REGIONAL TRAINING COURSE

MaB

WATERSHED RESOURCES MANAGEMENT AND ENVIRONMENTAL MONITORING

PROBLEM MANUAL

A

REGIONAL TRAINING COURSE

IN

NATURAL RESOURCES MANAGEMENT

AND

ENVIRONMENTAL MONITORING

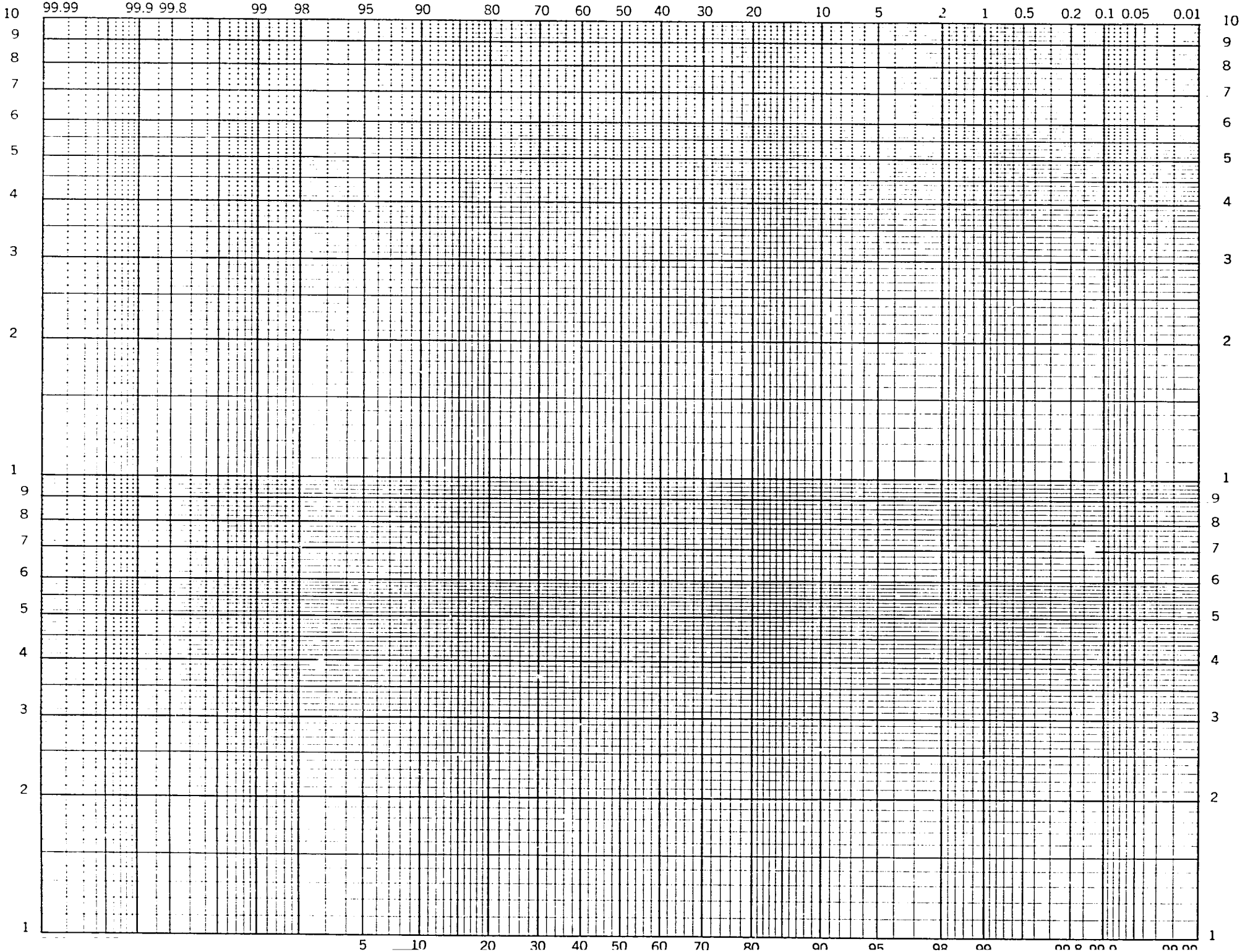
PROBLEM MANUAL

HYDROLOGY EQUIPMENTS

		Price
Box 1	1 3-foot wading rod,	$14.00
Box 2	1 DH-48 suspended sediment sampler	105.00
Box 3	1 recording hygrothermograph and package of 100 charts	406.80
Box 4	1 rain gage stakes	14.00
Box 5	1 8-inch rain gage,	169.00
Box 6	1 rain gage stand	32.00
Box 7	1 rain gage (Belfort recording) with chart drive for battery powered drive	585.00
Box 8 & Box 10	1 portable water testing portable laboratory with conductivity and pH	1225.00
Box 9	1 max. and min. thermometer mounted on stainless steel backs and townsend support	96.00
Box 11	1 pygmy type water current meter	525.00

K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 8043



$$T_a = \frac{L^{1.15}}{500\ H^{0.38}}$$

$$L = \frac{18{,}000^{1.15}}{500}$$

PREFACE

To supplement the syllabus entitled WATERSHED RESOURCES MANAGEMENT AND ENVIRONMENTAL MONITORING, this manual has been prepared to furnish relevant problem sets to natural resource managers and land use planners concerned with watershed resources. Through experiences obtained in solving these problem sets, a better understanding of watershed resources management should be acquired. This prototype manual includes problem sets on STATISTICAL METHODS AND INFERENCES, THE WATER BUDGET, RAINFALL-RUNOFF ANALYSIS, SOIL EROSION, RESERVOIR MANAGEMENT, MODELING, ROMOTE SENSING, and DECISION MAKING.

Specific materials for the problem manual have been provided by:

Kenneth N. Brooks, University of Minnesota
Peter F. Ffolliott, University of Arizona
William O. Rasmussen, University of Arizona
John L. Thames, University of Arizona

In essence, this problem manual consists of two parts -- one presenting the problem sets and the other giving solutions to the problems. This manual is an initial effort to develop problem sets for a Regional Training Course in WATERSHED RESOURCES MANAGEMENT AND ENVIRONMENTAL MONITORING, to be revised as necessary to accommodate the needs of subsequent courses.

Peter F. Ffolliott
October 1981

REGIONAL TRAINING COURSE

WATERSHED RESOURCES MANAGEMENT AND ENVIRONMENTAL MONITORING

PROBLEM MANUAL

PROBLEM SET I: STATISTICAL METHODS AND INFERENCES

A. Probability Theory
B. Frequency Distribution
C. Graphical Frequency Analysis - Example
D. Analytical Frequency Analysis - Example
E. Graphical Frequency Analysis Problem
F. Effects of Clearcutting on Annual Peak Discharge Frequency Curve

PROBLEM SET II: WATER BUDGET EXERCISE

PROBLEM SET III: RAINFALL-RUNOFF ANALYSIS

A. Mean Basin Precipitation
B. Loss Rates
C. Unit Hydrograph Derivation and Hydrograph Reconstruction
D. Estimating Runoff From Small Watersheds - A Summary of the Soil Conservation Service Method
E. Estimating Runoff Using the Soil Conservation Service Method

PROBLEM SET IV: SOIL EROSION

PROBLEM SET V: RESERVOIR MANAGEMENT

A. Single Reservoir Multi-Purpose Simulation Example
B. Reservoir Management Problem

PROBLEM SET VI: MODELING

PROBLEM SET VII: REMOTE SENSING

PROBLEM SET VIII: DECISION MAKING

PROBLEM I

STATISTICAL METHODS AND INFERENCES

A. Probability Theory

1. Find the probability of rolling exactly 2 sixes in 5 rolls of an unbiased die.

2. Find the probability of rolling no more than 2 sixes in 5 rolls of an unbiased die.

3. Find the probability of rolling at least 2 sixes in 5 rolls of an unbiased die.

4. What is the probability of:

   a. Getting exactly a 100-year flood next year?

   b. Not getting a flood greater than the 50-year flood in 50 years?

   c. Getting at least one flood greater than the 1000-year flood in 50 years?

B. Frequency Distribution

1. Compute the probabilities of all possible outcomes of one toss of a pair of unbiased die. Plot the probabilities as a relative frequency histogram. What is the name of the resulting probability distribution?

2. Plot the cumulative-frequency histogram on arithmic paper. What is the probability of rolling a nine or greater with one toss of the dice?

3. The following data were drawn randomly from a normally distributed parent population.

Observations
5
8
4
11
8
7
6
8
6

a. From the above sample, compute the following:

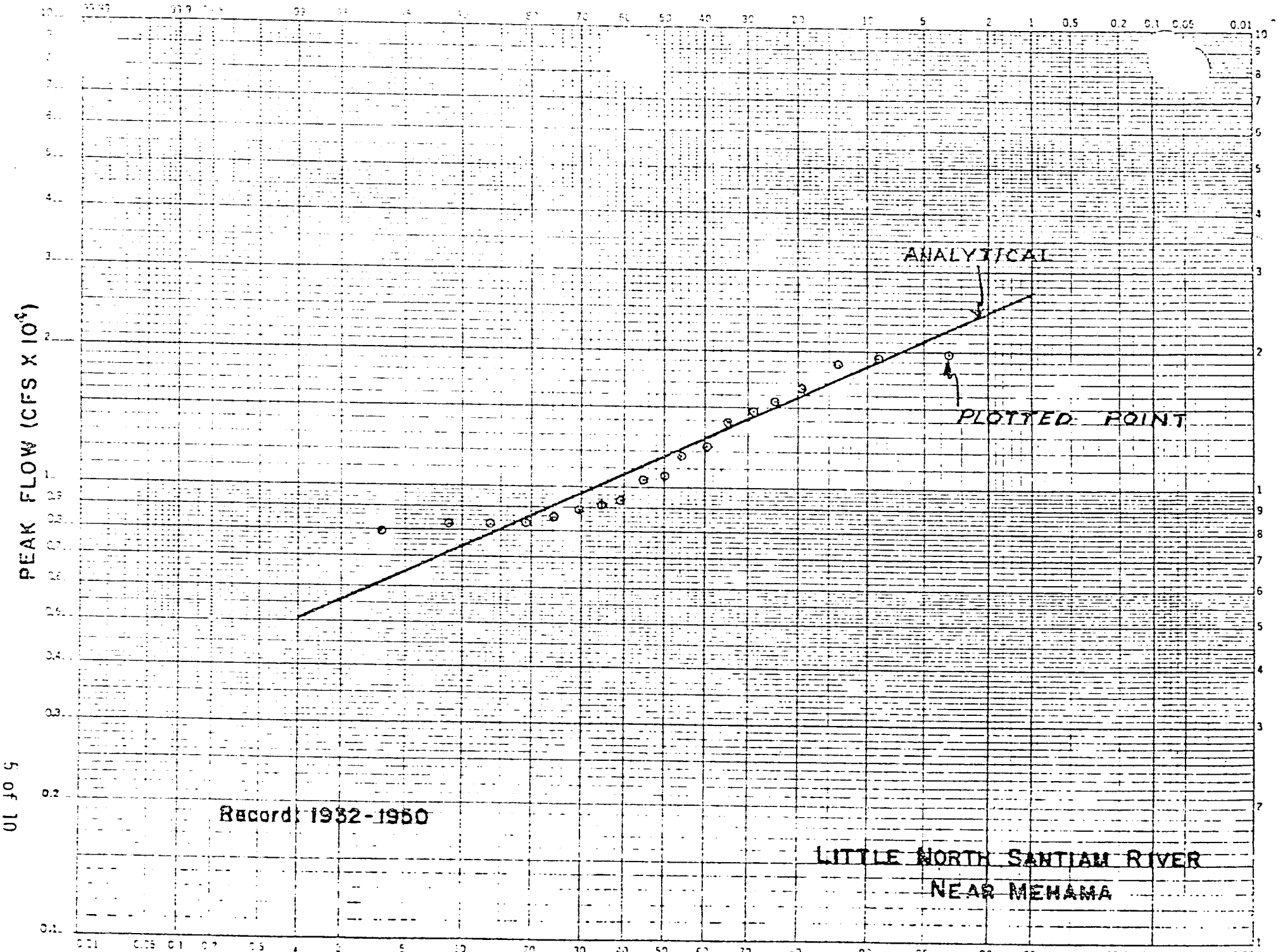
Mean	Variance
Median	Standard Deviation
Mode	Skew Coefficient

b. Estimate the probability of exceeding the following deviations from the mean: (Assume a normal distribution)

(1) 0

(2) +2

(3) +4


PEAK FLOW (CFS X 10³)
ANALYTICAL
PLOTTED POINT
Record: 1932-1950
LITTLE NORTH SANTIAM RIVER
NEAR MEHAMA

C. Example of Graphical Frequency Analysis

Little North Santiam River near Mehama, Oregon

N = 19 years

Water Year	Inst. Peak (cfs)	Order of Mag.	Plotting Position (%) (from Table I-1)
1932	13900	19900	3.58
33	10600	19400	8.8
34	18900	18900	13.9
35	10400	16500	19.1
36	12200	15300	24.2
37	8200	14800	29.4
38	16500	13900	34.5
39	8570	12200	39.7
1940	8200	11700	44.8
41	8330	10600	50.0
42	9300	10400	55.2
43	19400	9300	60.3
44	7900	9120	65.5
45	11700	8860	70.6
46	19900	8570	75.8
47	15300	8330	80.9
48	14800	8200	86.1
49	9120	8200	91.2
1950	8860	7990	96.4

Table I-1. Median plotting positions (from (Beard 1962).

PLOTTING POSITIONS IN PERCENT
(EXCEEDENCE FREQUENCY IN EVENTS PER HUNDRED YEARS)

(See par. 3-05)

m \ N	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	N \ m
	50.0	29.3	20.6	15.9	13.0	10.9	9.43	8.30	7.43	6.70	6.11	5.62	5.20	4.83	4.52	4.25	4.00	3.78	3.58	3.41	3.25	3.10	1
		70.7	50.0	36.6	31.5	26.5	22.9	20.2	18.1	16.3	14.9	13.7	12.7	11.8	11.0	10.3	9.8	9.2	8.8	8.3	7.9	7.6	2
			79.4	61.4	50.0	42.2	36.5	32.1	28.7	25.9	23.7	21.8	20.1	18.7	17.5	16.4	15.5	14.7	13.9	13.2	12.6	12.0	3
44	89.4			84.1	68.5	57.8	50.0	44.0	39.4	35.6	32.4	29.8	27.6	25.7	24.0	22.5	21.2	20.1	19.1	18.1	17.3	16.5	4
43	96.2	98.4			87.0	73.5	63.5	56.0	50.0	45.2	41.2	37.9	35.1	32.6	30.5	28.6	27.0	25.5	24.2	23.0	22.0	21.0	5
42	93.9	96.1	98.4			89.1	77.1	67.9	60.6	54.8	50.0	46.0	42.5	39.6	37.0	34.7	32.8	31.0	29.4	27.9	26.6	25.4	6
41	91.7	93.9	96.0	98.3			90.6	79.8	71.3	64.4	58.8	54.0	50.0	46.5	43.5	40.8	38.5	36.4	34.5	32.8	31.3	29.9	7
40	89.4	91.5	93.6	95.9	98.3			91.7	81.9	74.1	67.6	62.1	57.5	53.5	50.0	46.9	44.2	41.8	39.7	37.7	36.0	34.4	8
39	87.2	89.2	91.3	93.5	95.8	98.2			92.6	83.7	76.3	70.2	64.9	60.4	56.5	53.1	50.0	47.3	44.8	42.6	40.6	38.8	9
38	84.9	86.9	88.9	91.1	93.3	95.7	98.2			93.3	85.1	78.2	72.4	67.4	63.0	59.2	55.8	52.7	50.0	47.5	45.3	43.3	10
37	82.7	84.6	86.6	88.7	90.8	93.2	95.6	98.1			93.9	86.3	79.9	74.3	69.5	65.3	61.5	58.2	55.2	52.5	50.0	47.8	11
36	80.4	82.3	84.2	86.2	88.4	90.6	93.0	95.5	98.1			94.4	87.3	81.3	76.0	71.4	67.2	63.6	60.3	57.4	54.7	52.2	12
35	78.2	80.0	81.9	83.8	85.9	88.1	90.4	92.8	95.3	98.0			94.8	88.2	82.5	77.5	73.0	69.0	65.5	62.3	59.4	56.7	13
34	75.9	77.4	79.5	81.4	83.4	85.5	87.8	90.1	92.6	95.2	98.0			95.2	89.0	83.6	78.8	74.5	70.6	67.2	64.0	61.2	14
33	73.7	75.4	77.1	79.0	81.0	83.0	85.2	87.4	89.8	92.4	95.1	97.9			95.5	89.7	84.5	79.9	75.8	72.1	68.7	65.6	15
32	71.4	73.0	74.8	76.6	78.5	80.5	82.6	84.8	87.1	89.6	92.2	94.9	97.9			95.8	90.2	85.3	80.9	77.0	73.4	70.1	16
31	69.2	70.7	72.4	74.2	76.0	77.9	80.0	82.1	84.3	86.7	89.3	91.9	94.8	97.8			96.0	90.8	86.1	81.9	78.0	74.6	17
30	66.9	68.4	70.1	71.7	73.5	75.4	77.4	79.4	81.6	83.9	86.3	88.9	91.7	94.6	97.7			96.2	91.2	86.8	82.7	79.0	18
29	64.6	66.1	67.7	69.3	71.0	72.9	74.7	76.7	78.9	81.1	83.4	85.9	88.6	91.4	94.4	97.6			96.4	91.7	87.4	83.5	19
28	62.4	63.8	65.3	66.9	68.6	70.3	72.1	74.1	76.1	78.3	80.5	82.9	85.5	88.2	91.1	94.2	97.6			96.6	92.1	88.0	20
27	60.1	61.5	63.0	64.5	66.1	67.8	69.5	71.4	73.4	75.4	77.6	79.9	82.4	85.0	87.8	90.8	94.1	97.5			96.8	92.4	21
26	57.9	59.2	60.6	62.1	63.6	65.2	66.9	68.7	70.6	72.6	74.7	77.0	79.3	81.9	84.6	87.4	90.5	93.8	97.4			96.9	22
25	55.6	56.9	58.3	59.7	61.1	62.7	64.3	66.0	67.9	69.8	71.8	74.0	76.2	78.7	81.3	84.0	87.0	90.2	93.6	97.3			
24	53.4	54.6	55.9	57.2	58.7	60.2	61.7	63.4	65.1	67.0	68.9	71.0	73.2	75.5	78.0	80.6	83.5	86.5	89.8	93.3	97.2		
23	51.1	52.3	53.5	54.8	56.2	57.6	59.1	60.7	62.4	64.1	66.0	68.0	70.1	72.3	74.7	77.2	80.0	82.9	86.0	89.4	93.0	97.0	23
22	48.9	50.0	51.2	52.4	53.7	55.1	56.5	58.0	59.6	61.3	63.1	65.0	67.0	69.1	71.4	73.8	76.4	79.2	82.2	85.5	89.0	92.8	22
21	46.6	47.7	48.8	50.0	51.2	52.5	53.9	55.3	56.9	58.5	60.2	62.0	63.9	65.9	68.1	70.4	72.9	75.6	78.4	81.5	84.8	88.5	21
20	44.4	45.4	46.5	47.6	48.8	50.0	51.3	52.7	54.1	55.7	57.3	59.0	60.8	62.7	64.8	67.0	69.4	71.9	74.6	77.6	80.8	84.2	20
19	42.1	43.1	44.1	45.2	46.3	47.5	48.7	50.0	51.4	52.8	54.4	56.0	57.9	59.6	61.5	63.6	65.9	68.3	70.8	73.6	76.6	79.9	19
18	39.9	40.8	41.7	42.8	43.8	44.9	46.1	47.3	48.6	50.0	51.5	53.0	54.6	56.4	58.2	60.2	62.3	64.6	67.1	69.7	72.6	75.7	18
17	37.6	38.5	39.4	40.3	41.3	42.4	43.5	44.7	45.9	47.2	48.5	50.0	51.5	53.2	54.9	56.8	58.8	61.0	63.3	65.8	68.4	71.4	17
16	35.4	36.2	37.0	37.9	38.9	39.8	40.9	42.0	43.1	44.3	45.6	47.0	48.5	50.0	51.6	53.4	55.3	57.3	59.5	61.8	64.4	67.1	16
15	33.1	33.9	34.7	35.5	36.4	37.3	38.3	39.3	40.4	41.5	42.7	44.0	45.4	46.8	48.4	50.0	51.8	53.7	55.7	57.9	60.2	62.8	15
14	30.8	31.6	32.3	33.1	33.9	34.8	35.7	36.6	37.6	38.7	39.8	41.0	42.1	43.6	45.1	46.6	48.2	50.0	51.9	53.9	56.2	58.6	14
13	28.6	29.3	29.9	30.7	31.4	32.2	33.1	34.0	34.9	35.9	36.9	38.0	39.2	40.4	41.8	43.2	44.7	46.3	48.1	50.0	52.0	54.3	13
12	26.3	27.0	27.6	28.3	29.0	29.7	30.5	31.3	32.1	33.0	34.0	35.0	36.1	37.3	38.5	39.8	41.2	42.7	44.3	46.1	48.0	50.0	12
11	24.1	24.6	25.2	25.8	26.5	27.1	27.9	28.6	29.4	30.2	31.1	32.0	33.0	34.1	35.2	36.4	37.7	39.0	40.5	42.1	43.8	45.7	11
10	21.8	22.3	22.9	23.4	24.0	24.6	25.3	25.9	26.6	27.4	28.2	29.0	29.9	30.9	31.9	33.0	34.1	35.4	36.7	38.2	39.8	41.4	10
9	19.6	20.0	20.5	21.0	21.5	22.1	22.6	23.3	23.9	24.6	25.3	26.0	26.8	27.7	28.6	29.6	30.6	31.7	32.9	34.2	35.6	37.2	9
8	17.3	17.7	18.1	18.6	19.0	19.5	20.0	20.6	21.1	21.7	22.4	23.0	23.8	24.5	25.3	26.2	27.1	28.1	29.2	30.3	31.6	32.9	8
7	15.1	15.4	15.8	16.2	16.6	17.0	17.4	17.9	18.4	18.9	19.5	20.1	20.7	21.3	22.0	22.8	23.6	24.4	25.4	26.4	27.4	28.6	7
6	12.8	13.1	13.4	13.8	14.1	14.5	14.8	15.2	15.7	16.1	16.6	17.1	17.6	18.1	18.7	19.4	20.0	20.8	21.6	22.4	23.4	24.3	6
5	10.6	10.8	11.1	11.3	11.6	11.9	12.2	12.6	12.9	13.3	13.7	14.1	14.5	15.0	15.4	16.0	16.5	17.1	17.8	18.5	19.2	20.1	5
4	8.3	8.5	8.7	8.9	9.2	9.4	9.6	9.9	10.2	10.4	10.7	11.1	11.4	11.8	12.2	12.6	13.0	13.5	14.0	14.5	15.2	15.8	4
3	6.1	6.2	6.4	6.5	6.7	6.8	7.0	7.1	7.4	7.6	7.8	8.1	8.3	8.6	8.9	9.2	9.5	9.8	10.2	10.6	11.0	11.5	3
2	3.8	3.9	4.0	4.1	4.2	4.3	4.4	4.5	4.7	4.8	4.9	5.1	5.2	5.4	5.6	5.8	6.0	6.2	6.4	6.7	7.0	7.2	2
1	1.56	1.60	1.64	1.68	1.72	1.76	1.81	.86	1.91	1.96	2.02	2.08	2.15	2.22	2.29	2.36	2.45	2.54	2.64	2.74	2.85	2.97	1
m \ N	44	43	42	41	40	39	38	37	36	35	34	33	32	31	30	29	28	27	26	25	24	23	N \ m

