are determined and the equation for the stage discharge relationship is $Q = 1.2 (h - 0.9)^{2.2}$

It may be noted that the value of a = 0.9, which gives a straight line plot is the gauge reading for zero discharge. Now the abscissa of (h – a) may be replaced by the gauge reading (stage) h, by adding the value of 'a' to (h – a) values. For example the (h – a) values of 0.1, 1, 2, 4, 6, 8 and 10 may be replaced by the h values of 1, 1.9, 2.9, 4.9, 6.9, 8.9 and 10.9 respectively. Now for any gauge reading (stage) h, the discharge Q can be directly read from the graph and the stage discharge curve can be extended.

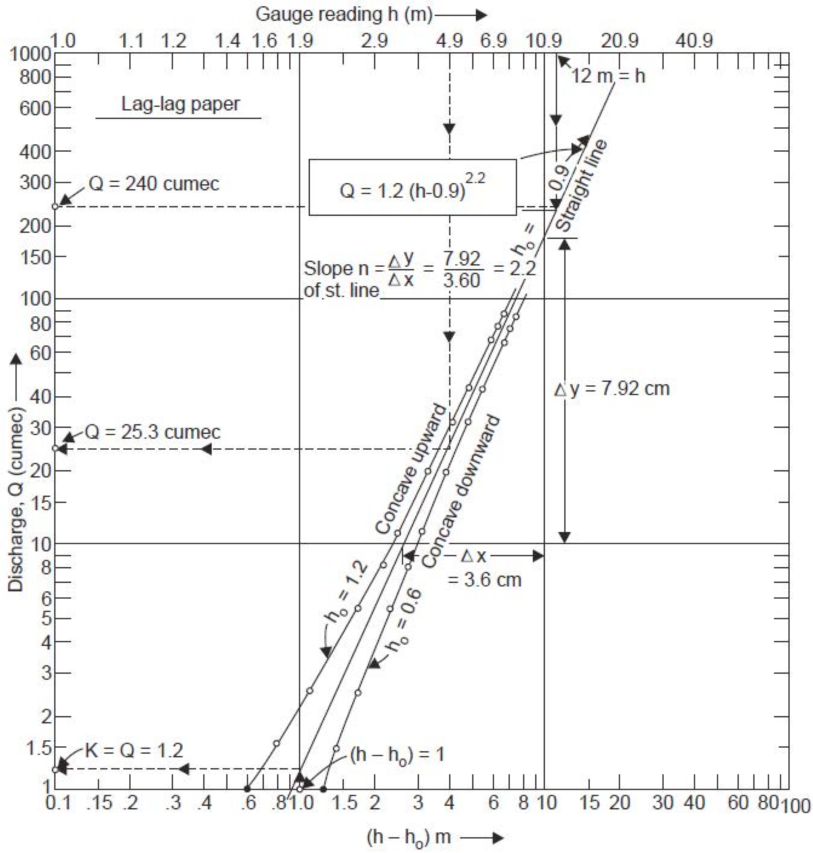
for h = 4.9 m, Q = 25.3 cumec

and for h = 12.0 m, Q = 240 cumec

which can be verified by the stage-discharge equation obtained as

for h = 4.9 m, $Q = 1.2 (4.9 - 0.9)^{2.2} = 25.3$ cumec

for h = 12 m, $Q = 1.2 (12 - 0.9)^{2.2} = 240$ cumec



48. The following data were obtained by stream gauging of a river:

Main gauge staff reading (m)	12.00	12.00
Auxiliary gauge staff reading (m)	11.65	11.02
Discharge (cumec)	9.50	15.20

what should be the discharge when the main gauge reads 12 m and the auxiliary gauge reads 11.37 m?

$\Delta h_0 = 12.00 - 11.65 = 0.35$ m

$\Delta h_a = 12.00 - 11.02 = 0.98$ m

$$\frac{Q_a}{Q_0} = \left(\frac{\Delta h_a}{\Delta h_0}\right)^n$$

$$\frac{15.20}{9.50} = \left(\frac{0.98}{0.35}\right)^n \rightarrow n = 0.5125$$

Again, when the auxiliary gauge reads 11.37 m,

Δh_a = 12.00 – 11.37 = 0.63 m

$$\frac{Q_a}{9.50} = \left(\frac{0.63}{0.35}\right)^{0.5125} \rightarrow Q_a = 12.85\, cumec$$

49. bridge has to be constructed over a river, which receives flow from three branches above the site. Compute the maximum flood discharge at the bridge site from the following data:

Branch 1 has a bridge:

Width of natural water way 324.0 m

Lineal water way under the bridge

(with Cd = 0.95 for rounded entry) 262.5 m

Depth upstream of bridge 4.6 m

Depth downstream of bridge 2.8 m

Branch 2 has a catchment area of 4125 km2

Ryve's C = 10

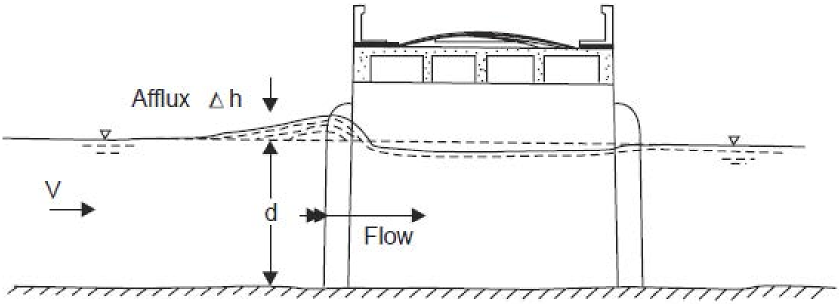
Branch 3 leveling cross sectio0n (c/s) data:

Distance from BM (m)	0	11	24	52	67	79	84
Rl ON C/S (m)	10.8	9.6	4.2	2.4	5.4	10.2	10.5

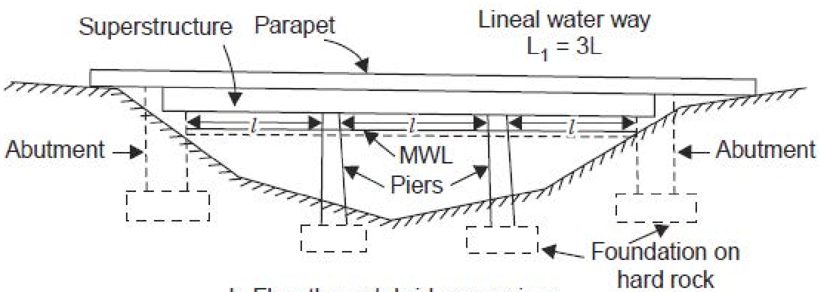
Leveling of longitudinal section (L/S) data:

Distance from bridge site L/S (m)	1 km upstream	At bridge site	1km downstream
HFL along L/S (m)	9.60	9.0	8.39
Mannings may be accused at 0.03			

(i) Discharge from Branch 1, i.e., Q_1 under bridge openings



a. Cross section



b. Flow through bridge openings

$$Q_1 = C_d A_1 \sqrt{2g\left(\Delta h + h_a\right)}$$

If L, d, V and L_1, d_1, V_1 refer to the length, mean depth and velocity of the normal stream (upstream of bridge site) and those under the contracted section of the bridge and also

$A_1 = L_1 d$

$Q_1 = LdV$

$$h_a = \frac{V^2}{2g}$$

$$Afflux\ \Delta h = \frac{V^2}{2g}\left(\frac{L^2}{C_d^2 L_1^2} - 1\right)$$

If the Branch 1, flow under bridge openings

$$4.6 - 2.8 = \frac{V^2}{2 \times 9.81}\left(\frac{324^2}{0.95^2 \times 262.5^2} - 1\right) \rightarrow V = 7.16\, m/\sec$$

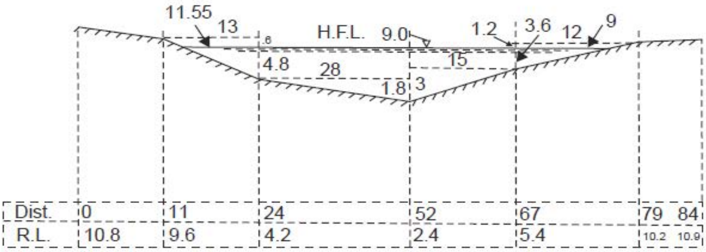
$Q_1 = LdV = 324 \times 2.8 \times 7.16 = 6500$ cumec

(ii) Discharge from Branch 2:

$Q_2 = CA^{2/3} = 10\ (4125)^{2/3} = 2580$ cumec

(iii) Discharge from Branch 3 (from slope-area method):

Area no.	Area shape	Area, A_1 (m^2)	Wetted perimeter, P_1 (m)
1.		$\frac{1}{2} \times 11.55 \times 4.8 = 27.75$	$\sqrt{11.55^2 + 4.8^2} = 12.5$
2.		$\frac{1}{2}\ (4.8 + 6.6)\ 28 = 159.60$	$\sqrt{28^2 + 1.8^2} = 28.2$
3.		$\frac{1}{2}\ (6.6 + 3.6)15 = 76.50$	$\sqrt{15^2 + 3^2} = 15.2$
4.		$\frac{1}{2} \times 9 \times 3.6 = 16.20$	$\sqrt{9^2 + 3.6^2} = 9.7$
		$A = 280.05$	$P = 65.6$



$$Hydraulic\ mean\ radius,\ R = \frac{A}{P} = \frac{280.05}{65.6} = 4.27\ m$$

$$Water\ surface\ slope,\ S = \frac{\Delta h}{L} = \frac{9.60 - 8.39}{2 \times 1000} = \frac{1}{1652}$$

By Manning's formula, the velocity of flow

$$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.03} (4.27)^{2/3} \sqrt{\frac{1}{1652}} = 2.16\ m/\sec$$

$Q_3 = AV = 280.05 \times 2.16 = 605$ cumec

Discharge at bridge site

$Q = Q_1 + Q_2 + Q_3 = 6500 + 2580 + 605 = 9685$ cumec

50. In a certain alluvial basin of 100 km², 90 Mm³ of ground water was pumped in a year and the ground water table dropped by about 5 m during the year. Assuming no replenishment, estimate the specific yield of the aquifer. If the specific retention is 12%, what is the porosity of the soil?

(i) Change in ground water storage

$\Delta GWS = A_{aq} \times \Delta GWT \times S_y$

$90 \times 10^6 = (100 \times 10^6) \times 5 \times S_y$

$S_y = 0.18$

(ii) Porosity $n = S_y + S_r = 0.18 + 0.12 = 30\%$

51. An artesian aquifer, 30 m thick has a porosity of 25% and bulk modulus of compression 2000 kg/cm². Estimate the storage coefficient of the aquifer. What fraction of this is attributable to the expansibility of water?

Bulk modulus of elasticity of water = 2.4 × 104 kg/cm².

$$S = \gamma_w nb\left(\frac{1}{K_w} + \frac{1}{nK_s}\right) = 1000 \times 0.25 \times 30\left(\frac{1}{2.14 \times 10^8} + \frac{1}{0.25 \times 2 \times 10^7}\right) = 1.54 \times 10^{-3}$$

Storage coefficient due to the expansibility of water as a percentage of S above

$$= \frac{7500 \times 0.467 \times 10^{-8}}{7500 \times 20.467 \times 10^{-8}} \times 100 = 2.28\%, \textit{ whic is negligible}$$

Note In less compressible formations like limestones for which Es ≈ 2×10^5 kg/cm², $S = 5 \times 10^{-5}$ and the fractions of this attributable to water and aquifer skeleton are 70% and 30%, respectively.

52. A 20-cm well penetrates 30 m below static water level (GWT). After a long period of pumping at a rate of 1800 lpm, the drawdowns in the observation wells at 12 m and 36 m from the pumped well are 1.2 m and 0.5 m, respectively.

Determine: (i) the transmissibility of the aquifer.

(ii) the drawdown in the pumped well assuming R = 300 m.

(iii) the specific capacity of the well.

$$Q = \frac{\pi K\left(h_2^2 - h_1^2\right)}{2.303\log_{10} r_2 / r_1}$$

$h_2 = H - s_2 = 30 - 0.5 = 29.5$ m; $h_1 = H - s_1 = 30 - 1.2 = 28.8$ m

$$\frac{1.800}{60} = \frac{\pi K\left(29.5^2 - 28.8^2\right)}{2.303\log_{10} 36/12}$$

$K = 2.62 \times 10^{-4}$ m/sec

(i) Transmissibility $T = KH = (2.62 \times 10^{-4})\ 30 = 78.6 \times 10^{-4}$ m²/sec

$$(ii)\, Q = \frac{2.72T\left(H - h_w\right)}{\log_{10} R / r_w}$$

$$\frac{1.800}{60} = \frac{2.72\left(78.6 \times 10^{-4}\right) S_w}{\log_{10} 300 / 0.10}$$

drawdown in the well, S_w = 4.88 m

(iii) The specific capacity of the well

$$= \frac{Q}{S_w} = \frac{1.800}{60 \times 4.88} = 0.0062\, m^3 / \sec - m$$

53. A tube well taps an artesian aquifer. Find its yield in litres per hour for a drawdown of 3 m when the diameter of the well is 20 cm and the thickness of the aquifer is 30 m. Assume the coefficient of permeability to be 35 m/day.

If the diameter of the well is doubled find the percentage increase in the yield, the other conditions remaining the same. Assume the radius of influence as 300 m in both cases.

$$Q = \frac{2.72T\left(H - h_w\right)}{\log_{10} R / r_w} = \frac{2.72\left\{\left(35 / 24\right) \times 30\right\} 3}{\log_{10}\left(300 / 0.10\right)} = 102.7\, m^3 / hr$$

The yield $\quad Q \propto \frac{1}{\log\left(R / r_w\right)}$

other things remaining same.

If the yield is Q′ after doubling the diameter, i.e.,

r_w' = 0.10 × 2 = 0.20 m

$$\frac{Q}{Q'} = \frac{\log R / r_w}{\log R / r_w}$$

$$\log\frac{300}{0.10} = 3.4771$$

$$\log\frac{300}{0.20} = 3.1761$$

$$\frac{102.7}{Q'} = \frac{3.1761}{3.4771} \rightarrow Q' = 112.4\, m^3 / hr$$

$$percentage\ increase\ in\ yield = \frac{Q' - Q}{Q} \times 100 = \frac{112.4 - 102.7}{102.7} \times 100 = 9.45\%$$

Thus, by doubling the diameter the percentage in yield is only about 10%, which is uneconomical. Large diameter wells necessarily do not mean proportionately large yields. The diameter of a tube well usually ranges from 20 to 30 cm so that the bowl assembly of a deep well or a submersible pump can easily go inside with a minimum clearance.

54. The following data are obtained from a cavity tube well:

Discharge	30 lps
Drawdown	4 m
Permeability of cavity	50 m/day
Depth of cavity	20 cm
Radius of influence	150 m

Determine the radius and width of cavity.

$$Well\ yield, Q = \frac{2\pi K_y \left(H - h_w\right)}{1 - \frac{r_w}{R}}$$

$$\frac{30}{1000} = 2\pi \times \frac{50}{24 \times 60 \times 60} \times \frac{0.20 \times 4}{1 - \frac{r_w}{150}}$$

Radius of cavity, r_w = 135.5 m

Width of cavity, r_e =[(2r_w − y)y]$^{0.5}$ = [(2 × 4.5 − 0.2) 0.2]$^{0.5}$ = 7.36 m

55. A well of size 7.70 × 4.65 m and depth 6.15 m in lateritic soil has its normal water level 5.08 m below ground level (bgl). By pumping for 1.5 hours, the water level was depressed to 5.93 m bgl and the pumping was stopped. The recuperation rates of the well during 4 hours after the pumping stopped are given below. The total volume of water pumped during 1.5 hours of pumping was 32.22 m^3. (no well steining is provided)

Time since pumping stopped (min)	Water level bgl (m)

0	5.930
15	5.890
30	5.875
45	5.855
60	5.840
90	5.820
120	5.780
180	5.715
240	5.680

Determine
(i) Rate of seepage into the well during pumping.
(ii) Specific yield of the soil and specific capacity of the well.
(iii) Yield of the well under a safe working depression head of 0.85 m.
(iv) The area of crop that can be irrigated under the well (assume a peak consumptive use of 4 mm and irrigation efficiency of 75%).
(v) Diameter of the well in such a soil to get an yield of 3000 lph under a safe working depression head of 0.8 m.

(i) Seepage into the well—from pumping data:
Volume of water pumped out = 32.22 m^3
Volume of water stored in the well (that was pumped out)= (7.70 × 4.65) (5.93 – 5.08) = 30.5 m^3

$$Rate\ of\ seepage\ into\ the\ well = \frac{32.22 - 30.5}{1.5} = 1.15\ m^3/hr$$

(ii) Specific yield of the soil

$$C = \frac{2.303}{T}\log_{10}\frac{s_1}{s_2} = \frac{2.303}{4}\log_{10}\frac{5.93 - 5.08}{5.68 - 5.08} = 0.09\ hr^{-1}$$

Specific capacity of the well is its yield per unit drawdown

Q = CAH

Specific capacity = Q/H = CA = 0.09 (7.70 × 4.65)= 3.58 $m^3\ hr^{-1}/m$

Safe yield of the well

Q = CAH = 0.09 (7.70 × 4.65) 0.85 = 3.04 m^3/hr

which is more than twice the seepage into the well during pumping.

(iv) Area of crop that can be irrigated under the well:

Data to draw the curve s_1/s_2 vs. t (s_1 = total drawdown, s_2 = residual drawdown): SWL =5.08 m, s_1 = 5.93 – 5.08 = 0.85 m

Time since pumping stopped (min)	Water level bgl (m)	Residual drawdown s_2= W_l – SWL (m)	Ratio (s_1/s_2)
0	5.930	0.850 (=s_1)	1.0
15	5.890	0.810	1.05
30	5.875	0.795	1.07

45	5.855	0.775	1.09
60	5.840	0.760	1.11
90	5.820	0.740	1.15
120	5.780	0.700	1.21
180	5.715	0.635	1.33
240	5.680	0.600	1.41

From the plot of 's_1/s_2 vs. time' on a semi-log paper, it is seen that s_1/s_2 = 9.5 after 24 hours of recovery (by extending the straight line plot), and the residual drawdown after 24 hours, s_{24} =0.85/9.5≈ 0.09 m; hence the depth of recuperation per day = 0.85 – 0.09 = 0.76 m and the volume of water available per day ≈ (7.70 × 4.65) ≈ 27.2 m3. With an average peak consumptive use of 4 mm for the type of crops grown and irrigation efficiency of 75%, the area of crop (A_{crop}) that can be irrigated under one well in lateritic soils is

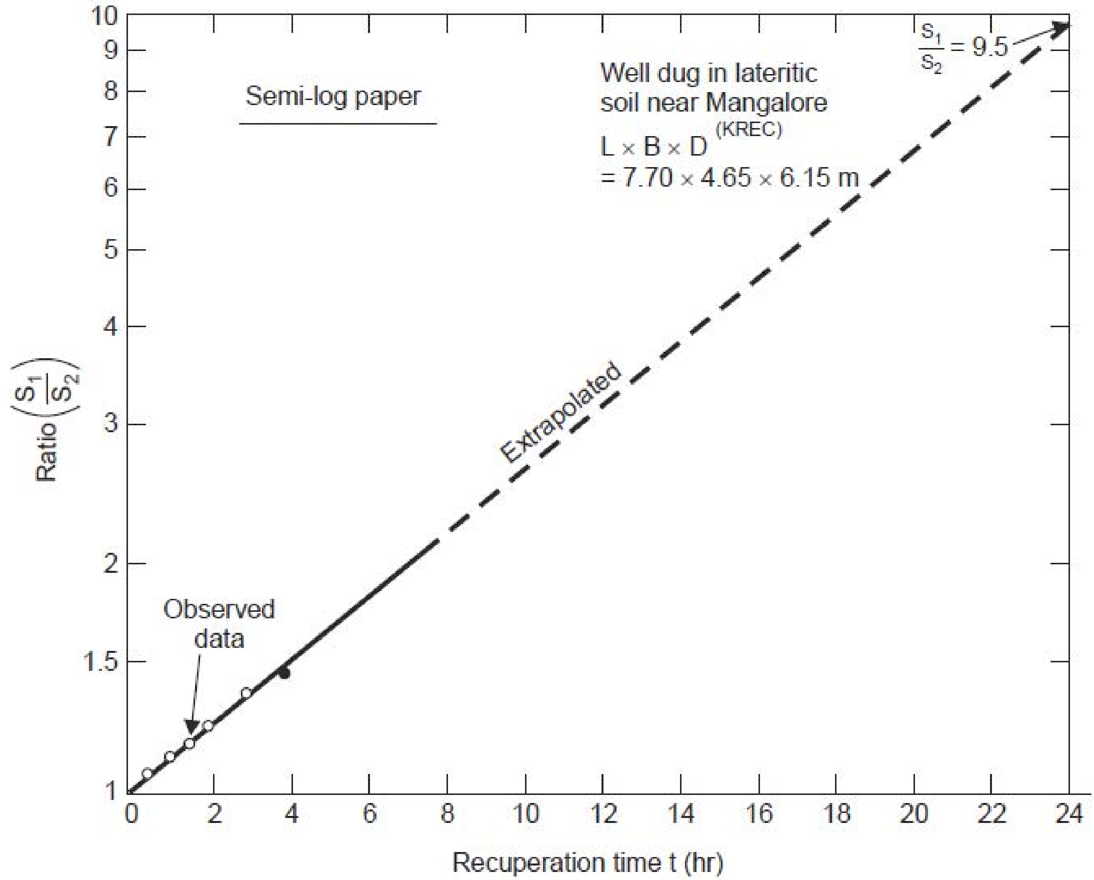
$$\frac{4}{1000 \times 0.75} \times A_{crop} = 27.4 \rightarrow A_{crop} = 5100\, m^2$$

(v) Diameter of the well to yield 3000 lph:

Q = CAH

$$\frac{3000}{1000} = 0.09 \times \pi \times \frac{D^2}{4} \times 0.8 \rightarrow D = 7.3\, m, \textit{ which is too big}$$

It may be noted that it is not advisable to go deeper in these areas otherwise salt water instrusion takes place.



56. Determine the peak discharge at the concentration point for a basin of 80 hectares having a time of concentration of 30 minutes due to a 5-cm flash storm, if the duration of the storm is (i) 60 min, (ii) 30 min, and (iii) 15 min. Assume a φ-index of 2.5 cm/hr for the entire basin. When the storm duration is 15 minutes, only drainage from 60% of the area of the basin reaches the concentration point.

Q = (i – φ) A, where i = intensity of rainfall (cm/hr)

(i) Q = (5 – 2.5) 80 = 200 ha-cm = 200 × 0.028 = 5.6 cumec

(ii) Q = (5/30 × 60 – 2.5) 80 = 600 ha-cm = 600 × 0.028 = 16.8 cumec

(iii) Q = (5/15 × 60 – 2.5) (0.60 × 80) = 840 ha-cm = 840 × 0.028 = 23.52 cumec

It is seen from (i) and (ii) that the peak discharge at the concentration point is maximum when the duration of storm is equal to the time of concentration, (iii) gives the highest flood, since only 60% of the area drains, the concentration time becomes less and the intensity of rainfall is very high during this time.

57. For an area of 20 hectares of 20 minutes concentration time, determine the peak discharge corresponding to a storm of 25-year recurrence interval. Assume a runoff

coefficient of 0.6. From intensity-duration-frequency curves for the area, for T = 25-yr, t = 20 min, i = 12 cm/hr.

For $t = t_c = 20$ min, T = 25-yr, $i = i_c = 12$ cm/hr

Q = CiA = 0.6 × 12 × 20 = 144 ha-cm/hr = 144 × 0.028 = 4 cumec

58. Determine the design flood discharge (allowing an increase of one-third) for a bridge site with the following data:

Catchment area	= 2 x 10^5 ha
Duration of storm	= 8 hours
Storm precipitation	= 3 cm
Time of concentration	= 2 hr

Gauged discharge for a past flood with average maximum daily rainfall of 18 cm was 3400 cumec.

From the past flood,

$$Runoff\ coefficient, C = \frac{Actual\ discharge}{Theoretical\ discharege} = \frac{3400}{2\times10^5\times10^4\times\frac{18}{100}\times\frac{1}{24\times60\times60}} = 0.815$$

Design or critical intensity of rainfall

$$i = i_c = \frac{P}{t_R}\left(\frac{t_R+1}{t_c+1}\right) = \frac{3}{8}\left(\frac{8+1}{2+1}\right) = 1.125\, cm/hr$$

Q = CiA = 0.815 × 1.125 × (2 × 10^5) = 1.83 × 10^5 ha-cm/hr

From Inglis formula

$$Q = \frac{124A}{\sqrt{A+10.4}} = \frac{124\times2000}{\sqrt{2000+10.4}} = 5520\, cumec$$

Design flood discharge = 5520 × 1.33 = 7350 cumec

The Unit Hydrograph Method. For small and medium size basins (A < 5000 km^2,i.e., when a single unit hydrograph could be applied to the entire basin) in developing design flood hydrographs by applying the unit hydrograph for the basin, the design storm estimates are made by the following methods.

(i) Selection of major storms

(ii) Maximization of selected storms

(iii) Plotting the depth-area-duration curves and their analysis

(iv) Moisture adjustment

(v) Storm transposition to a critical position

(vi) Envelopment of the transposed adjusted storms

(vii) Use of minimum infiltration indices

In the depth-area-duration analysis of a particular storm, the maximum average depths of rainfall over various sizes of area during certain periods of storm (hr or days), say cm over 1000 km^2 in 1 day, 2 days or 3 days from the isohyetal maps contructed. Such values determined for all the transposable storms provide the basic data to estimate the PMP over the basin.

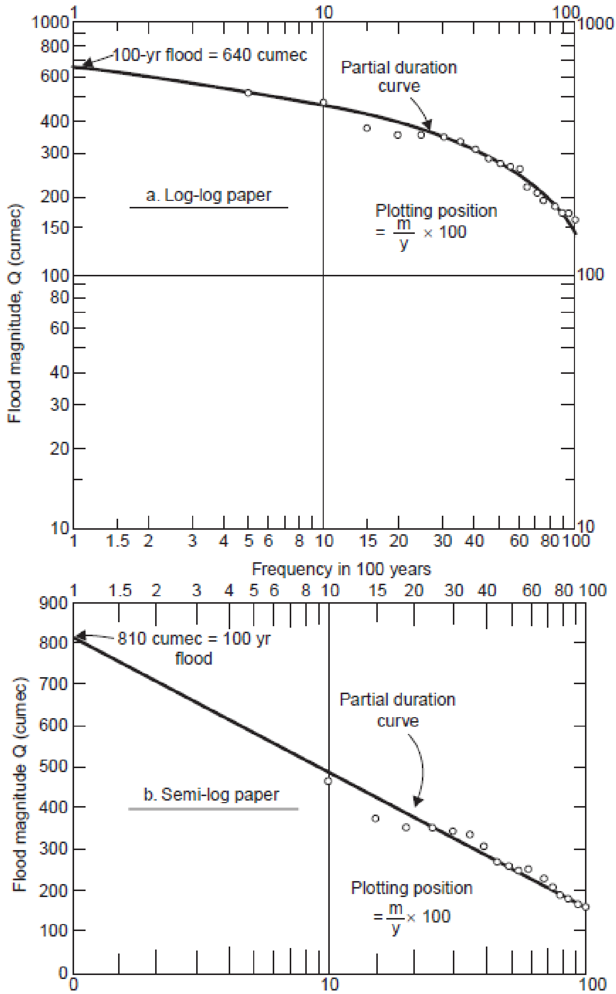
59. Twenty largest one-day floods (without respect to time) are selected in a period of 20 years arranged in the descending order of magnitude (cumec). Draw the partial duration curve:

501, 467, 371, 351, 351, 345, 334, 311, 283, 273, 266, 264, 221, 214, 194, 193, 182, 175, 173, 163.