D. Analytical Frequency Computation: Little North Santiam River Near Mahama, Oregon (1932-1950).

Log Q = X	X^2	X^3
4.2989	18.4805	79.4460
4.2878	18.3852	78.8322
4.2765	18.2884	78.2106
4.2175	17.7873	75.0179
4.1847	17.5117	73.2813
4.1703	17.3914	72.5274
4.1430	17.1644	71.1123
4.0864	16.6987	68.2374
4.0682	16.5502	67.3297
4.0253	16.2030	65.2221
4.0170	16.1363	64.8195
3.9685	15.7490	62.4999
3.9599	15.6808	62.0944
3.9474	15.5820	61.5084
3.9330	15.4685	60.8376
3.9206	15.3711	60.2639
3.9138	15.3178	59.9509
3.9138	15.3178	59.9509
3.9025	15.2295	59.4331
$X = 77.2351$	$\Sigma X^2 = 314.3136$	$\Sigma X^3 = 1280.5755$

$(\Sigma X)^2 = 5965.2607$

$(\Sigma X)^2/N = 313.9611$

$\overline{X} = \Sigma X/N = 4.0650$

$$S = \sqrt{\frac{\Sigma X^2 - (\Sigma X)^2/N}{N - 1}} = \sqrt{\frac{(314.3136 - 313.9611)}{18}} = .1399$$

$S^3 = .00274$

$$g = \frac{N^2 \Sigma X^3 - 3N\ \Sigma X \Sigma X^2 + 2\ (\Sigma X)^3}{N\ (N-1)\ (N-2)\ S^3}$$

$$= \frac{361\ (1280.5755) - 57\ (77.2351)\ (314.3136) + 2\ (460727.5067)}{15.9304}$$

$$= \frac{462287.7555 - 1383734.413 + 921455.0134}{15.9304}$$

$= -.6001$

Analytical Frequency Computations (Continued).

P_∞	1.0	10	50	90	99	
K (g=0)[a]	2.33	1.28	0	-1.28	-2.33	
$K \cdot S$	.3260	.1791	0	- .1791	- .3260	
$\text{Log } Q = K \cdot S + \overline{X}$	4.3910	4.2441	4.0650	3.8859	3.7390	
Q (cfs)	24,600	17,500	11,600	7,690	5,484	
P_N[b]	1.79	11.4	50.0	88.6	98.2	Plotted

[a] From Table I-2 with skew = 0

[b] From Table I-3 (N-1 = 18); the flow values are then plotted at these adjusted exceedence frequency values.

Table I-2. Pearson Type III Coordinates (from Beard 1962).

PEARSON TYPE III COORDINATES

g (Skew coefficient)	k = Magnitude in standard deviations from mean for exceedence percentages of:												
	0.01	0.1	1.0	5	10	30	50	70	90	95	99	99.9	99.99
1.0	5.92	4.54	3.03	1.87	1.34	0.38	-0.16	-0.61	-1.12	-1.31	-1.59	-1.80	-1.88
0.8	5.48	4.25	2.90	1.83	1.34	0.42	-0.13	-0.60	-1.16	-1.38	-1.74	-2.03	-2.18
0.6	5.04	3.96	2.77	1.79	1.33	0.45	-0.09	-0.58	-1.19	-1.45	-1.88	-2.28	-2.53
0.4	4.60	3.67	2.62	1.74	1.32	0.48	-0.06	-0.57	-1.22	-1.51	-2.03	-2.54	-2.92
0.2	4.16	3.38	2.48	1.69	1.30	0.51	-0.03	-0.55	-1.25	-1.58	-2.18	-2.81	-3.32
0.0	3.73	3.09	2.33	1.64	1.28	0.52	0.00	-0.52	-1.28	-1.64	-2.33	-3.09	-3.73
-0.2	3.32	2.81	2.18	1.58	1.25	0.55	0.03	-0.51	-1.30	-1.69	-2.48	-3.38	-4.16
-0.4	2.92	2.54	2.03	1.51	1.22	0.57	0.06	-0.48	-1.32	-1.74	-2.62	-3.67	-4.60
-0.6	2.53	2.28	1.88	1.45	1.19	0.58	0.09	-0.45	-1.33	-1.79	-2.77	-3.96	-5.04
-0.8	2.18	2.03	1.74	1.38	1.16	0.60	0.13	-0.42	-1.34	-1.83	-2.90	-4.25	-5.48
-1.0	1.88	1.80	1.59	1.31	1.12	0.61	0.16	-0.38	-1.34	-1.87	-3.03	-4.54	-5.92
Skew Coefficients Commonly Used													
.00	3.73	3.09	2.33	1.64	1.28	0.52	0.00	-0.52	-1.28	-1.64	-2.33	-3.09	-3.73
-.04	3.65	3.03	2.30	1.63	1.27	0.53	0.01	-0.52	-1.28	-1.65	-2.36	-3.15	-3.82
-.12	3.48	2.92	2.24	1.60	1.26	0.54	0.02	-0.51	-1.29	-1.67	-2.42	-3.26	-3.99
-.23	3.26	2.77	2.16	1.57	1.25	0.55	0.03	-0.50	-1.30	-1.70	-2.50	-3.42	-4.23
-.32	3.08	2.68	2.09	1.54	1.23	0.56	0.05	-0.49	-1.31	-1.72	-2.56	-3.55	-4.42
-.37	2.98	2.58	2.05	1.52	1.22	0.57	0.06	-0.48	-1.32	-1.73	-2.60	-3.63	-4.53
-.40	2.92	2.54	2.03	1.51	1.22	0.57	0.06	-0.48	-1.32	-1.74	-2.62	-3.67	-4.60

NOTE: Approximate transformations between normal deviate (X) and Pearson Type III deviate k can be accomplished with the following equation:

$$k = \frac{2}{g}\left\{\left[\frac{g}{6}\left(X - \frac{g}{6}\right) + 1\right]^3 - 1\right\}$$

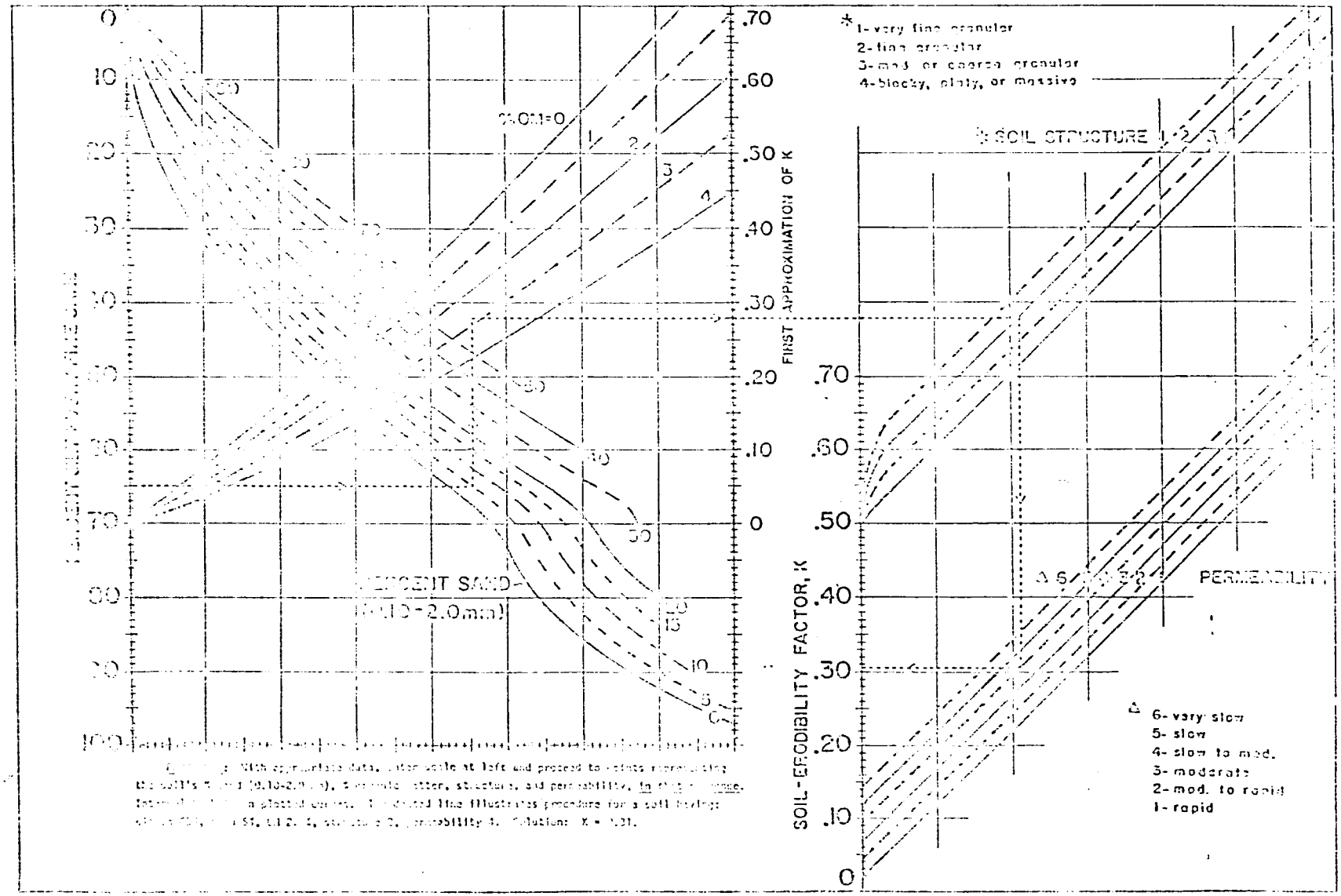


FIGURE 3.-40. Soil-erodibility nomograph. Where the silt fraction does not exceed 70 percent, the equation is $100\,K = 2.1\,M^{1.14}\,(10^{-4})\,(12 - a) + 3.25\,(b - 2) + 2.5\,(c - 3)$ where M = (percent si + vfs) (100 − percent c), a = percent organic matter, b = structure code, and c = profile permeability class.

E. Graphical Frequency Analysis Problem

Background

Streamflow records for the Columbia River at The Dalles, Oregon, have been kept since 1858, making it the longest record station in the region. Reservoir system regulation studies of the Columbia River Basin, however, utilize a 30-year sample period from 1929-1958. This period is evaluated because it contains rather long sequences of low-runoff events. System evaluations of reservoir operations are thus made considering requirements for irrigation, hydropower, flood control, and water quality, under the most severe recorded low-flow events. From such studies, it has been determined that an April-August volume of 74 $m^3 \times 10^9$ or less would cause critical shortages.

1. For this workshop, you are to develop graphical frequency curves for:

   a. April-August volumes (m^3) for the 30-year period, and

   b. Annual peak discharges (cumecs) for the same 30-year period. In addition to the 30-year record, assume that you have information (from newspaper reports, photographs, and high water marks) that the flood of record was 35,300 cumecs and occurred in 1894. How would you use this additional information in developing your annual peak frequency curves?

2. Based on your frequency curves, answer the following questions:

   a. What is the probability of having an April-August flow volume equal to or less than 74 $m^3 \times 10^9$?

   b. What is the magnitude of the annual peak flood that you would expect to be equalled or exceeded once in 100 years? How does your answer compare with the .01 exceedence frequency value of 34,000 cumecs determined from the longer record? Do you feel that this 30-year peak-streamflow record is representative of the longer record?

Table I-3. **TABLE OF P_N VERSUS P_∞ IN PERCENT** (from Beard 1962).

For use with samples drawn from a normal population

N-1 \ P_∞	50.0	30.0	10.0	5.0	1.0	0.1	0.01
1	50.0	37.2	24.3	20.4	15.4	12.1	10.2
2	50.0	34.7	19.3	14.6	9.0	5.7	4.3
3	50.0	33.6	16.9	11.9	6.4	3.5	2.3
4	50.0	33.0	15.4	10.4	5.0	2.4	1.37
5	50.0	32.5	14.6	9.4	4.2	1.79	.92
6	50.0	32.2	13.8	8.8	3.6	1.38	.66
7	50.0	31.9	13.5	8.3	3.2	1.13	.50
8	50.0	31.7	13.1	7.9	2.9	.94	.39
9	50.0	31.6	12.7	7.6	2.7	.82	.31
10	50.0	31.5	12.5	7.3	2.5	.72	.25
11	50.0	31.4	12.3	7.1	2.3	.64	.21
12	50.0	31.3	12.1	6.9	2.2	.58	.18
13	50.0	31.2	11.9	6.8	2.1	.52	.16
14	50.0	31.1	11.8	6.7	2.0	.48	.14
15	50.0	31.1	11.7	6.6	1.96	.45	.13
16	50.0	31.0	11.6	6.5	1.90	.42	.12
17	50.0	31.0	11.5	6.4	1.84	.40	.11
18	50.0	30.9	11.4	6.3	1.79	.38	.10
19	50.0	30.9	11.3	6.2	1.74	.36	.091
20	50.0	30.8	11.3	6.2	1.70	.34	.084
21	50.0	30.8	11.2	6.1	1.67	.33	.078
22	50.0	30.8	11.1	6.1	1.63	.31	.073
23	50.0	30.7	11.1	6.0	1.61	.30	.068
24	50.0	30.7	11.0	6.0	1.58	.29	.064
25	50.0	30.7	11.0	5.9	1.55	.28	.060
26	50.0	30.6	10.9	5.9	1.53	.27	.057
27	50.0	30.6	10.9	5.9	1.51	.26	.054
28	50.0	30.6	10.9	5.8	1.49	.26	.051
29	50.0	30.6	10.8	5.8	1.47	.25	.049
30	50.0	30.6	10.8	5.8	1.45	.24	.046
40	50.0	30.4	10.6	5.6	1.33	.20	.034
60	50.0	30.3	10.4	5.4	1.22	.16	.025
120	50.0	30.2	10.2	5.2	1.11	.13	.017
∞	50.0	30.0	10.0	5.0	1.00	.10	.010

NOTE: P_N values above are usable approximately with Pearson Type III distributions having small skew coefficients.

Table I-4. Columbia River at the Dalles; Streamflow Volumes (April-August) and Annual Peaks for Water Years 1929-1958.

Water Year	April-August Volume ($m^3 \times 10^9$)	Annual Peak Streamflow (m^3/sec)
1929	80	14190
30	81	10140
31	74	9570
32	122	17840
33	130	21440
34	109	13422
35	104	14750
36	106	16310
37	84	11720
38	120	18460
39	93	11980
1940	89	11640
41	76	10000
42	105	13510
43	134	16590
44	70	10340
45	93	15350
46	125	17610
47	116	16420
48	152	30530
49	111	19820
1950	141	24470
51	132	19650
52	127	18120
53	117	20390
54	141	18090
55	113	18690
56	154	27520
57	128	24580
1958	115	22260

Note: 1894 = 35300 (cumecs) flood of record-determined from high water marks.

F. Effects of Clearcutting on Annual Peak Discharge Frequency Curve

Problem:

A paired watershed approach was used to determine the effects of clearcutting on water yield and flooding. Meteorological and hydrological data were collected on a 52.6 ha watershed (control) and a 34 ha watershed which was to be partially clearcut. Nine years of pre-harvest or calibration data were collected (from 1962 to 1970). Eighty percent of the hardwood forest on the 34 ha watershed was clearcut in 1971 and data were collected on both watersheds until 1979. Statistical analyses showed that the magnitude of peaks and stormflow volumes (from rainfall) from the clearcut watershed were increased for 8 years following clearcutting.

Requirements:

Your job is to perform a frequency analysis of the clearcut watershed and the control watershed to determine how the annual peak streamflow-frequency relationship changed. In performing this analysis, do the following:

1. Construct before and after clearcutting frequency curves for both the control and clearcut watersheds.

    a. Plot annual peaks according to the graphical plotting procedure illustrated in the handout.

    b. Construct analytical frequency curves assuming the skew coefficient is zero ($g = 0$).

2. Discuss the effects of clearcutting on the frequency distribution of annual peak streamflow. How reliable are these results? Discuss briefly.

Annual Peak Streamflow

Year	Control Watershed	Treated Watershed
	$m^3\ sec^{-1}\ km^{-2}$	
1962	0.156	0.10
1963	.026	.047
1964	.147	.121
1965	.032	.082
1966	.067	.091
1967	.010	.017
1968	.017	.042
1969	.018	.064
1970	.013	.030
-Clearcut-		
1971	.032	.175
1972	.057	.183
1973	.021	.077
1974	.067	.127
1975	.071	.196
1976	.010	.012
1977	.018	.038
1978	.064	.115
1979	.354	.617

PROBLEM II

WATER BUDGET EXERCISE

One hundred ninety hectares of mixed hardwoods are to be clearcut on a watershed which drains into a water supply reservoir. The city which receives water from this reservoir is interested in determining how much of an increase in water yield might be expected as a result of this cut. In order to provide a conservative estimate of possible increases, a water budget was calculated on a dry 21-month period. A water budget has been calculated for this dry period under pre-cut conditions (Table II).

1. Using the following information and the same rainfall and potential ET data in Table II, estimate the change in water yield in cubic meters for the 21-month period.

   a. Soils are clay-loam with an average depth of about 1.7 m; plant available water (field capacity minus permanent wilting point) averages 164 mm per meter of soil depth.

   b. The root systems of the mature hardwood stand fully occupy the soil, therefore, the total soil water which can be depleted by ET is 279 mm (1.7 m x 164 mm/m).

   c. Studies have shown that under light herbaceous plant cover, a condition similar to a clearcut area, soil water is depleted by ET only to a depth of about 0.6 m.

2. Discussion questions:

   a. What assumptions were made in applying the water budget in this study?

   b. What factors would be important in determining the quantities of increased water yield which would be available at the reservoir site?

Table II- WATER BUDGET EXERCISE FOR A HARDWOOD-COVERED WATERSHED - BEFORE CLEARCUTTING

	Year 1												Year 2								
	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN
	mm																				
1. 1/ Average rainfall	58	103	133	98	100	48	27	4	31	42	36	12	50	120	140	105	90	95	65	20	46
2. 2/ Initial soil moisture	56	57	140	273	279	279	279	248	163	67	0	0	0	0	100	240	279	279	279	279	210
3. Total avail. moisture	114	160	273	371	379	328	306	252	194	109	36	12	50	120	240	345	369	374	344	299	256
4. 3/ Potential ET	57	20	0	0	3	13	58	89	127	173	157	107	57	20	0	0	3	13	58	89	127
5. 4/ Actual ET	57	20	0	0	3	13	58	89	127	109	36	12	50	20	0	0	3	13	58	89	127
6. Remaining available moisture	57	140	273	371	376	314	248	163	67	0	0	0	0	100	240	345	366	361	286	210	129
7. 5/ Final soil moisture	57	140	273	279	279	279	248	163	67	0	0	0	0	100	240	279	279	279	279	210	129
8. 6/ Runoff	0	0	0	92	97	35	0	0	0	0	0	0	0	0	0	66	87	82	7	0	0

1/ Average over the watershed for each month of record.

2/ At start of each month. Same as "final soil moisture" of previous month.

3/ Average annual values for the month, as estimated by Thornthwaite's method.

4/ Total available moisture, or potential ET, whichever is smaller.

5/ At end of month. Same as "initial soil moisture" for next month. This value cannot be larger than the soil-water holding capacity determined for the watershed, for this watershed 279 mm.

6/ Runoff occurs when the remaining available moisture exceeds the water holding capacity for the watershed (279 mm).

PROBLEM III

RAINFALL-RUNOFF ANALYSIS

A. Mean Basin Precipitation

Problem Setting

A hydrologic study is to be performed on a watershed in the Rocky Mountains of the western U.S. The purpose of the study is to determine the effects of clearcutting lodgepole pine on runoff, erosion, and sedimentation, and selected water quality constituents. A mature stand of lodgepole pine (*Pinus contorta* Dougl.) occupies 85 percent and quaking aspen (*Populus tremuloides* Michx.) 10 percent of the watershed, respectively.

The intermittent stream flowing from the watershed (Figure 1) is gauged at the outlet. A recording (weighing) rain gauge is located at Station A. Stations B and C are nonrecording (standard) rain gauges.

As part of this study, a large storm is to be analyzed in terms of rainfall depth over the watershed, its recurrence interval, and the net rainfall available for either runoff or for satisfying soil water deficits.

Requirements

As a forest hydrologist you are to present a brief report to your supervisor, containing the information below. Computations and explanations are to be presented in a well-organized and concise format.

1. Precipitation:

You are to determine the average depth of rainfall over the watershed, which resulted from an intense storm on September 3-4, 1980. The precipitation for this period for Station A is on the attached recording chart. The daily rainfall measured at Stations B and C (gauges are measured each day at 1700 hours) for this storm were:

	September 1980			Storm Total (in.)
	2	3	4	
	- - - -	inches	- - - -	
Station B	0.0	1.5	2.8	4.3
Station C	0.0	1.5	2.6	4.1

a. Determine the average depth of rainfall for the above storm by means of:

(1) Arithmetic average

(2) Thiessen polygon method (show polygons on map), and

(3) Isohyetal method (use intervals of 0.2 inches and illustrate your isohyets on the map).

b. Select the value that would provide the best estimate of average rainfall depth for this watershed and explain, briefly, why it is the best value.

c. Using the recording chart for Station A, distribute the average rainfall depth over time and construct a hyetograph (a graph with rainfall intensity in inches per hour vs. time in hours). Use graph paper.

d. Using the attached frequency curve for Station A, determine the exceedence probability for the storm. What is the corresponding recurrence interval for this storm? Write a sentence explaining what this storm represents in terms of either probability or recurrence interval.

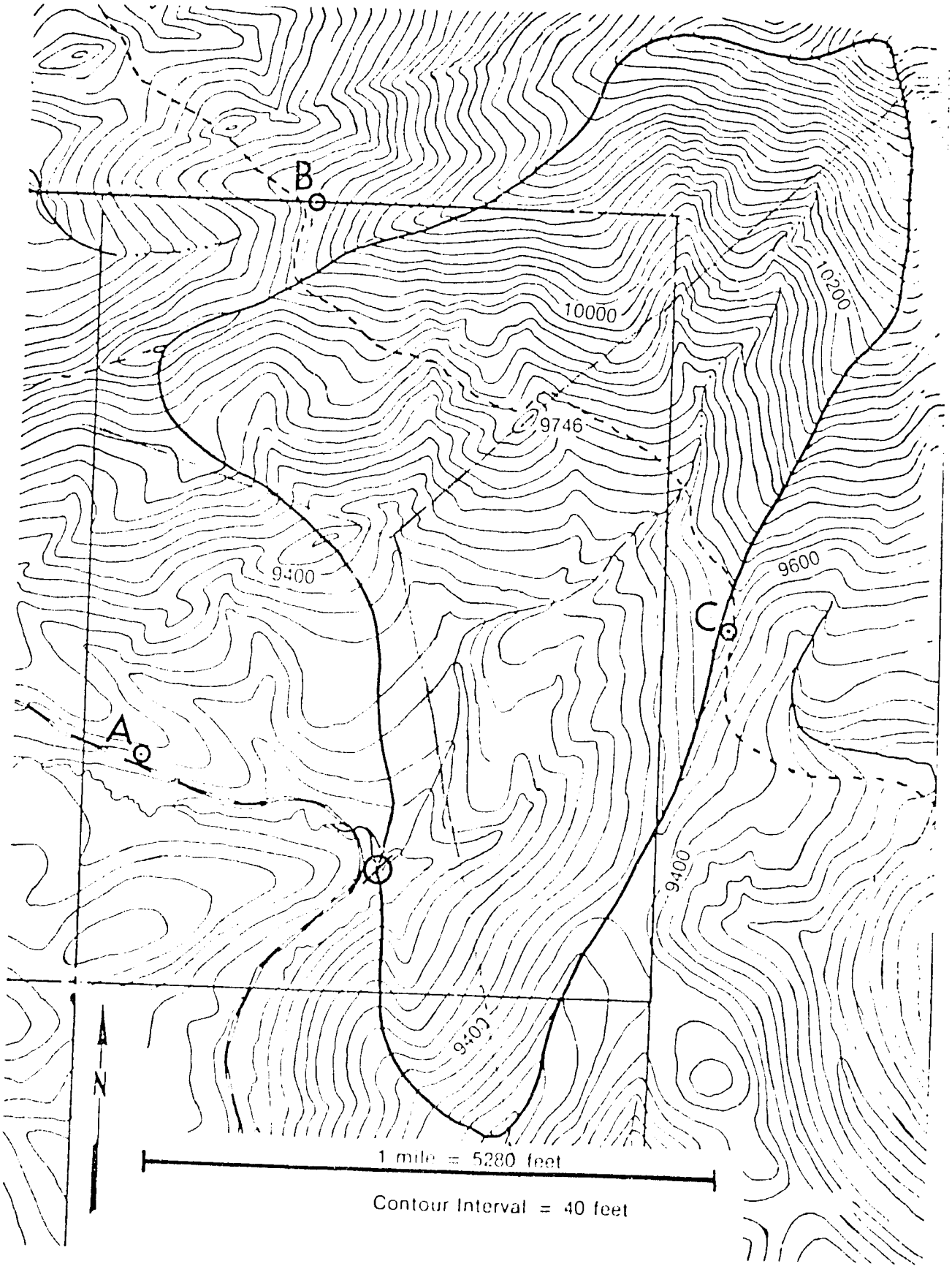
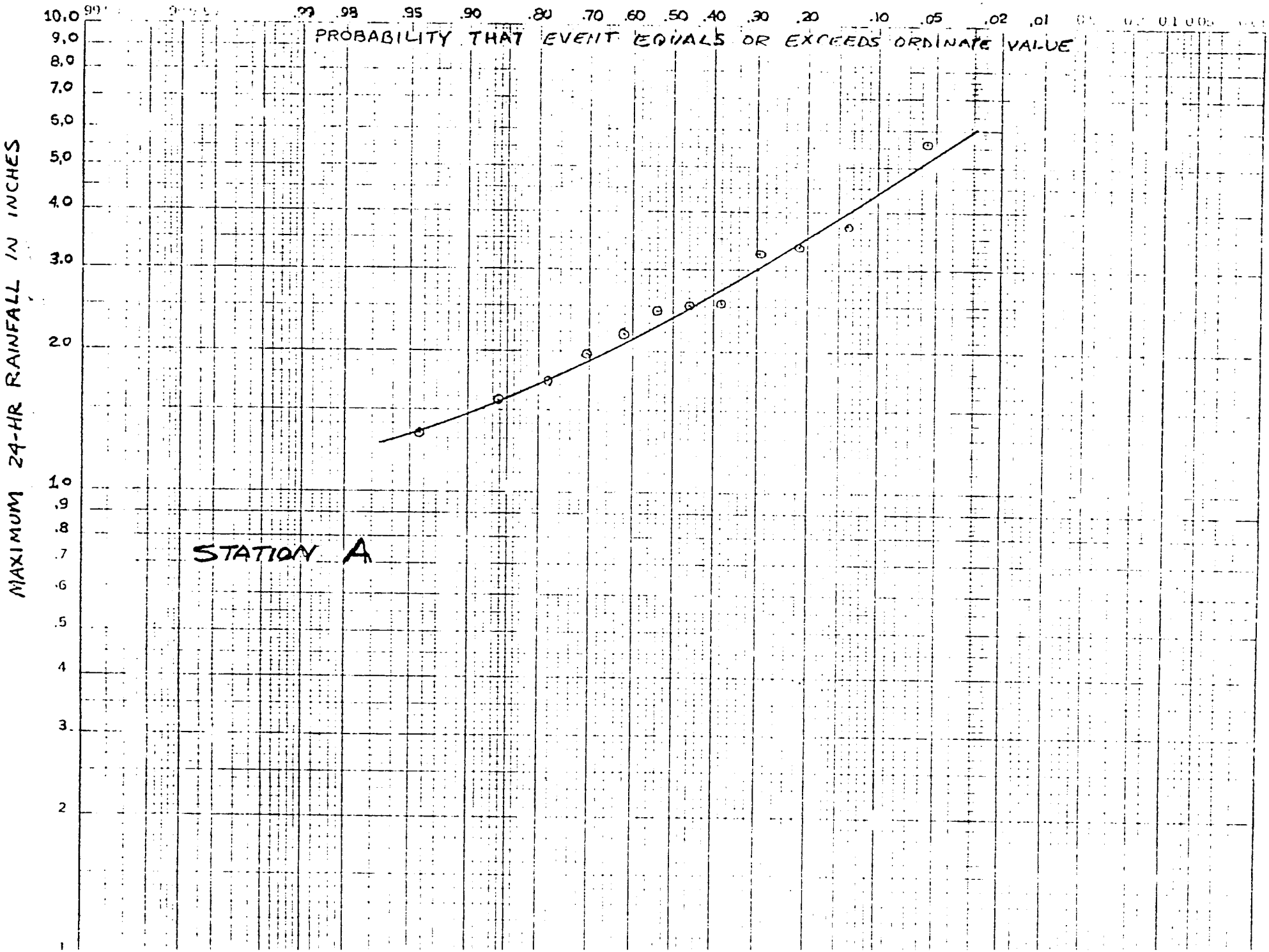

B
10000
10200
9746
9400
9600
C
A
9400
9400
N
1 mile = 5280 feet
Contour Interval = 40 feet

Figure 1


PROBABILITY THAT EVENT EQUALS OR EXCEEDS ORDINATE VALUE
.99 .98 .95 .90 .80 .70 .60 .50 .40 .30 .20 .10 .05 .02 .01
MAXIMUM 24-HR RAINFALL IN INCHES
10.0 9.0 8.0 7.0 6.0 5.0 4.0 3.0 2.0 1.0 .9 .8 .7 .6 .5 .4 .3 .2 .1
STATION A

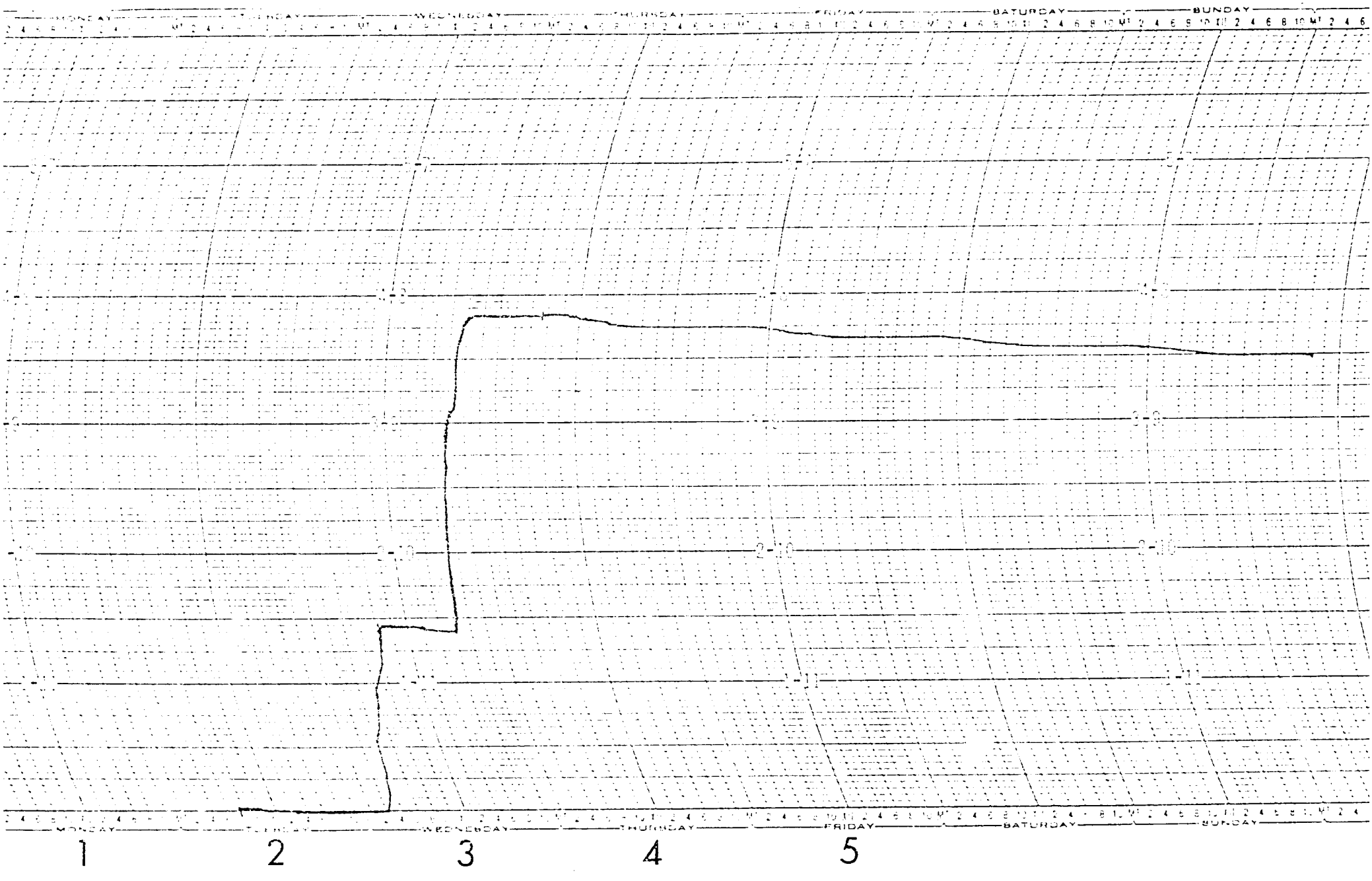

MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
1
2
3
4
5
Sept.