Sl. no.	*Flood flow (cumec)*	*Probable frequency in 100 years*
1	501	5*
2	467	10
3	371	15
4	351	20
5	351	25
6	345	30
7	334	35
8	311	40
9	283	45
10	273	50
11	266	55
12	264	60
13	221	65
14	214	70
15	194	75
16	193	80
17	182	85
18	175	90
19	173	95
20	163	100

*Probable frequency $= \frac{m}{y} \times 100 = \frac{1}{20} \times 100 = 5$

The partial duration curves are plotted on both log-log paper and semi-log paper as shown in the figures; the 100-yr flood is extrapolated as 640 and 810 cumec from the curves (a) and (b), respectively.



60. The highest annual floods for a river for 60 years were statistically analysed. The sixth largest flood was 30,000 cumec (30 tcm).

Determine:

(i) The period in which the flood of 30 tcm may reoccur once

(ii) The percentage chance that this flood may occur in any one year

(iii) The percentage chance that this flood may not occur in the next 20 years

(iv) The percentage chance that this flood may occur once or more in the next 20 years

(v) The percentage chance that a 50-yr flood may occur (a) once in 50 years, (b) one or more times in 50 years

$$(i)\ Weibull;\ T = \frac{n+1}{m} = \frac{60+1}{6} = 10\ yr$$

$$(ii)\ Percentage\ chance,\ i.e.,\ P = \frac{1}{T}\times 100 = \frac{1}{10.1}\times 100 = 10\%$$

$$(iii)\ Encounter\ probabilty,\ P_{(N,0)} = (1-P)^N = \left(1-\frac{1}{10.1}\right)^{20} = 12.4\%$$

$$(iv)\ P_{Ex} = 1-(1-P)^N = 1-P_{(N,0)} = 1-0.124 = 87.6\%$$

$$(v)(a)\ P = \frac{1}{T}\times 100 = \frac{1}{50}\times 100 = 2\%$$

$$(b)\ P_{(N,0)} = \left(1-\frac{1}{50}\right)^{50} = 0.3631$$

$$P_{Ex} = 1-P_{(N,0)} = 1-0.3631 = 64\%$$

61. Determine the percentage chance that a 25-yr storm may occur
(a) In the next 10 years
(b) In the next year itself
(c) May not occur in another 15 years

$$(a)\ T = 25,\ P_{Ex} = 1-(1-P)^N,\ P = \frac{1}{T}$$

$$P_{Ex} = 1-\left(1-\frac{1}{25}\right)^{10} = 33.5\%$$

$$(b)\ P_{Ex} = 1-\left(1-\frac{1}{25}\right)^{10} = 4\%$$

$$(c)\ P_{(N,0)} = (1-P)^N = \left(1-\frac{1}{25}\right)^{15} = 54.2\%$$

62. Determine the return period (recurrence interval T) of a flood, which has a 10% risk of being flooded (a) in the next 100 years, (b) in the next 50 years.

$P_{Ex} = 1 - (1 - P)^N$, for risk of being exceeded

i.e., $P_{Ex} = 10\% = 0.1$

(a) $0.1 = 1 - (1 - P)^{100}$, $(1 - P)^{100} = 1 - 0.1 = 0.9$

$1 - P = 0.9^{0.01} = 0.99895$, $P = 0.00105 = 1.05 \times 10^{-3}$

$$T = \frac{1}{P} = \frac{1}{1.05 \times 10^{-3}} = 1000 - yr\ flood$$

(b) $0.1 = 1 - (1 - P)^{50}$, $(1 - P)^{50} = 1 - 0.1 = 0.9$

$1 - P = 0.9^{0.02} = 0.9979$

$P = 0.0021 = 2.1 \times 10^{-3}$

$$T = \frac{1}{P} = \frac{1}{2.1 \times 10^{-3}} = 500 - yr\ flood$$

Note. If a structure has a life period of 50 years and if we can accept a 10% risk of its being flooded during its life, then we have to design the structure for a return period of T-Yr as follows:

$P_{Ex} = 1 - (1 - P)^{50}$; for 10% risk, $P_{Ex} = 0.1$

$0.1 = 1 - (1 - P)^{50}$, $(1 - P)^{50} = 1 - 0.1 = 0.9$

$(1 - P) = (0.9)^{0.02} = 0.9979$

$$P = 0.0021 = 2.1 \times 10^{-3}, T = \frac{1}{P} = \frac{1}{2.1 \times 10^{-3}} = 476\ yr$$

i.e., we have to design the structure for a 476-yr flood and not for a 50-yr flood; if it is designed for a 50 yr flood, the risk of failure

$$P_{Ex} = 1 - (1 - P)^{50} = 1 - \left(1 - \frac{1}{50}\right)^{50} = 63.6\%$$

63. The maximum annual floods for the river Tapti at Ukai were statistically analysed for a period of 93 years (1876-1968). The mean annual flood and the standard deviation are 14210 and 9700 cumec, respectively.

Determine:

(i) The recurrence interval of the highest flood 42500 cumec (in 1968) by Weibull's method and what its percentage chance of occurring in (a) in any year, (b) in 10 years ?

(ii) What is the recurrence interval of the design flood adopted by CWPC (49500 cumec) and the highest flood (42500 cumec) by Gumbels method?

For the highest flood, its rank m = 1

(ii) From Gumbel's Eqn.

(a) Design flood Q_{DF} = 49500 cumec = Q_T , Q_{ave} = 14210 cumec

$Q_T = Q_{ave} + \sigma\,(0.78 \ln T - 0.45)$, for $n > 50$

$$(i)\ T=\frac{n+1}{m}=\frac{93+1}{1}=94\ yr$$

$$(a)\ P=\frac{1}{T}=\frac{1}{94}=1.065\%$$

$$(b)\ P_{Ex}=1-(1-P)^{N}=1-\left(1-\frac{1}{94}\right)^{10}=10.14\%$$

49500 = 14210 + 9700 (0.78 ln T – 0.45)

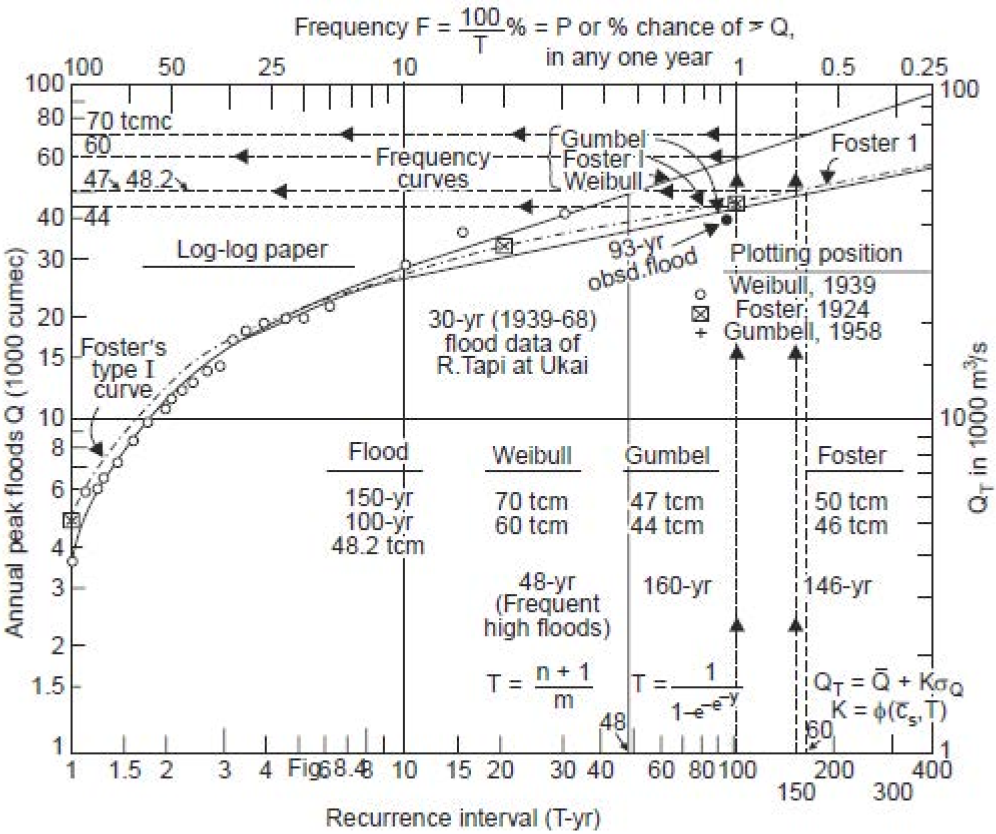
ln T = 5.241, T = 189, say 190 yr

against Weibull's 50 yr

(b) Highest flood Q_{MF} = 42500 cumec = Q_T

42500 = 14210 + 9700 (0.78 ln T – 0.45)

ln T = 4.316, T = 75 yr, against Weibull's 30 yr



64. Statistical analysis of the annual floods of the river Tapti (1876-1968) using

Gumbel's method yielded the 100-yr and 10-yr floods as 42800 and 22700 cumec, respectively.

Determine:

(a) the magnitude of a 20-yr flood.

(b) the probability of a flood of magnitude 35000 cumec (i) occurring in the next 10 years, (ii) in the next year itself.

$$y_{10} = -\ln.\ln\frac{10}{9} = 2.25,\ X_{10} = 22.7\ tcm$$

$$y_{100} = -\ln.\ln\frac{100}{99} = 4.6,\ X_{100} = 42.8\ tcm$$

$$2.25 = \frac{a\left(22.7 - \overline{X}\right)}{\sigma} + b$$

$$4.6 = \frac{a\left(42.8 - \overline{X}\right)}{\sigma} + b$$

$$2.35 = 20.1\frac{a}{\sigma}$$

$$\sigma = 20.1 \times \frac{1.2835}{2.35} = 10.969\ tcm$$

$$22.7 - \overline{X} = \left(2.25 - 0.577\right)\frac{10.969}{1.2825} = 14.309$$

$$\overline{X} = 22.7 - 14.309 = 8.391\ tcm$$

$$y_{20} = -\ln.\ln\frac{20}{19} = 2.97$$

$$2.97 = \frac{1.2825}{10.969}\left(X_{20} - 8.391\right) + 0.577$$

$$X_{20} = 28.856\ tcm$$

$$y_T = \frac{1.2825}{10.969}\left(35 - 8.391\right) + 0.577 = 3.6881$$

$$-\ln.\ln\frac{T}{T-1} = 3.6881$$

$$T = 41\ yr$$

$$P = \frac{1}{T} = 0.0244$$

$$Alternatively,\ P = 1 - e^{-e^{-y}} = 1 - e^{-e^{-3.6881}} = 0.0247$$

(i) $P_{Ex} = J_{(41, 10)} = 1 - (1 - 0.0244)^{10} = 0.2188$, say 22%

(ii) $P_{Ex} = J_{(41, 1)} = - (1 - 0.0244)^{1} = 0.0244$, or 2.44% chance

65. The annual floods for a large period were statistically analysed by Gumbel's methods, which yielded Q_{ave} = 19000 cumec, σ = 3200 cumec.

Determine

(a) the probability of a flood magnitude of 30000 cumec occurring in the next year.

(b) the flood magnitude of 5-yr return period.

$$y = \frac{1.2825}{3200}(30000 - 19000) + 0.577 = 5.5$$

$$P = 1 - e^{-e^{-y}} = 1 - e^{-0.0067} \cong 1\%$$

$P_{Ex} = 1 - (1 - 0.0099)^1 = 0.0099 \sim 1\%$

$$T = 5, P = \frac{1}{5} = 1 - e^{-e^{-y}}, y = 0.079$$

$$0.079 = \frac{1.2825}{3200}(X - 19000) + 0.577, X = 17758\, cumec$$

66. A channel has a bottom width of 200 m, depth 6 m and side slopes 1:1. If the depth is increased to 9 m by dredging, determine the percentage increase in velocity of flow in the channel. For the same increase in cross sectional area, if the channel is widened (instead of deepening), what is the percentage increase in the velocity of flow.

Case (i) Increasing the depth to 9 m by dredging.

Putting the subscript 'o' for the original area of cross section (A), wetted perimeter (P) and the hydraulic mean radius (R),

$A_0 = (200 + 1 \times 6)6 = 1236\ m^2$

$$P_0 = 200 + 2 \times 6\sqrt{1^2 + 1} = 217\, m$$

$$R_0 = \frac{A_0}{P_0} = \frac{1236}{217} = 5.7\, m$$

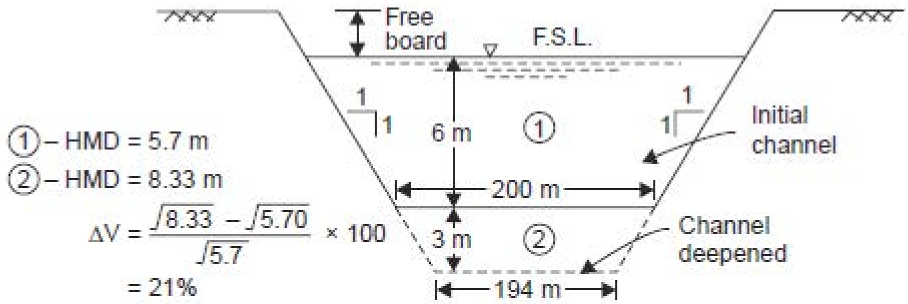
After deepening from 6 m to 9 m,

$A = (194 + 1 \times 9)9 = 1827\ m^2$

$$P = 194 + 2 \times 9\sqrt{1^2 + 1} = 219.4\, m$$

$$R = \frac{A}{P} = \frac{1827}{219.4} = 8.33\, m$$

$$Velocity\ increase\ by\ deepening = \frac{\sqrt{8.33} - \sqrt{5.70}}{\sqrt{5.7}} \times 100 = 21\%$$



Case (ii) For the same increase in the cross sectional area, widening the channel, Let the bottom width after widening be b′.

1827 = (b′ + 1 × 6)6

b′ = 298.5 m

$$After\ widening\ P = 298.5 + 2 \times 9\sqrt{1^2 + 1} = 315.42\ m$$

$$R = \frac{A}{P} = \frac{1827}{315.42} = 5.8\ m$$

$$Velocity\ increase\ by\ deepening = \frac{\sqrt{5.8} - \sqrt{5.70}}{\sqrt{5.7}} \times 100 = 0.84\%$$

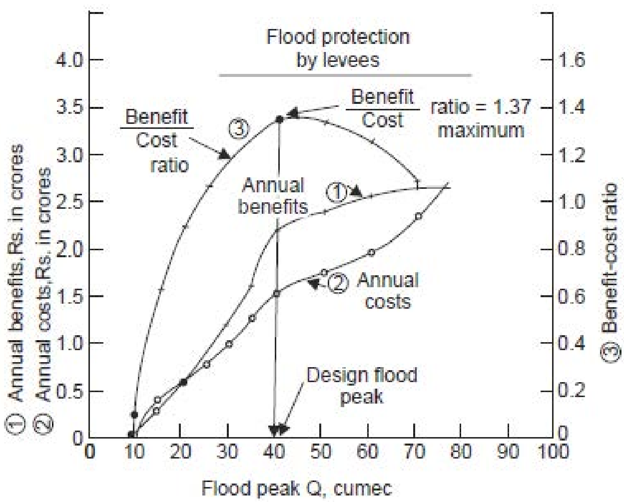
Thus, the velocity increase will be only 0.84% on widening as against 21% by deepening. Hence, exploding the river channels at the mouths at the start and ebbing of floods will be logical proposition.

67. The costs of construction of levees for flood protection for various flood peaks are given below. From this and other data given, make an economic analysis of the flood control project and determine the flood peak for which the levees have to be designed.

Flood peak (1000 cumec)	Total damage under the flood peak (Rs. In crores)	Recurrence interval of flood peak (yr)	Annual project cost upto the flood peak (Rs. In crores)
10	0	2	0.2
15	2	10	0.4
20	5	20	0.6
25	8	30	0.8
30	12	42	1.0
35	20	60	1.3
40	32	80	1.6
50	46	150	1.8
60	70	300	2.0
70	98	600	2.4

The economic analysis is made as shown in the table on the basis of benefit-cost ratio.
The ratio of benefit to cost is a maximum of 1.39 when the levees are constructed to safely pass a flood peak of 40000 cumec. Hence, the levees designed for this flood peak will be most economical.

Flood peak (1000 cumec)	Total damage under the flood peak (Rs. In crores)	Increment of Damage (Rs. crores)	Recurrence interval of flood peak (yr)	Increment in recurrence interval (Yr)	Annual benefit from protection of incremental damage (Rs. Crores) 3 + 5	Total annual benefits from protection for flood peak (Rs. Crores)	Annual project cost for the flood peak (Rs. Crores)	Ratio of benefit to cost
1	2	3	4	5	6	7	8	9
10	0	2	2	8	0.25	0	0.2	0
15	2	3	10	10	0.30	0.25	0.4	0.62
20	5	3	20	10	0.30	0.55	0.6	0.92
25	8	4	30	12	0.33	0.85	0.8	1.06
30	12	8	42	18	0.44	1.18	1.0	1.18
35	20	12	60	20	0.60	1.62	1.3	1.25
40	32	14	80	70	0.20	2.22	1.6	1.39
50	46	24	150	150	0.16	2.42	1.8	1.34
60	70	28	300	300	0.09	2,58	2.0	1.29
70	98		600			2.67	2.4	1.10



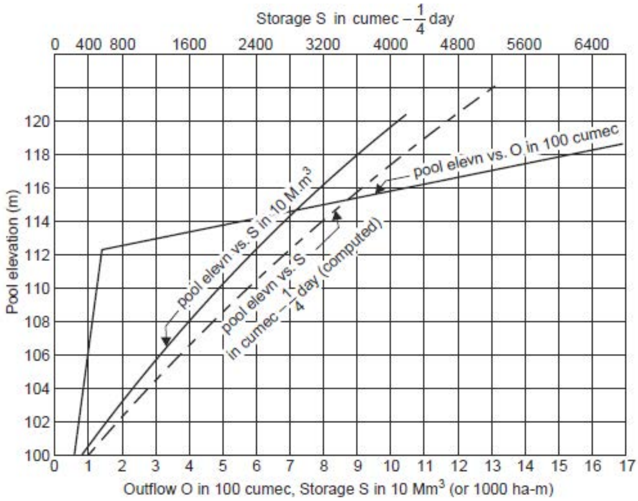
68. For a reservoir with constant gate openings for the sluices and spillway, pool elevation vs storage and discharge (outflow) curves are shown in the figure. The inflow hydrograph into the reservoir is given below:

Time (hr)	0	6	12	18	24	30	36	42
Inflow (cumec)	50	70	160	300	460	540	510	440
Time (hr)	48	54	60	66	72	78	84	90
Inflow (cumec)	330	250	190	150	120	90	80	70

Pool elevation at the commencement = 110 m

Discharge at the commencement = 124 cumec

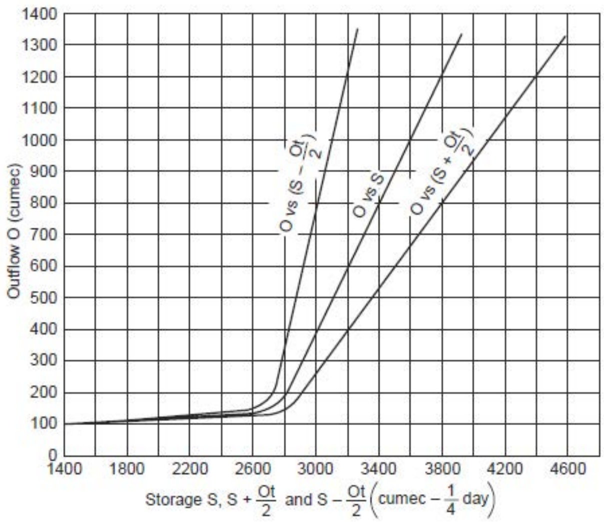
Route the flood through the reservoir by (a) ISD method, and (b) modified Puls method, and compute the outflow hydrograph, the maximum pool elevation reached, the reduction in the flood peak and the reservoir lag.



(a) Flood routing by ISD method Take the routing period as 6 hr or 0.25 day. It is easier to work the flow rates in cumec and the storage volumes in terms of cumec – 0.25 day. Hence, the storage in Mm^3 is converted to cumec –0.25 day by multiplying by 46.3. Corresponding to an initial pool elevation of 110 m, O = 124 cumec, S = 49.1 Mm^3 = 49.1 × 46.3 = 2270 cumec -0.25 day, Ot/2=(O/2)×t(124/2) cumec×0.25 day=62 cumec-0.25 day, S+(Ot/2)+2270+62=2332 cumec-0.25 day, and S-(Ot/2)=2270-62=2208 cumec-0.25 day. First 'O vs. S' curve is drawn. For a particular O on the S curve, O/2 abscissa units may be set off on either side of the S curve and this is repeated for other values of O. The points obtained on either side of S curve plot S+(Ot/2) and S-(Ot/2) curves as shown in the figure.

Pool elevation	*Outflow O*	*Storage S*		*Computation for I.S.D. method ($t = 6hr = \frac{1}{4}$ day)*			*Computation for modified Puls method*		
(m)	*(cumec)*	*(Mm³)*	*(cumec – $\frac{1}{4}$ day*)*	$\frac{Ot}{2}$ *(cumec – $\frac{1}{4}$ day)*	$S + \frac{Ot}{t}$ *(cumec – $\frac{1}{4}$ day)*	$S - \frac{Ot}{2}$ *(cumec – $\frac{1}{4}$ day)*	$\frac{2S}{t}$ *(cumec)*	$\frac{2S}{t} + O$ *(cumec)*	$\frac{2S}{t} - O$ *(cumec)*
100	60	8.7	400	30	430	370	800	860	740
102	70	15.1	700	35	735	665	1400	1470	1330
104	86	23.4	1480	43	1123	1037	2160	2246	2074
106	100	32.0	1480	50	1530	1430	2960	3060	2860
108	110	40.0	1850	55	1905	1795	3700	3810	3590
110	124	49.1	2270	62	2332	2208	4540	4664	4416
112	138	58.3	2700	69	2769	2631	5400	5538	5262
113	310	63.0	2920	155	3075	2765	5840	6150	5530
114	550	68.3	3160	275	3435	2885	6320	6870	5770
115	800	73.5	3400	400	3800	3000	6800	7600	6000
116	1030	78.8	3650	515	4165	3135	7300	8330	6270
117	1280	83.8	3880	640	4520	3240	7760	9040	6480
118	1520	90.0	4160	760	4920	3400	8320	9840	6800
120	—	101.0	4680	—	—	—	—	—	—

*1 cumec – $\frac{1}{4}$ day = 1 × 6 × 60 = 21600 m³. 1 million m³ (Mm³) = $10^6/21600$ = 46.3 cumec – $\frac{1}{4}$ day.



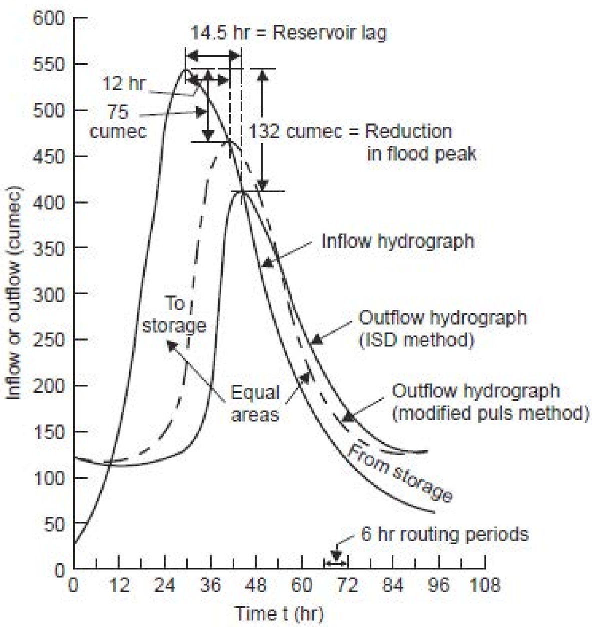
For routing the flood by the I.S.D. method, Table 9.2, for the known outflow at the commoncement of 124 cumec, S-$\frac{Ot}{2}$ is read from the curve as 2208 cumec-$\frac{1}{4}$ day and to this $\frac{I_1 + I_2}{2}t = \frac{50+70}{2} cumec \times \frac{1}{4}$ *day*= 60 cumec $-\frac{1}{4}$ day is added to get the right hand side of *the Eq.* i.e., S +$\frac{Ot}{2}$ = 2268 and corresponding to this O = 120 cumec is read from the graph which is the outflow at the beginning of the next routing period. Corresponding to this O = 120 cumec, the pool elevation of 109.2 m is read from the 'pool elevations vs. O' curve. Corresponding to this O = 120 cumec, S $-\frac{Ot}{2}$ = 2040 is read from the graph and $\frac{I_1 + I_2}{2}t = \frac{70+160}{2}t = 115\, cumec - \frac{1}{4}$ day is added to get S +$\frac{Ot}{2}$ = 2155 for which O is read as 116 cumec and pool elevation as 108.4 m. Thus the process is repeated till the flood is completely routed through the reservoir and the outflow hydrograph is obtained as shown in *the figure.*

(b) Flood routing by modified Puls method: Corresponding to the initial pool elevation of 110 m,

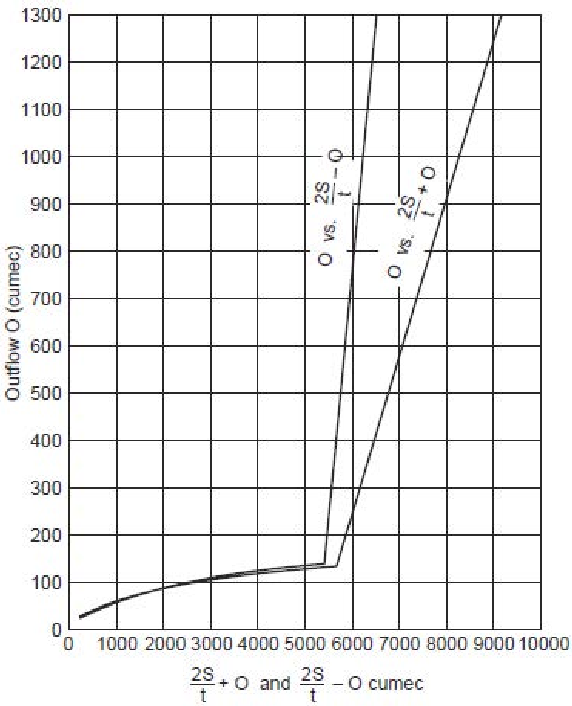
O = 124 cumec, S = 2270 cumec-$\frac{1}{4}$ day, $\frac{2S}{t} = \frac{2 \times 2270\, cumec - \frac{1}{4} day}{\frac{1}{4} day} = 4540\, cumec, \frac{2S}{t} + O =$

4540 + 124 = 4664 cumec and $\frac{2S}{t} - O = 4540 - 124 = 4416$ *cumec. Thus, for other* values of O, values of $\frac{2S}{t} + O$ *and* $\frac{2S}{t} - O$ are computed and 'O *vs.* $\frac{2S}{t} + O$ *and* $\frac{2S}{t} - O$' curves are drawn as shown in the figure.