Table 2. Runoff curve numbers for hydrologic soil-cover complexes (Antecedent moisture condition II, and $I_a = 0.2\ S$)

Land use and treatment or practice	Hydrologic condition	Hydrologic soil group A	B	C	D
Fallow					
Straight row	--	77	86	91	94
Row crops					
Straight row	Poor	72	81	88	91
Straight row	Good	67	78	85	89
Contoured	Poor	70	79	84	88
Contoured	Good	65	75	82	86
Contoured and terraced	Poor	66	74	80	82
Contoured and terraced	Good	62	71	78	81
Small grain					
Straight row	Poor	65	76	84	88
Straight row	Good	63	75	83	87
Contoured	Poor	63	74	82	85
Contoured	Good	61	73	81	84
Contoured and terraced	Poor	61	72	79	82
Contoured and terraced	Good	59	70	78	81
Close-seeded legumes or rotation meadow					
Straight row	Poor	66	77	85	89
Straight row	Good	58	72	81	85
Contoured	Poor	64	75	83	85
Contoured	Good	55	69	78	83
Contoured and terraced	Poor	63	73	80	83
Contoured and terraced	Good	51	67	76	80
Pasture or range					
No mechanical treatment	Poor	68	79	86	89
No mechanical treatment	Fair	49	69	79	84
No mechanical treatment	Good	39	61	74	80
Contoured	Poor	47	67	81	88
Contoured	Fair	25	59	75	83
Contoured	Good	6	35	70	79
Meadow	Good	30	58	71	78
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	25	55	70	77
Farmsteads	--	59	74	82	86
Roads[a]					
Dirt	--	72	82	87	89
Hard surface	--	74	84	90	92

[a] Including rights-of-way.

Table 2 for AMC II, the average antecedent moisture condition. The soil types are described in Table 3 while Table 4 presents a description of the hydrologic condition for different land uses.

Table 1. Antecedent moisture conditions for estimating runoff curve numbers.

Antecedent Moisture Condition (AMC)	Rain in previous 5 days
I	Less than 0.50 inches
II	0.50 to 1.50 inches
III	More than 1.50 inches

Table 5 relates the runoff curve number (CN) for wet (AMC III) and dry (AMC I) antecedent moisture conditions to the CN for AMC II.

Table 2 was developed primarily for humid and subhumid watersheds. For forest-range complexes in the more arid western United States, Figure 1 is preferred to estimate runoff curve numbers for various soil types and ground cover densities.

A graphical solution to the runoff equation (10) is shown in Figure 2.

Substituting 9 in 8 gives:

$$Q = \frac{(P - 0.2\ S)^2}{P + 0.8\ S} \tag{10}$$

Equation 10 is the rainfall-runoff relation used in the SCS method of estimating direct runoff from storm rainfall where the units generally used are

Q = storm runoff in inches

P = storm rainfall in inches

S = potential maximum retention in inches

Curve Numbers

The potential maximum retention S is related to a curve number (CN) by the empirical expression

$$CN = \frac{1000}{10 + S} \tag{11}$$

Curve numbers are dependent on

- Soil type
- General hydrologic condition of watershed
- Land use and treatment or practice
- Antecedent moisture condition (AMC)

Curve numbers for various hydrologic soil-cover complexes were obtained from an analysis of the historical records of many watersheds. First, the antecedent moisture condition was grouped into three levels according to the total mount of rainfall in a 5-day period preceding a storm (see Table 1). Then, relationships were obtained between rainfall and runoff for a variety of on-site watershed parameters. Results are shown in

The initial abstraction is brought into the relation by subtracting it from the rainfall. The equivalent of equation 1 becomes:

$$\frac{F}{S} = \frac{Q}{P - I_a} \tag{5}$$

where I_a is the initial abstraction $F \leq S$ and $Q \leq (P - I_a)$. The parameter S includes I_a; that is, $S = S' + I_a$.

Equation 2 becomes:

$$F = (P - I_a) - Q \tag{6}$$

equation 3 becomes:

$$\frac{(P - I_a) - Q}{S} = \frac{Q}{(P - I_a)} \tag{7}$$

and equation 4 becomes:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{8}$$

which is the rainfall-runoff relation with the initial abstraction taken into account.

The initial abstraction consists mainly of interception, infiltration, and surface storage, all of which occur before runoff beings. The insert on Figure 2 shows the position of I_a in a typical storm. To remove the necessity for estimating these variables in equation 8, the relation between I_a and S (which includes I_a) was developed by means of rainfall and runoff data from experimental small watersheds. The empirical relationship is:

$$I_a = 0.2\ S \tag{9}$$

where

F = actual retention

S' = potential maximum retention ($S' \geq F$)

Q = actual runoff

P = potential maximum runoff ($P \geq Q$)

Equation 1 applies to on-site runoff; for large watersheds there is a lag in the appearance of the runoff at the stream gage, and the double-mass curve produces a different relation. But if storm totals for P and Q are used equation 1 does apply even for large watersheds because the effects of the lag are removed.

The parameter S' in equation 1 does not contain the initial abstraction and differs from the parameter S to be used later. The retention S' is a constant for a particular storm because it is the maximum that can occur under the existing conditions of the storm continues without limit. The retention F varies because it is the difference between P and Q at any point on the mass curve, or:

$$F = P - Q \tag{2}$$

Equation 1 can therefore be rewritten:

$$\frac{P - Q}{S'} = \frac{Q}{P} \tag{3}$$

Solving for Q produces the equation:

$$Q = \frac{P^2}{P + S'} \tag{4}$$

which is a rainfall-runoff relation in which the initial abstraction is ignored.

of the runoff curve number (CN), which is another indicator of the probability of flow types: the larger the CN the more likely that the estimate is of surface runoff.

The Rainfall-Runoff Relation

The most generally available rainfall data in the United States are the amounts measured at nonrecording rain gages, and it was for the use of such data or their equivalent that the rainfall-runoff relation was developed. The data are totals for one or more storms occurring in a calendar day, and nothing is known about the time distributions. The relation therefore excludes time as an explicit variable; this means that rainfall intensity is ignored.

If records of natural rainfall and runoff for a large storm over a small area are used, a plotting of accumulated runoff versus accumulated rainfall will show that runoff starts after some rain accumulates (there is an "initial abstraction" of rainfall) and that the double-mass line curves, becoming asymptotic to a straight line has a 45-degree slope. The relation between rainfall and runoff can be developed from this plotting, but a better understanding of the relation is had by first studying a storm in which rainfall and runoff begin simultaneously (the initial abstraction does not occur). For the simpler storm the relation between rainfall, runoff, and retention (the rain not converted to runoff) at any point on the mass curve can be expressed as:

$$\frac{F}{S'} = \frac{Q}{P} \qquad (1)$$

Surface runoff occurs only when the rainfall rate is greater than the infiltration rate. The runoff flows on the watershed surface to the point of reference. This type appears in the hydrograph after the initial demands of interception, infiltration, and surface storage have been satisfied. It varies during the storm and ends during or soon after it. Surface runoff flowing down dry channels of watersheds in arid, semiarid, or subhumid climates is reduced by transmission losses, which may be large enough to eliminate the runoff entirely.

Subsurface flow occurs when infiltrated rainfall meets an underground zone of low transmission, travels above the zone to the soil surface downhill, and appears as a seep or spring. This type is often called "quick return flow" because it appears in the hydrograph during or soon after the storm.

Base flow occurs when there is a fairly steady flow from natural storage. The flow comes from lakes or swamps, or from an aquifer replenished by infiltrated rainfall or surface runoff, or from "bank storage," which is supplied by infiltration into channel banks as the stream water level rises and which drains back into the stream as the water level falls. This type seldom appears soon enough after a storm to have any influence on the rates of the hydrograph for that storm, but base flow from a previous storm will increase the rates. Base flow must be taken into account in the design of the principal spillway of a floodwater-retarding structure.

All types do not regularly appear on all watersheds. Climate is one indicator of the probability of the types. In arid regions the flow on smaller watersheds is nearly always surface runoff, but in humid regions it is generally more of the subsurface type. But a long succession of storms produces subsurface or base flow even in dry climates although the probability of this occurring is less in dry climates than in wet climates.

In flood hydrology it is customary to deal separately with base flow and to combine all other types into direct runoff, which consists of channel runoff, surface runoff, and subsurface flow in unknown proportions. The SCS method estimates direct runoff, but the proportions of surface runoff and subsurface flow (channel runoff is ignored) can be appraised by means

D. Estimating Runoff From Small Watersheds (A Summary of the Soil Conservation Service Method*)

Introduction

The SCS method of estimating direct runoff from storm rainfall is based on methods developed by SCS hydrologists in the last three decades, and it is in effect a consolidation of these earlier methods. The hydrologic principles of the method are not new, but they are put to new uses. Because most SCS work is with ungaged watersheds (not gaged for runoff) the method was made to be usable with rainfall and watershed data that are ordinarily available or easily obtainable for such watersheds.

The principal application of the method is in estimating quantities of runoff in flood hydrographs or in relation to flood peak rates. These quantities consist of one or more types of runoff. An understanding of the types is necessary to apply the method properly in different climatic regions. The classification of types used in this handbook is based on the time from the beginning of a storm to the time of the appearance of a type in the hydrograph. Four types are distinguished:

> Channel runoff occurs when rain falls on a flowing stream or on the impervious surfaces of a streamflow-measuring installation. It appears in the hydrograph at the start of the storm and continues throughout it, varying with the rainfall intensity. It is generally a negligible quantity in flood hydrographs, and no attention is given to it except in special studies.

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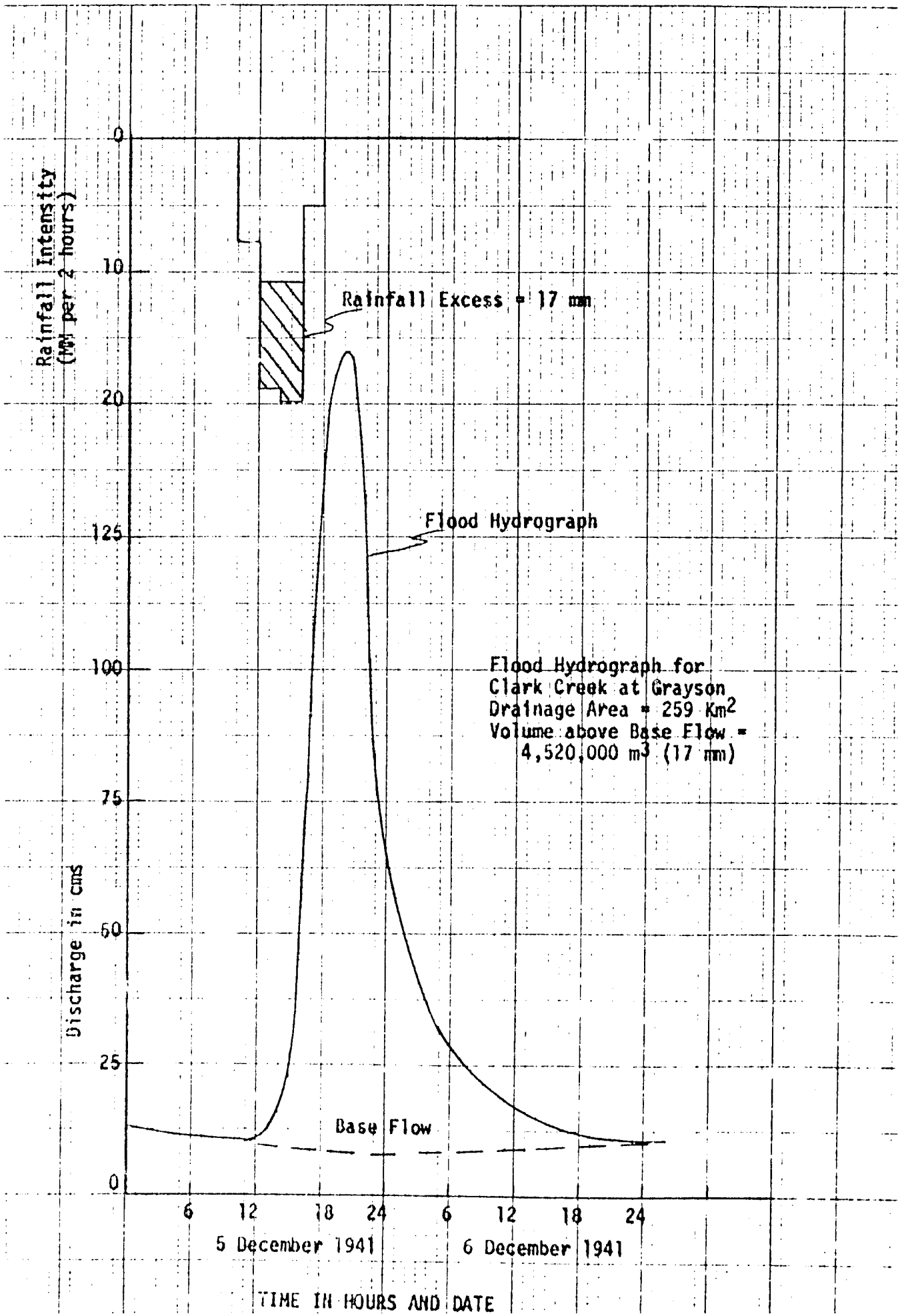
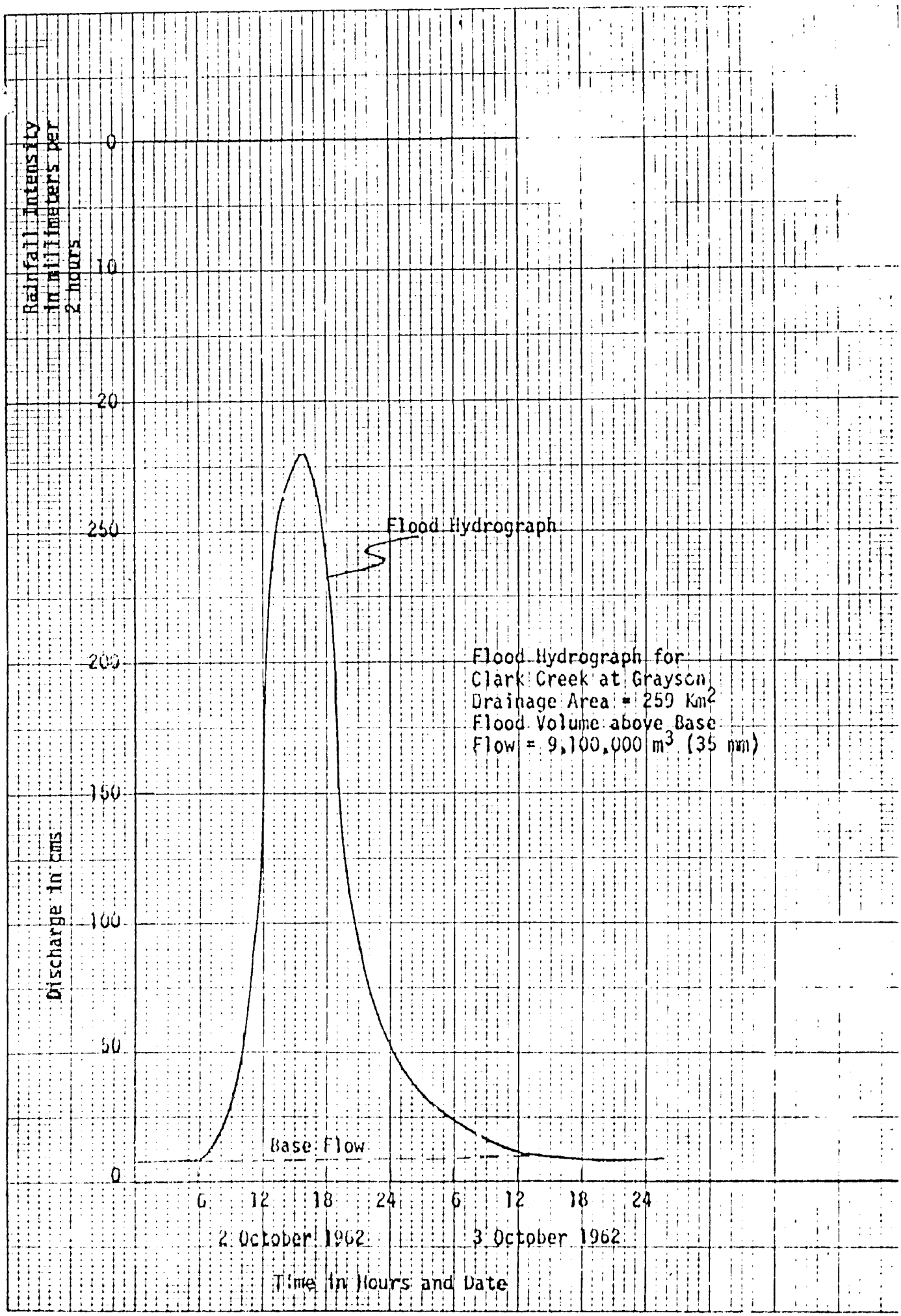
* Soil Conservation Service. 1972. SCS National Engineering Handbook. Section 4. Hydrology. U.S. Department of Agriculture, Washington, D.C.


Rainfall Intensity
(MM per 2 hours)
0
10
20
Rainfall Excess = 17 mm
Flood Hydrograph
125
100
Flood Hydrograph for
Clark Creek at Grayson
Drainage Area = 259 Km2
Volume above Base Flow =
4,520,000 m3 (17 mm)
75
Discharge in cms
50
25
Base Flow
0
6
12
18
24
6
12
18
24
5 December 1941
6 December 1941
TIME IN HOURS AND DATE


Rainfall Intensity
in millimeters per
2 hours
0
10
20
250
200
150
100
50
0
Discharge in cms
Flood Hydrograph
Flood Hydrograph for
Clark Creek at Grayson
Drainage Area = 259 Km2
Flood Volume above Base
Flow = 9,100,000 m3 (35 mm)
Base Flow
6
12
18
24
6
12
18
24
2 October 1962
3 October 1962
Time in Hours and Date

C. Unit Hydrograph Derivation and Hydrograph Reconstruction

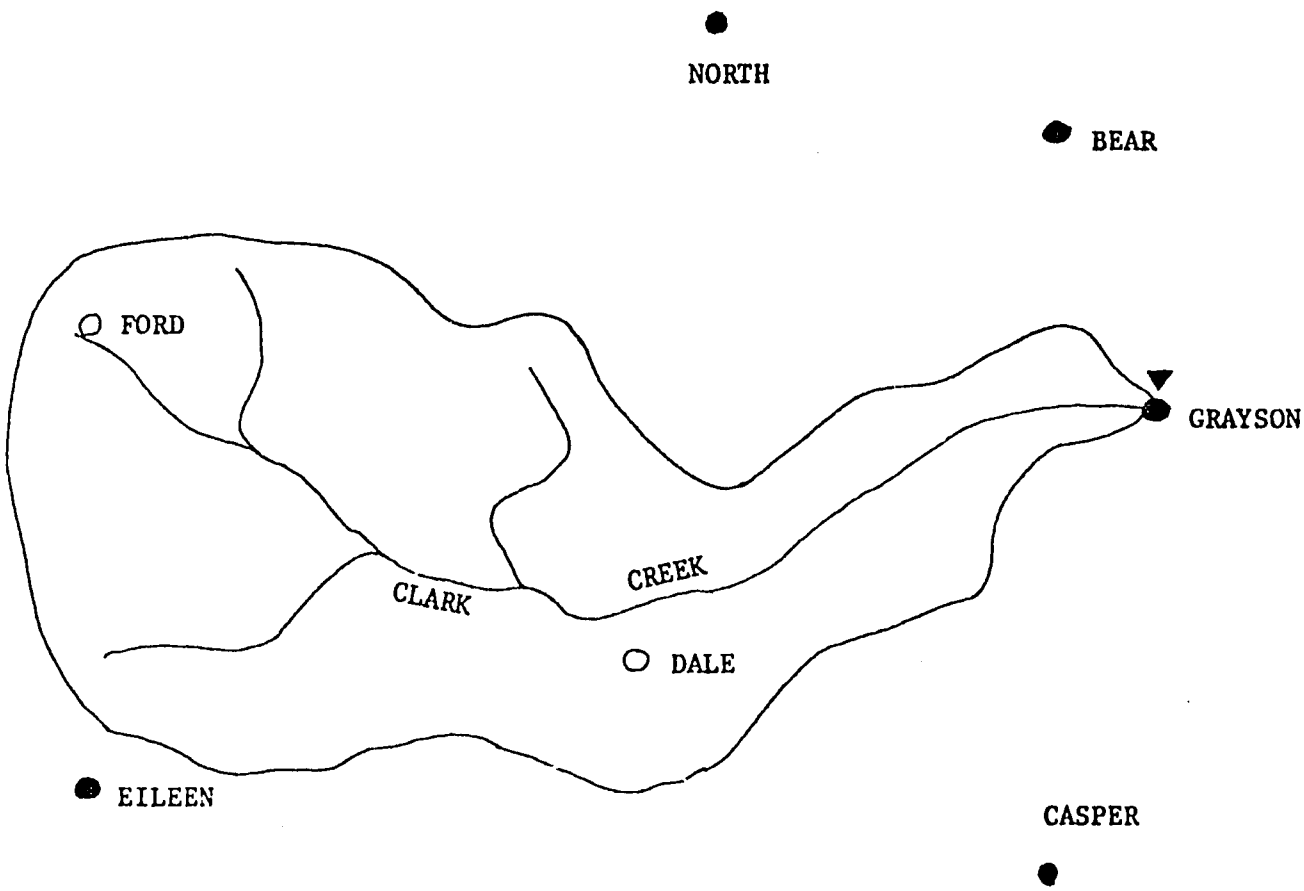
Problem Description

Recorded flood hydrographs and basin average hyetographs for the October 1962 and December 1941 floods on Clark Creek at Grayson are given on the following pages.

Problem

1. Using the rainfall excesses and flood hydrograph for the October 1962 flood, derive a unit graph representing 1 mm of direct runoff. Assume that the rainfall excess is uniform over its duration. What is the duration of your unit hydrograph?

2. Given a rainfall excess of 17 mm for the December 1941 flood, how would you apply your unit hydrograph in order to reconstitute the December 1941 flood? Outline in detail the steps you would take, however, do not go through the calculations to actually reconstitute this flood.



CLARK CREEK BASIN

Scale 1:20,000

LEGEND

▼ Stream Gage

● Daily Rainfall Gage

○ Hourly Recording Rainfall Gage

Clark Creek Basin

Rainfall Data
2 October 1962

	Bihourly Rainfall Chart Readings - Millimeters											
Time (hour)	2	4	6	8	10	12	14	16	18	20	22	24
Station												
Dale	0	0	6	22	43	64	67	67	67	67	67	67
Ford	0	0	11	35	68	99	115	115	115	115	115	115

Daily Rainfall in Millimeters

Date: October	1	2
Station		
Grayson	0	36
Bear	0	63
Casper	0	48
Eileen	0	84
North	0	102

B. Loss Rates

Problem Description

The bihourly precipitation distribution for Dale Station is considered representative of the distribution for the Clark Creek Basin for the October 2, 1962, storm. The mean depth of precipitation over the basin for this storm was 82 millimeters.

Problem

1. Using the Dale Station as a guide, determine the bihourly basin rainfall.

2. Determine a uniform loss rate for the storm so that the rainfall excess is 35 millimeters.

Table 3. Soil groups for estimating runoff curve numbers.

Soil Group:	Description of soil characteristics
A	Soils having very low runoff potential. For example, deep sands with very little silt or clay.
B	Light soils and/or well structured soils having above-average infiltration when thoroughly wetted. For example, light sandy loams, silty loams.
C	Medium soils and shallow soils having below-average infiltration when thoroughly wetted. For example, clay loams.
D	Soils having high runoff potential. For example, heavy soils, particularly clays of high swelling capacity, and very shallow soils underlain by dense clay horizons.

Table 4. Characterization of hydrologic condition for various land uses.

Land Use	Hydrologic condition
Native pastures	Pasture in poor condition is sparse, heavily grazed pasture with less than half the total watershed area under plant cover. Pasture in fair condition is moderately grazed and with between half and three-quarters of the catchment under plant cover. Pasture in good condition is lightly grazed and with more than three-quarters of the catchment area under plant cover.
Timbered areas	Poor areas are sparsely timbered and heavily grazed with no undergrowth. Fair areas are moderately grazed, with some undergrowth. Good areas are densely timbered and ungrazed, with considerable undergrowth.
Improved permanent pastures	Densely sown permanent legume pastures subject to careful grazing management are considered to be in good hydrologic condition.
Rotation pastures	Dense, moderately grazed pastures used as part of a well planned, crop-pasture-fallow rotation are considered to be in good hydrologic condition. Sparse, overgrazed or "opportunity" pastures are considered to be in poor condition.
Crops	Good hydrologic condition refers to crops which form part of a well-planned and managed crop-pasture-fallow rotation. Poor hydrologic condition refers to crops managed according to a simple crop fallow rotation.

Table 5. Curve numbers (CN) for wet (AMC III) and dry (AMC I) antecedent moisture conditions corresponding to average antecedent moisture condition (AMC II).

CN for AMC II	Corresponding CN's	
	AMC I	AMC III
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
20	9	37
15	6	30
10	4	22
5	2	13

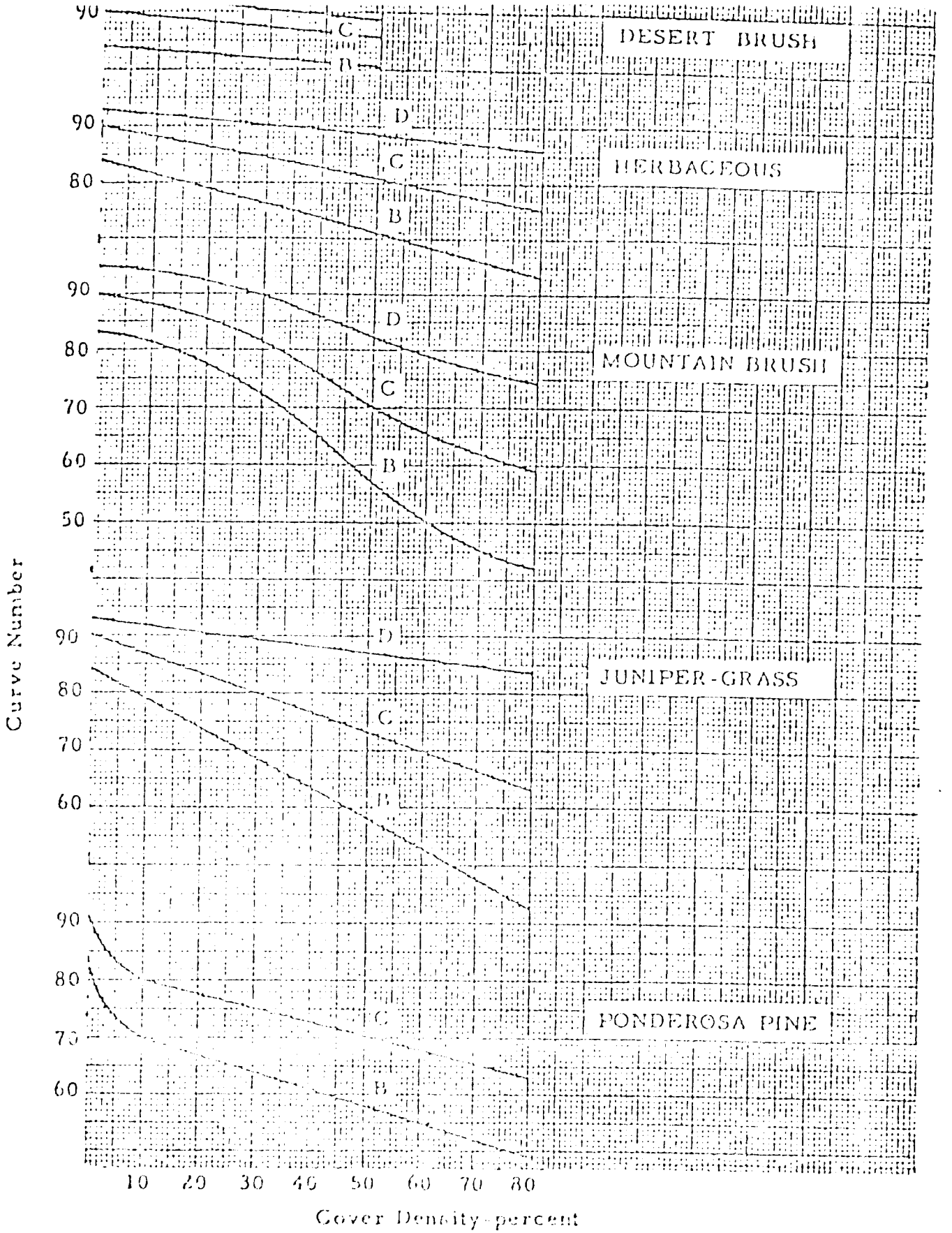


Fig. 1

HYDROLOGIC SOIL - COVER COMPLEXES
AND ASSOCIATED CURVE NUMBERS

Synthetic Hydrograph

After the runoff volume has been estimated, the next step is to distribute this quantity in time. This is done by developing a synthetic hydrograph. Using the schematic representation of the hydrograph shown in Figure 3, various relationships between the hydrograph components can be determined as follows:

$$Q = \frac{q_p T_p}{2} + \frac{q_p T_r}{2} \tag{11}$$

from which

$$q_p = \frac{2Q}{T_p + T_r} \tag{12}$$

Letting $T_r = HT_p$, where H is a constant to be determined for a particular watershed

$$q_p = \frac{2Q}{(1+H)\ T_p} \tag{13}$$

Converting from inches per hour to cubic feet per second by introducing the drainage area A in square miles (1 inch per hour = 645.3 cfs per square mile) gives the peak rate as

$$q_p = \frac{1290.6\ A\ Q}{(1+H)\ T_p} \tag{14}$$

Analyses by the SCS have resulted in their adoption of H = 1.67 as a general average value for ungaged watersheds. Thus, the equation for the peat rate is expressed as

$$q_p = \frac{484\ A\ Q}{T_p} \tag{15}$$

From Figure 3, it can be seen that

$$T_p = 0.5\ D + T_L \tag{16}$$

where

D = duration of excess rainfall often taken to be, as a first approximation, storm duration

T_L = time lag constant generally taken to be the time from the center of mass excess rainfall to the peak flow

Recession time (tr) can be taken as

$$T_r = 5/3\ T_p$$

A study of many hydrographs has led to the empirical relationship for time lag, namely

$$T_L = 0.6\ T_c \tag{17}$$

where T_c is the time of concentration for a watershed, or the time it takes runoff to travel from the hydraulically most distant part of the storm area to the watershed outlet. In hydrograph analysis, T_c is the time from the end of excessive rainfall to the point on the falling link of the hydrograph (inflection point) where the recession curve begins. T_c may be estimated by the following formula:

$$T_c = \frac{L^{1.15}}{7{,}700\ H^{0.38}} \tag{18}$$

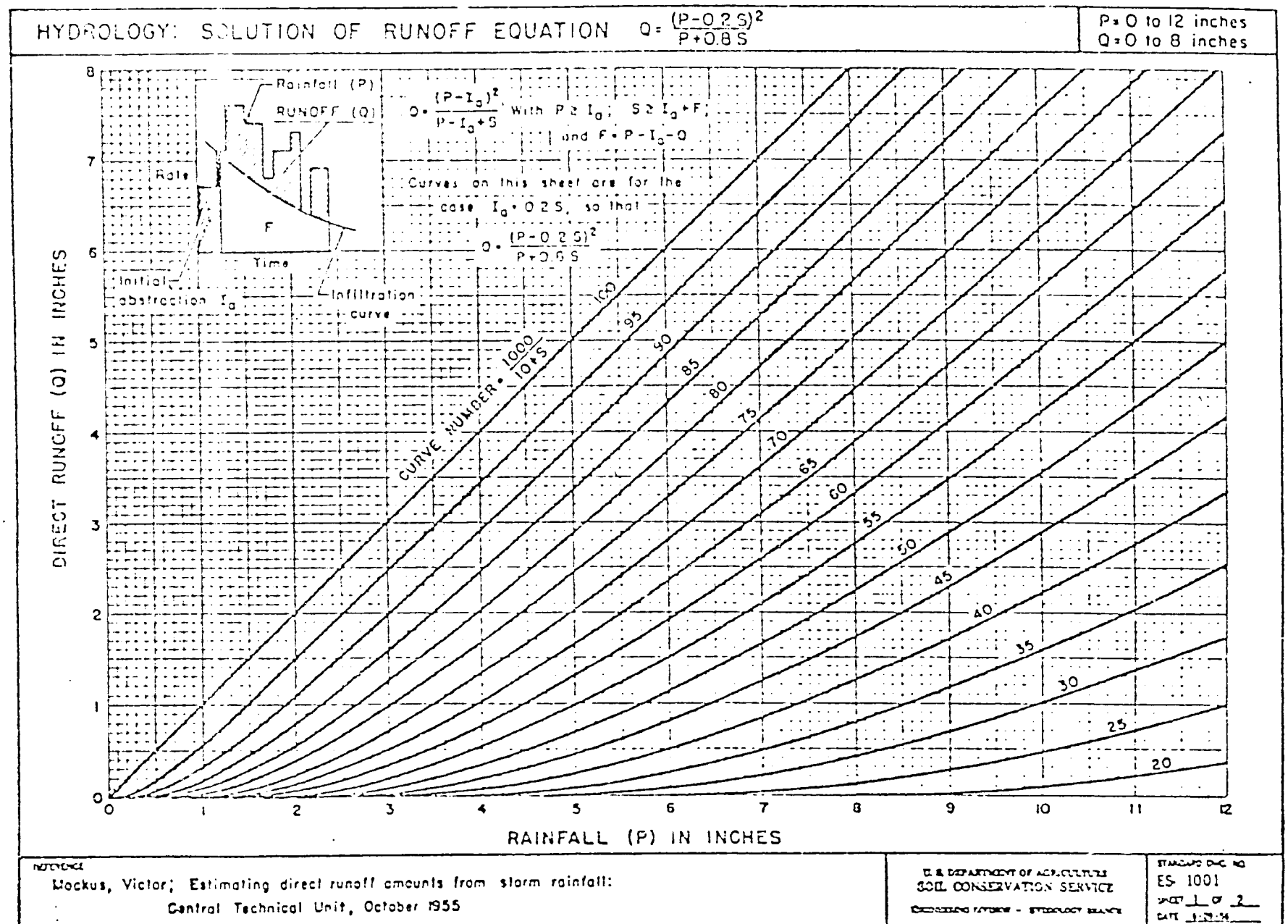
where

T_c = time of concentration in hours

L = length of the watershed along the mainstream from the outlet to the most distant ridge in feet

H = the difference in elevation between the watershed outlet and the most distant ridge in feet

A solution to equation 18 is presented in the nomograph shown in Figure 4.