Time (hr)	Inflow I (cumec)	$\frac{I_1 + I_2}{2} t$ (cumec-$\frac{1}{4}$ day)	Outflow O (cumec)	$S - \frac{Ot}{2}$ (cumec-$\frac{1}{4}$ day)	$S + \frac{Ot}{2}$ (cumec-$\frac{1}{4}$ day)	Pool elevation (m)
0	50		124			110.0
		60	+	2208	2268	
6	70		120			109.2
		115	+	2040	2155	
12	160		116			108.4
		230	+	1960	2190	
18	300		119			109.1
		380		2020	2400	
24	460		122			109.6
		500		2080	2580	
30	540		130			110.8
		525		2380	2905	
36	510		195			112.5
		475		2730	3205	
42	440		395			113.4
		385		2820	3205	
48	330		395			113.4
		290		2920	3110	
54	250		335			113.1
		220		2790	3010	
60	190		265			112.8
		170		2760	2930	
66	150		210			112.6
		135		2740	2875	
72	120		170			112.4
		105		2720	2825	
78	90		145			112.3
		85		2700	2785	
84	80		132			111.2
		75		2650	2725	
90	70		130			110.8



14.5 hr = Reservoir lag
12 hr
75
cumec
132 cumec = Reduction
in flood peak
Inflow hydrograph
To
storage
Outflow hydrograph
(ISD method)
Equal
areas
Outflow hydrograph
(modified puls method)
From storage
6 hr routing periods
Inflow or outflow (cumec)
Time t (hr)



O vs. 2S/t − O
O vs. 2S/t + O
Outflow O (cumec)
2S/t + O and 2S/t − O cumec

For routing the flood by the modified Puls method, Table 9.3, corresponding to the initial pool elevation of 110 m, O = 124 cumec, $\frac{2S}{t}+O=4664$ *cumec and* $\frac{2S}{t}-O=4416$ cumec are read off. For this $\frac{2S}{t}-O=4416$ cumec, $I_1+I_2=50+70=120$ cumec is added to get the right hand side of *the Eq.* i.e., $\frac{2S}{t}+O=4416+120=4536$ cumec. For this value of $\frac{2S}{t}+O$, O = 123 cumec, and $\frac{2S}{t}-O=4290$ cumec are read off from the curves. For O = 123 cumec, the pool elevation of 109.8 m is read off from the 'O vs pool elevation curve'. These values become the initial values for the next routing period. Again, for $\frac{2S}{t}-O=4290$ cumec, $I_1+I_2=70+160=230$ cumec is added to get the right hand side of *the Eq.* i.e., $\frac{2S}{t}+O=4290+230=4520$ cumec for which O and $\frac{2S}{t}-O$ values are read off and pool elevation obtained, which become the initial values for the next routing period. Thus the process is repeated till the flood is completely routed through the reservoir and the outflow hydrograph is obtained as shown in *the figure* by dashed line.

Time (hr)	Inflow O (cumec)	$\frac{2S}{t} - O^*$ (cumec)	$\frac{2S}{t} + O$ (cumec)	Outflow O (cumec)	Pool elevation (m)
0	50	4416	4464	124	110.0
6	70	4290	4536	123	109.8
12	160	4276	4520	122	109.6
18	300	4482	4736	126	111.8
24	460	4986	5248	131	111.0
30	540	5506	5986	240	112.7
36	510	5696	6556	430	113.5
42	440	5716	6646	465	113.6
48	330	5666	6486	410	113.4
54	250	5586	6246	330	113.0
60	190	5526	6026	250	112.7
66	150	5466	5866	200	112.5
72	120	5436	5736	150	112.3
78	90	5476	5646	135	111.6
84	80	5278	5546	134	111.4
90	70		5428	130	110.8

$$* \frac{2S}{t} - 0 = \left(\frac{2S}{t} + 0\right) - 2O$$

	ISD method	Modified plus method
Maximum pool elevn reached	1113.5 m*	113.6 m
Reduction in flood peak	132 cumec	75 cumec
Reservoir lag	14 ½ hr	12 hr

***to pass the crest of the outflow hydrograph**

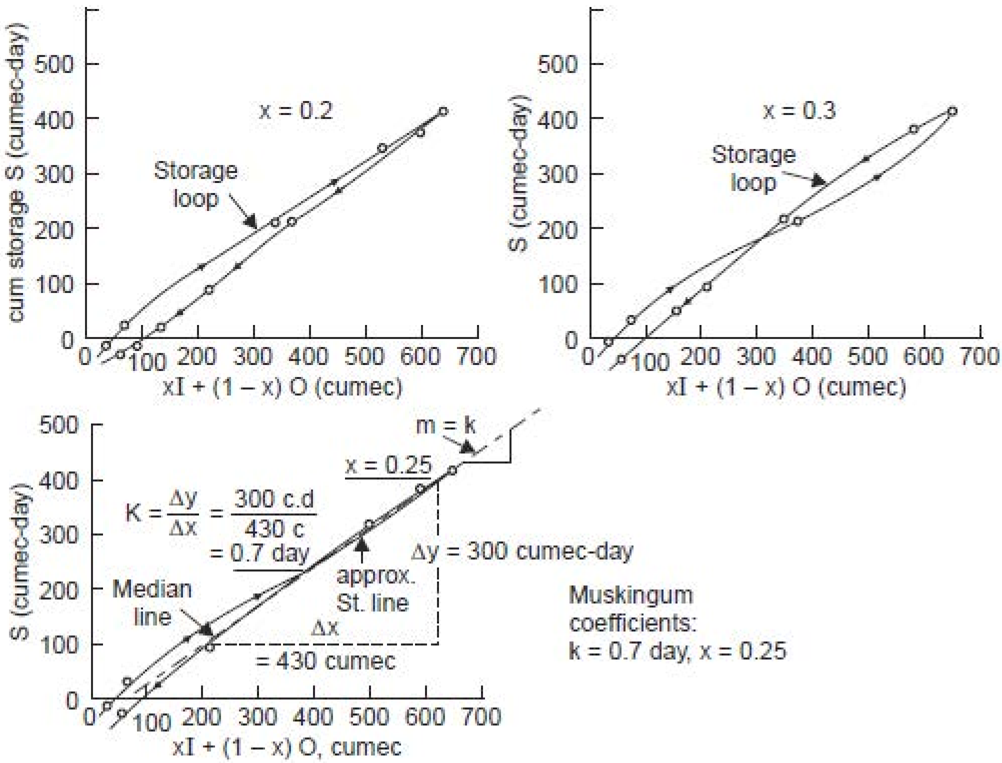
69. The inflow and outflow hydrographs for a reach of a river are given below. Determine the value of the Muskingum coefficients K and x for the reach.

Time (hr)	0	24	48	72	96	120	144	168	192	216
Inflow (cumec)	35	125	575	740	456	245	144	95	67	50
Outflow (cumec)	39	52	287	624	638	394	235	142	93	60

From the daily readings of the inflow and outflow hydrographs, a routing period t =24 hr = 1 day is taken. The mean storage is determined and then the cumulative storage S is tabulated. For trial values of x = 0.2, 0.25 and 0.3, the values of [xI + (1 – x) O] are computed in the table. Storage loops for the reach, i.e., curves of S vs. [xI + (1 – x) O] for each trial value of x are plotted as shown in the figure. By inspection, the middle value of x = 0.25 approximates a straight line and hence this value of x is chosen. K is determined by measuring the slope of the median straight line which is found to be 0.7 day. Hence, for the given reach of the river, the values of the Muskingum coefficients are

x = 0.25, K = 0.7 day

Time (hr)	Inflow I (cumec)	Outflow O (cumec)	I-O (cumec)	Mean storage (cumec-day)	Cumulative Storage (cumec-day)	X = 0.2			X = 0.25			X = 0.3		
						0.2 I	0.80	Total (cumec)	0.25 I	0.75 O	Total (cumec)	0.3 I	0.7 O	Total (cumec)
0	35	39	-4	-2	-2	7	31.2	38.2	8.75	29.25	38.0	10.5	27.3	37.8
24	125	52	73	34	32	25	41.6	66.6	31.25	39.0	70.25	37.4	36.4	73.9
48	575	287	288	180	212	115	229.6	344.6	143.75	215	358.75	172.5	200.9	373.4
72	740	624	116	202	414	148	499.2	647.2	185.0	468	653.0	222.0	436.8	658.8
96	456	638	-182	-33	381	91.2	510.4	601.6	114.0	478	592.0	136.8	446.6	583.4
120	245	394	-149	-166	216	49	315.2	364.2	61.25	295.5	356.75	73.5	275.8	349.3
144	144	235	-91	-120	96	28.8	188.0	216.8	36.0	176.3	212.3	43.2	164.5	207.7
168	95	142	-47	-69	27	19.0	113.6	132.6	23.75	101.64	125.39	28.5	99.4	127.9
192	67	93	-26	-37	-10	13.4	74.4	87.8	16.75	69.7	86.45	20.1	65.1	85.2
216	50	60	-10	-18	-28	10	48.0	58.0	12.5	45.0	57.5	15.0	42.0	57.0



70 The inflow hydrograph readings for a stream reach are given below for which the Muskingum coefficients of K = 36 hr and x = 0.15 apply. Route the flood through the reach and determine the outflow hydrograph. Also determine the reduction in peak and the time of peak of outflow.
Outflow at the beginning of the flood may be taken as the same as inflow.

Time (hr)	0	12	24	36	48	60	72	84	96	108	120
Inflow (cumec)	42	45	88	272	342	288	240	198	162	133	110
Time (hr)	132	144	156	168	180	192	204	216	228	240	
Inflow (cumec)	90	79	68	61	56	54	51	48	45	42	

$O_2 = C_0I_2 + C_1I_1 + C_2O_1$

x = 0.15, K = 36 hr = 1.5 day; take the routing period (from the inflow hydrograph readings) as 12 hr = 1/2 day. Compute C_0, C_1 and C_2 as follows:

$$C_0 = -\frac{Kx - 0.5t}{K - Kx + 0.5t} = -\frac{1.5 \times 0.15 - 0.5 \times \frac{1}{2}}{1.5 - 12 \times 0.15 + 0.5 \times \frac{1}{2}} = 0.02$$

$$C_1 = \frac{Kx + 0.5t}{K - Kx + 0.5t} = \frac{1.5 \times 0.15 + 0.5 \times \frac{1}{2}}{1.5 - 12 \times 0.15 + 0.5 \times \frac{1}{2}} = 0.31$$

$$C_2 = \frac{K - Kx - 0.5t}{K - Kx + 0.5t} = \frac{15 - 1.5 \times 0.15 - 0.5 \times \frac{1}{2}}{1.5 - 12 \times 0.15 + 0.5 \times \frac{1}{2}} = 0.67$$

Check: $C_0 + C_1 + C_2 = 0.02 + 0.31 + 0.67 = 1$

$O_2 = 0.02\ I_2 + 0.31\ I_1 + 0.67\ O_1$

In the table, I_1, I_2 are known from the inflow hydrograph, and O_1 is taken as I_1 at the beginning of the flood since the flow is almost steady.

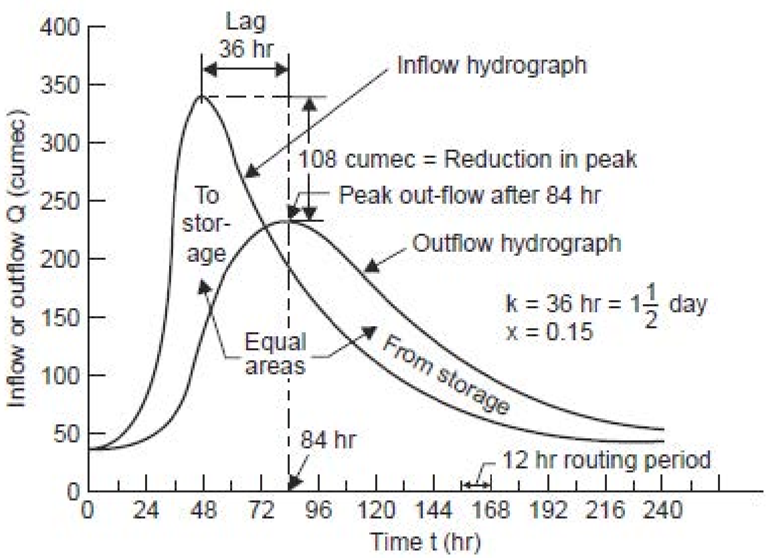
Time (hr)	Inflow I (cumec)	$0.02\ I_2$ (cumec)	$0.31\ I_1$ (cumec)	$0.67\ O_1$ (cumec)	Outflow O (cumec)
0	42	-	-	-	42*
12	45	0.90	13.0	28.2	42.1
24	88	1.76	14.0	28.3	44.0
36	272	5.44	27.3	29.5	62.2
48	342	6.84	84.3	41.7	132.8
60	288	5.76	106.0	89.0	200.7
72	240	4.80	89.2	139.0	233.0
84	198	3.96	74.4	156.0	234.0
96	162	3.24	61.4	157.0	221.6
108	133	2.66	50.2	148.2	201.0
120	110	2.20	41.2	134.5	178.9
132	90	1.80	34.1	119.8	155.7

144	79	1.58	27.9	104.0	133.5
156	68	1.36	24.4	89.5	115.3
163	61	1.22	21.1	77.4	99.7
180	56	1.12	18.9	66.8	86.8
192	54	1.08	17.4	58.2	76.7
204	51	1.02	16.7	51.4	69.1
216	48	1.00	15.8	46.3	63.1
228	45	0.90	14.8	42.3	58.0
240	42	0.84	13.9	38.9	53.6

*O_1 is assumed equal to I1 = 42 cumec

$O_2 = 0.02 \times 45 + 0.31 \times 42 + 0.67 \times 42 = 42.06$ cumec

This value of O_2 becomes O_1 for the next routing period and the process is repeated till the flood is completely routed through the reach. The resulting outflow hydrograph is plotted as shown in the figure. The reduction in peak is 108 cumec and the lag time is 36 hr, i.e., the peak outflow is after 84 hr (= 3.5 days) after the commencement of the flood through the reach.



71. The following is a record of the mean monthly discharges of a river in a dry year. The available fall is 80 m.

Determine

(i) the minimum capacity of a reservoir if the entire annual inflow is to be drawn off at a uniform rate (with no flow going into waste over the spillway).

(ii) the amount of water which must be initially stored to maintain the uniform draw off.

(iii) the uniform power output assuming a plant efficiency of 70%.

(iv) If the amount of water initially stored is 125 Mm^3, the maximum possible draw off rate and the amount of water wasted over the spillway (assuming the same reservoir capacity determined in (i) above.

(v) if the largest reservoir that can be economically constructed is of capacity 125 Mm^3, the maximum possible output and the amount of water wasted over the spillway.

(vi) the capacity of the reservoir to produce 22.5 megawatts continuously throughout the year.

month	Mean flow (cumec)	month	Mean flow (cumec)
Jan	29.7	July	68.0
Feb	75.3	Aug	50.2
March	66.8	Sept	74.5
April	57.2	Oct	66.8
May	23.2	Nov	40.5
june	26.3	dec	26.3

Take each month as 30 days for convenience; 1 month = 30 days × 86400 sec = 2.592 × 10^6 sec. Inflow volume in each month = monthly discharge × 2.592 Mm^3; and monthly inflow and cumulative inflow are tabulated in the table.

month	Mean flow (cumec)	Inflow volume (Mm^3)	Cumulative inflow (Mm^3)	month	Mean flow (cumec)	Inflow volume (Mm^3)	Cumulative inflow (Mm^3)
Jan	29.7	77	77	July	68.0	176	897
Feb	75.3	195	272	Aug	50.2	130	1027
March	66.8	173	445	Sept	74.5	193	1220
April	57.2	148	593	Oct	66.8	173	1393
May	23.2	60	653	Nov	40.5	105	1498
June	26.3	68	721	dec	26.3	68	1566

Plot the mass curve of flow as cumulative inflow vs month as shown in the figure.

(i) Join OA by a straight line; the slope of OA, i.e., 1566 Mm^3/yr or (1566 × 10^6)m^3/(365 × 86400) sec = 49.7 cumec is the uniform draw off throughout the year with no spill over the spillway. Draw BC | | OA, GH | | OA, B, G being the crests of the mass curve; EH = FG

Minimum capacity of reservoir = DE + EH = 150 + 20 = 170 Mm^3

Note If the capacity is less than this, some water will be wasted and if it is more than this, the reservoir will never get filled up.

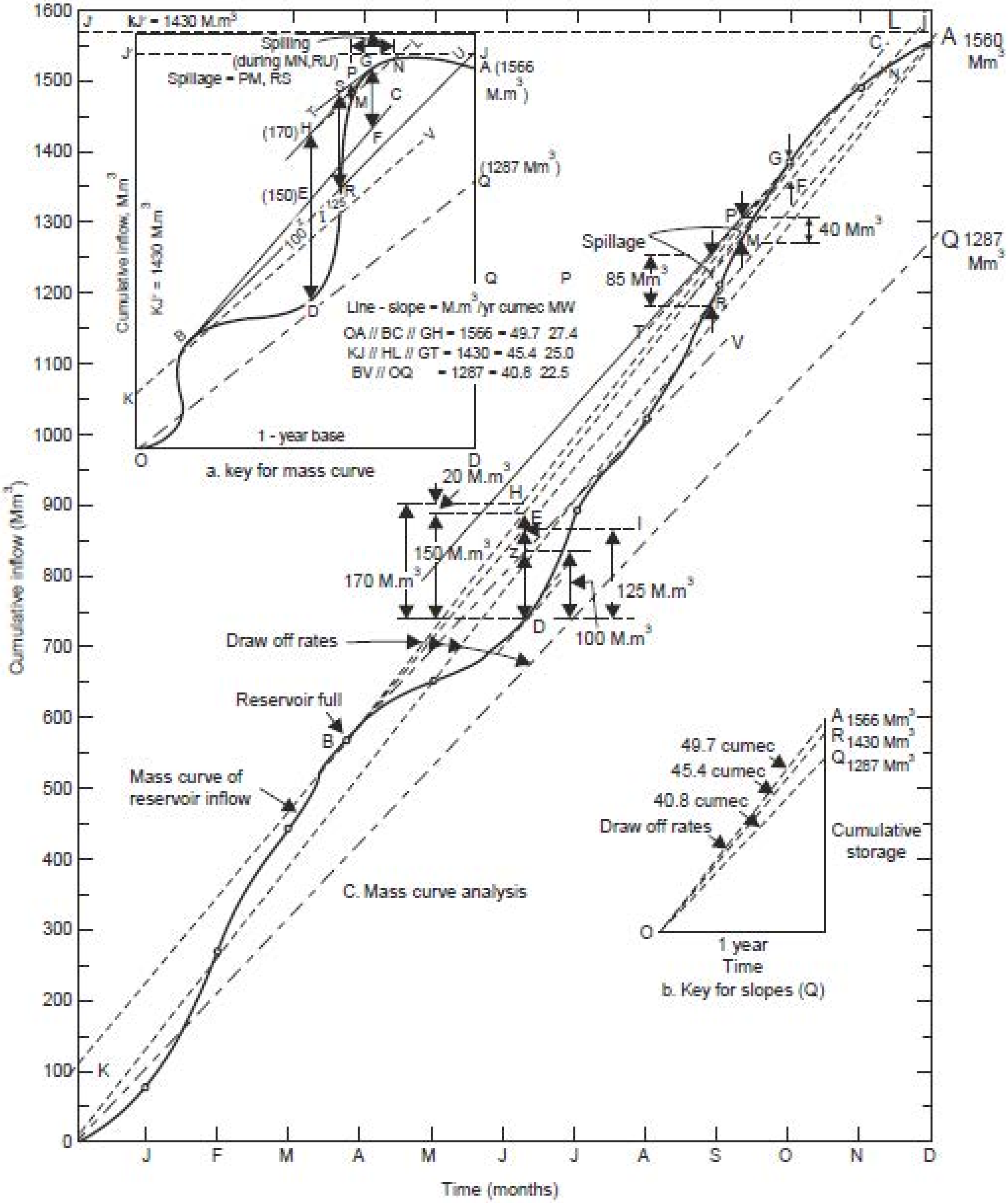
(ii) Amount of water to be initially stored for the uniform draw off of 49.7 cumec = DE= 150 Mm^3

$$(iii)\ \text{Continuous uniform power output in kW, } P = \frac{\rho_w g Q H}{1000} \times \eta_0$$

$$P = \frac{1000 \times 9.81 \times 49.7 \times 80}{1000} \times 0.70 = 27.4\ MW$$

(iv) If the amount of water initially stored is only 125 M.m^3, measure DI = 125 M.m^3, join BI and produce to J. The slope of the line BJ is the maximum possible draw off rate. Let the line BJ intersect the ordinate through O (i.e., the cumulative inflow axis) at K. The vertical intercept KJ′ = 1430 Mm^3 and the slope of this line = 1430 Mm^3/yr = 45.4 cumec which is the maximum possible draw off rate.

OMICS Group eBooks



To maintain the same reservoir capacity of 170 M.m³, draw the straight line HL || KJ intersecting the mass curve of flow at M and N. Draw the straight line GT || HL. The vertical intercept PM gives the amount of water wasted over the spillway (during the time period MN) which is 40 Mm³.

(v) If the reservoir capacity is limited to 125 $M.m^3$ from economic considerations, the line KJ intersects the mass curve of flow at R. Let the vertical at R meet the line GT (GT | | KJ) at S. In this case the amount of water wasted over the spillway = RS = 85 Mm^3. The maximum possible output in this case for a uniform draw off rate of 45.4 cumec is

$$P^{'} = 27.4 \times \frac{45.4}{49.7} = 25\,MW$$

(vi) For a continuous power output of 22.5 MW the uniform draw off rate can be determined from the equation

$$22500\,kW = \frac{1000 \times 9.81 \times Q \times 80}{1000} \times 0.70 \rightarrow Q = 40.8\,cumec$$

which can also be calculated as 49.7 ×22 5/27 4= 40.8 cumec = 40.8 (365 × 86400 sec) = 1287 Mm^3/yr.

On the 1-year base, draw the ordinate at the end of December = 1287 $M.m^3$ and join the line OQ (dashed line). The slope of this line gives the required draw off rate (40.8 cumec) to produce a uniform power output of 22.5 mW. Through B and D, i.e., the crest and the trough draw tangents parallel to the dashed line OQ (BV | | OQ). The vertical intercept between the two tangents DZ gives the required capacity of the reservoir as 100 Mm^3.

72. The following data are obtained from the records of the mean monthly flows of a river for 10 years. The head available at the site of the power plant is 60 m and the plant efficiency is 80%.

Mean monthly flow range (cumec)	No. of occurrences (in 10 yr period)
100-149	3
150-199	4
200-249	16
250-299	21
300-349	24
350-399	21
400-499	20
450-499	9
500-549	2

(a) Plot

(i) The flow duration curve (ii) The power duration curve

(b) Determine the mean monthly flow that can be expected and the average power that can be developed.

(c) Indicate the effect of storage on the flow duration curve obtained.

(d) What would be the trend of the curve if the mean weekly flow data are used instead of monthly flows.

The mean monthly flow ranges are arranged in the ascending order as shown in the table. The number of times that each mean monthly flow range (class interval, C.I.) has been equalled or exceeded (m) is worked out as cumulative number of occurrences starting from the bottom of the column of number of occurrences, since the C.I. of the monthly flows, are arranged in the ascending order of magnitude. It should be noted that the flow values are arranged in the ascending order of magnitude in the flow duration analysis, since the minimum continuous flow that can be expected almost throughout the year (i.e., for a major percent of time) is required particularly in drought duration and power duration studies, while in flood flow analysis the CI may be arranged in the descending order of magnitude and m is worked out from the top as cumulative number of occurrences since the high flows are of interest. The percent of time that each CI is equalled or exceeded is worked out as the percent of the total number of occurrences (m) of the particular CI out of the 120 (= 10 yr × 12 = n) mean monthly flow values, i.e., =(m/n)× 100. The monthly power developed in megawatts,

Mean monthly flow class interval C.I. (cumec)	*No. of occurrences (in 10-yr period)*	*No. of time equalled or exceeded (m)*	*Percent of time lower value of CI equalled or exceeded* $= \frac{m}{n} \times 100\%$	*Monthly power P = 0.472 Q (MW) Q = lower value of C.I.*
100-149	3	120	100	47.2
150-199	4	117	97.5	70.8
200-249	16	113	94.2	94.4
250-299	21	97	80.8	118
300-349	24	76	63.3	142
350-399	21	52	43.3	165
400-499	20	31	25.8	189
450-499	9	11	9.2	212
500-549	2	2	1.7	236
	Total n = 120			

Note: For drought–duration studies, m = No. of times equal to or less than the flow value and has to be worked from the top; percent of time ≤ the flow value is $\frac{m}{n} \times 100$. In this example, m = 3, 7, 23, 44, ... and % of time ≤ Q are 2.5, 5.83, 19.2, 36.7. ...., respectively (from top).

$$P = \frac{gQH}{1000} \times \eta_0 = \left(\frac{9.81 \times 60}{1000} \times 0.80 \right) Q$$

where Q is the lower value of the CI Thus, for each value of Q, P can be calculated.

(i) The flow duration curve is obtained by plotting Q vs. percent of time, (Q = lower value of the CI).

(ii) The power duration curve is obtained by plotting P vs. percent of time.

(b) The mean monthly flow that can be expected is the flow that is available for 50% of the time i.e., 357.5 cumec from the flow duration curve drawn. The average power that can be developed i.e., from the flow available for 50% of the time, is 167 MW, from the power duration curve drawn.

(c) The effect of storage is to raise the flow duration curve on the dry weather portion and lower it on the high flow portion and thus tends to equalise the flow at different times of the year, as indicated in the figure.

(d) If the mean weekly flow data are used instead of the monthly flow data, the flow duration curve lies below the curve obtained from monthly flows for about 75% of the time towards the drier part of the year and above it for the rest of the year as indicated in the figure.

In fact the flow duration curve obtained from daily flow data gives the details more accurately (particularly near the ends) than the curves obtained from weekly or monthly flow data but the latter provide smooth curves because of their averaged out values. What duration is to be used depends upon the purpose for which the flow duration curve is intended.