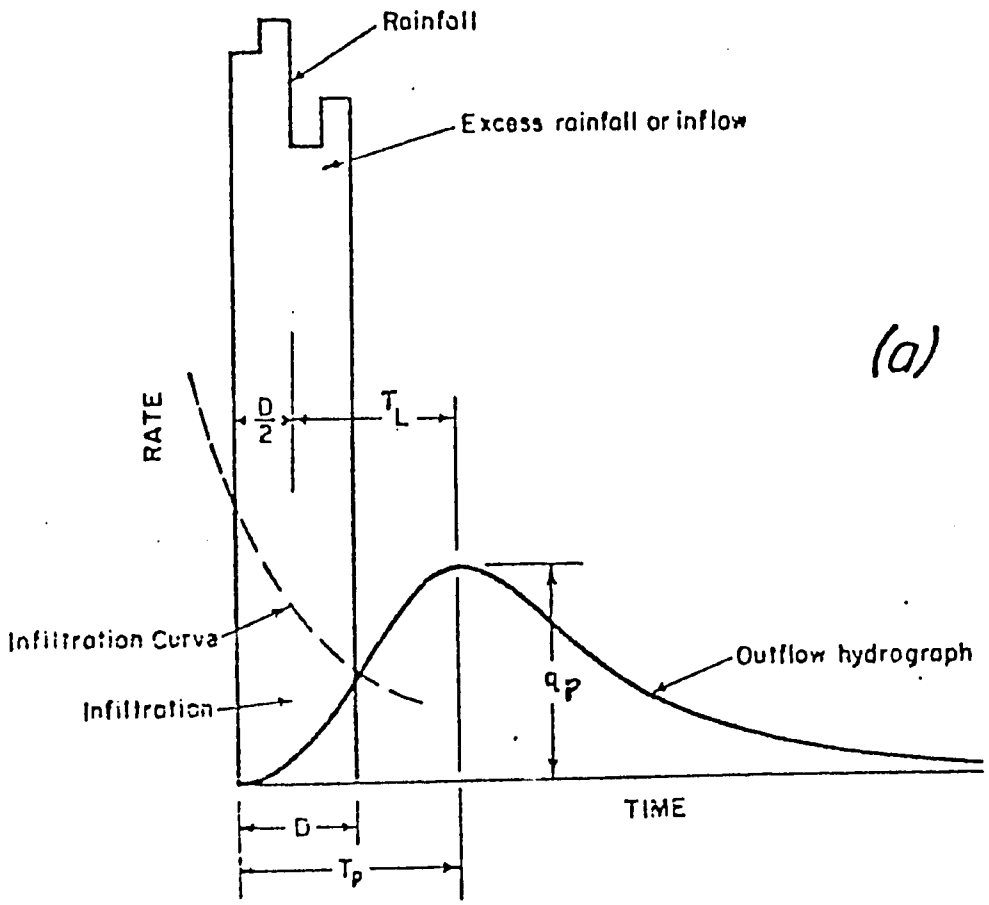
REFERENCE

Mockus, Victor; Estimating direct runoff amounts from storm rainfall: Central Technical Unit, October 1955

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING NOTEBOOK - HYDROLOGY BRANCH

STANDARD DWG. NO. ES 1001
SHEET 1 OF 2
DATE 1-27-54

Fig. 2 SOLUTION OF RUNOFF EQUATION



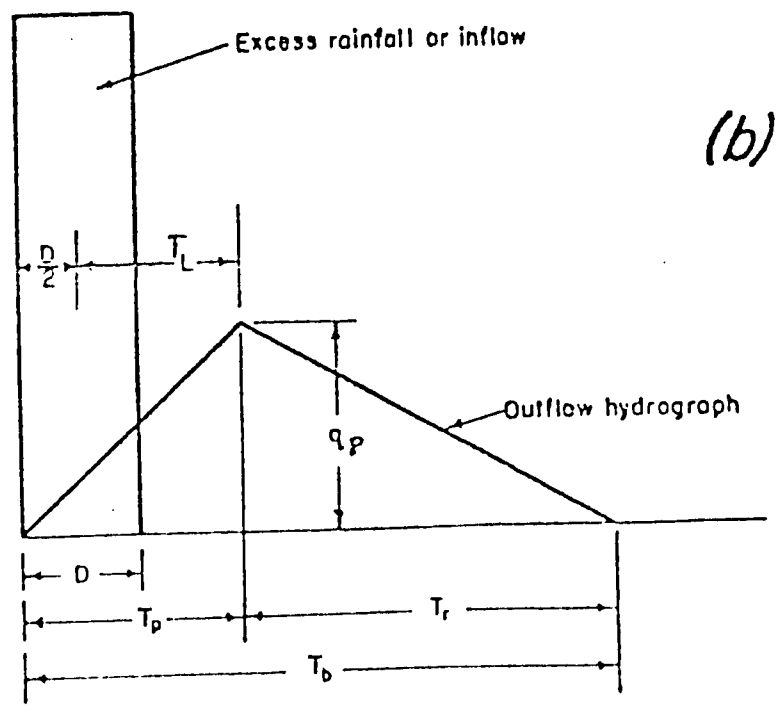
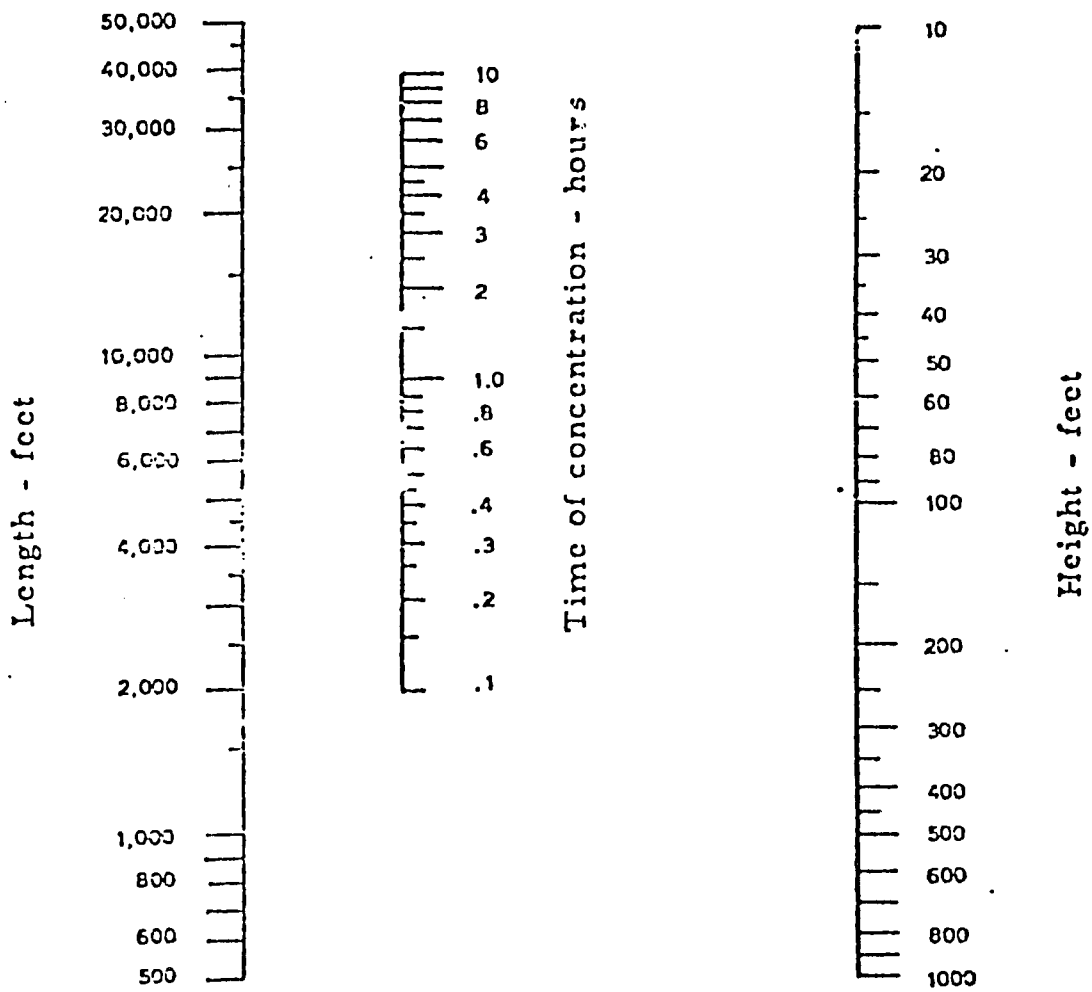


Fig. 3 DEVELOPMENT OF TRIANGULAR HYDROGRAPH



$$T_c = \frac{L^{1.15}}{7700\ H^{.38}}$$

T_c = time of concentration - hours
L = length of drainage area - feet
H = elevation - feet

Fig. 4. Nomograph for solving time of concentration equation.

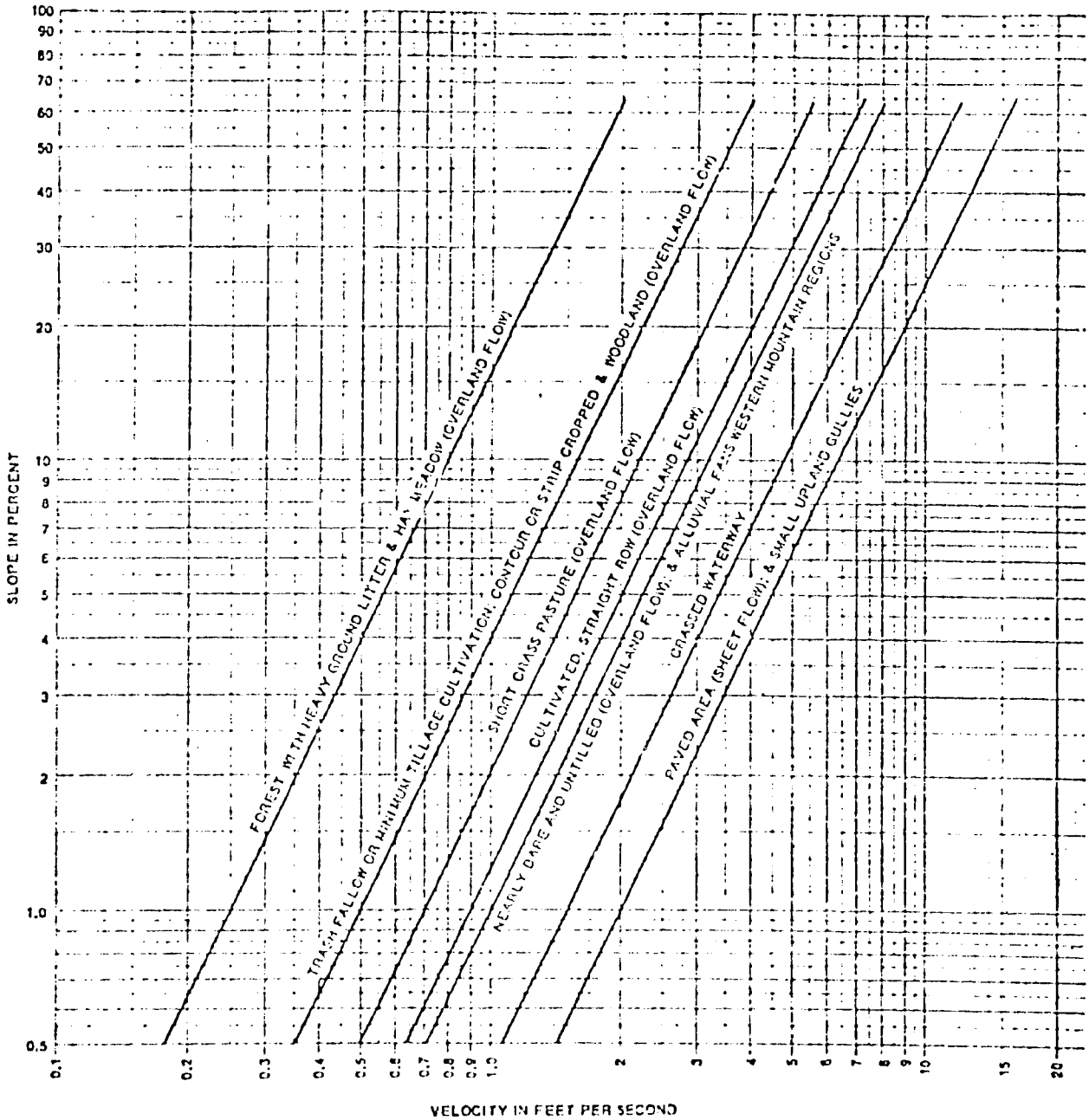


Figure 5. -Velocities for upland method of estimating T_c

Substituting equations 16 and 17 into equation 15 results in the following expression for estimating peak runoff rates for ungaged watersheds:

$$q_p = \frac{484\ AQ}{0.5\ D + 0.6\ T_c}$$

where

q_p = peak runoff rate in cfs

A = drainage area in square miles

Q = runoff volume in inches

D = duration of excess rainfall in hours

T_c = time of concentration in hours

Thus, with estimates of T_p and q_p and with the assumption that T_r = 2.67 T_p, a synthetic hydrograph can be obtained for a specific storm falling on a specific watershed.

E. Estimating Runoff Using the Soil Conservation Service Method

You are asked to determine the peak flow and total discharge from a watershed that would be produced by a 2 hour storm which has a 20-year recurrence interval. The information is needed to construct a diversion dam to alleviate flooding of a small community downstream.

You calculate that the two hour, 20-year storm is 4 inches. In order to be safe you decide to assume that this storm would have been preceeded within the past few days by a storm of two inches.

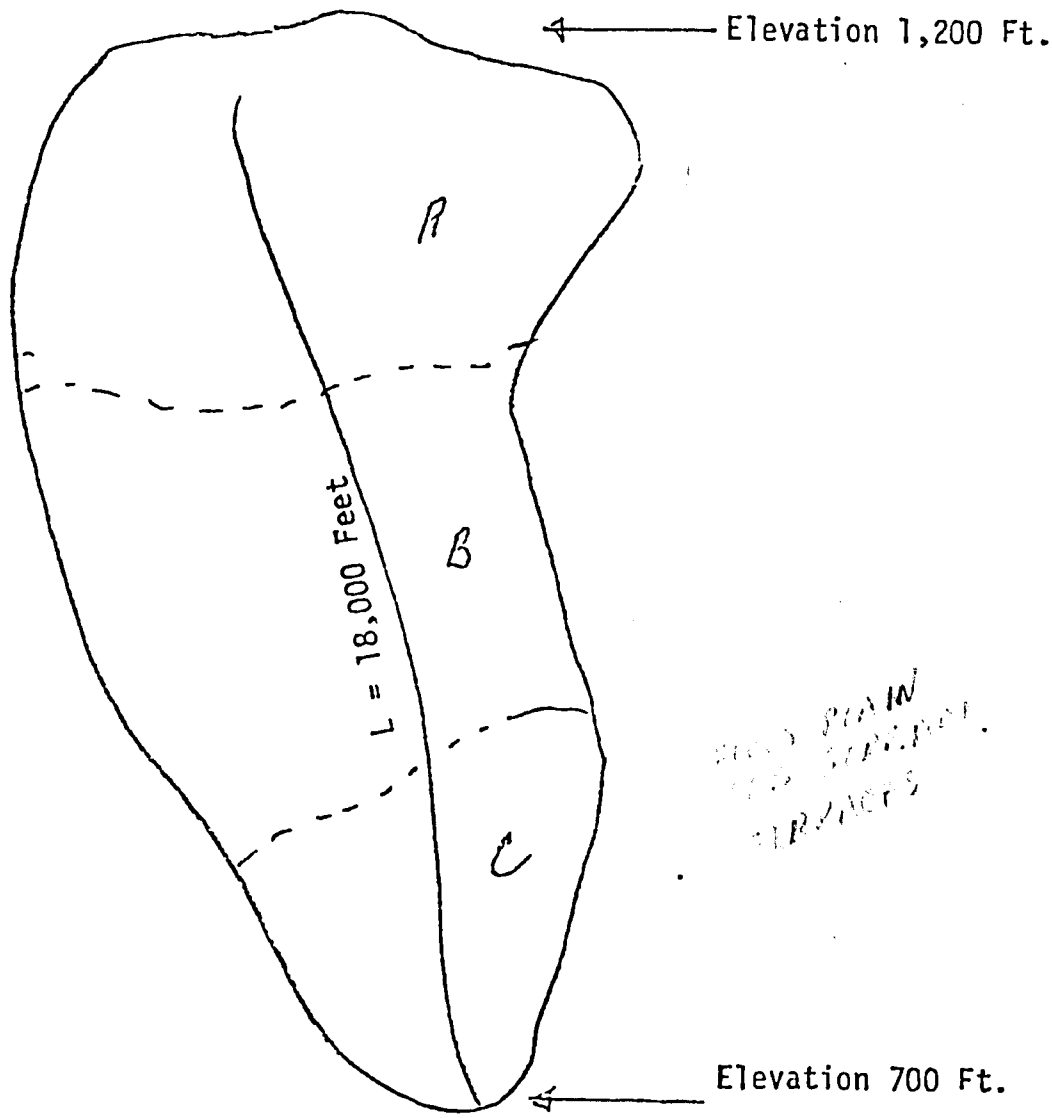
Field Data

A field survey (see Figure 1) revealed that:

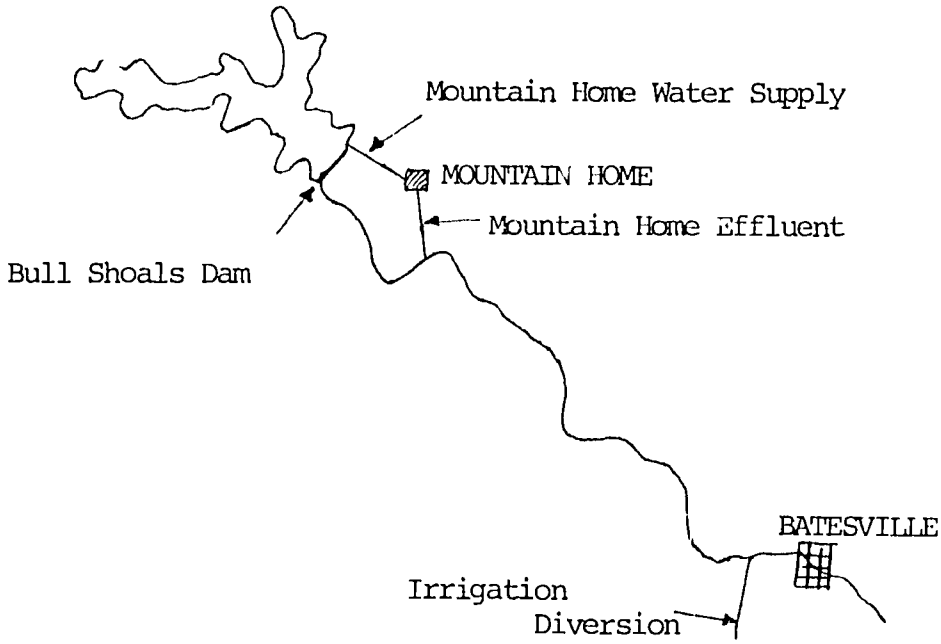
1. The upper portion of the watershed was sparsely forested. There were a few shifting cultivators in the area who also had considerable livestock. The soils were well structured silt loams.

2. The central portion of the watershed was in heavily overgrazed native pastures. The soils were shallow clay loams.

3. The lower portion of the watershed was an area of flood plain and old stream terraces. The soils, though moderately deep, were heavy clay loams. Nearly all of the area was devoted to straight row grain crops.



Area		Ha
A	=	520
B	=	346
C	=	213
Total		1079



BULL SHOALS PROJECT

A. Problem

1. Using the routing form provided, perform monthly sequential routing to supply the demands through November 1953. For each month through this period, indicate what factor controlled the operation of the reservoir.

2. Upon completing part 1, obtain a completed routing form from one of the instructors and answer the following questions:

   a. During the critical low flow period between March 1953 and February 1955, what was the lowest pool elevation computed? Did a shortage occur at this time?

   b. If hydropower was included as a project purpose, it would become necessary to prohibit drawdown below elevation 192 meters. What modifications to the operation plan would you make to satisfy this constraint without changing the top of conservation storage or decreasing required demands?

However, preliminary studies reveal that in extremely dry years it will not be possible to provide all of the desired flow; therefore, a study of minimum required flows below Batesville has been made and the following schedule adopted:

Mininum
Required Flow at Batesville
in $m^3 \times 10^6$

Jan	37.24	Jul	223.41
Feb	37.24	Aug	223.41
Mar	74.47	Sep	156.39
Apr	74.47	Oct	74.47
May	148.94	Nov	37.24
Jun	223.41	Dec	37.24

A plan of operation is proposed whereby a buffer zone in Bull Shoals Reservoir will be employed to assure required releases for water supply, irrigation, fish and wildlife, navigation, and minimum required flow at Batesville. All purposes, including desired flow at Batesville, will be supplied when the reservoir level is above the top of the buffer zone. However, when the reservoir level falls below the top of buffer zone, releases will not be made for desired flow at Batesville, but releases will be made for required flow at Batesville and for all other project purposes. Preliminary estimates indicate that the top of buffer zone should be at elevation 194.10 and that the critical low flow period is between March 1953 and February 1955. Evaporation is important and is to be considered in the routing. Nondamaging channel capacities are 380 cumecs below Bull Shoals damsite and 765 cumecs at Batesville.

The Batesville Irrigation District has been formed for the purpose of obtaining water for irrigation. The diversion would be located about 1.5 kilometers upstream from Batesville and the expected demand is as follows (assume no return flow):

Irrigation Demands in $m^3 \times 10^6$

Jan	0	Jul	48.41
Feb	0	Aug	74.47
Mar	0	Sep	55.85
Apr	0	Oct	18.62
May	0	Nov	0
Jun	22.34	Dec	0

Commercial and sport fisheries in the 190 kilometers reach between Bull Shoals damsite and Batesville are an important factor in the regional economy. Fish and wildlife experts have stated that a minimum flow of 14 cumecs will be required throughout the entire reach at all times in order to maintain fish life.

The master plan for development of the White River indicates that it would be desirable to maintain the following schedule of flow at Batesville in order to facilitate small craft navigation below Batesville.

Navigation
Desired Flow at Batesville
in $m^3 \times 10^6$

Jan	223.41	Jul	372.35
Feb	223.41	Aug	372.35
Mar	297.88	Sep	335.12
Apr	297.88	Oct	223.41
May	335.12	Nov	223.41
Jun	335.12	Dec	223.41

PROBLEM V

RESERVOIR MANAGEMENT

A. Single Reservoir Multi-Purpose Simulation Example[1]

Problem Description

A multi-purpose project is to be designed for the Bull Shoals damsite on the main stem of the White River. Flood-control studies have indicated that the bottom of the flood control pool will be at Elevation 198.00m (Figure 1). Initial conservation studies for the Bull Shoals project will consider only water supply, irrigation, fish and wildlife, and navigation.

The city of Mountain Home has requested that storage for municipal and industrial water supply be provided. The following water supply demand data are supplied:

Water Supply
Average Monthly Demand in $m^3 \times 10^6$

Jan	.2	Jul	.8
Feb	.3	Aug	.7
Mar	.4	Sep	.3
Apr	.5	Oct	.2
May	.8	Nov	.2
Jun	.9	Dec	.2

The city plans to take the water directly from the reservoir via a pipeline at the dam. It is estimated that 75 percent of the water withdrawn will be returned to the White River about 16 kilometers below the proposed damsite.

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1 Taken from the Hydrologic Engineering Center Training Course "Second International Workshop in Hydrologic Engineering" 5-30 August 1974, Davis, California.

Table 15-3 Cropping-management factors, *C* in the Universal Soil-Loss Equation, for woodland. (From U.S. Soil Conservation Service 1975b.)

TREE CANOPY (% OF AREA)	% OF AREA COVERED BY > 2 INCHES OF FOREST LITTER	UNDERGROWTH	*C*
100-75	100-90	Grazing and burning controlled	0.001
		Heavily grazed and burned	0.003-0.011
70-40	80-75	Grazing and burning controlled	0.002-0.004
		Heavily grazed and burned	0.01-0.04
35-20	70-40	Grazing and burning controlled	0.003-0.009
		Heavily grazed and burned	0.02-0.09
<20	Treated as grassland or cropland		

Table 15-4 Cropping-management factors, *C* in the Universal Soil-Loss Equation, for pasture, rangeland, and idle land. (From U.S. Soil Conservation Service 1975b.)

TYPE OF CANOPY AND AVERAGE FALL HEIGHT OF WATER DROPS	CANOPY COVER (%)	GROUND COVER*	PERCENT GROUND COVER 0	20	40	60	80	95-100
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.090	.043	.011
Canopy of tall weeds or short brush (0.5 m fall ht)	25	G	.36	.17	.09	.038	.012	.003
		W	.36	.20	.13	.082	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.075	.039	.011
	75	G	.17	.10	.06	.031	.011	.003
Appreciable brush or brushes (2 m fall ht)	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.085	.042	.011
	50	G	.34	.16	.085	.038	.012	.003
		W	.34	.19	.13	.081	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.077	.040	.011
Trees but no appreciable low brush (4 m fall ht)	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.087	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.085	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.083	.041	.011

*G = Cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 inches deep.
W = Cover at surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface, undecayed residue, or both).

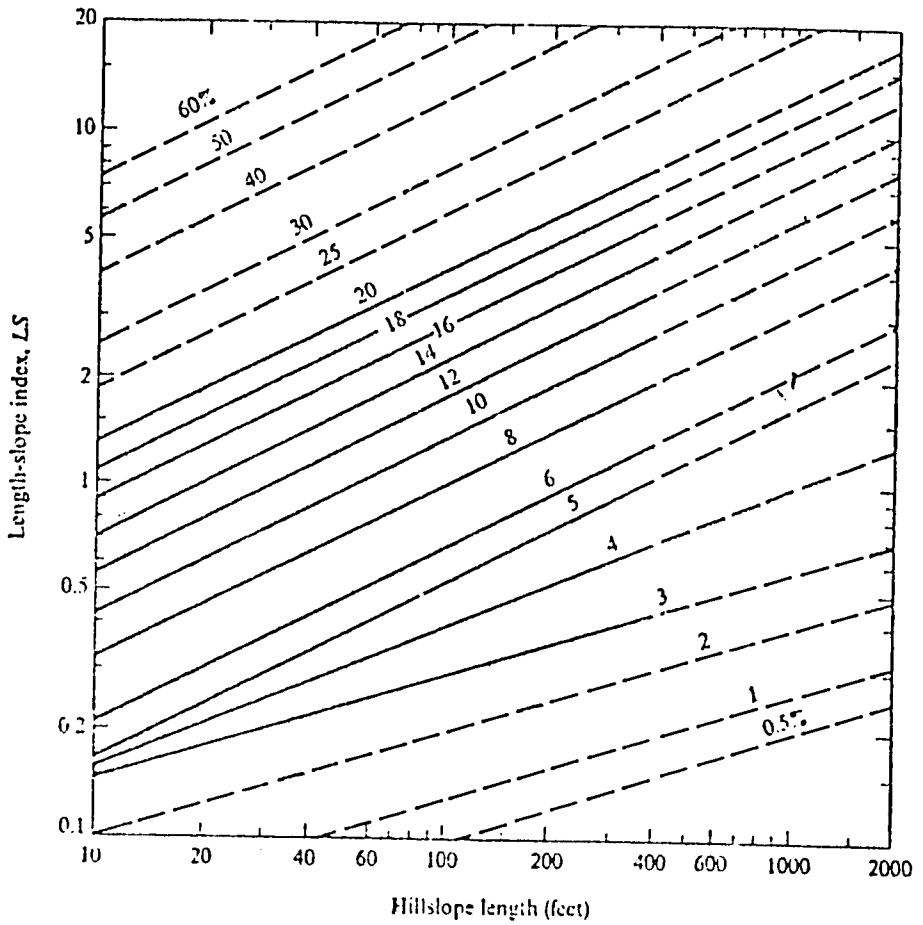


Figure 15-19 Chart for evaluating the length-slope factor, *LS*, in the Universal Soil-Loss Equation. The solid lines represent conditions within the range of data from which the curves were derived. The dashed lines are based on extrapolations and should be used with care. (From U.S. Soil Conservation Service 1975b.)

Table 15-2 Examples of cropping-management factors, *C* in the Universal Soil-Loss Equation, for agricultural land. Many other crop combinations are listed in the original reference as well as in technical releases for particular regions. The crop stages are also defined in the original source. (From Wischmeier and Smith 1965.)

	CROP STAGE				
CROP	FALLOW	SEEDLING	ESTABLISHMENT	MAJOR GROWTH	RESIDUE
Corn with conventional tillage in rotation with small grains and grass	0.36	0.63	0.50	0.26	0.30
Alfalfa					0.02

PROBLEM IV

SOIL EROSION

Problem

Calculate the potential annual soil loss for the watershed given in the SCS runoff problem.

Assume: R = 350

Average slope length and gradient

Area A = 80 ft, 40%
Area B = 100 ft, 20%
Area C = 150 ft, 8%

Soils

Area A: 60 percent silt and fine sand with fine structure and moderate to rapid permeability with 3 percent organic matter (silt loams).

Area B: 40 percent silt and fine sand with very fine granular structure, moderate to rapid permeability and 2 percent organic matter (clay loams).

Area C: 30 percent silt, clay and fine sand with massive structure and 4 percent organic matter (heavy clay loams), slow permeability.

Cover

Area A: Heavily grazed and burned, tree canopy approximately 20%. Litter cover 45% of area.

Area B: Grass ground cover, 60% density.

Area C: Small grains, row cropped, 80% cover.

1. The completed routing form is attached. Column 17 indicates the factor which controlled the reservoir operation for each period. The controlling factors correspond to the following:

Controlling Factor	
1	Release restricted by reservoir outlet capacity
2	Release restricted by damsite channel capacity
3	Release restricted by downstream channel capacity
4	Release to satisfy downstream water requirement
5	Release to satisfy water requirements at dam
6	Release to maintain fish life in downstream reach
7	Release to empty flood control pool
8	Release restricted by buffer pool

2a. The lowest pool elevation (191.50 meters) was in September 1954. A shortage did not occur in terms of downstream requirements since the diversion and minimum flow requirements at Batesville were met. However, the desired flow at Batesville was not met since the reservoir was being operated in the buffer zone.

2b. The elevation of the top of the buffer zone could be raised to prohibit drawdown below elevation 192.00 meters. An estimate of the required raise can be made by first computing the increased storage required to raise the pool from 191.50 meters (lowest pool elevation in critical drought period) to 192.00 meters in September 1954.

$$\text{Increased storage} = 2585.0 - 2515.0$$
$$= 70.0 \text{ m}^3 \times 10^6$$

By adding this storage to that corresponding to the top of the buffer zone, we get:

$$2890.0 + 70.0 = 2960.0 \text{ m}^3 \times 10^6$$

which corresponds to the elevation of 194.55 meters.

Max Release = 380 cumecs
= 1000.6 $m^3 x 10^6$

Max D.S. Flow = 765 cumecs
= 2014.3 $m^3 x 10^6$

1. SINGLE RESERVOIR MULTIPURPOSE SIMULATION

Month and Year	Inflow	Pipe-line	Net Evaporation		Release	Change in Storage	End of Period			Required Flow	Return Flow	Local Inflow	Diversion	Desired Flow at Bates-ville	Minimum Required Flow at Bates-ville	Actual Flow at Batesville	Control Factor
							Storage	Area	Pool Elevation								
	$M^3 X 10^6$	$M^3 x 10^6$	MM	$M^3 X 10^6$	$M^3 X 10^6$	$M^3 X 10^6$	$M^3 X 10^6$	Hectares	M	$M^3 X 10^6$	$M^3 X 10^6$	$M^3 X 10^6$	$M^3 X 10^6$	$M^3 X 10^6$	$M^3 X 10^6$	$M^3 X 10^6$	M X
(1)	(2)	(3)	(4a)	(4b)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
INITIAL							3525.0	17,500	198.00								
1953 MAR	1[illegible]8.2	.4	- 19	- 3.3	1[illegible]	[illegible]	3[illegible]	17,700	198.72	37.0	.3	687.1	0	297.9	74.5	1638.0	2
APR	5[illegible]7.7	.5	42	7.4	833.3	[illegible]	3[illegible]	17,925	198.70	37.0	.4	1180.6	0	297.9	74.5	2014.3	3
MAY	534.2	.6	24	4.3	9[illegible].1	-127.[illegible]	3525.0	17,500	198.00	37.0	.6	820.5	0	335.1	148.9	1777.5	7
JUN	8[illegible].1	.9	149	26.4	174.6	-12[illegible]	3403.2	17,040	197.30	37.0	.7	182.1	22.3	335.1	223.4	335.1	4
JUL	77.3	.8	100	18.3	318.6	-287.4	3145.8	16,050	195.70	37.0	.6	104.6	48.4	372.4	223.4	372.4	4
AUG	26.2	.7	124	20.5	259.8	-2[illegible]	2890.0	15,060	194.10	37.0	.5	81.2	74.5	372.4	223.4	281.7	8
SEP	14.2	.3	138	21.5	158.9	-1[illegible]	2723.5	14,400	193.00	37.0	.2	53.2	55.9	335.1	156.4	156.4	8
OCT	15.9	.2	54	7.9	37.0	- 29.2	2694.3	14,300	192.80	37.0	.2	69.3	18.6	223.4	74.5	87.9	6
NOV	16.0	.2	13	1.6	37.0	- 22.3	2672.0	14,200	192.62	37.0	.2	87.4	0	223.4	37.2	124.6	6
DEC	24.9	.2	- 6	-.9	37.0	- 11.4	2660.6	14,180	192.58	37.0	.2	90.2	0	223.4	37.2	127.4	6
1954 JAN	62.2	.2	- 86	-12.2	37.0	37.2	2697.8	14,330	192.82	37.0	.2	205.8	0	223.4	37.2	243.0	6
FEB	66.7	.3	- 31	- 4.4	37.0	25.8	2723.6	14,420	193.00	37.0	.2	235.7	0	223.4	37.2	272.9	6
MAR	167.0	.4	- 19	- 2.7	37.0	162.3	2885.9	15,000	194.00	37.0	.3	242.9	0	297.9	74.5	280.2	6
APR	23[illegible].0	.5	42	6.2	37.0	188.9	3064.8	15,740	195.23	37.0	.4	267.0	0	297.9	74.5	304.4	6
MAY	316.4	.8	24	3.7	37.0	273.9	3338.7	16,760	196.90	37.0	.6	498.5	0	335.1	148.9	535.1	6
JUN	6[illegible].2	.9	149	24.2	246.7	-269.6	3129.1	16,000	195.65	37.0	.7	110.0	22.3	335.1	223.4	335.1	4
JUL	26.2	.8	100	17.4	247.1	-239.1	2890.0	15,050	194.10	37.0	.6	75.2	48.4	372.4	223.4	274.5	8
AUG	20.7	.7	124	19.2	229.0	-2[illegible]	2661.8	14,160	192.56	37.0	.5	68.4	74.5	372.4	223.4	223.4	8
SEP	19.0	.3	138	20.2	142.9	-144.4	2517.4	13,640	191.50	37.0	.2	69.2	55.9	335.1	156.4	156.4	8
OCT	220.7	.2	54	7.5	37.0	182.0	2699.4	14,340	192.83	37.0	.2	103.0	18.6	223.4	74.5	121.6	6
NOV	120.3	.2	13	1.8	37.0	89.3	2788.7	14,680	193.45	37.0	.2	70.8	0	223.4	37.2	108.0	6
DEC	524.6	.2	- 6	-.9	37.0	488.3	3277.0	16,530	196.52	37.0	.2	246.6	0	223.4	37.2	283.8	6
1955 JAN	400.3	.2	- 86	-13.4	170.6	248.0	3525.0	17,500	198.00	37.0	.2	205.1	0	223.4	37.2	375.8	7
FEB	842.7	.3	- 31	- 5.3	847.7	0	3525.0	17,500	198.00	37.0	.2	392.4	0	223.4	37.2	1240.3	7

Description of Routing Form for Single Reservoir
Multi-Purpose Simulation

Column

1 Year and month of the routing period.

2 Average inflow to Bull Shoals Reservoir in $m^3 \times 10^6$ per month.

3 Monthly water supply demand (pipeline) for Mountain Home in $m^3 \times 10^6$.

4 Net reservoir evaporation in mm and $m^3 \times 10^6$ per month. Conversion from mm to monthly $m^3 \times 10^6$ is accomplished by multiplying the average end of period area, column 8, by the number of mm of evaporation, column 4, and dividing by the necessary conversion constant:

$$\text{Net evaporation } (m^3 \times 10^6) = \frac{(\text{Col. 4a}) * (\text{Ave. Col. 8})}{0.1}$$

5 Reservoir release in monthly $m^3 \times 10^6$.

6 Change in monthly storage in $m^3 \times 10^6$, Col. 2 - (Col. 3 + 4b + Col. 5).

7 End of monthly reservoir storage in $m^3 \times 10^6$, which is the previous period storage (Col. 7) plus the present period change in storage (Col. 6).

8 End of period area in hectares taken from the area-capacity table.

9 End of period pool elevation in meters above mean sea level taken from the area-capacity table.

10 Minimum monthly flow in $m^3 \times 10^6$ required in the White River to maintain fish life.

11 Monthly flow returned to the White River from Mountain Home in $m^3 \times 10^6$, Col. 3 x .75.

12 Local monthly inflow to the White River between Bull Shoals Dam and Batesville in $m^3 \times 10^6$.

13 Monthly diversion demand to Batesville Irrigation District in $m^3 \times 10^6$.

14 Desired monthly flow at Batesville for navigation in $m^3 \times 10^6$.

15 Minimum monthly required flow at Batesville in $m^3 \times 10^6$.

16 Actual monthly flow at Batesville in $m^3 \times 10^6$ (Max. allowable = 2,014,300,000 m^3)
Col. 5 + Col. 11 + Col. 12 - Col. 13.

B. Reservoir Management Problem

1. Using the information below and the situation presented in the handout, determine a demand curve for the estimated storage requirements.

Volumes in $m^3 \times 10^6$

Demands	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
Water supply	0.2	0.3	0.4	0.5	0.8	0.9	0.8	0.7	0.3	0.2	0.2	0.2
Irrigation	0	0	0	0	0	22.3	48.4	74.5	55.9	18.6	0	0
Hydro-electric Power	290	290	290	300	300	300	300	300	300	290	290	290
Minimum flow downstream	37.2	37.2	74.5	74.5	148.9	223.4	223.4	223.4	156.5	74.5	37.2	37.2

2. Develop a matrix to compare the complementary or conflicting potentials for storage use. Include with the above information, a flood storage requirement of 450, 900, 900, and 450 $m^3 \times 10^6$ for the months FEB through MAY.