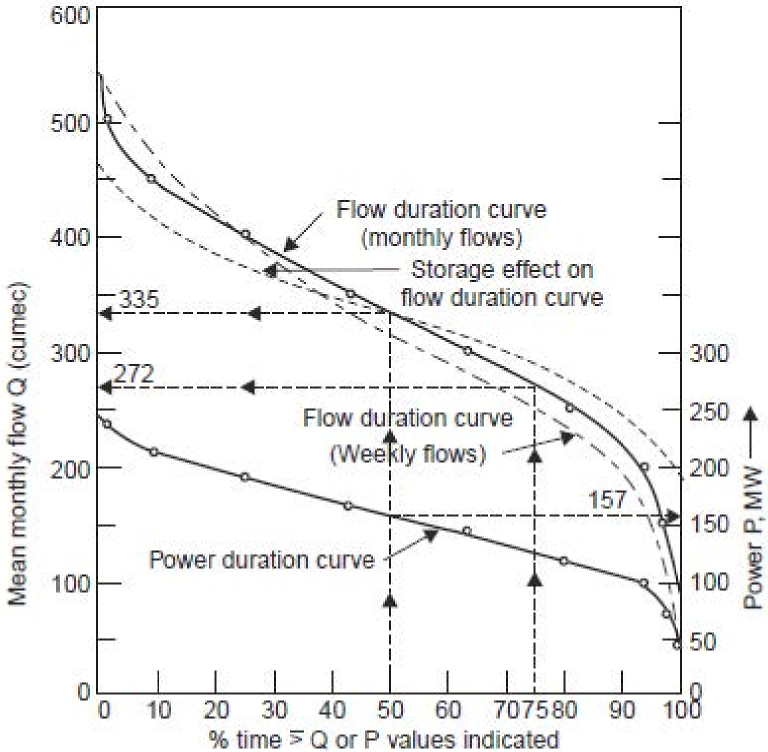
St. No 1	Year 2	Cumec 3	June 4	Dt. 5	July 6	Dt. 7	Aug. 8	Dt. 9	Sept. 10	Dt. 11	Oct. 12	Dt. 13
1	1885	Q			5814	16	7241	9				
		x			1481		2908					
2	1886	Q			4665	15	9163	5	6018	2		
		x			332		4831		1685			
		Q			4848	22						
		x			515							
		Q			4369	27						
		x			36							
3	1887	Q					5882	2				
		x					1549					
		Q					7404	22				
		x					3074					
4	1888	Q			5417	22	4848	2	5417	3		
		x			1084		515		1084			
		Q			5034	27			6870	20		
		x			701				2537			
		Q			4909	31						
		x			576							
5	1889	Q	5680	25	7936	7	7546	3	4971	9		
		x	1374		3603		3213		638			
		Q			4848	17	9855	15				
		x			515		5522					
		Q			8827	29	7857	29				
		x			4494		3524					
6	1890	Q			5841	19						
		x			1481							
		Q			11887	30	6508	7				

		x			7554		2175					
		Q					9249	19				
		x					4916					
		Q					5417	31				
		x					1084					
7	1891	Q			4786	26	8827	14				
		x			453		4494					
8	1892	Q			4605	26	5747	15	4971	3		
		x			272		1414		608			
		Q					7546	25				
		x					3213					
9	1893	Q	5287	29	5482	26	6087	4	8498	5	5128	19
		x	955		1149		1754		4165		795	
		Q					6870	10				
		x					2537					
10	1894	Q			5160	8	10032	6	7376	6		
		x			827		5699		3043			
		Q			6018	21	9680	12				
		x			1685		5347					
		Q			9767	27	16757	26				
		x			5434		12424					
		Q			13179	30						
		x			8846							
11	1895	Q			5747	2	9680	13				
		x			1414		5347					
12	1896	Q					14336	10				
		x					10003					
13	1897	Q			7623	22	7407	5	4427	3		
		x			3290		3074		94			
		Q					8174	23				
		x					3841					
14	1898	Q			5814	10	8953	12	6366	7		
		x			1481		4620		2033			
		Q			5882	24	7241	18				
		x			1549		2908					
15	1899	Q			7546	23						
		x			3213							
16	1900	Q			4486	17	6651	11	5950	6		
		x			153		2318		1617			
		Q					5097	26	4786	12		
		x					764		453			
17	1901	Q			4427	19	7700	8				
		x			94		3367					
		Q					11409	24				
		x					7076					
18	1902	Q					9163	3	4848	14		
		x					4830		515			

19	1903	Q					7407	14				
		x					3074					
		Q					6296	27				
		x					1963					
20	1904	Q			8579	29	7092	3				
		x			4246		2759					
		Q					7091	12				
		x					2758					
		Q					7017	21				
		x					2684					
21	1905	Q					9362	13				
		x					5029					
22	1906	Q			7092	28	6870	3	6226	16		
		x			2759		2537		1893			
		Q					5950	19				
		x					1617					
23	1907	Q			7546	30	7241	3				
		x			3213		2908					
24	1908	Q			6651	13	11504	2				
		x			2318		7171					
		Q					5949	21				
		x					1616					
		Q					6018	29				
		x					1685					
25	1909	Q			6870	16	7407	12				
		x			2537		3074					
		Q			8335	21	6724	20				
		x			4002		2391					
		Q					5949	27				
		x					1616					
26	1910	Q			10121	28	7469	3	6156	11	15077	3
		x			5788		3136		1823		10744	
		Q					11887	13				
		x					7554					
		Q					7091	18				
		x					2759					
27	1911	Q					6943	18	4369	13		
		x					2610		36			
28	1912	Q					7700	16	8335	2		
		x					3367		4002			
29	1914	Q			5417	2			9249	19		
		x			1084				4916			
		Q			6018	28						
		x			1685							
30	1915	Q					6579	2				
		x					2246					
		Q					7407	13				

		x					3074					
		Q					4725	29				
		x					392					
31	1916	Q					4725	26				
		x					392					
32	1917	Q			8416	11	5352	5	5482	9		
		x			4083		1019		1149			
		Q			6870	25	4909	23				
		x			2537		576					
33	1918	Q					4665	22				
		x					332					
34	1919	Q			6296	13			5160			
		x			1963				827			
35	1920	Q			4848	23	8174	14				
		x			515		3841					
36	1921	Q	5680	26			7623	4	6508	9		
		x	1347				3290		2175			
							9079	18	5950	15		
							4746		1617			
37	1922	Q			5814	20	7407	17	5034	3		
		x			1481		3074		701			
38	1923	Q					5482	5				
		x					1149					
39	1924	Q			5160	27	6087	3	4848	5		
		x			827		1754		515			
		Q					5097	19	19136	29		
		x					764		14803			
40	1925	Q			5160	21	9670	12				
		x			827		5347					
41	1927	Q					7236	5				
		x					2903					
		Q					7241	20				
		x					2908					
42	1929	Q					4545	15				
		x					212					
43	1930	Q			5443	27	5997	21				
		x			1110		1664					
44	1932	Q					5532	14				
		x					1199					
							6155	24				
							1822					
45	1933	Q			4692	25	5267	15				
		x			359		934					
46	1934	Q					6193	21				
		x					1860					
47	1935	Q					5289	4				
		x					956					

48	1942	Q			4887	24	6650	9				
		x			554		2317					
49	1943	Q					4442	22				
		x					109					
50	1945	Q					4836	19	5101			
		x					503		768			
51	1946	Q			4629	28						
		x			296							
52	1947	Q							4345			
		x							12			
53	1948	Q					4890	25				
		x					557					
54	1950	Q			4562	26	5899	18				
		x			229		1566					
55	1951	Q					4458	22	4339			
		x					125		6			
56	1953	Q					5470	13				
		x					1137					
57	1954	Q					5978	20				
		x					1645					
58	1955	Q							4644	5		
		x							311			
59	1956	Q							6381	11		
		x							2048			
60	1957	Q							4548	15		
		x							213			
61	1959	Q					4493	3				
		x					160					
62	1961	Q					4855	17				
		x					522					
63	1962	Q			5760	28						
		x			1427							
64	1963	Q					5574	21	9192	17		
		x					1241		4859			
65	1966	Q			4741	26						
		x			408							
66	1967	Q					5919	27				
		x					1586					
67	1969	Q					4546	20				
		x					213					
68	1971	Q					4542	7				
		x					209					



73. The available flow for 97% of the time (i.e., in a year) in a river is 30 cumec. A run-of-river plant is proposed on this river to operate for 6 days in a week round the clock. The plant supplies power to a variable load whose variation is given below:

Period (hr)	*0–6*	*6–12*	*12–18*	*18–24*
$\frac{\textit{Load during period}}{\textit{24-hr average load}}$ *ratio*	*0.6*	*1.4*	*1.5*	*0.5*

The other relevant data are given below:

Head at full pond level = *16 m*

Maximum allowable fluctuation of pond level = *1 m*

Plant efficiency = *80%*

Pondage to cover inflow fluctuations = *20% of average daily flow*

Pondage to cover wastage and spillage = *10%*

Determine:

(i) the average load that can be developed

(ii) daily load factor

(iii) plant capacity

(iv) weekly energy output

(v) pondage required and the surface area of the pond for satisfactory operation

(i) 7 days flow has to be used in 6 days

Average flow available for power development

$$Q = 30 \times \frac{7}{6} = 35\, cumec$$

Since maximum allowable fluctuation of pond level is 1 m, average head

$$H = \frac{16+15}{2} = 15.5\, m$$

The average load that can be developed

$$P = \frac{gQH}{1000} \times \eta_0 = \left(\frac{9.81 \times 35 \times 15.5}{1000} \times 0.80 \right) = 4.27\, MW$$

$$(ii)\, Daily\ load\ \ factor = \frac{average\, load}{peak\, load} = \frac{1}{1.5} = 0.67$$

(iii) Plant capacity = 4.27 × 1.5 = 6.4 MW

(iv) Weekly energy output = Average load in kW × No. of working hours = (4.27 × 1000)(6 × 24) = 6.15 × 10^5 kWh

It should be noted that the installed capacity has to be equal to the peak load and the number of units (kWh) generated will be governed by the average load.

(v) Pondage required

(a) to store the idle day's flow = 30 × 86400 = 2.592 × 10^6 m^3, or 2.592 Mm^3

(b) to store the excess flow during low loads to meet the peak load demand. Since power developed is proportional to discharge (assuming constant average head of 15.5 m), flow required during peak load periods of 6.00 to 12.00 hr is (1.4 – 1) 35 cumec and from 12.00 to 18.00 hr is (1.5 – 1) 35 cumec.

pondage to meet peak load demand= (0.4 + 0.5) 35 cumec for 6 hr= (0.9 × 35)(6 × 60 × 60)= 6.81 × 10^5 m^3

(c) pondage to cover inflow fluctuations= (0.20 × 30) 86400= 5.18 × 10^5 m^3

Total of (a), (b) and (c) = 3.791 Mm^3

Add 10% for wastage and spillage = 0.379 Mm^3

Total pondage required = 4.170 Mm^3

Since the maximum fluctuation of pond level is 1 m
the surface area of pond = 4.170 × 106 m^2

74. A run-of-river hydroelectric plant with an effective head of 22 m and plant efficiency of 80% supplies power to a variable load as given below:

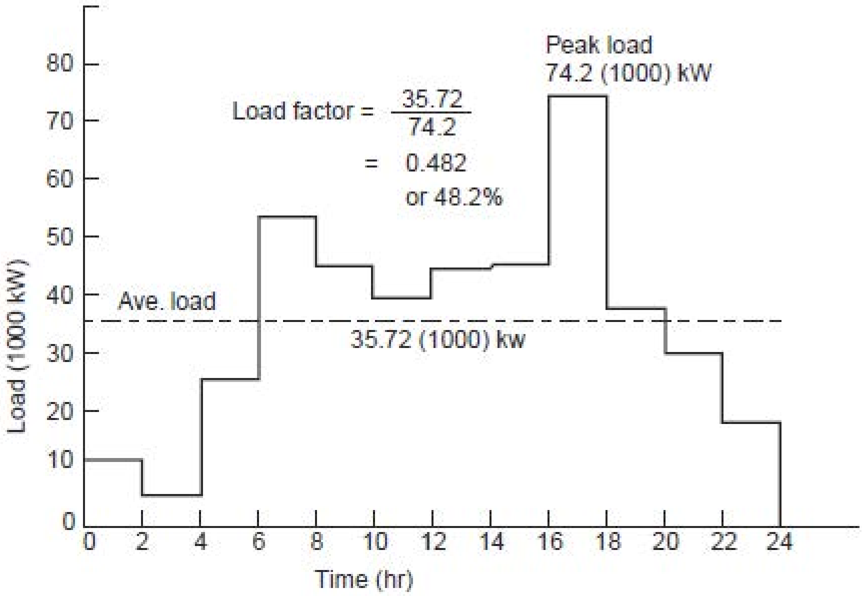
	Time (hr)	Load (1000 Kw)	Time (hr)	Load (1000 Kw)
MN	0-2	11.4	12-14	44.2
	2-4	5.6	14-16	44.4
	4-6	25.6	16-18	74.2
	6-8	53.2	18-20	37.8
	8-10	44.8	20-22	30.0
N	10-12	39.4	22-24	18.0

Draw the load curve and determine:

(i) the minimum average daily flow to supply the indicated load.

(ii) pondage required to produce the necessary power at the peak.

(iii) the plant load factor.



Total of loads at 2 hr intervals = 428.6 kW

$$Average\ load = \frac{428.6 \times 1000kW \times 2hr}{24\ hr} = 35.72 \times 1000\ kW$$

Flow (Q) required to develop the average load

$$\frac{1000 \times 9.81 \times Q \times 22}{1000} \times 0.8 = 35.72 \times 1000\ \text{kW} \rightarrow \text{Q} = 207\ \text{cumec}$$

(ii) Flow required to produce the required load demand

$$Q = \frac{207}{35.72} \times Load\ in\ 1000\ kW$$

To determine the pondage capacity the following table is prepared:

	Time (hr)	Load (1000 kW) P	Required flow (cumec) 5.8 P	Deviation from the average flow of 207 cumec)	
				Deficiency (cumec)	Excess (cumec)
MN	0-2	11.4	66.10		140.90
	2-4	5.6	32.46		174.54
	4-6	25.6	148.40		58.60
	6-8	53.2	308.20	101.20	
	8-10	44.8	260.00	53.00	
N	10-12	39.4	228.50	21.50	

	12-14	44.2	256.00	49.00	
	14-16	44.4	257.00	50.40	
	16-18	74.2	430.00	223.00	
	18-20	37.8	219.40	12.40	
	20-22	30.0	174.00		33.00
	22-24	18.0	104.30		102.70
	TOTAL	428.6		510.50	509.74

Total deficiency = total excess = 510 cumec

Pondage capacity required = 510 cumec for 2 hr= 510 (2 × 60 × 60) = 3.67 Mm^3

$$(iii)\ Plant\ Load\ \ factor = \frac{average\ peak}{peak\ load} = \frac{35.72}{74.20} = 48.2\%$$

75. A proposed reservoir has a capacity of 400 ha-m. The catchment area is 130 km^2 and the annual stream flow averages 12.31 cm of runoff. If the annual sediment production is 0.03 ha-m/km^2, what is the probable life of the reservoir before its capacity is reduced to 20% of its initial capacity by sediment deposition. The relation between trap efficiency and capacity-inflow ratio is given below.

Capacity-inflow ratio, $\frac{C}{I}$	*Trap efficiency,* η_{trap} *(%)*	*Capacity-inflow ratio,* $\frac{C}{I}$	*Trap efficiency,* η_{trap} *(%)*
0.1	87	0.002	2
0.2	93	0.003	13
0.3	95	0.004	20
0.4	95.5	0.005	27
0.5	96	0.006	31
0.6	96.5	0.007	36
0.7	97	0.008	38
1.0	97.5	0.01	43
		0.015	52
		0.02	60
		0.03	68
		0.04	74
		0.05	77
		0.06	80
		0.07	82

The useful life may be computed by determining the number of years required for each incremental loss of reservoir capacity (i.e., for the decreasing values of capacity-inflow ratios) upto the critical storage volume of 400 × 0.20 = 80 ha-m as tabulated below:

Capacity C (ha-m)	Capacity* inflow ratio $\frac{C}{I}$	Trap efficiency η_{trap} (%) for the $\frac{C}{I}$ ratio	Trap efficiency η_{trap} (%) Ave. for *increment	Annual** sediment trapped $V_s = Q_s \times \eta_{trap}$	Loss of reservoir capacity ΔC (ha-m)	No. of years for the capacity loss $\Delta C \div V_s$
400	0.25	94				
320	0.20	93	93.5	3.64	80	22.0
240	0.15	90	91.5	3.57	80	22.4
160	0.10	87	88.5	3.45	80	23.2
80	0.05	77	82.0	3.20	80	25.0
						Total = 92.6 say, 93 yr

*Average annual inflow, $I = \frac{12.31}{100} \times \frac{130 \times 10^6}{10^4} = 1600$ ha-m

For reservoir capacity $C = 400$ ha-m, $\frac{C}{I} = \frac{400}{1600} = 0.25$

**Annual sediment inflow into the reservoir

$$Q_s = 0.03 \times 130 = 3.9 \text{ ha-m}$$

Note: If the average annual sediment inflow Q_s is given in tons, say $Q_s = 43600$ tons and for $\eta_{trap} = 93.5\%$ (for the first incremental loss), assuming a specific gravity of 1.12 for the sediment deposits, annual sediment trapped $W_s = 43600 \times 0.935 = 40750$ tons.

$$V_s = \frac{W_s}{\gamma_s} = \frac{40750 \times 1000 \text{ kg}}{1.12 \times 1000 \text{ kg/m}^3} = \frac{40750}{1.12} \text{ m}^3 = 3.64 \text{ ha-m.}$$

Usually the specific gravity of sediments deposits ranges from 1 to 1.4.

76. Annual rainfall and runoff data for the Damodar river at Rhondia (east India) for 17 years (1934-1950) are given below. Determine the linear regression line between rainfall and runoff, the correlation coefficient and the standard error of estimate.

year	Rainfall (mm)	Runoff(mm)
1934	1088	274
35	1113	320
36	1512	543
37	1343	437
38	1103	352
39	1490	617
40	1100	328
41	1433	582
42	1475	763
43	1380	558
44	1178	492

45	1223	478
46	1440	783
47	1165	551
48	1271	565
49	1443	720
1950	1340	730

The regression line computations are made in the table and is given by
R = 0.86 P – 581
where P = rainfall (mm) and R = runoff (mm)

Rainfall (mm) x	*Runoff (mm) y*	$x^2 \times 10^4$	$xy \times 10^4$	$\Delta x = x - \bar{x}$	$\Delta y = y - \bar{y}$	$(\Delta x)^2 \times 10^4$	$(\Delta y)^2 \times 10^4$	$\Delta x.\Delta y \times 10^4$	*Working*
1088	274	118.4	29.8	– 212	– 261	4.50	6.81	5.54	$\bar{x} = \frac{\Sigma x}{n} = \frac{22097}{17} = 1300$ mm
1113	320	124.0	35.6	– 187	– 215	3.50	4.63	4.02	
1512	543	229.0	82.2	+ 212	+ 8	4.50	0.01	0.17	$\bar{y} = \frac{\Sigma y}{n} = \frac{9093}{17} = 535$ mm
1343	437	180.2	58.8	+ 43	– 98	0.18	0.96	0.42	
1103	352	122.0	38.9	– 197	– 183	3.88	3.35	3.61	Normal equations:* Eqs. (13.2)
1490	617	222.0	92.0	+ 190	+ 82	3.62	0.67	1.56	$9093 = 17\,a + 22097\,b$ (*i*)
1100	328	121.0	36.0	– 200	– 207	4.00	4.28	4.14	$1213.3 \times 10^4 = 22097\,a$
1433	582	205.0	83.5	+ 133	+ 47	1.77	0.22	0.65	$+ 2910 \times 10^4 b$
1475	763	217.0	112.6	+ 175	+ 228	3.06	5.10	3.99	Dividing throughout by 10^4,
1380	558	190.5	77.0	+ 80	+ 23	0.64	0.05	0.18	$1213 = 2.2\,a + 2910b$ (*ii*)
1178	492	138.6	57.8	– 122	– 43	1.49	0.19	0.52	Solving (*i*) and (*ii*)
1223	478	149.8	58.5	– 77	– 57	0.59	0.33	0.44	$a = -581,\ b = 0.86$
1140	783	207.0	112.8	+ 140	+ 248	1.96	6.15	3.48	
1165	551	136.0	64.2	– 135	+ 16	1.82	0.03	0.22	∴ Regression line is
1271	565	162.0	71.8	– 29	+ 30	0.08	0.09	0.09	$y = 0.86x - 581$
1443	720	208.0	104.0	+ 143	+ 185	2.05	3.43	2.65	or $R = 0.86P - 581$
1340	730	179.5	97.8	+ 40	+ 195	0.16	3.80	0.78	where *R* and *P* are in mm
Σ = 22097	9093	2910×10^4	1213.3×10^4			37.81×10^4	40.09×10^4	32.46×10^4	

*Matrix form for computer solution

$$y = a + bx$$

$$\begin{bmatrix} \Sigma i & \Sigma x \\ \Sigma x & \Sigma x^2 \end{bmatrix} \begin{bmatrix} a \\ b \end{bmatrix} = \begin{bmatrix} \Sigma y \\ \Sigma xy \end{bmatrix}, \quad \Sigma i = n \text{ (or } m\text{)}$$

$$b = \frac{\Sigma \Delta x . \Delta y}{\sqrt{\Sigma \Delta x^2 . \Sigma \Delta y^2}} = \frac{32.46 \times 10^4}{37.81 \times 10^4} = 0.86$$

$$a = \bar{y} - b\bar{x} = 535 - 0.86 \times 1300 = -581$$

Correlation coefficient

$$r = \frac{\Sigma \Delta x . \Delta y}{\sqrt{\Sigma \Delta x^2 . \Sigma \Delta y^2}}$$

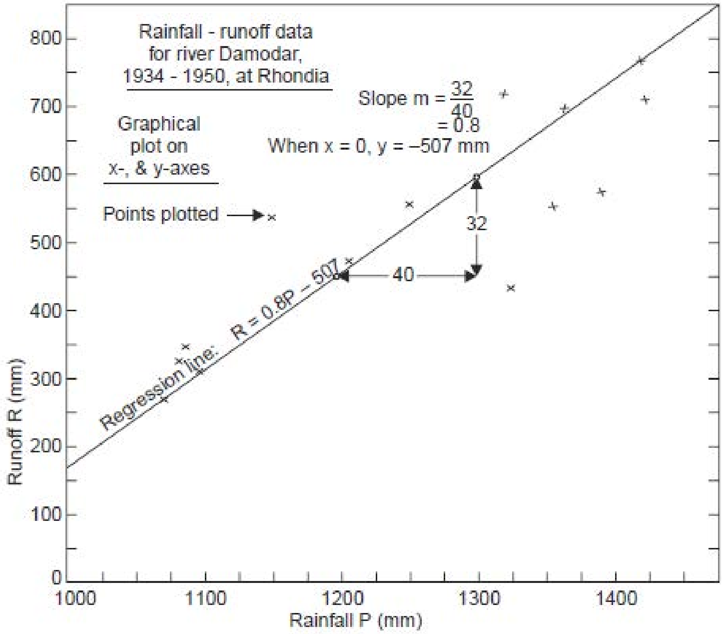
$$= \frac{32.46 \times 10^4}{\sqrt{37.81 \times 40.09 \times 10^8}} = 0.835$$

also, $b = r\left(\frac{\sigma_y}{\sigma_x}\right) = r\sqrt{\frac{\Sigma \Delta y^2}{\Sigma \Delta x^2}}$

$$0.86 = r\sqrt{\frac{40.09 \times 10^4}{37.81 \times 10^4}}$$

$$r = 0.835$$

The correlation coefficient r = 0.835, which indicates a close linear relation and the straight line plot is shown in the figure, the relation is very close.



Standard error of estimate

$$S_{y,x} = \sigma_y \sqrt{1 - r^2}$$

$$\sigma_y = \sqrt{\frac{\sum \left(y - \bar{y}\right)^2}{n-1}} = \sqrt{\frac{\sum (\Delta y)^2}{n-1}} = \sqrt{\frac{40.10 \times 10^4}{17-1}}$$

$$S_{y,x} = 160\sqrt{1 - (0.835)^2} = 90.24\,mm$$

77. The following are the data of the monthly Ground Water Table (GWT) fluctuations, precipitation and ground water pumping in the Cauvery delta in Thanjavur, TN Obtain the regression line connecting GWT fluctuations with the precipitation and pumping.

month	GWR below MP (m)	Precipitation	G.W. pumping rate (Mm^3)
Jan	3.60	30	14.0
Feb	4.05	52	23.4
March	4.12	95	32.4
April	4.57	90	51.2
May	4.80	200	62.3
June	4.95	280	79.5
July	5.02	168	61.4
Aug	4.80	51	47.4

Sept	4.42	18	34.4
Oct	4.20	27	18.9
Nov	3.90	52	1.8
dec	3.30	57	5.2

The regression line computations are made in the table and the normal equations are given below:

$12a + 1120b + 432c = 51.73$

$1120a + 17.15 \times 10^4 b + 5.83 \times 10^4 c = 5138.8$

$432a + 5.83 \times 10^4 b + 1.68 \times 10^4 c = 1997.1$

Simultaneous solution of the three equations gives

$a = 4.02$, $b = 0.00865$, $c = -0.0144$

Month	GWT x_1 (m)	Precipitation x_2 (mm)	Pumping x_3 (Mm^3)	x_2^2	x_3^2	x_1x_2	x_1x_3	x_2x_3	$x_1-\bar{x}_1$	$(x_1-\bar{x}_1)^2$	$x_2-\bar{x}_2$	$(x_2-\bar{x}_2)^2$	$x_3-\bar{x}_3$	$(x_3-\bar{x}_3)^2$
Jan.	3.60	30	14.0	900	196	108	50.4	420	–0.71	0.504	–63.33	4000	–22	484
Feb.	4.05	52	23.4	2700	550	210	94.7	1217	–0.26	0.068	–41.33	1710	–12.6	160
Mar.	4.12	95	32.4	9030	1050	391	133.3	3080	–0.13	0.017	1.67	3	–3.6	13
April	4.57	90	51.2	8100	2620	412	234	4610	0.26	0.068	–3.33	11	15.2	230
May	4.80	200	62.3	40000	3890	960	299	12460	0.49	0.240	106.67	11370	26.3	700
June	4.95	280	79.5	78500	6320	1387	394	22240	0.64	0.410	186.67	34800	43.5	1890
July	5.02	168	61.4	28300	377	843	308	10300	0.71	0.504	74.67	5490	25.4	640
Aug.	4.80	51	47.4	2610	225	244	228	2420	0.49	0.240	42.33	1797	11.4	130
Sept.	4.42	18	34.4	325	1182	79.5	152	620	0.11	0.012	–75.33	5670	–1.6	3
Oct.	4.20	27	18.9	730	358	113.3	79.5	510	–0.11	0.012	–66.33	4400	17.1	294
Nov.	3.90	52	1.8	271	3.3	203	7	93.5	–0.41	0.168	–41.33	1710	–34.2	1180
Dec.	3.30	57	5.2	325	27.1	188	17.2	296	–1.01	1.020	–36.33	1320	–30.8	950
$\Sigma n = 12$	51.73	1120	431.9	17.15 × 10^4	1.68 × 10^4	5138.8	1997.1	5.83 × 10^4		3.263		72281		6674

$$\bar{x} = \frac{\Sigma x}{n},\ \bar{x}_1 = 4.31,\ \bar{x}_2 = 93.33,\ \bar{x}_3 = 36,\ \sigma = \sqrt{\frac{\Sigma(x-\bar{x})^2}{n-1}},\ \sigma_1 = 0.545,\ \sigma_2 = 81,\ \sigma_3 = 24.6$$

and the regression line is given by

$x_1 = 4.02 + 0.00865x_2 + 0.0144x_3$

or calling GWT as y (m), precipitation as P (mm) and pumping rate Q (Mm^3), the linear multiple regression line is given by

y = 4.02 + 0.00865 P + 0.0144 Q

from which the GWT corresponding to a known precipitation and pumping rate can be computed.

To compute the multiple correlation coefficient $r_{1.23}$

$$r_{12} = \frac{\sum x_1x_2 - n\overline{x_1}\,\overline{x_2}}{(n-1)\sigma_1\sigma_2} = \frac{5138.8 - 12(4.31)(93.33)}{(12-1)(0.545)81} = 0.66$$

$$r_{13} = \frac{\sum x_1x_3 - n\overline{x_1}\,\overline{x_3}}{(n-1)\sigma_1\sigma_3} = \frac{1997.1 - 12(4.31)36}{(12-1)(0.545)24.6} = 0.92$$

$$r_{23} = \frac{\sum x_2x_3 - n\overline{x_2}\,\overline{x_3}}{(n-1)\sigma_2\sigma_3} = \frac{5.83\times10^4 - 12(93.33)36}{(12-1)(81)24.6} = 0.82$$

$$r_{1.23} = \sqrt{\frac{0.66^2 + 0.92^2 - 2(0.66)(0.92)(0.82)}{1-(0.82)^2}} = 0.94$$

$r_{1.23}$ = 0.94 indicates a close linear correlation

$$r_{1.23} = \sqrt{1-\left(1-0.66^2\right)\left(1-0.92\right)^2} = 0.95$$

The standard error of estimate

$$S_{1.23} = \sigma_1\sqrt{1-r_{1.23}^2} = 0.545\sqrt{1-(0.94)^2} = 0.2$$

78. The API for a station was 50 mm on 1st July 1995; 40 mm rain fell on 6th July, 25 mm on 8th July and 30 mm on 9th July. Assuming a recession constant of 0.9, compute the API

(i) on 15th July.

(ii) on 15th July, assuming no rainfall during 1-15 July.

$I_t = I_0 K^t$, I_0 = 50 mm, K = 0.9

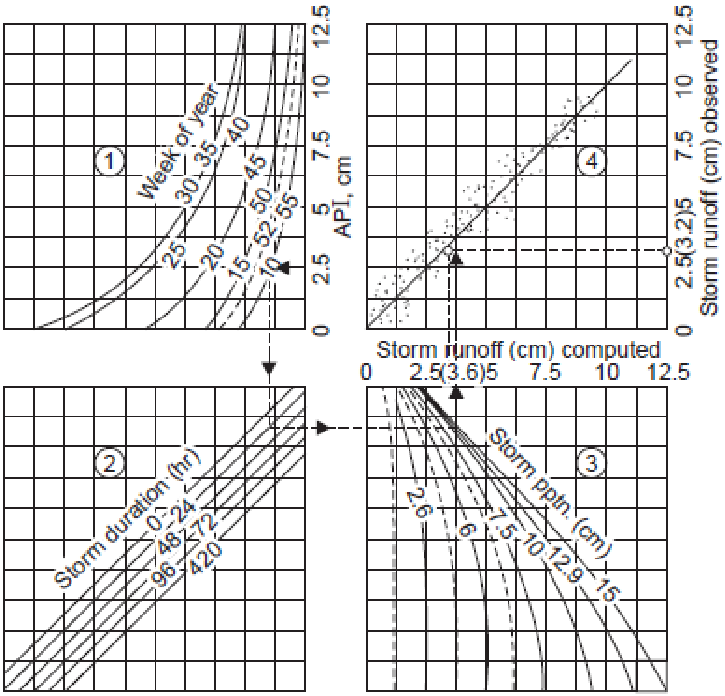
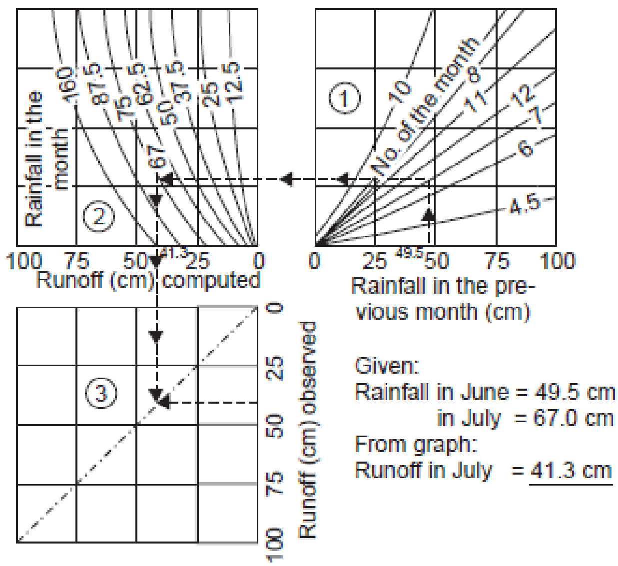


Fig. 13.2 Coaxial correlation for Monocacy river, USA (US National Weather Service)



on 6-July, $I_{6-1} = 50 \times 0.9^5 + 40 = 69.52$ mm

on 8-July, $I_{8-6} = 69.52 \times 0.9^2 + 25 = 81.31$ mm

on 9-July, $I_{9-8} = 81.31 \times 0.9 + 30 = 103.18$ mm

on 15-July, $I_{15-9} = 103.18 \times 0.9^6 = 54.84$ mm = API

(ii) $I_{15-1} = 50 \times 0.9^{14} = 11.44$ mm = API

Depending upon the API, the time of the year, duration and magnitude of the storm and the altitude, the estimation of runoff can be made by following the data as indicated by the dotted line on the graphical plot for the Monacacy river, USA. Thus, a catchment with an API of 2.5 cm, in the 10th week of the year with the occurrence of storm of 24 hr duration and 12 cm depth of precipitation, will yield a runoff of 3.6 cm. This graphical approach is called coaxial correlation and is preferred to the multivariate linear correlation since many complex characteristics of the basin and storm is involved.

Another coaxial graphical correlation for estimating the monthly runoff from a catchment of the river Kallada in south Kerala (south India) as given by Pillai N.N. (1964) is shown in the figure. Here the API has been taken as the precipitation of the previous month and the runoff for a particular month can be read on the graphical plot if the precipitation in the previous month is known. Thus, if the surface runoff in the month of July (7th month) is required, given the rainfalls for the months of June and July as 49.5 cm and 67.0 cm, respectively, then the runoff during the month of July is 41.3 cm, as indicated by the dotted line. If however, the total yield from the catchment is to be found out, the base flow (estimated as 30 cm) is to be added to the cumulative surface runoff of the whole year.

Though the correlation graph was developed for river Kallada (basin area = 874 km²) during 1952-57, it was also applied to compute the yield of river Pamba (basin area = 1700 km²) in 1953 and of river Achenkoil (basin area = 847 km²) in 1955 and was found to be within ±4% of the observed yield.

79. Rainfall (P) and Runoff (R) data for a small catchment are given below:

P (mm)	22	26	14	4	30	12
R (mm)	6	12	4	0	18	6

Develop a linear regression equation and find the coefficient of correlation; write a computer program in C-language.

R = aP + b   x = P, y = R, m = no. of data pairs = 6

$$a = \frac{m.\sum xy - \sum x.\sum y}{m.\sum x^2 - \left(\sum x\right)^2}$$

$$b = \frac{\sum y - a\sum x}{m}$$

$$r = \frac{m\sum xy - \sum x.\sum y}{\sqrt{\left[m\sum x^2 - \left(\sum x\right)^2\right]\left[m\sum y^2 - \left(\sum y\right)^2\right]}}$$

$\Sigma x = 108$, $\Sigma y = 46$, $(\Sigma x)^2 = 11664$, $(\Sigma y)^2 = 1116$

$\Sigma x^2 = 484 + 676 + 196 + 16 + 900 + 144 = 2416$

$\Sigma y^2 = 36 + 144 + 16 + 0 + 324 + 36 = 556$

$\Sigma xy = 132 + 312 + 56 + 0 + 540 + 72 = 1112$

a = 0.6, b = – 3.16, r = 0.917 → 1, Good fit

Regression equation: R = 0.6P = 3.16

80. For the grouped data of the annual floods in the river Ganga at Hardwar (1885-1971), find the mean, median, and mode. Determine the coefficients of skew and the coefficient of variation.

Class interval (1000 cumec)	Frequency
0-2*	0
2-4*	17
4-6	27
6-8	18
8-10	18
10-12	3
12-14	0
14-16	2
16-18	1
18-20	1

***from 0 to <2**

From 2 to <4 and like that.

Class interval CI (1000 cumec)	*Mid-point of CI x*	*Frequency f*	*Product f.x*	$x-\bar{x}$	$(x-\bar{x})^2$	$f.(x-\bar{x})^2$	$(x-\bar{x})^3$	$f.(x-\bar{x})^3$
0-2	1	0	0	–5.6	31.4	0	–176	0
2-4	3	17	51	–3.6	13.0	221.0	–46.8	–796.00
4-6	5	27	135	–1.6	2.56	69.2	–4.1	–110.50
6-8	7	18	126	0.4	0.16	2.9	0.064	1.15
8-10	9	18	162	2.4	5.76	103.8	13.82	249.00
10-12	11	3	33	4.4	19.40	58.2	85.30	255.90
12-14	13	0	0	6.4	41.00	0	262.60	0.00
14-16	15	2	30	8.4	70.50	141.0	593.00	1186.00
16-18	17	1	17	10.4	108.00	108.0	1123.00	1123.00
18-20	19	1	19	12.4	154.00	154.0	1910.00	1910.00
		$\Sigma f = n = 87$	$\Sigma fx = 573$			$\Sigma f(x-\bar{x})^2 = 858.1$		$\Sigma f(x-\bar{x})^3 = 4725\ 05$ $-\ 906.50$ $= 3818.55$

$$\text{Mean } \bar{x} = \frac{\Sigma fx}{n} = \frac{573}{87} = 6.6 \text{ tcm}$$

$$\text{Standard deviation, } \sigma = \sqrt{\frac{\Sigma f\,(x-\bar{x})^2}{n-1}} = \sqrt{\frac{858.1}{87-1}} = 3.16 \text{ tcm}$$

(i) Mean x = 6.6 tcm

(ii) Standard deviation, σ = 3.16 tcm

$$(iii)\ Median = L_{md} + \left(\frac{\frac{n}{2} - CF}{f_{md}}\right) CI = 4 + \left(\frac{\frac{87}{2} - 17}{27}\right) 2 = 6\ tcm$$

$$(iv)\ Mode = L_{mo} + \left(\frac{d_1}{d_1 + d_2}\right) CI = 4 + \left(\frac{10}{10+9}\right) 2 = 5\ tcm$$

(v) Coefficients of skew (C_s)

$$Pearsons\ first\ coefficient, C_{s1} = \frac{\bar{x} - \text{mode}}{\sigma} = \frac{6.6 - 5}{3.16} = 0.507$$

$$Pearsons\ second\ coefficient, C_{s2} = \frac{3(\bar{x} - \text{median})}{\sigma} = \frac{3(6.6 - 6)}{3.16} = 0.57$$

$$For\ flood\ data\ (Foster), C_S = \frac{\sum f(x - \bar{x})^3}{(n-1)\sigma^3} = \frac{3818.55}{(87-1)3.16^3} = 1.4$$

Adjustment for the period of record,

$$C_{s(adj)} = C_s\left(1 + \frac{k}{n}\right) = 1.4\left(1 + \frac{6}{87}\right) = 1.5$$

All the coefficients of skew are positive and the skew is to the right; if the coefficients were negative, the skew would have been to the left.

$$(vi)\ Coefficient\ of\ \text{variation}, C_v = \frac{\sigma}{\bar{x}} \times 100 = \frac{3.16}{6.6} \times 100 = 47.8\%$$

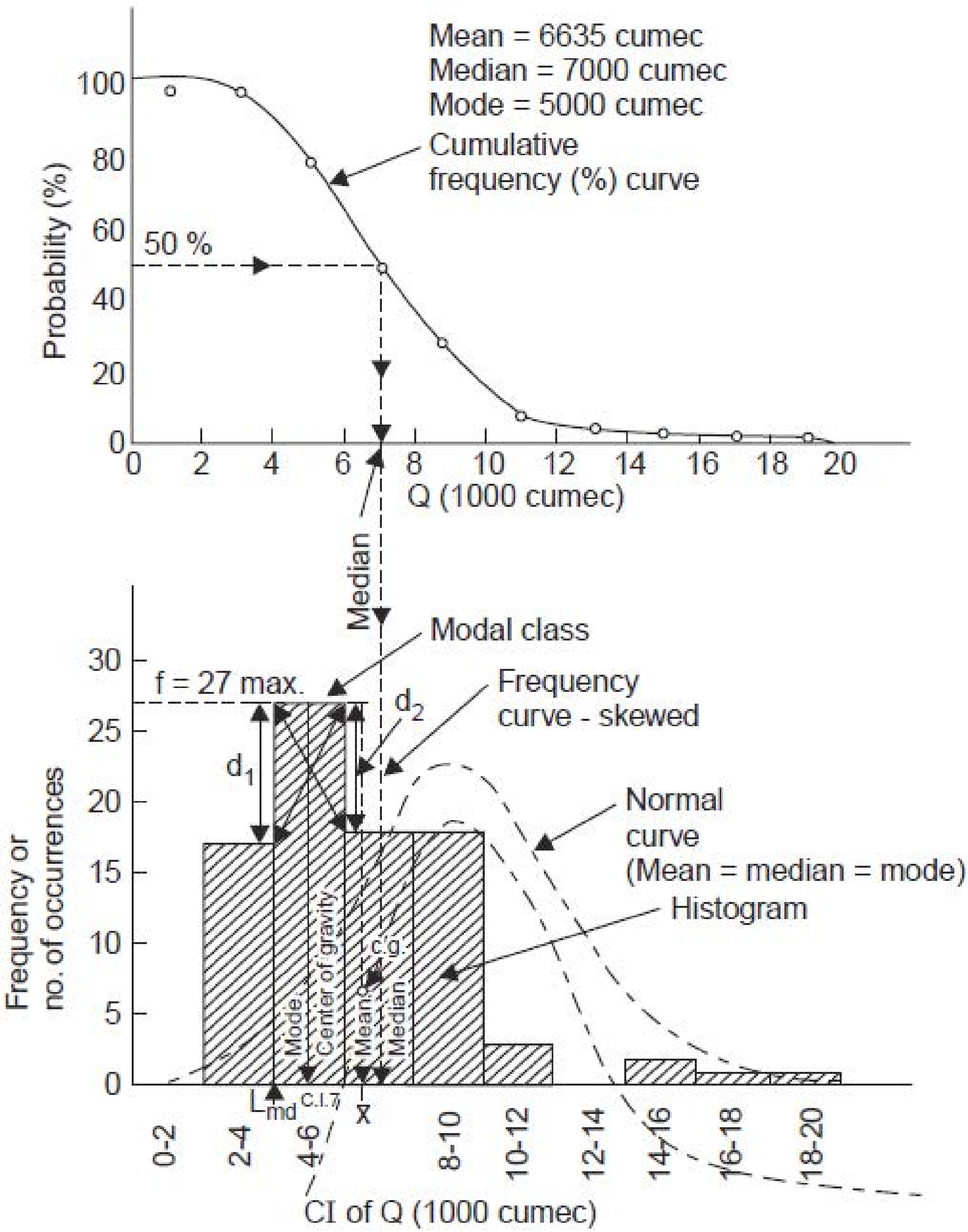
81. Flood data in the form of Partial-Duration Series and Annual-Flood Peaks for the Ganga river at Hardwar for a period of 87 years (1885-1971) are given in the tables. The base flow for the partial duration series may be taken as 4333 cumec (which was accepted as the bankfull discharge in the design of weir at Bhimgoda).

Derive the flood-frequency curves based on the two series by using the stochastic models. Make a comparative study with the other methods based on annual floods discussed earlier.

Sl No.	year	Ann. Peak Q, cumec	Log_{10} Q	Sl No.	year	Ann. Peak Q cumec	Log_{10} Q
1	1885	7241	3.8598	45	1929	4545	3.6576

2	1886	9164	3.9621	46	1930	5998	3.7780
3	1887	7407	3.8696	47	1931	3470	3.5403
4	1888	6870	3.8370	48	1932	6155	3.7893
5	1889	9855	3.9936	49	1933	5267	3.7216
6	1890	11887	4.0752	50	1934	6193	3.7919
7	1891	8827	3.9458	51	1935	5289	3.7223
8	1892	7546	3.8777	52	1936	3320	3.5211
9	1893	8498	3.9293	53	1937	3232	3.5095
10	1894	16757	4.2242	54	1938	3525	3.5471
11	1895	9680	3.9859	55	1939	2341	3.3694
12	1896	14336	4.1565	56	1940	2429	3.3854
13	1897	8174	3.9124	57	1941	3154	3.4989
14	1898	8953	3.9518	58	1942	6650	3.8228
15	1899	7546	3.8777	59	1943	4442	3.6476
16	1900	6652	3.8229	60	1944	4229	3.6262
17	1901	11409	4.0573	61	1945	5101	3.7077
18	1902	9164	3.9621	62	1946	4629	3.6654
19	1903	7404	3.8694	63	1947	4345	3.6380
20	1904	8579	3.9335	64	1948	4890	3.6893
21	1905	9362	3.9714	65	1949	3619	3.5586
22	1906	7092	3.8507	66	1950	5899	3.7708
23	1907	7546	3.8777	67	1951	4458	3.6492
24	1908	11504	4.0607	68	1952	3919	3.5932
25	1909	8335	3.9209	69	1953	5470	3.7380
26	1910	15077	4.1783	70	1954	5978	3.7766
27	1911	6943	3.8416	71	1955	4644	3.6669
28	1912	8335	3.9209	72	1956	6381	3.8049
29	1913	3579	3.5538	73	1957	4548	3.6579
30	1914	9299	3.9684	74	1958	4056	3.6081
31	1915	7407	3.8696	75	1959	4493	3.6525
32	1916	4726	3.6744	76	1960	3884	3.5893
33	1917	8416	3.9251	77	1961	4855	3.6861
34	1918	4668	3.6698	78	1962	5760	3.7604
35	1919	6296	3.7991	79	1963	9192	3.9634
36	1920	8147	3.9124	80	1964	3024	3.4806
37	1921	9079	3.9580	81	1965	2509	3.3994
38	1922	7407	3.8696	82	1966	4741	4.6759
39	1923	5482	3.7390	83	1967	5919	3.7725
40	1924	19136	4.2818	84	1968	3798	3.5795
41	1925	9680	3.9859	85	1969	4546	3.6577
42	1926	3698	3.5680	86	1970	3842	3.5845
43	1927	7241	3.8598	87	1971	4542	3.6573
44	1928	3698	3.5680				

The histogram of annual flood peaks for the Ganga river at Hardwar for the period 1885-1971, 87 years, is shown in the figure. The computation of the cumulative frequency curve is made in the table.



Mean = 6635 cumec
Median = 7000 cumec
Mode = 5000 cumec
Cumulative
frequency (%) curve
Probability (%)
100
80
60
40
20
0
50 %
0
2
4
6
8
10
12
14
16
18
20
Q (1000 cumec)
Median
Modal class
f = 27 max.
d2
d1
Frequency
curve - skewed
Normal
curve
(Mean = median = mode)
Histogram
Frequency or
no. of occurrences
30
25
20
15
10
5
0
Mode
Center of gravity
Mean
Median
c.g.
Lmd
C.I.
x̄
0-2
2-4
4-6
8-10
10-12
12-14
14-16
16-18
18-20
CI of Q (1000 cumec)

Annual flood peak C.I. (1000 cumec)	*No. of occurrences or frequency, f*	*Cumulative occurrences or frequency, CF*	*Probability* $\left(=\frac{CF}{\Sigma f}\times 100\right)\%$
0-2*	0	87	100
2-4*	17	87	100
4-6	27	70	80.5
6-8	18	43	49.5
8-10	18	25	28.8
10-12	3	7	8.05
12-14	0	4	4.6
14-16	2	4	4.6
16-18	1	2	2.3
18-20	1	1	1.15
	$\Sigma f = 87$		

*0-<2.

2- <4, and like that.

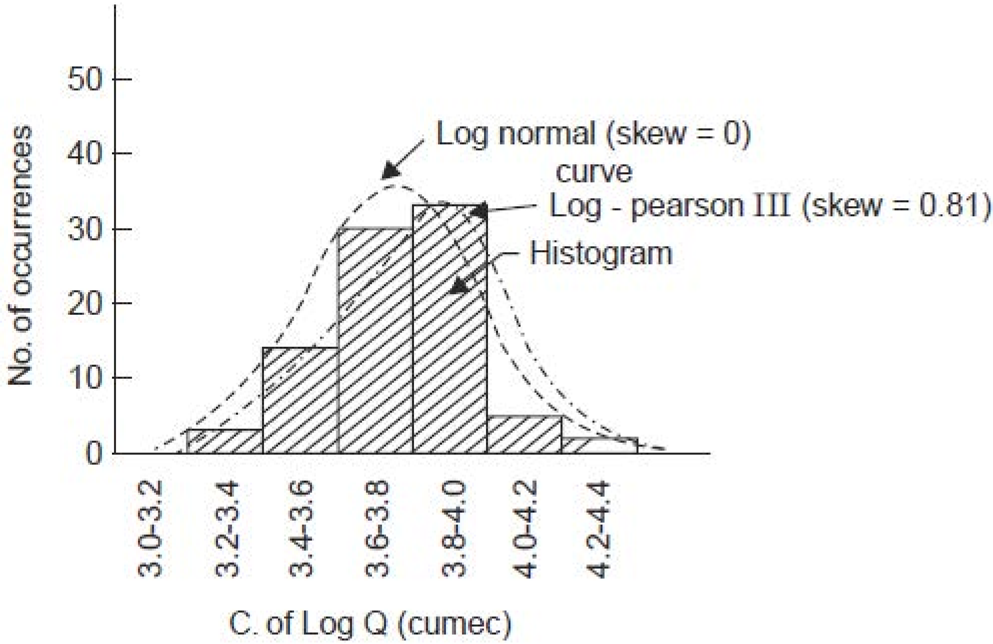
It is seen that the distribution of floods do not have the normal bell-shaped curve but they are skewed. However, the data can be transformed by plotting the common logarithm of the flood peaks so that the distribution density curve is approximately normal as shown in the figure. This is then called a log normal distribution and the standard deviation is in logarithmic units. The histogram of the partial-duration series of the flood peaks above the selected base of 4333 cumec is shown in the figure, which also represents skewed data.

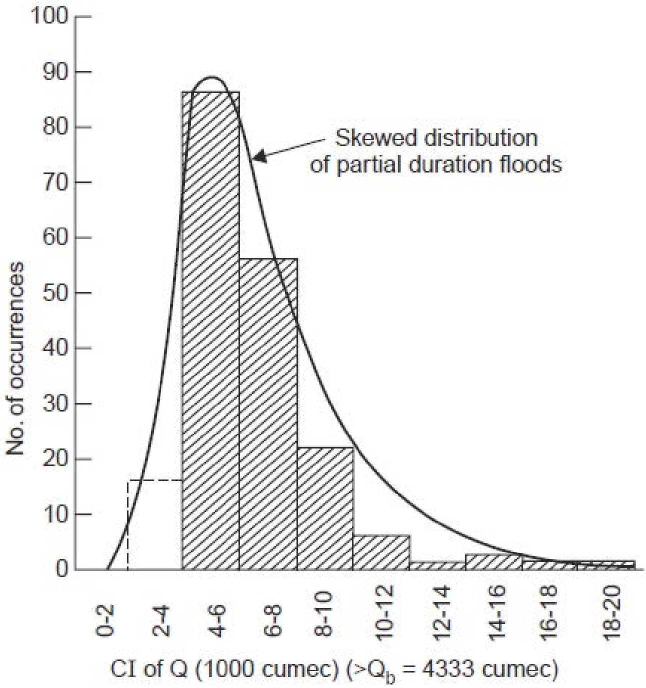
(a) Partial duration series. There are 175 flood exceedances (above Q_b) during 87 years. Average number of exceedances per year.

$$\lambda = \frac{175}{87} = 2.01$$

Parameter β is estimated in the following table.

Sl. no.	Flood peak exceedance x_i (cumec) CI	Variable	Observed frequency	Cumulative frequency CF	$H(x) = \frac{CF}{175}$	$1 - H(x)$	$\beta = \frac{-\ln\{1 - H(x)\}}{x}$ ...× 10^{-4}
1.	below-2500	2500	107	107	0.6114	0.3886	3.362
2.	2500-5000	5000	51	158	0.9029	0.0971	4.649
3.	5000-7500	7500	10	168	0.9600	0.0400	4.282
4.	7500-10000	10000	3	171	0.9771	0.0229	3.762
5.	10000-12500	12500	3	174	0.9443	0.0057	4.119
6.	12500-15000	15000	1	175	1.0000	0.0000	—





The average value of $\beta = \bar{\beta} = 4.05 \times 10^{-4}$

$$Q_T = Q_b + x_T$$

$$Q_T = 4333 + \frac{10^4}{4.05}\left[\ln(2.01T) - \ln\{\ln(2.01T)\}\right]$$

T-yr:	1000	500	200	100	50
Q_T *(cumec):*	18103	16605	14728	13296	11938
Q_T *by* $T = \frac{n+1}{m}$:	23600	21600	18900	17000	15100

ANNUAL FLOOD PEAKS—RIVER GANGA

(i) Gumbel's method

$$Q_T = \overline{Q} + K\sigma$$

$$\overline{Q} = 6635.63\ cumec$$

$$\sigma = 3130.8\ cumec$$

$$K = \frac{y - \overline{y_n}}{\sigma_n}$$

$$For\ n = 87, \overline{y_n} = 0.55815, \sigma_n = 1.1987$$

T-yr	$X_T = log\left(log\dfrac{T}{T-1}\right)$	$Y = -0.834 - 2.3\,X_T$	$K = \dfrac{y - \bar{y}_n}{\sigma_n}$	$Q_T = \overline{Q} + K\sigma$
1000	– 3.361	6.907	5.29	23185
500	– 3.060	6.213	4.7	21335
200	– 2.662	5.295	3.95	19005
100	– 2.360	4.600	3.36	17155
50	– 2.056	3.901	2.79	15365

(ii) Stochastic Method

$$Q_{\min} = 2341\ cumec; \overline{Q} = 6635.63\ cumec; n_f = 77$$

$$Q_T = Q_{\min} + 2.3\left(\overline{Q} - Q_{\min}\right)\log\left(\frac{n_f}{n}\times T\right) = 2341 + 2.3\left(6635.63 - 2341\right)\log\left(\frac{77}{87}\times T\right)$$

T-yr	0.885 T	Log (0.885 T)	9890 log (0.885 T)	Q_T cumec
1000	885	2.947	29200	31541
500	442.5	2.646	26200	28541
200	177	2.248	22200	24541
100	88.5	1.947	19200	21541
50	44.25	1.646	16260	18601

(iii) Log-Pearson Type III distribution.

$$Mean : \overline{\log x} = \frac{\sum f(\log x)}{\sum f} = \frac{67.3856}{87} = 0.7750$$

$$Std.dev.\,\sigma_{\log x} = \sqrt{\frac{\sum f\left(\log x - \overline{\log x}\right)^2}{n-1}} = \sqrt{\frac{3.3315}{87-1}} = 0.1962$$

CI (1000 cumec)	Mid-pt. of CI x (1000 cumec)	Frequency f	$\log x$	$f \log x$	$\log x - \overline{\log x}$	$(\log x - \overline{\log x})^2$	$(\log x - \overline{\log x})^3$	$f(\log x - \overline{\log x})^2$	$f(\log x - \overline{\log x})^3$
0-2	1	0	0	0	0	0	0	0	0
2-4	3	17	0.4771	8.1000	–0.2979	–0.0890	–0.0265	1.5120	–0.4500
4-6	5	27	0.6990	18.9000	–0.0760	0.0058	–0.00044	0.1560	–0.0119
6-8	7	18	0.8451	15.2000	0.0701	0.0049	0.0003	0.0885	0.0062
8-10	9	18	0.9542	17.2000	0.1792	0.0340	0.0057	0.5790	0.1025
10-12	11	3	1.0414	3.1242	0.2664	0.0710	0.0190	0.2130	0.0570
12-14	13	0	1.1139	0.0000	0.3389	0.1150	0.0390	0.0000	0.0000
14-16	15	2	1.1761	2.3522	0.4011	0.1610	0.0644	0.3220	0.1288
16-18	17	1	1.2304	1.2304	0.4554	0.2070	0.0940	0.2070	0.0940
18-20	19	1	1.2788	1.2788	0.5038	0.2540	0.1280	0.2540	0.1280
Σ		$n = \Sigma f = 87$	$\Sigma f. \log(x) = 67.3856$					3.3315	0.5165

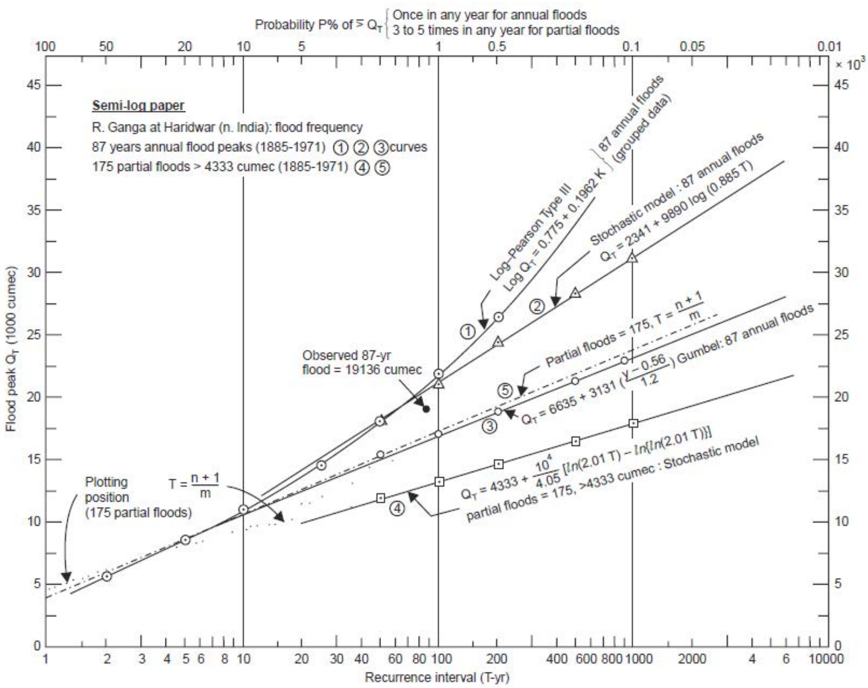
For grouped data: $\overline{\log x} = \dfrac{\Sigma f.\log(x)}{\Sigma f} = \dfrac{67.3856}{87} = 0.7750$

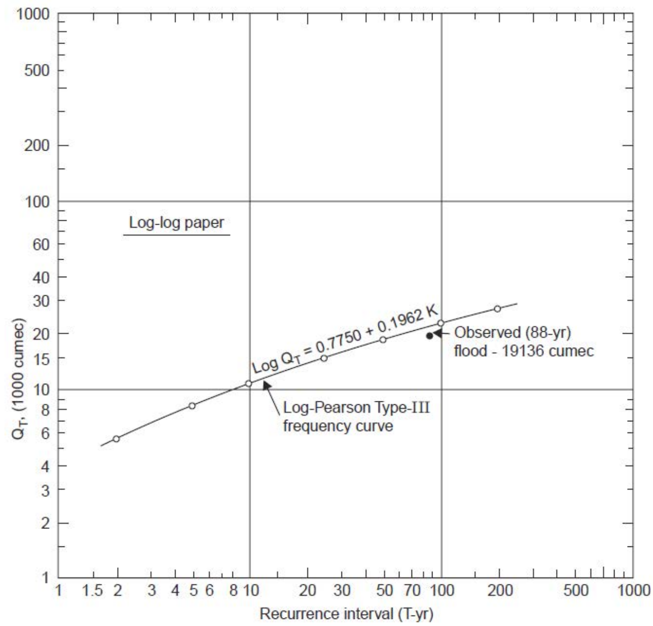
$$Skew: g = \frac{n\sum f\left(\log x - \overline{\log x}\right)^3}{(n-1)(n-2)\left(\sigma_{\log x}\right)^3} = \frac{87(0.5165)}{(87-1)(87-2)(0.1962)^3} = 0.81$$

$$\log Q_T = \overline{\log Q} + K\sigma_{\log Q}$$

and QT for any desired T can be computed by knowing the value of K for g = 0.81 and desired T from the table.

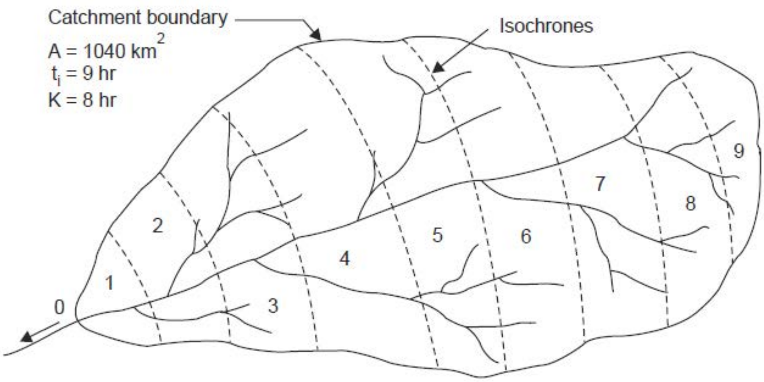
T-yr	*K = f(g, T)* *from Table 14.2*	*K . $\sigma_{log\ Q}$* *($\sigma_{log\ Q}$ = 0.1962)*	*log Q_T* = $\overline{log\ Q} + K\sigma_{log\ Q}$ ($\overline{log\ Q}$ = 0.7750)	*Q_T* *(1000 cumec)*
2	– 0.132	– 0.0259	0.7491	5.611
5	0.779	0.1530	0.9280	8.472
10	1.336	0.2620	1.0370	10.89
25	1.996	0.3910	1.1660	14.66
50	2.458	0.4820	1.2570	18.07
100	2.898	0.5670	1.3420	21.98
200	3.321	0.6500	1.4250	26.61





The flood frequency curves by the above four methods have been plotted on semi-log paper. It can be seen that the highest annual flood peak of 19136 cumec during a period of 87 years (T= (87+1)/1=88-yr) has exceeded the 100-yr flood given by Gumbel's method and that computed by the new stochastic model based on the partial duration series. However, in this case, the stochastic method using annual flood data and Log-Pearson Type-III distribution give safe design values.

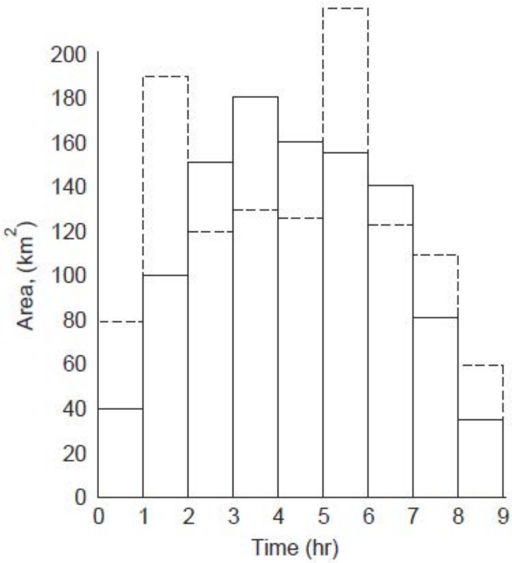
82. A catchment of area 1040 km² is divided into 9-hourly divisions by isochrones (lines of equal travel time) in the figure. From the observation of a hydrograph due to a short rain on the catchment, t_i = 9 hr and K = 8 hr. Derive: (a) the IUH for the catchment. (b) a 3-hr UG.



(i) It will be assumed that the catchment is divided into sub-areas such that all surface runoff from each of these areas will arrive during a 1-hr period at the gauging point. The areas are measured by planimetering each of the hourly areas as:

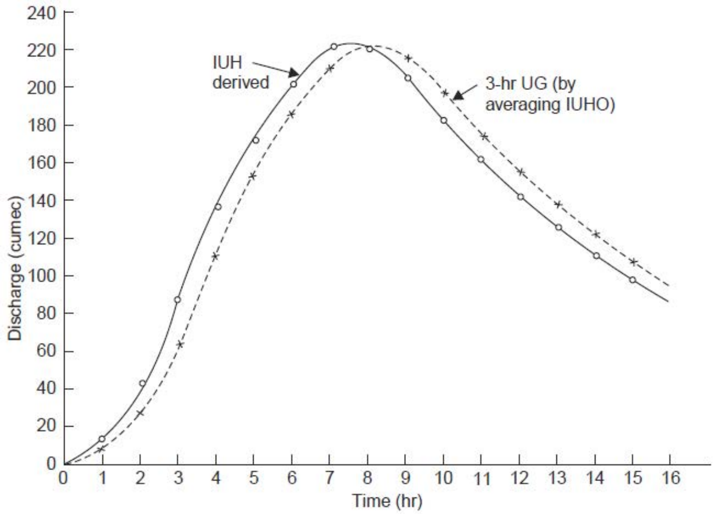
Hour	1	2	3	4	5	6	7	8	9
Area (km^2)	40	100	150	180	160	155	140	80	35

(ii) The time-area graph (in full lines) and the distribution graph of runoff (in dotted lines) are drawn as shown in the figure. The dotted lines depict the non-uniform areal distribution of rain.



Time (hr)	Time-area diagram Area (km^2)	0.1177 I = 2.78 × 0.1177 × col (2) (cumec)	0.882 × col (5) previous (cumec)	O_2 = IUH = col (3) + col (4) (cumec)	3-hr UGO (cumec)
1	2	3	4	5	6
0	0	0	0	0	0
1	40	13.1	0	13.10	
2	100	32.7	11.54	44.24	
3	150	49.1	39.00	88.10	44.00
4	180	58.9	77.70	136.60	
5	160	52.3	120.40	172.70	
6	155	50.7	152.00	202.70	145.40
7	140	45.8	179.00	224.80	
8	80	26.2	197.00	223.20	
9	35	11.4	196.50	208.00	205.30
10	0	0	184.50	184.50	
11	0	0	163.70	163.70	
12	0	0	145.00	145.00	176.50
13	0	0	128.60	128.60	
14	0	0	114.00	114.00	
15	0	0	101.00	101.00	123.00

Plot col (1) vs. col (5) to get the IUH, and col (1) vs. col (6) to get the 3-hr UGO, as shown in the figure.



(iii) $O_2 = C'I + C_2O_1$

$$C' = \frac{t}{K + \frac{1}{2}t} = \frac{1}{8 + \frac{1}{2} \times 1} = \frac{1}{8.5} = 0.1177$$

$$C_2 = \frac{K - \frac{1}{2}t}{K + \frac{1}{2}t} = \frac{8 - \frac{1}{2} \times 1}{8 + \frac{1}{2} \times 1} = \frac{7.5}{5} = 0.882, Check : C' + C_2 = 1$$

Hence, the routing equation becomes

$O_2 = 0.1177\ I + 0.882\ O_1$

O_2 vs. time gives the required synthetic IUH from which the 3-hr UGO are obtained as computed in the table. The conversion constant for Col (3) is computed as

$$1 - cm\ rain\ on\ 1\ km^2\ in\ 1\ hr = \frac{10^6 \times 10^{-2}}{3600} = 2.78\ m^3 / s$$

The 3-hr UGO is obtained by averaging the pair of IUH ordinates at 3-hr intervals and writing at the end of the intervals.

83. The mean monthly flow data for a proposed reservoir site are given below:

Month	jan	feb	mar	april	may	June
Mean monthly flow (cumec)	6	3	1	2	7	1
month	july	aug	sept	oct	nov	dec
Mean monthly flow (cumec)	27	29	30	27	31	15

Determine the average discharge that can be expected throughout the year. Draw the residual mass curve and obtain an expression for the range as developed by Hurst on the basis of the monthly flow data.

month	Mean monthly flow, x (cumec)	Monthly flow volume (ha-m)	Cumulative monthly inflow (ha-m)	Cumulative mean flow throughout the year (ha-m)	Residual mass curve (ha-m)
1	2	3	4	5	6
Jan	6	1575	1575	3931	-2356
Feb	3	790	2365	7862	-5497
Mar	1	262	2627	11793	-9166
April	2	525	3152	15725	-12573
May	7	1840	4992	19656	-14664
June	1	262	5254	23587	-18333
Nuly	27	7100	12354	27518	-15164
Aug	29	7750	20104	31450	-11346
Sept	30	7880	27984	35381	-7397
Oct	27	7100	35084	39312	-4228
Nov	31	8150	43234	43243	-0009
dec	15	3940	47174	47174	0
n =	179	47174			

$$\bar{x} = \frac{179}{12} = 15\,cumec$$

$$Mean\ \ flow\,(per\ month)\ throught\ the\ year = \frac{47174}{12} = 3931.2\ ha - m$$

The average discharge that can be expected throughout the year

$$Q = \frac{47174 \times 10^4\ m^3}{365 \times 86400\ S} = 15\,cumec = \bar{x}$$

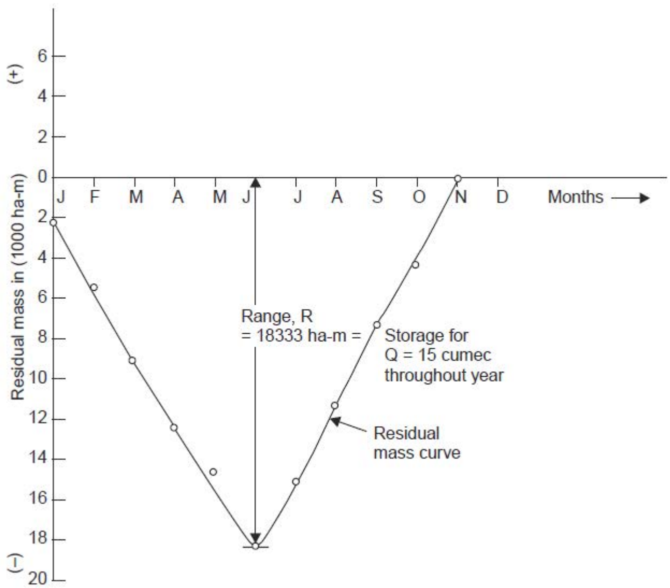
The residual mass curve is plotted in Fig. 16.16 and the range, R = 18333 ha-m, which is the storage capacity of the reservoir to maintain the mean flow of 15 cumec throughout the year.

x:	6	3	1	2	7	1	27	29	30	27	31	15
x- x̄:	-9	-12	-14	-13	-8	-14	12	14	15	12	16	0
(x- x̄)^2:	81	144	196	169	64	196	144	196	225	144	256	0
Σ(x- x̄)^2 = 1815												

$$\sigma = \sqrt{\frac{\sum\left(x-\bar{x}\right)^2}{n-1}} = \sqrt{\frac{1815}{12-1}} = 12.84\,cumec$$

$$Let\ R = \sigma\left(\frac{n}{2}\right)^k$$

$$18333 \times 10^4 = 12.84\left(30.4 \times 24 \times 60 \times 60\right)\left(\frac{12}{2}\right)^k \rightarrow k = 0.945$$



Thus, the expression for range (on the basis of 12 months data) is

$$R = \sigma \left(\frac{n}{2} \right)^{0.945}$$

Usually k varies from 0.5 to 1.0, the average value being 0.73. Usually, a number of years of observation are required.

84. Given in the table (Col. 1, 2, 3, 5 and 6) are the monthly inflows during low water period at the site of a proposed dam, the corresponding monthly precipitation and pan evaporation at a nearby station, and the estimated monthly demand for water. Prior water rights downstream require a special release of 6 cumec or the natural inflow, whichever is less. Assuming that only 24% of the rainfall on the land area to be flooded by the proposed reservoir has reached the stream in the past, reservoir area as 6000 ha on an average, and a pan coefficient of 0.7, construct the sequent peak alogrithm and determine the required storage capacity of the reservoir.

Month	Mean monthly flow, Q (cumec)	Monthly flow volume (ha-m)	Precipi tation, P (mm)	Pan Eva-poration, E_p (mm)	Demand (ha-m)	D/s Release (ha-m)	Change in storage, Δs (ha-m) = (3) + (4) – (5)-(6)-(7)	Cumulative storage, $\Sigma\Delta s$ (ha-m)	Reservoir capacity (ha-m)
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*
July	27	6998†	135	155	650	1555†	+4758*	4758	
Aug.	29	7257	175	75	975	1555	+5210	9968	
Sept.	30	7776	140	80	1200	1555	+5071	15039	
Oct.	27	6998	25	125	1750	1555	+3282	18321	
Nov.	31	8035	5	65	2500	1555	+3730	22051	
Dec.	15	3888	0	40	2500	1555	–335	21716	
Jan.	6	1555	0	50	2500	1555	–2710	19006	
Feb.	3	777	0	80	2400	777	–2736	16270	
March	1	259	0	100	2250	259	–2670	13600	
April	2	518	20	130	1500	518	–1955	11645	
May	7	1814	45	195	1250	1555	–1605	10040	
June	1	259	100	200	650	259	–1034	9006	**13045**

(July–Nov.) Σ Surplus = 22051 ha-m

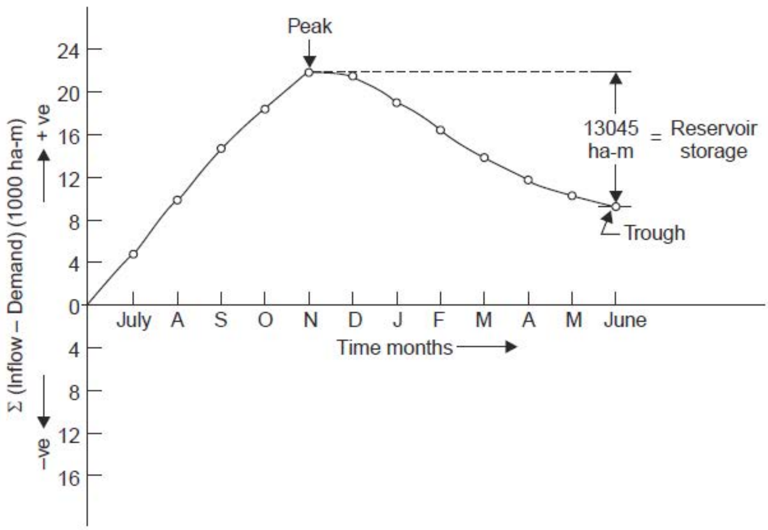
(Dec.–June) Σ Deficit = 13045 ha-m

22051 – 9006 = 13045 ha-m

1. †27 cumec × 30 days = 27(30 × 86400)/10^4 = 6998 ha-m
2. †56 cumec × 30 days = 6 (30 × 86400)/10^4 = 1555 ha-m
3. $*6998 + \frac{135 \times 0.75 - 155 \times 0.7}{1000} \times 6000 - 650 - 1555 = +4758$ ha-m
4. Reservoir capacity = sum of negative quantities in col. (8).

Since 24% of the rainfall (P) is runoff, which is already included in the monthly inflows into the reservoir, only 100 – 24 = 76% of the rainfall on the reservoir area is to be included. Reservoir evaporation = 0.7 × Pan Evaporation (EP). ($0.76P - 0.7\,E_p$) values have to be multiplied by the average reservoir area at the beginning and end of each month.

The monthly change in storage and cumulative storage (at the end of each month) are worked out in the table and the sequent peak algorithm is drawn as shown in the figure and the required storage capacity of the Reservoir (difference between the initial peak and the lowest trough in the interval) is 13045 ha-m, which is also indicated in the col. (10) of the table. Actually this process has to be done for 4-5 consesecutive years and the difference between the highest peak and the succeeding lowest trough gives the required storage capacity to meet the specified demand. The required storage capacity is also equal to the sum of the negative quantities (ΣDeficit) in Col (8) of the table, which is less than the sum of the positive quantities (ΣSurplus) col (8), thus ensuring the filler of the reservoir during monsoons.



85. The effective rainfall due to a 4-hr storm in the successive hours are: 2.6, 2.5, 2.3 and 2.4 cm. The resulting DRO's in the successive hours are: 3, 15, 26, 40, 50, 35, 25, 20, 15, 10, 7, 4, 3 and 1 cumec. Determine the values of n and k.

Step 1 Evaluate the first and second moments of P_{net} about the origin, i.e., t = 0 (commencement of P_{net} and DRO)

$$MI_1 = \frac{2.6\times0.5+2.5\times1.5+2.3\times2.5+2.4\times3.5}{2.6\times2.5\times2.3\times2.4} = \frac{19.2\,cm.hr}{9.8\,cm} \cong 2\,hr$$

$$MI_2 = \frac{2.6\times0.5^2+2.5\times1.5^2+2.3\times2.5^2+2.4\times3.5^2}{2.6\times2.5\times2.3\times2.4} = \frac{49.75\,cm.hr^2}{9.8\,cm} \cong 5\,hr^2$$

Step 2 Evaluate the first and second moments of Q_i about the origin;

$$Q_i = DRO_i$$

$$Q\bar{t} = \sum Q_1 t_1$$

$$MQ_1 = \frac{\sum Q_1 t_1}{Q}$$

$$Q = \sum Q_1 = \sum DRO$$

$$Q\overline{t^2} = \sum Q_1 t_1^{\ 2}$$

$$MQ_2 = \frac{\sum Q_1 t_1^{\ 2}}{Q}$$

Time t (hr)	DRO (cumec) (given) = Q	Q x t (cumec-hr)	Q x t^2 (cumec-hr^2)
0	0	0	0
1	3	3	3
2	15	30	60
3	26	78	234
4	40	160	640
5	50	250	1250
6	35	210	1260
7	25	175	1225
8	20-	160	1280
9	15	135	1215
10	10	100	1000
11	7	77	847
12	4	48	576
13	3	39	507
14	1	14	196
15	0	0	0
	∑Q = 254	∑Qt = 1472	∑Qt^2 = 102.93

$$MQ_1 = \bar{t} = \frac{1472}{254} = 5.8\ hr$$

$$MQ_2 = \bar{t}^2 = \frac{10293}{254} = 40.5\ hr^2$$

nk = $MQ_1 - MI_1$ = 5.8 – 2 = 3.8 hr

$MQ_2 - MI_2 = n(n + 1)k^2 + 2\ nk\ MI_1$

40.5 – 5 = nk^2 (n + 1) + 2 × 3.8 × 2

35.5 = $n^2k^2 + nk^2$ + 15.2

nk^2 = 35.5 – 15.2 – $(3.8)^2$ = 5.87

$$k = \frac{nk^2}{nk} = \frac{5.87}{3.8} = 1.55$$

$$n = \frac{nk}{n} = \frac{3.8}{1.55} = 2.45 \cong 2\left(whole\ number\right)$$

86. Derive an IUH and a 2-hr UG (UGO at 2-hr intervals) for a catchment of 240 km², having n = 3 and k = 5 hr.

$$u\left(t\right) = \frac{1}{k\Gamma n} \times e^{-\frac{t}{k}} \times \left(\frac{t}{k}\right)^{n-1} = \frac{1}{5 \times 2} \times e^{-\frac{t}{5}} \times \left(\frac{t}{5}\right)^{3-1}$$

Time t(hr)	$\frac{t}{5}$	$e^{-\frac{t}{5}}$	$\left(\frac{t}{5}\right)^2$	*IUHO* u(t) (cm/hr) $= \frac{(3) \times (4)}{10}$	*IUHO* u(t) (cumec) (5) × 2.78 × 240 = (5) × 668	2-hr UGO (cumec) (by averaging)
1	2	3	4	5	6	7
0	0	1	0	0	0	0
2	0.4	0.67	0.16	0.0107	7.17	$\frac{0 + 7.17}{2} = 3.58$
4	0.8	0.45	0.64	0.0288	19.4	$\frac{19.4 + 7.17}{2} = 13.3$

6	1.2	0.30	1.44	0.0432	28.8	24.1
8	1.6	0.20	2.56	0.0512	34.2	31.5
10	2.0	0.135	4.00	0.0542	36.2	35.2
12	2.4	0.091	5.76	0.0524	34.9 peak of IUH	35.5 peak of UG
14	2.8	0.061	7.84	0.0480	33.2	34.0
16	3.2	0.041	10.24	0.0420	28.0	30.6
18	3.6	0.027	12.96	0.0350	24.4	26.2
20	4.0	0.0183	16.00	0.0293	19.6	22.0
22	4.4	0.0122	19.36	0.0236	15.7	17.6
24	4.8	0.0082	23.04	0.019	12.7	14.2
26	5.2	0.0055	27.04	0.0149	9.95	11.32
28	5.6	0.0037	31.36	0.0116	7.75	8.85
30	6.0	0.0025	36.00	0.009	6.0	6.87
32	6.4	0.0017	40.96	0.007	4.66	5.33
34	6.8	0.0011	46.4	0.005	3.34	4.00
36	7.2	0.00075	51.8	0.004	2.67	3.00

Nash, from his study on some gauged catchments in UK, established a correlation between the IUH parameters n and k, and the basin parameters like length of main stream (L, miles), slope of the basin (S, parts per 1000) and the area (A, sq. miles), as

$$n = 2.4L^{0.1}, K = \frac{11A^{0.3}}{L^{0.1}S^{0.3}}$$

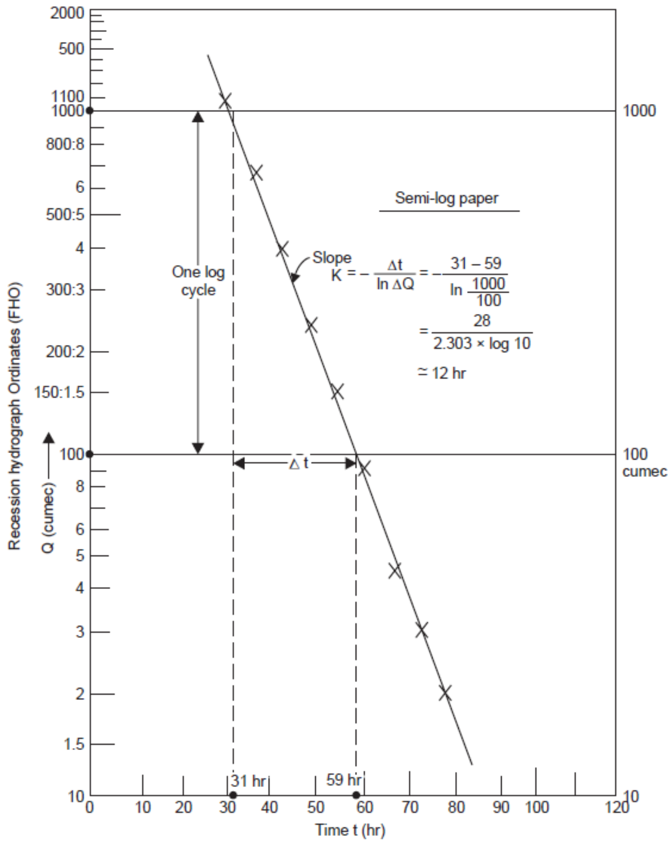
Using the above relations, the IUH of any ungauged basin in a hydro meteorologically homogeneous region can be obtained.

87. The recession ordinates of the flood hydrograph (FHO) for the Lakhwar dam site across river Yamuna are given below. Determine the value of K.

Time (hr)	30	36	42	48	54	60	66	72	78
FHO(cumec)	1070	680	390	240	150	90	45	30	20

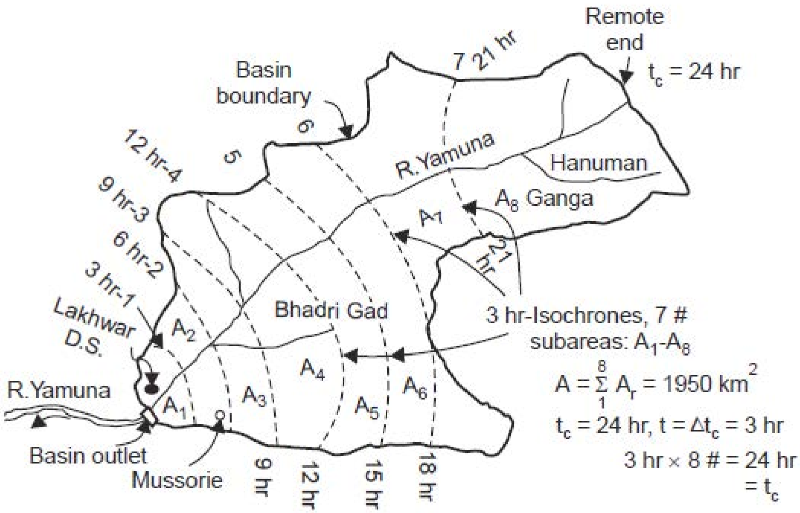
$$Q_t = Q_0 e^{-\frac{t}{k}}, \text{ when } K = \frac{t}{\ln\left(\frac{Q_0}{Q_t}\right)}$$

'Q vs. t' is plotted on the semi-log paper. K is the slope of the recession-flood hydrograph plot.



$$K = \frac{\Delta t}{\Delta \ln Q} = \frac{\Delta t}{2.303 \log \frac{1000}{100}} = \frac{31-59}{1.303 \times 1} = -12.15, say\, 12\, hr$$

88. The isochronal map of Lakhwar damsite catchment, the figure has areas between successive 3 hr isochrones as 32, 67, 90, 116, 135, 237, 586 and 687 km². Taking k = 12 hr, derive the IUH of the basin by Clark's approach and hence a 3-hr UG.



(a) Isochronal map of Lakhwar DS Catchment.

$A = \Sigma A_r = 1950\ km^2$

$t_c = t \times N = 3 \times 8 = 24$ hr, K = 12 hr

No. of isochrones = N – 1 = 8 – 1 = 7#

$$Computation\ \text{interval}\ t = \Delta t_c\ between\ successive\ isochrones = 3\ hr = \frac{24}{8} = \frac{t_c}{N}$$

$Q_2 = C'I + C_2Q_1$

$$C' = \frac{t}{k + \frac{t}{2}} = \frac{3}{12 + \frac{3}{2}} = 0.2222$$

$$C_2 = \frac{k - \frac{t}{2}}{k + \frac{t}{2}} = \frac{12 - \frac{3}{2}}{12 + \frac{3}{2}} = 0.7778$$

Check: $C' + C_2 = 0.2222 + 0.7778 = 1$

From the sub areas A_r,

$$I = 2.78\frac{A_r}{t} = 2.78 \times \frac{A_r}{3}$$

Clark's: $Q_2 = C'I + C_2Q_1$, $C_2Q_1 = 0.7778\ Q_1$ $\quad$ Q_2 = IUHO

C'I = 0.2222 × 0.9267 A_r = 0.203 A_r

1	2	3	4	5	6
Time (hr)	A_r, (km^2) (from TAD)	C'I = 0.203 A_r = (2) × 0.203	C_2Q_1 = 0.7778 Q_1 = (5) ×0.7778 previous	IUHO Q_2(cumec) = $C'I + C_2Q_1$ = (3) + (4)	3-hr UGO (cumec) (by averaging)
0	0	0		0	0
3	32	6.4	0	6.4	$\frac{0+6.4}{2} = 3.2$
6	67	13.5	5.1	18.6	$\frac{6.4+18.6}{2} = 12.5$
9	90	18.0	14.9	33.0	25.3
12	116	23.3	26.4	49.7	41.3
15	135	27.0	39.7	66.7	58.2
18	237	47.5	53.0	100.5	83.6
21	586	117	80.0	197.0	148.8
24	687	137.5	157	294.5 peak of IUH	245.7
27	ΣA_r = 1950 km^2	0	230	230	262.2 (peak of UG)
30		0	179	179	204.5
33		0	139.5	139.5	159.2

Plot Col. (5) vs. col (1) to get IUH, and Col (6) vs. col. (1) to get 3-hr UG. Note that the two peaks are staggered by 3 hr; i.e., IUH is more skewed.

89. During a snow survey, the data of a snow sample collected are given below:
Depth of snow sample 2 m
Weight of tube and sample 25 N
Weight of sample tube 20 N
Diameter of tube 40 mm
Determine
(i) the density of snow
(ii) the water equivalent of snow
(iii) the quality of snow, if the final temperature is 5 °C when 4 lit. of water at 15 °C is added.

(i) Density of snow is the same as its specific gravity

$$Sp.\,gr.\,of\ snow,\ G_s = \frac{\gamma_s}{\gamma_w} = \frac{\frac{W_s}{V_s}}{\gamma_w} = \frac{\frac{25-20}{\pi(0.020)^2 \times 2}}{1000 \times 9.81} = 0.203$$

$$(ii)\ Density\ of\ snow,\ G_s = \frac{Depth\ of\ melt\ water\ (d_w)}{Depth\ of\ snow\ (d_s)}$$

Water equivalent of snow, $d_w = G_s d_s = 0.203 \times 2 = 0.406$ m

(iii) If the actual weight of ice content in the sample is W_c gm, then

Heat gained by snow = Heat lost by water

Heat required to melt + to rise temperature to 5 °C

$$W_c \times 80 + \frac{5}{9.81} \times 1000 \times 5 = 4000(15-5)$$

Solving, $W_c = 468.2$ gm $= 0.4682 \times 9.81 = 4.6$ N

$$Quality\ of\ snow = \frac{4.6}{5} = 0.92$$

90. If the density of a snow pack 1.2 m depth is 20%, determine its weight density, mass density, sp. gr. and water equivalent.

The density is the percentage of snow volume, which its water equivalent would occupy.

$$Snow\ density = \frac{Depth\ of\ melt\ water\ (d_w)}{Depth\ of\ snow\ (d_s)}$$

$$0.20 = \frac{d_w}{d_s}$$

Water equivalent of snow, $d_w = 0.20 \times 1.2 = 0.24$ m

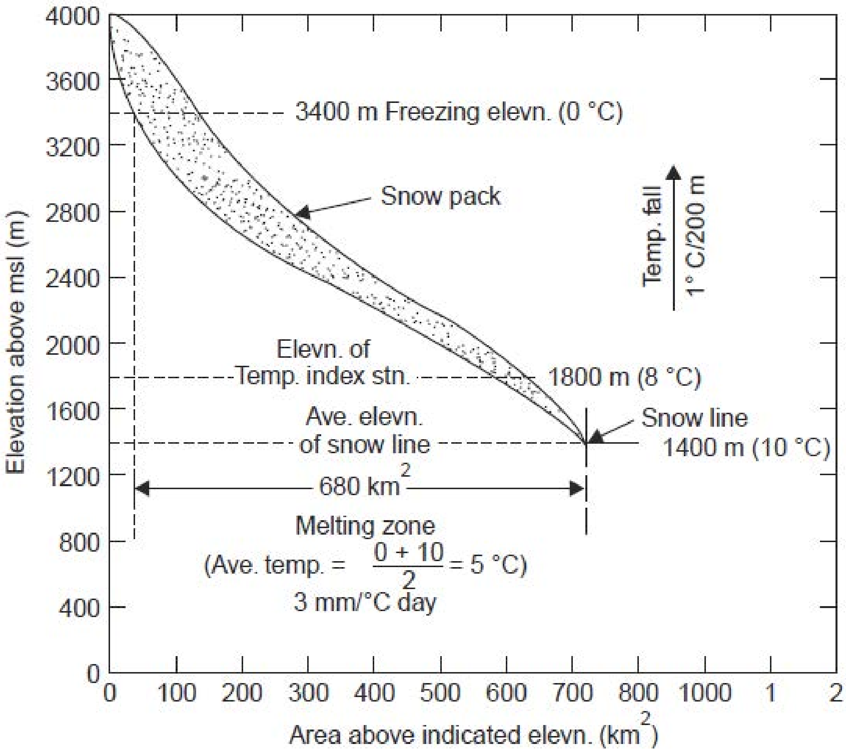
$$Weirht\ density, \gamma_s = \frac{W_s}{V_s} = \frac{W_w}{V_s} = \frac{V_w \gamma_w}{V_s} = \frac{d_w}{d_s}\gamma_w = 0.20(1000 \times 9.81) = 1962\ N/m^3$$

$$Mass\ density, \rho_s = \frac{M_s}{V_s} = \frac{\frac{W_s}{g}}{V_s} = \frac{\gamma_s}{g} = \frac{1962}{9.81} = 200\ kg/m^3$$

$$Sp.\ gr., G_s = \frac{\gamma_s}{\gamma_w} = \frac{1962}{1000 \times 9.81} = 0.2$$

Note, that the specific gravity is the same as the snow density.

91. The average snow line is at 1400 m elevation and a temperature index station located at 1800 m elevation indicated a mean daily temperature of 8 °C on a certain day. Assuming a temperature decrease of 1 °C per 200 m increase in elevation and a degree-day factor of 3 mm/degree-day, compute the snowmelt runoff for the day. An area elevation curve for the snowpack is shown in the figure.



Freezing occurs at higher altitudes when the temperature falls to 0 °C.

Freezing elevation = 1800 + (8 – 0) × 200 = 3400 m. The area between the snow line elevation of 1400 m and the freezing elevation of 3400 m is read out from the area-elevation curve, the figure as 680 km². The average temperature over this area is

$$\frac{1}{2}\left[\underset{\text{freezing elevn.}}{0°C\ at} + \underset{\text{at snow line elevn.}}{\left\{8°C + \frac{1800-1400}{200}\right\}}\right] = \frac{1}{2}(0+10) = 5°C$$

Snowmelt runoff for the day= 0.003 × 5 °C (680 × 10^6) = 10.2 × 10^6 m³= 10.2 km²-m

92. Equilibrium overland flow occurs over a rectangular area 100 m long due to a uniform net rainfall of 50 mm/hr. At what distance from the upper edge of the area the flow changes from laminar to turbulent if the temperature is 20 °C and the critical Reynolds number is 800.

$$R_e = \frac{vd}{\nu} = \frac{q}{\nu}$$

$$800 = \frac{q}{1\times10^{-6}} \rightarrow q = 8\times10^{-4}\ cumec/\text{m}$$

$$q = i_{net} l$$

$$8\times10^{-4} = \frac{50}{1000\times60\times60}\times l$$

l = 57.6 m, beyond which the flow becomes turbulent.

93. A concrete-paved area is 200 m long by 100 m wide and has surface slope of 0.005. The design storm is given by

i=250/t$^{0.4}$

Construct the outflow hydrograph for a 1-hr storm using Izzard's method.

Equilibrium discharge, $q_e = i_{net}$ (l × 1); for t = 60 min,

$$i = \frac{250}{60^{0.4}} = 48.5\ mm/hr$$

Assuming i = i_{net} for the concrete pavement (initially wet),

$$q_e = \frac{48.5}{1000\times60\times60}(200\times1) = 0.0027\ cumec/m = Kd_e^3 S$$

$$d_e = \left[\frac{q_e}{KS}\right]^{\frac{1}{3}}$$

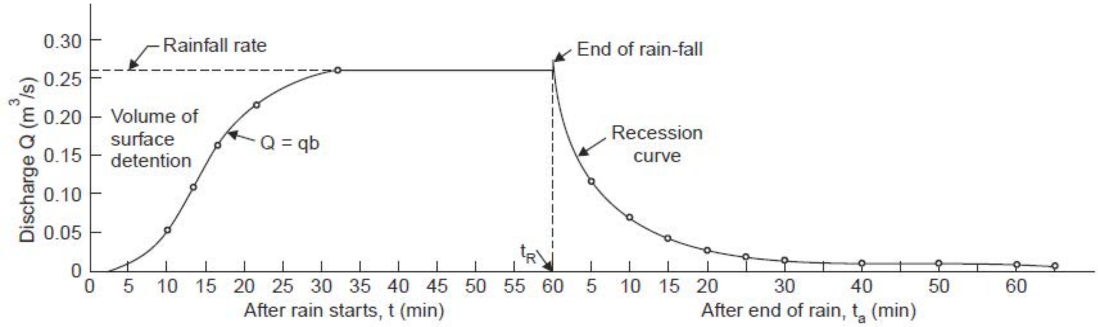
$$K^{\frac{1}{3}} = \frac{1}{(2.8\times10^{-5})i + C} = \frac{1}{(2.8\times10^{-5})(48.5)+0.012} = \frac{1}{0.01336}$$

$$d_e = 0.01336\left(\frac{0.0027}{0.005}\right)^{\frac{1}{3}} = 13.1\ mm$$

$$t_e = \frac{2d_e l}{q_e} = \frac{2\times0.0131\times200}{0.0027} = 1940\ \text{sec}$$

Data to plot the rising time				Data to plot the recession curve			
q/q_e	Q x q_eb (x 0.27) = (m^3/s)	t/t_e	T x t_e (x 32.33) = (min)	t_a (min)	β= 0.069 t_a	q/q_e (from graph)	(x 0.27) = Q (m^3/s)
0	0	0	0	5	0.345	0.45	0.121
0.2	0.054	0.31	10.0	7.5	0.517	0.30	0.081
0.4	0.108	0.42	13.6	10	0.690	0.27	0.073

0.6	0.162	0.52	16.8	15	1.035	0.18	0.049
0.8	0.216	0.67	21.7	20	1.38	0.13	0.035
0.97	0.261	1.00	32.3	25	1.72	0.11	0.030
				30	2.07	0.09	0.0243
				40	2.76	0.06	0.0162
				50	3.45	0.04	0.0108
				60	4.14	0.035	0.0094
				65	4.50	0.03	0.0081



$$\frac{q_e t_a}{d \quad l}$$

$$\left(\frac{}{KS}\right)^{-}$$

$$K^{-} = \frac{}{0.012}\left(with\ i = \quad\right)$$

$$= 0.012\left(\frac{0.0027}{0.005}\right)^{-} = 0.01175$$

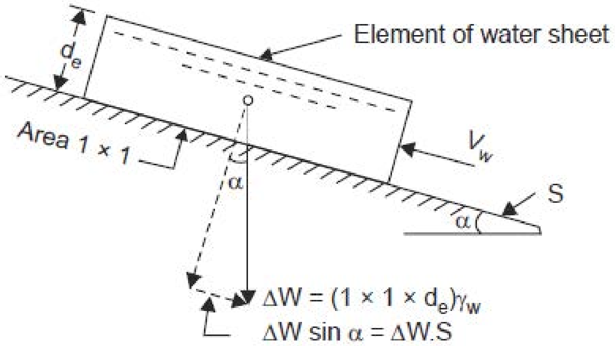
$$\frac{0.0027(\quad \times 60)}{0.01175 \times 200}$$

where t_a = time after the end of rain in min.

$\beta = 0.69\ t_a$

94. If the depth of surface detention on a smooth surface is 4 mm and the slope is 0.01, determine the wind velocity in the upslope direction required to counterbalance the component of gravity force down slope, if the rainfall rate is 60 mm/hr.

(a) Let the wind velocity be V_w towards upslope



$$Q = iA = \frac{0.060}{60 \times 60}(1 \times 1) = \frac{0.001}{60} m^3 / S$$

Force exerted by wind

$$F = \rho Q(\Delta V) = 1000 \times \frac{0.001}{60} V_w = \frac{V_w}{60} N$$

Force downslope due to gravity

$W s = \Delta W \sin \alpha = (1 \times 1 \times d_e)\, \gamma_w . S = 0.004\ (1000 \times 9.81)\ 0.01 = 0.3924$ N

Equating F = Ws, velocity of wind

$V_w = 0.3924 \times 60 = 23.544$ m/s

In the case of moderate slopes, such as airport runway surfaces, wind may play an important part in determining the rate of overland flow.

95. The runoff data for a river during a lean year along with the probable demands are given below. Can the demands be met with the available river flow? If so, how?

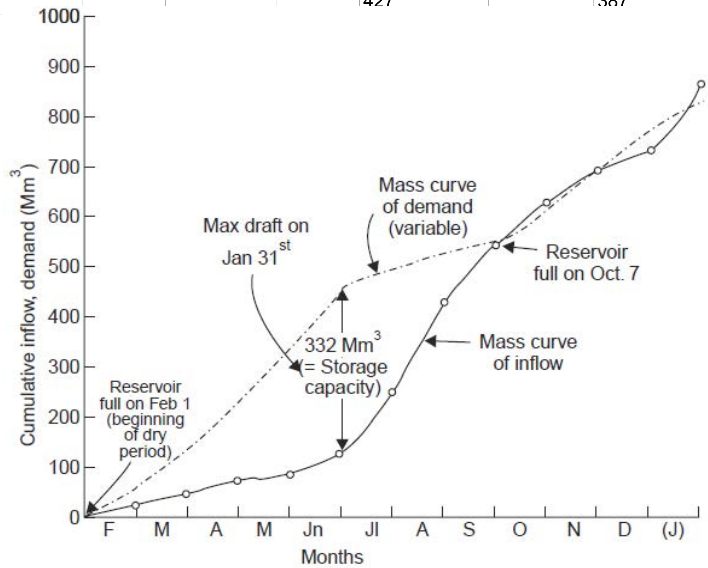
(b) What is the maximum uniform demand that can be met and what is the storage capacity required to meet this demand?

Month:	J	F	M	A	M	J	J	A	S	O	N	D
river flow (Mm^3)	135	23	27	21	40	120	185	112	87	63	42	
Demand (Mm^3)	60	55	80	102	100	121	38	30	25	59	85	75

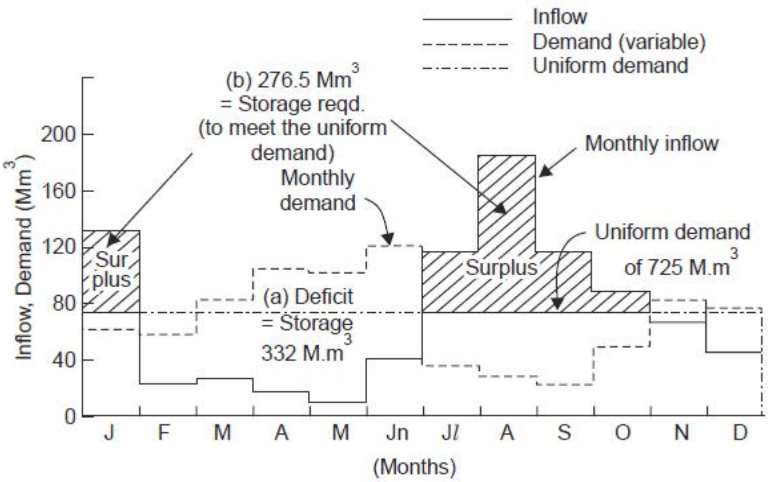
(a) Evaporation losses and the prior water rights of the downstream user are not given and hence not considered. The computation is made in the table. Since the cumulative surplus is more than the cumulative deficit the demands can be met with the available river flows, by constructing a reservoir with minimum storage capacity of 352 Mm^3, which is also the maximum departure of the mass curves (from the beginning of the severe dry period) of inflow and demand.

Month	Inflow (Mm^3)	Cumulative inflow (Mm^3)	Demand (Mm^3)	Cumulative demand (Mm^3)	Surplus (Mm^3)	Cumulative surplus (Mm^3)	Deficit (Mm^3)	Cumulative deficit (Mm^3)	Remarks
Jan.	135	870	60	830	75	75			Reservoir full by end of Jan
Feb.	23	23	55	55			32		Start of dry period

March	27	50	80	135			53		
April	21	71	102	237			81		
May	15	86	100	337			85		
June	40	126	121	458			81	332	Max. Draft = storage
July	120	246	38	496	82				
Aug.	185	431	30	526	155				
Sept.	112	543	25	551	87				
Oct.	87	630	59	610	28	352			
Nov.	63	693	85	695			22		
Dec.	42	735	75	770			33	55	
total	870					427		387	



In the bar graph, the monthly inflow and demand are shown by full line and dashed line, respectively. The area of maximum deficit (i.e., demand over surplus) is the storage capacity required and is equal to 332 Mm^3.



(b) The cumulative inflow in the lean year is 870 Mm^3. The maximum uniform demand that can be met is 870/12=72.5 Mm^3 per month. In the bar graph, the line of uniform demand is drawn at 72.5 $M.m^3$/month. The shaded area represents the surplus over the uniform demand (during the months of January, and July to October), which is the storage capacity required to meet the uniform demand, and is equal to

$(135) + (120 + 185 + 112 + 87) - 72.5 \times 5 = 276.5\ Mm^3$

96. The following are the data for a proposed medium size reservoir in Maharashtra. Determine LSL, FRL, HFL (MWL). What is the total length of the spillway fitted with crest gates assuming a pier width of 1.5 m (10 m span), flood detention of 4 hr and C = 2.2.

Catchment area 1200 km^2

Rainfall of 75% dependability 900 mm

Gross commanded area 25000 ha

Crpping pattern (proposed) and their water requirement (Δ)

(i) Kharif : Jowar—30% (0.45 m), Cotton—15% (0.75 m),

Rice—10% (1.20 m), Sugar cane—10% (1.90 m)

(ii) Rabi: Rice—20% (1.20 m), Wheat—20% (0.45 m)

(iii) Hot Weather: Vegetables—20% (0.60 m)

Area Capacity of Reservoir Site

Contour (m)	471	475	495	500	505
Area (ha)	0	36	178	242	323
Capacity (m^3)	0	0.90	19.35	29.85	43.98
Contour (m)	530	535	540	545	
Area (ha)	841	1002	1224	1480	
Capacity (m^3)	186.62	232.69	288.34	355.95	

River bed level 471.0

Top of bound level (TBL) 550.0

Silt load (expected) 250 $m^3/km^2/yr$ with a life of 100 yr

Evaporation losses 1.5 m over the mean area

Empirical formula for yield and flood of the region Inglis formula

(a) Yield from the basin:

$$Dependable\ runoff, R = \frac{(P-17.8)P}{254}$$

$$P = 95\ cm, R = \frac{(90-17.8)90}{254} = 25.6\ cm$$

Yield = AR = 1200 × 10^6 (0.256) = 307.2 × 10^6 m^3= 307 Mm^3

(b) Irrigation water requirement:

(i) Kharif: Jowar 25000 × 0.3 × 0.45 = 3380 ha-m

Cotton 25000 × 0.15 × 0.75 = 2820 ha-m

Rice 25000 × 0.10 × 1.20 = 3000 ha-m

Sugarcane 25000 × 0.10 × 1.90 = 4750 ha-m

(ii) Rabi: Rice 25000 × 0.20 × 1.20 = 6000 ha-m

Wheat 25000 × 0.20 × 0.45 = 2250 ha-m

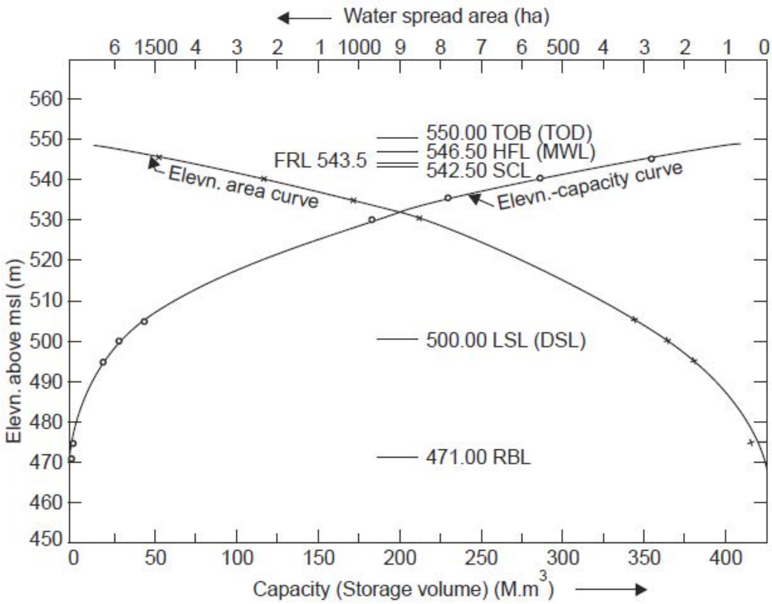
(iii) Hot weather:
Vegetables 25000 × 0.20 × 0.60 = 3000 ha-m

Total for the three crop seasons = 25200 ha-m.

Allowing 20% for conveyance losses, 10% for evaporation and seepage losses in the reservoir, 5% for overlap, and 5% as carryover storage-a total of 40%.

Live storage = 25200 × 1.40 = 35280 ha-m or = 352.8 Mm^3

While the annual yield is only 307 Mm^3 which limits the area irrigated unless supplemented by natural rainfall. Hence, a live storage of 307 $M.m^3$ is possible. Curves of eleven. vs. capacity and eleven. vs. water spread area are shown in the figure.



Dead storage = 250 × 1200 × 100 = 30 × 10^6 m^3 or 30 Mm^3

for which from the elevn-capacity curve, the lowest sill level of the sluice, LSL = 500.00 m

Gross storage = Dead storage + Live storage = 30 + 307 = 337 Mm^3

for which from the elevn-capacity curve, the full reservoir level

FRL = 543.50 m

Allowing a flood lift of 3 m, the maximum water level (MWL) or

HFL = 546.50 m

With a freeboard of 3.5 m, top of bound level or top of dam, TBL or

TOD = 550.00 m

Height of dam = TOD – RBL = 550.00 – 471.00 = 79 m

Length of spillway

Assuming the crest of the spillway is at FRL, the head on the spillway.

HFL – FRL = 546.5 – 543.5 = 3 m

Effective length of spillway per span

L_e = L – 0.1 nH= 10 – 0.1 × 2 × 3 = 9.4 m

Discharge over spillway per span

FAC = Capacity at HFL – capacity at FRL= 374–337 = 37 Mm^3

$$Spillway\ design\ flood, Q_D = MPF - \frac{FAC}{T}$$

where T = flood detention time in the reservoir and usually varies between 2.5 to 16 hr, and here given as T = 4 hr. The Maximum Flood discharge (MPF) may be calculated from the Inglis formula applicable for the region.

$$MPF \approx \frac{124A}{\sqrt{A+10.24}} \approx \frac{124\times 1200}{\sqrt{1200-10.24}} \approx 4280\, cumec$$

$$Q_D = 4280 - \frac{37\times 10^6}{4\times 60\times 60} = 1710\, cumec$$

$$No.\, of\ spans\ required = \frac{1710}{108} = 15.8, say, 16$$

Total length of spillway = 16 × 10 + 1.5 × 15=182.5 m

This length of the spillway can be reduced if the spillway crest (i.e., the sill of the crest gates) is kept at R.L. 542.50 m, so that the crest gates (height = 4 m) conserve water upto R.L. 543.50 m (FRL) or even above this level, as the floods subside (i.e., towards the end of flood season).

H = 546.5 – 542.5 = 4 m

q = 2.2 × 9.4 × $4^{3/2}$ = 165.5 cumec/span

FAC =374 – 322.5 = 51.5 M.m^3

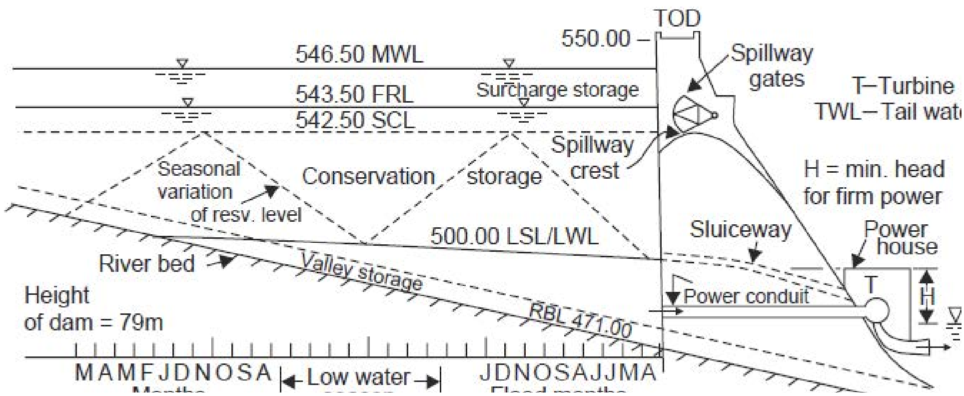
(since when the floods are forecast, the FRL is lowered to the spillway crest level by opening the crest agates)

$$Q_D = 4280 - \frac{51.5\times 10^6}{4\times 60\times 60} = 700\, cumec$$

$$No.\, of\ spans\ required = \frac{700}{165.5} = 4.23, say\ 5\ spans$$

Total length of spillway = 5 × 10 + 1.5 × 4 = 56 m

The various control levels are shown in the figure.

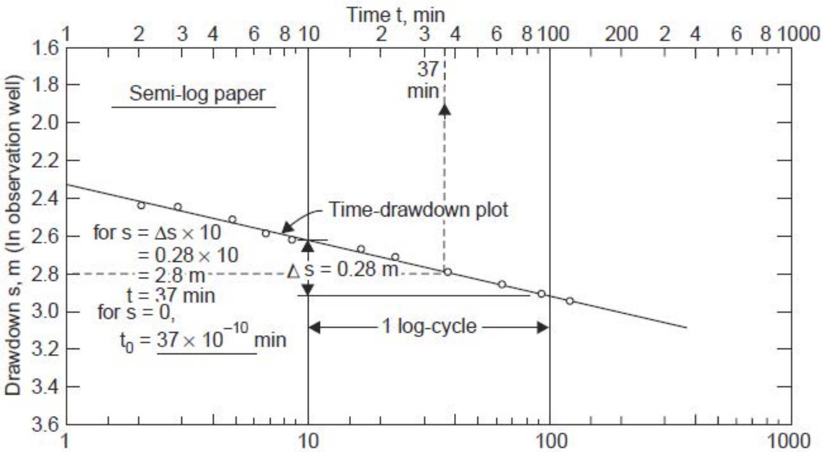


The flood absorption capacity thus reduces the peak of the MPF. Actually the MPF hydrograph into the reservoir (inflow hydrograph) is first obtained and then routed (for an assumed eleven of spillway crest, RL of sluice outlets) by making use of eleven-capacity-discharge relationship, and the peak of the outflow hydrograph thus derived gives the spillway design flood. For small reservoirs (catchment area < 100 km^2) the flood absorbing capacity is neglected as additional safety factor.

97. A 200 mm-well is pumped at the rate 1150 lpm. The drawdown data on an observation well 12.3 away from the pumped well are given below. Determine the transmissibility and storage coefficients of the aquifer. What will be the drawdown at the end of 180 days (a) in the observation well, (b) in the pumped well? Use the modified Theis method; under what conditions is this method valid?

Time (min)	2	3	5	7	9	12
Drawdown(m)	2.42	2.46	2.52	2.58	2.61	2.63
Time (min)	15	20	40	60	90	120
Drawdown(m)	2.67	2.71	2.79	2.85	2.91	2.94

The time-drawdown plot is shown in the figure, from which Δs = 0.28 m per log-cycle of t, and t_0 (for s = 0) is 37 × 10^{-10} min.



$$T = \frac{2.3Q}{4\pi\Delta s} = \frac{2.3 \times \frac{1.150}{60}}{4\pi(0.28)} = 0.0125\, m^2/s$$

$$S = \frac{2.25Tt_0}{r^2} = \frac{2.25(0.0125)37 \times 10^{-10} \times 60}{(12.3)^2} = 4.12 \times 10^{-11}$$

(a) Drawdown in the observation well after 180 days,

$$s=\frac{2.3Q}{4\pi T}\log\frac{2.25Tt}{r^2S}, u<0.01$$

$$s=\frac{2.3\left(\frac{1.150}{60}\right)}{4\pi(0.0125)}\log\frac{2.25(0.0125)180\times86400}{(12.3)^2\,4.12\times10^{-11}}=3.89\,m$$

(b) Drawdown in the pumped well after 180 days

$$s_w=\frac{2.3\left(\frac{1.150}{60}\right)}{4\pi(0.0125)}\log\frac{2.25(0.0125)180\times86400}{(0.100)^2\,4.12\times10^{-11}}=5.06\,m$$

The Jacob's method is valid for

$$u<0.01$$

$$\frac{r^2S}{4Tt}<0.01$$

98. A production well was pumped for 2 hr at a constant rate of 1600 lpm and the drawdowns in the seven nearby observation wells are given below. Determine the aquifer constants S and T.

Observation well	A	B	C	D	E	F	G
Distance from pumped well (m)	5	10	20	40	80	120	200
Drawdown (m)	5.35	4.35	3.35	2.35	1.4	0.8	0.3

The distance-drawdown plot is shown in Fig. D-4 from which Δs = 3.25 m per log cycle of r, and r_0 (for s = 0) is 210 m.

$$T=\frac{2.3Q}{2\pi\Delta s}=\frac{2.3\left(\frac{1.600}{60}\right)}{2\pi(3.25)}=0.003\,m^2/s\ for\ 2.6\times10^6\ lpd/m$$

$$S=\frac{2.25Tt}{r_0^2}=\frac{2.25(0.03)2\times60\times60}{210^2}=0.0011$$

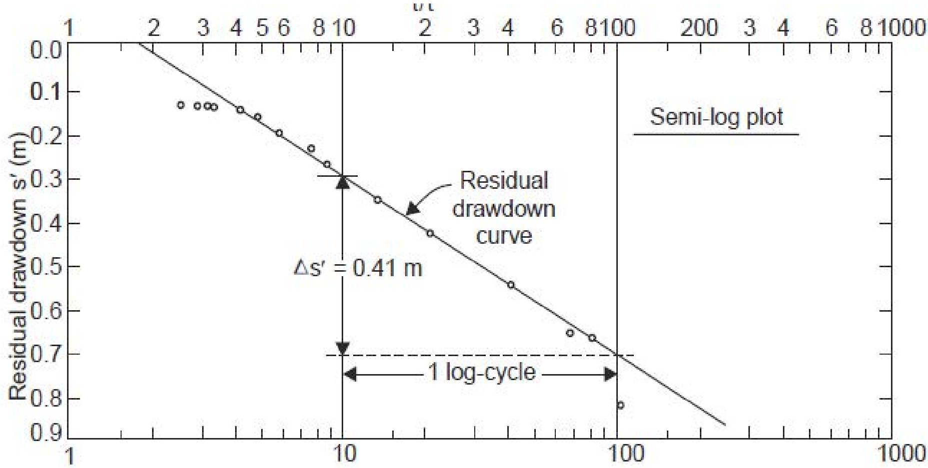
99. A 400-mm well was pumped at the rate of 2000 lpm for 200 min and the drawdown in an observation well 20 m from the pumping well was 1.51 m. The pumping was stopped and the residual draw downs during recovery in the observation well for 2

hr are given below. Determine the aquifer constants S and T.

Time since pumping stopped (min)	Residual drawdown (m)	Time since pumping stopped (min)	Residual drawdown (m)
2	0.826	45	0.180
3	0.664	50	0.159
5	0.549	55	0.155
10	0.427	60	0.149
16	0.351	70	0.146
20	0.305	80	0.140
25	0.271	90	0.134
30	0.241	100	0.131
35	0.220	110	0.131
40	0.201	120	0.131

The time-residual drawdown data are processed in the table and the Theis recovery curve is plotted on a semi-log paper as shown in the figure.

Time since pumping stopped t' (min)	Residual draw down s' (m)	Time since pumping started t= t_1 + t' (min)	Ratio (t/t')
2	0.826	202	101
3	0.664	203	68
5	0.549	205	41
10	0.427	210	21
16	0.351	216	13.5
20	0.305	220	11
25	0.271	225	9
30	0.241	230	7.7
35	0.220	235	6.7
40	0.201	240	6
45	0.180	245	5.45
50	0.159	250	5
55	0.155	255	4.65
60	0.149	260	4.33
70	0.146	270	3.86
80	0.140	280	3.5
90	0.134	290	3.22
100	0.131	300	3.00
110	0.131	310	2.82
120	0.131	320	2.66



From the recovery plot, Δs′ = 0.41 m per log-cycle of t/t′ and

$$T = \frac{2.3Q}{4\pi\Delta s'} = \frac{2.3\left(\frac{2.000}{60}\right)}{4\pi(0.41)} = 0.0149\, m^2 / s = 1.284\times10^6\ lpd / m$$

and S can be obtained from s_1 = 1.51 m after 200 min of pumping as

$$s_1 = \frac{2.3Q}{4\pi T}\log\frac{2.25Tt_1}{r^2 S}$$

$$\log\frac{2.25Tt_1}{r^2 S} = \frac{4\pi(0.0149)1.51}{2.3\left(\frac{2.000}{60}\right)} = 3.69$$

$$Anti\log of\ 3.69 = 4898$$

$$\frac{2.25(0.0149)200\times60}{20^2 S} = 4898$$

$$S = 0.000206$$

100. For a particular location the average net radiation is 185 W/m², air temperature is 28.5 °C, relative humidity is 55 percent, and wind speed is 2.7 m/s at a height of 2 m. Determine the open water evaporation rate in mm/d using the energy method.

Latent heat of vaporization in joules (J) per kg varies with T (°C), or $l_r = 2.501\times10^6 - 2370T$, so l_r = 2501-2.37×28.5 = 2433 kJ/kg, ρ_w = 996.3 kg/m³. The evaporation rate by the energy balance method is determined with R = 185 W/m²:

$E_r = R/(l_r\rho_w) = 185/(2433\times10^3 \times 996.3) = 7.63 \times 10^{-8}$ m/s

References

1. American Society of Civil Engineers (1996) Hydrology Handbook. ISBN 0784470146, 9780784470145.
2. Anderson, Malcolm G, Tim P Burt (1985) Hydrological Forecasting. Wiley John + Son. The University of Michigan. ISBN 047190614X, 9780471906148.
3. Banihabib, ME, Valipour M, Behbahani SMR, (2012) Comparison of Autoregressive Static and Artificial Dynamic Neural Network for the Forecasting of Monthly Inflow of Dez Reservoir. Journal of Environmental Sciences and Technology 13: 1-14.
4. Bedient, Philip B, Wayne Charles Huber (2002) Hydrology and floodplain analysis. Prentice Hall. The University of California. ISBN 0130322229, 9780130322227.
5. Beven Keith J (2004) Rainfall - Runoff Modelling: The Primer. Wiley. ISBN 0470866713, 9780470866719.
6. Bras, Rafael L (1990) Hydrology: an introduction to hydrologic science. Addison-Wesley. The University of California. ISBN 0201059223, 9780201059229.
7. Chow, Ven Te (1964) Handbook of applied hydrology: a compendium of water-resources technology, Volume 1. McGraw-Hill. The University of Michigan.
8. Chow Ven Te, David R Maidment, Larry W Mays (1988) Applied Hydrology. Tata McGraw-Hill Education. ISBN 007070242X, 9780070702424.
9. Dingman SL (2009) Physical hydrology. Macmillan Pub. Co. the University of California. ISBN 002329745X, 9780023297458.
10. Eagleson Peter S (1970) Dynamic hydrology. McGraw-Hill. The University of California.
11. Gray Donald M (1973) Handbook on the principles of hydrology: with special emphasis directed to Canadian conditions in the discussions, applications, and presentation of data, Volume 1. Water Information Center, inc. the University of Michigan. ISBN 0912394072, 9780912394077.
12. Grigg Neil S (1996) Water Resources Management: Principles, Regulations, and Cases. McGraw Hill Professional. ISBN 007024782X, 9780070247826.
13. Grigg Neil S (1985) Water resources planning. McGraw-Hill Ryerson, Limited. The University of Michigan. ISBN 0070247714, 9780070247710.
14. Goodman, Alvin S, David C Major (1984) Principles of water resources planning. Prentice-Hall. The University of California.
15. Gupta Ram S (2001) Hydrology and hydraulic systems. Waveland Press Incorporated. The University of Michigan. ISBN 1577660307, 9781577660309.
16. Haan Charles Thomas, Howard P Johnson, Donald L Brakensiek (1982) Hydrologic modeling of small watersheds. American Society of Agricultural Engineers. The University of Michigan. ISBN 0916150445, 9780916150440.
17. Han D (2010) Concise Hydrology. Bookboon. ISBN 978-87-7681-536-3.
18. Kuo Chin Y (1993) Engineering hydrology: proceedings of the symposium. American Society of Civil Engineers. the University of Michigan. ISBN 087262921X, 9780872629219.
19. Linsley Ray K, Max Adam Kohler, Joseph LH Paulhus (1982) Hydrology for engineers. McGraw-Hill. The University of Michigan. ISBN 0070379564, 9780070379565.
20. Loucks Daniel P, Jery R Stedinger, Douglas A Haith (1981) Water resource systems planning and analysis. Prentice-Hall. The University of California. ISBN 0139459235, 9780139459238.
21. Mays, Larry W (1996) Water Resources Handbook. McGraw-Hill Professional Publishing. ISBN 0070411506, 9780070411500.
22. Mays Larry W, Yeou-Koung Tung (2002) Hydrosystems Engineering and Management. Water Resources Publication. ISBN 1887201327, 9781887201322.
23. Mays Larry W (2010) Water Resources Engineering. John Wiley & Sons. ISBN 0470460644, 9780470460641.
24. McCuen Richard H (1998) Hydrologic analysis and design. Prentice Hall PTR. the University of Michigan. ISBN 0131349589, 9780131349582.
25. Maidment David R (1993) Handbook of hydrology. McGraw-Hill, the University of Michigan, ISBN 0070397325, 9780070397323.
26. Němec Jaromír (1972) Engineering hydrology. McGraw-Hill. The University of Wisconsin – Madison.
27. Ponce Victor Miguel (1994) Engineering Hydrology: Principles and Practices. Prentice Hall PTR. ISBN 0133154661, 9780133154665.
28. Raghunath HM (2006) Hydrology: Principles Analysis Design. New Age International (P) Ltd. ISBN (13):978-81-224-2332-7.
29. Rao K Nageswara (2006) Water resources management: realities and challenges. New Century Publications. ISBN 8177081063, 9788177081060.
30. Raudkivi AJ (1979) Hydrology: an advanced introduction to hydrological processes and modeling. Pergamon Press. The University of California.
31. Serrano Sergio E (1997) Hydrology for engineers, geologists, and environmental professionals: an integrated treatment of surface,

subsurface, and contaminant hydrology. HydroScience. ISBN 0965564398, 9780965564397.
32. Singh VP (1995) Environmental Hydrology. Springer. ISBN 079233549X, 9780792335498.
33. Singh Vijay P (1992) Elementary hydrology. Prentice Hall PTR. The University of California. ISBN 0132493845, 9780132493840.
34. Stephenson David (2010) Water Resources Management. Taylor & Francis. ISBN 9058095738, 9789058095732.
35. Valipour M, Banihabib ME, Behbahani SMR (2013) Comparison of the ARMA, ARIMA, and the autoregressive artificial neural network models in forecasting the monthly inflow of Dez dam reservoir. Journal of Hydrology 476: 433-441.
36. Valipour M (2013) INCREASING IRRIGATION EFFICIENCY BY MANAGEMENT STRATEGIES: CUTBACK AND SURGE IRRIGATION. ARPN Journal of Agricultural and Biological Science 8(1): 35-43.
37. Valipour M (2013) Necessity of Irrigated and Rainfed Agriculture in the World. Irrigation & Drainage Systems Engineering 9: 1-3.
38. Valipour M (2013) Evolution of Irrigation-Equipped Areas as Share of Cultivated Areas. Irrigation & Drainage Systems Engineering 2(1): 114-115.
39. Valipour M (2013) USE OF SURFACE WATER SUPPLY INDEX TO ASSESSING OF WATER RESOURCES MANAGEMENT IN COLORADO AND OREGON, US. Advances in Agriculture, Sciences and Engineering Research 3(2): 631-640.
40. Valipour M (2013) Estimation of Surface Water Supply Index Using Snow Water Equivalent. Advances in Agriculture, Sciences and Engineering Research 3(1): 587-602.
41. Valipour M (2013) Scrutiny of Inflow to the Drains Applicable for Improvement of Soil Environmental Conditions. In: The 1st International Conference on Environmental Crises and its Solutions, Kish Island, Iran.
42. Valipour M (2013) Comparison of Different Drainage Systems Usable for Solution of Environmental Crises in Soil. In: The 1st International Conference on Environmental Crises and its Solutions, Kish Island, Iran.
43. Valipour M, Mousavi SM, Valipour R, Rezaei E (2013) A New Approach for Environmental Crises and its Solutions by Computer Modeling. In: The 1st International Conference on Environmental Crises and its Solutions, Kish Island, Iran.
44. Valipour M, Banihabib ME, Behbahani SMR (2012) Monthly Inflow Forecasting Using Autoregressive Artificial Neural Network. Journal of Applied Sciences 12(20): 2139-2147.
45. Valipour M, Banihabib ME, Behbahani SMR (2012) Parameters Estimate of Autoregressive Moving Average and Autoregressive Integrated Moving Average Models and Compare Their Ability for Inflow Forecasting. Journal of Mathematics and Statistics 8(3): 330-338.
46. Valipour M (2012) Critical Areas of Iran for Agriculture Water Management According to the Annual Rainfall. European Journal of Scientific Research 84(4): 600-608.
47. Valipour M, Montazar AA (2012) Optimize of all Effective Infiltration Parameters in Furrow Irrigation Using Visual Basic and Genetic Algorithm Programming. Australian Journal of Basic and Applied Sciences 6(6): 132-137.
48. Valipour M, Montazar AA (2012) Sensitive Analysis of Optimized Infiltration Parameters in SWDC model. Advances in Environmental Biology 6(9): 2574-2581.
49. Valipour M (2012) Comparison of Surface Irrigation Simulation Models: Full Hydrodynamic, Zero Inertia, Kinematic Wave. Journal of Agricultural Science 4(12): 68-74.
50. Valipour M (2012) Sprinkle and Trickle Irrigation System Design Using Tapered Pipes for Pressure Loss Adjusting. Journal of Agricultural Science 4(12): 125-133.
51. Valipour M (2012) HYDRO-MODULE DETERMINATION FOR VANAEI VILLAGE IN ESLAM ABAD GHARB, IRAN. ARPN Journal of Agricultural and Biological Science 7(12): 968-976.
52. Valipour M, Montazar AA (2012) An Evaluation of SWDC and WinSRFR Models to Optimize of Infiltration Parameters in Furrow Irrigation. American Journal of Scientific Research 69: 128-142.
53. Valipour M (2012) Number of Required Observation Data for Rainfall Forecasting According to the Climate Conditions. American Journal of Scientific Research 74: 79-86.
54. Valipour M, Mousavi SM, Valipour R, Rezaei E (2012) Air, Water, and Soil Pollution Study in Industrial Units Using Environmental Flow Diagram. Journal of Basic and Applied Scientific Research 2(12): 12365-12372.
55. Valipour M (2012) Scrutiny of Pressure Loss, Friction Slope, Inflow Velocity, Velocity Head, and Reynolds Number in Center Pivot. International Journal of Advanced Scientific and Technical Research 2(5): 703-711.
56. Valipour M (2012) Ability of Box-Jenkins Models to Estimate of Reference Potential Evapotranspiration (A Case Study: Mehrabad Synoptic Station, Tehran, Iran). IOSR Journal of Agriculture and Veterinary Science (IOSR-JAVS) 1(5): 1-11.
57. Valipour M (2012) Effect of Drainage Parameters Change on Amount of Drain Discharge in Subsurface Drainage Systems. IOSR Journal of Agriculture and Veterinary Science (IOSR-JAVS) 1(4): 10-18.
58. Valipour M (2012) A Comparison between Horizontal and Vertical Drainage Systems (Include Pipe Drainage, Open Ditch Drainage, and Pumped Wells) in Anisotropic Soils. IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE) 4(1): 7-12.

59. Valipour M, Mousavi SM, Valipour R, Rezaei E (2012) SHCP: Soil Heat Calculator Program. IOSR Journal of Applied Physics (IOSR-JAP) 2(3): 44-50.

60. Valipour M (2012) Determining possible optimal values of required flow, nozzle diameter, and wetted area for linear traveling laterals. The International Journal of Engineering and Science (IJES) 1(1): 37-43.

61. Viessman, Warren, Gary L Lewis, John W Knapp (1989) Introduction to hydrology. Harper & Row. The University of California. ISBN 006046822X, 9780060468224.

62. Wanielista Martin P (1990) Hydrology and water quantity control, Volume 1. Wiley. The University of Michigan. ISBN 0471624047, 9780471624042.

63. Valipour M (2014) Importance of solar radiation, temperature, relative humidity, and wind speed for calculation of reference evapotranspiration. Archives of Agronomy and Soil Science. doi: 10.1080/03650340.2014.925107

64. Valipour M (2014) Temperature analysis of reference evapotranspiration models. Meteorological Applications. doi: 10.1002/met.1465

65. Valipour M (2014) Handbook of Irrigation Engineering Problems. OMICS.

66. Valipour M (2014) Handbook of Hydraulic Engineering Problems. OMICS.

67. Valipour M (2014) Future of agricultural water management in Americas. Journal of Agricultural Research 54(2):245-268.

68. Valipour M (2014) Land use policy and agricultural water management of the previous half of century in Africa. Applied Water Science. doi: 10.1007/s13201-014-0199-1

69. Valipour M (2014) Handbook of Water Engineering Problems. OMICS.

70. Valipour M (2014) Future of agricultural water management in Europe based on socioeconomic indices. Acta Advances in Agricultural Sciences 2(7):1-18.

71. Valipour M (2014) Application of new mass transfer formulae for computation of evapotranspiration. Journal of Applied Water Engineering and Research 2(1):33-46.

72. Valipour M (2014) Use of average data of 181 synoptic stations for estimation of reference crop evapotranspiration by temperature-based methods. Water Resources Management. doi: 10.1007/s11269-014-0741-9

73. Valipour M (2014) Study of different climatic conditions to assess the role of solar radiation in reference crop evapotranspiration equations. Archives of Agronomy and Soil Science. doi: 10.1080/03650340.2014.941823

74. Valipour M (2014) Comparison of mass transfer-based models to predict reference crop evapotranspiration Meteorological Applications. Accepted.

75. Valipour M (2014) Analysis of potential evapotranspiration using limited weather data. Applied Water Science. Accepted.

76. Valipour M (2014) Pressure on renewable water resources by irrigation to 2060. Acta Advances in Agricultural Sciences 2(8).

77. Valipour M (2014) Comparative evaluation of radiation-based methods for estimation of reference evapotranspiration. Journal of Hydrologic Engineering. Accepted.

78. Valipour M (2014) Drainage, waterlogging, and salinity. Archives of Agronomy and Soil Science. doi: 10.1080/03650340.2014.905676

79. Valipour M (2013) Need to update of irrigation and water resources information according to the progresses of agricultural knowledge. Agrotechnology. S10:e001. doi: 10.4172/2168-9881.S10-e001

80. Valipour M, Mousavi SM, Valipour R, Rezaei E, (2013) Deal with environmental challenges in civil and energy engineering projects using a new technology. Journal of Civil & Environmental Engineering S4:127. doi: 10.4172/2165-784X.1000127

81. Valipour M (2014) Future of the area equipped for irrigation. Archives of Agronomy and Soil Science. doi: 10.1080/03650340.2014.905675

82. Ward (1967) Principles Of Hydrology 4e. McGraw-Hill Education (India) Pvt Limited. ISBN 1259002241, 9781259002243.