Purpose	Water Supply	Irrigation ...
Water Supply	-	CF ...
Irrigation	CF	-
.	.	
.	.	
.	.	

CF = conflicting
CO = complimentary

3. Given the following information, determine the volume of dead storage needed for the reservoir if it is to last for 40 years. Sediment delivery from the 200 ha watershed is presently 0.9 tons/ha/year. Assume the bulk density of sediments is 1.6 g cm^{-3}.

If poor cultivation and overgrazing on the watershed above the reservoir are allowed, and result in sediment deliveries of 35 tons ha^{-1} $year^{-1}$, how many years will the dead storage space (previously determined) last?

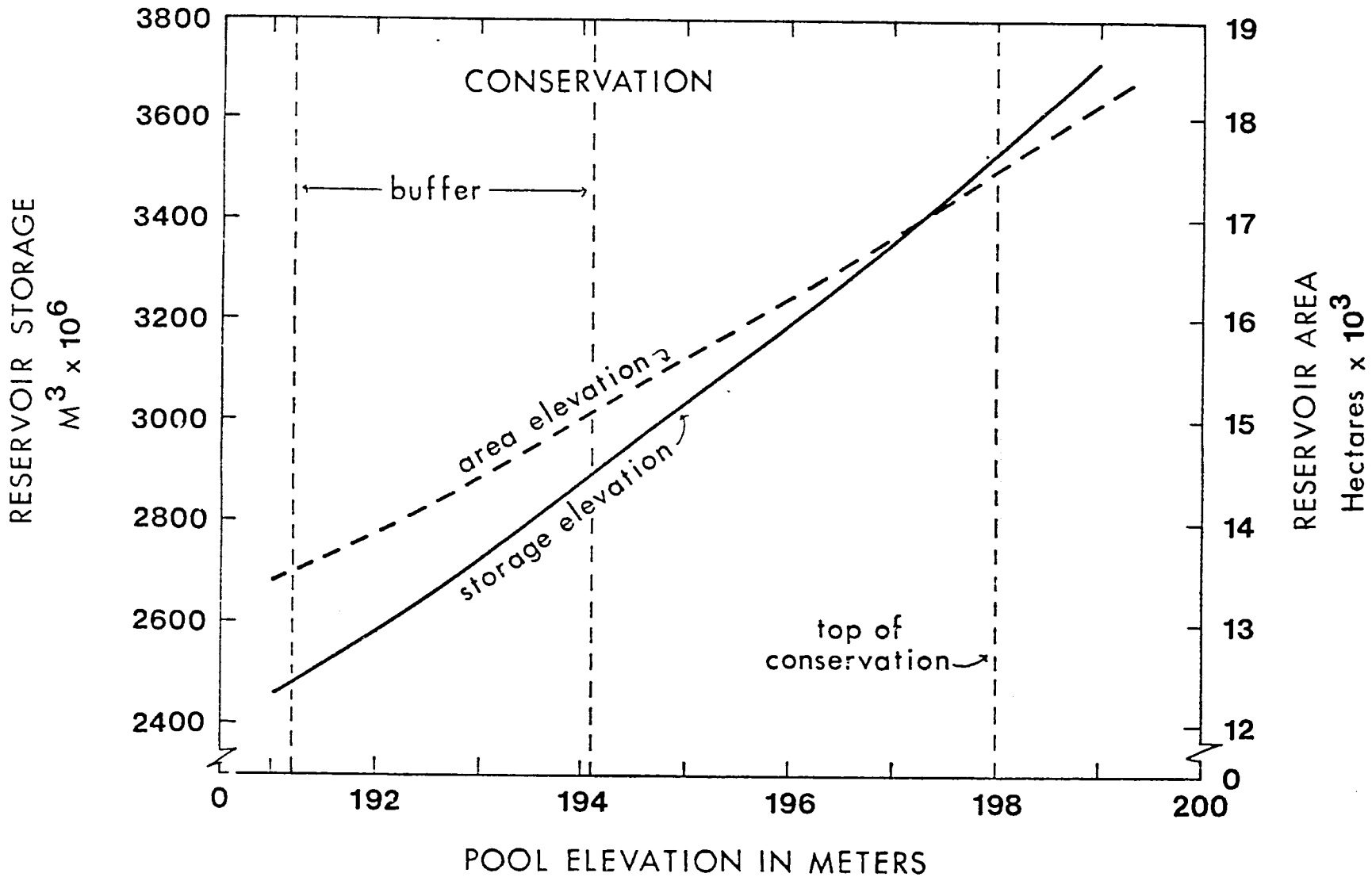


Figure 1. Bull Shoals Reservoir

PROBLEM VI

MODELING

Assume the attached data were measured in the field. From other considerations we expect the data to be represented by the functional form

$$Y = A \sin (Dx-E) + Bx + C$$

Determine the values of A, B, C, D, and E.

PROBLEM VII

REMOTE SENSING

Design a remote sensing project to determine the change in acreage of forest covered land to agriculture in an area 30 km by 30 km. Give specific details. The change is over a five year period between now and 5 years in the future.

PROBLEM VIII

PROBLEM ON DECISION MAKING

Objective: Maximize Benefits

Constraints:

1. Maintain timber production at (as a minimum) 3.0 cubic meters per hectare.

2. Provide enough forage to achieve a livestock gain (as a minimum) of 0.0040 kilograms per animal.

3. Improve wildlife habitat over existing conditions.

4. Increase water yield by (as a minimum) at least 10 percent.

Problems:

Given the above, and assuming that you are responsible for the management of a 70,000 hectare watershed, answer the following questions:

1. What additional biophysical and socio-economic information is required to select the "best alternative" in order to satisfy the objective?

2. Assuming that you had all of the required information, which alternative would you select, and why? (To answer this question, you may assume hypothetical biophysical and socio-economic information.)

Table 1. Product Mix Vectors for Southwestern Ponderosa Pine Forests 1/

Item	T_0	T_1	T_2	T_3
	As is	Convert	Uneven-aged	Even-aged
Timber cut (cubic meters)	0.0	9.0	4.9	3.8
Timber growth (cubic meters)	4.2	2.5	5.5	5.2
Livestock (kilograms gain)	0.068	0.48	0.0045	0.27
Wildlife (number of deer)	0.021	0.034	0.032	0.033
Water (centimeters)	15.0	22.0	16.0	18.0

1/ Adapted from efficient development and use of forest lands: An outline of a prototype computer-oriented system for operational planning by R.J. McConnon, 1965.

REGIONAL TRAINING COURSE

WATERSHED RESOURCES MANAGEMENT AND ENVIRONMENTAL MONITORING

Solutions to Problems

SOLUTIONS PROBLEM SET I

A. Probability Theory

1. The probability of rolling exactly 2 sixes in 5 rolls of an unbiased die: Here we can use the binomial distribution with N = 5 and X = 2.

$$\Pr(\text{"1" six in one roll}) = \frac{1}{6} = \frac{\text{No. ways of success}}{\text{Total No. possible outcomes}}$$

$$\Pr(\text{No six in one roll}) = \frac{5}{6}$$

What is Pr (X=2), i.e., 2 events (6's) in 5 rolls

$$\Pr(X=2) = {}_5C_2 \left(\frac{1}{6}\right)^2 \left(\frac{5}{6}\right)^{5-2}$$

$$= \frac{5!}{2!3!} \left(\frac{1}{6}\right)^2 \left(\frac{5}{6}\right)^3$$

$$= 10 \left(\frac{1}{36}\right) \left(\frac{125}{216}\right) = \underline{.161}$$

2. The probability of rolling no more than 2 sixes in 5 rolls of an unbiased die:

$$\Pr(X \leq 2) = \Pr(X=0) + \Pr(X=1) + \Pr(X=2)$$

We know Pr(X=2) = .161 from (1).

$$\Pr(X=0, N=5) = \frac{5!}{0!(5-0)!} \left(\frac{1}{6}\right)^0 \left(\frac{5}{6}\right)^5 = \left(\frac{5}{6}\right)^5 = \frac{3125}{7776} = \underline{.402}$$

$$\Pr(X=1, N=5) = \frac{5!}{1!(5-1)!} \left(\frac{1}{6}\right)^1 \left(\frac{5}{6}\right)^4 = 5\left(\frac{1}{6}\right)^1 \left(\frac{5}{6}\right)^4 = \frac{3125}{7776} = \underline{.402}$$

So $\Pr(X \leq 2) = .402 + .402 + .161 = \underline{.965}$

3. Pr (at least 2 sixes, N=5), know this is equal to:

$$\Pr(X \geq 2) = \Pr(X=2) + \Pr(X=3) + \Pr(X=4) + \Pr(X=5)$$

or

$$\Pr(X \geq 2) = 1 - [\Pr(X=0) + \Pr(X=1)]$$

since we know $(\Pr(X \geq 2) + \Pr(X<2) = 1$

So $\Pr(X \geq 2) = 1 - [(.402) + (.402)] = \underline{.196}$

4a. The probability of getting <u>exactly</u> a 100-year flood next year is = 0, since we have a continuous distribution. Recall, by definition, the probability of getting a flood next year that is equal to or greater than the 100-year flood is = .01.

Pr (exceeding 100 year flood in 1 yr) = .01

4b. The probability of not getting a flood greater than the 50-year flood in 50 years is determined as follows:

Pr ($X \geq$ 50-yr flood in any one year) = .02 by definition

So the Pr(X<50-yr flood) = 1 - .02 = .98 for any given year.

Thus, Pr(X<50-yr in 50 years)

X = 0 occurrence, N = 50

$$Pr(\bar{X}=0) = \frac{50!}{0!(50-0)!} (.02)^0 (.98)^{50}$$

$$= (.98)^{50}$$

$$= 50 \times \log .98$$

$$= 9.9912 - 10$$
$$\underline{\times\ 50}$$
$$499.5600 - 500$$

Antilog of .5600 = <u>.363</u>

4c. Pr(X>1000-yr flood, N=50)

p = .001, 1 - p = .999

Pr(X>1000-yr flood) = 1 - Pr(X<1000-yr flood)

$$Pr(X<1000\text{-yr flood}, N=50) = \frac{50!}{0!(50-0)!} (.001)^0 (.999)^{50}$$

$$= (.999)^{50}$$

$$= \text{Antilog } (50 \times \log .999)$$

$$= \text{Antilog } (50 \times (9.9996 - 10))$$

$$= \text{Antilog } (499.9785 - 500)$$

$$= \underline{.952}$$

So Pr(X>1000-yr flood) = 1 - .952 = <u>.048</u> ≈ <u>5%</u>

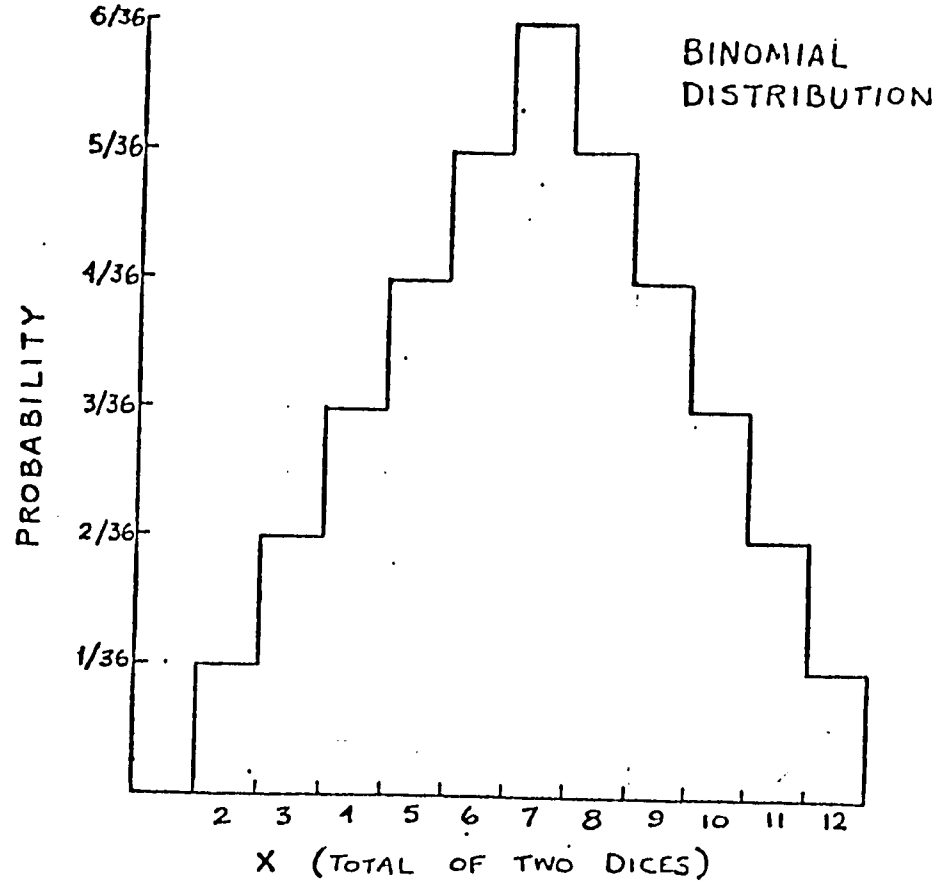
B. Frequency Distribution

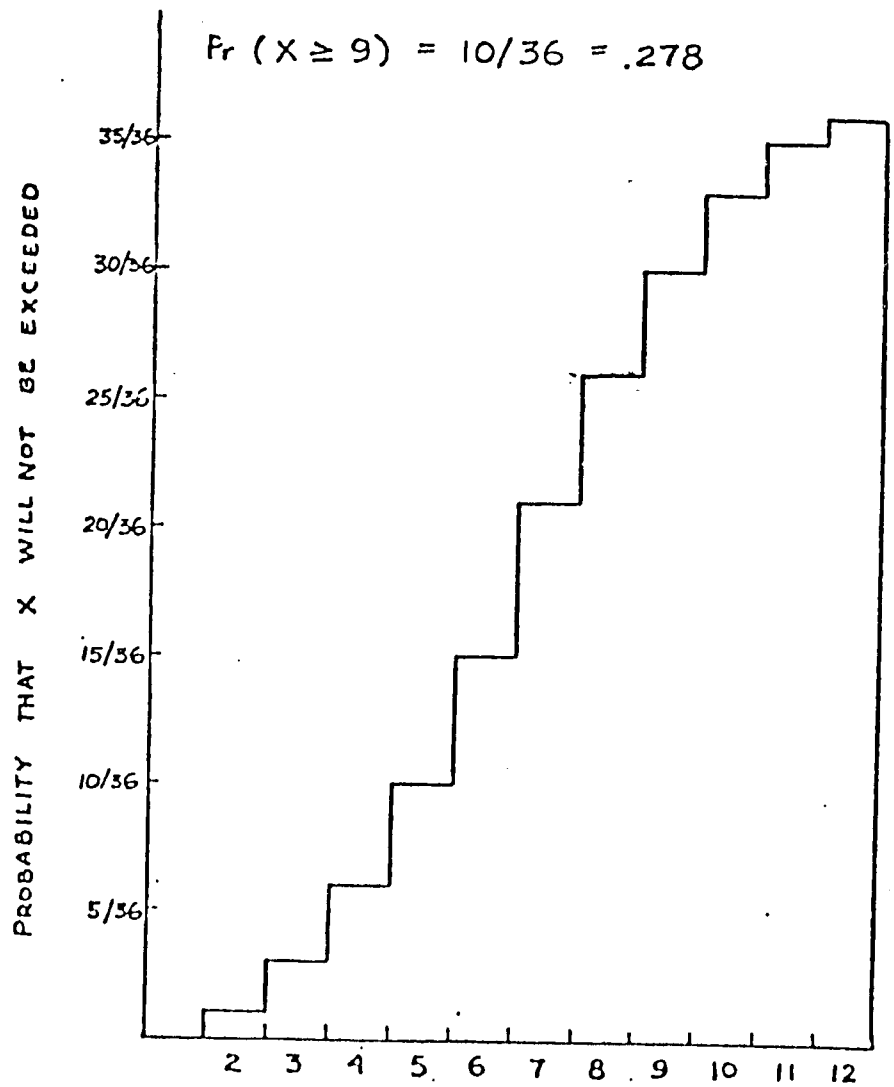
1. Compute the probabilities of all possible outcomes of one toss of a pair of unbiased dice:

Number on second die	Number on first die: 1	2	3	4	5	6
1	2	3	4	5	6	7
2	3	4	5	6	7	8
3	4	5	6	7	8	9
4	5	6	7	8	9	10
5	6	7	8	9	10	11
6	7	8	9	10	11	12

There are 36 possible outcomes, 6 of which produce the number 7. The probabilities are:

Outcome	Probability
2	1/36
3	2/36
4	3/36
5	4/36
6	5/36
7	6/36
8	5/36
9	4/36
10	3/36
11	2/36
12	1/36
	36/36


BINOMIAL DISTRIBUTION
PROBABILITY
6/36
5/36
4/36
3/36
2/36
1/36
2 3 4 5 6 7 8 9 10 11 12
X (TOTAL OF TWO DICES)


Pr (X ≥ 9) = 10/36 = .278
PROBABILITY THAT X WILL NOT BE EXCEEDED
35/36
30/36
25/36
20/36
15/36
10/36
5/36
2 3 4 5 6 7 8 9 10 11 12

3a. List the data in descending order.

Order No.	Data (X)	X^2	X^3
1	11	121	1331
2	8	64	512
3	8	64	512
4	8	64	512
5	7	49	343
6	6	36	216
7	6	36	216
8	5	25	125
9	4	16	64
	$\Sigma = 63$	475	3831

$$\text{Mean} = \frac{\Sigma X}{n} = \frac{63}{9} = 7$$

Median = middle value = 7

Mode = 8 (most frequent)

$$\text{Variance} = S^2 = \frac{\Sigma X^2 \quad (\Sigma X)^2/N}{} = \frac{475-441}{8} = 4.25$$

$$\text{Standard Deviation} = S = \sqrt{4.25} = 2.06$$

$$\text{Skew Coefficient} = \frac{N^2\Sigma X^3 - 3N\Sigma X\Sigma X^2 + 2\ (\Sigma X)^3}{N(N-1)\ (N-2)S^3}$$

$$= \frac{81(3831) - 27(63)(475) + 2(250047)}{9(8)(7)(8.75)}$$

$$= \underline{0.551}$$

3b. Assume Normal Distribution:

The normal standard deviate

$$Z = \frac{X - \mu}{\sigma}$$

Since this sample is our only knowledge of the parent population $\overline{X}$ is our unbiased estimate of μ and s is our unbiased estimate of σ. The probability of exceeding deviations $(X - \mu)$ are found in the normal distribution table corresponding to Z.

(1) $(X - \mu) > 0 = \frac{0}{S} = 0$

for $\frac{X-\mu}{\sigma} = 0$, $P = \underline{.50}$

(2) $(X - \mu) \geq + 2 = \frac{2}{2.06} = .97$

$P = .166$

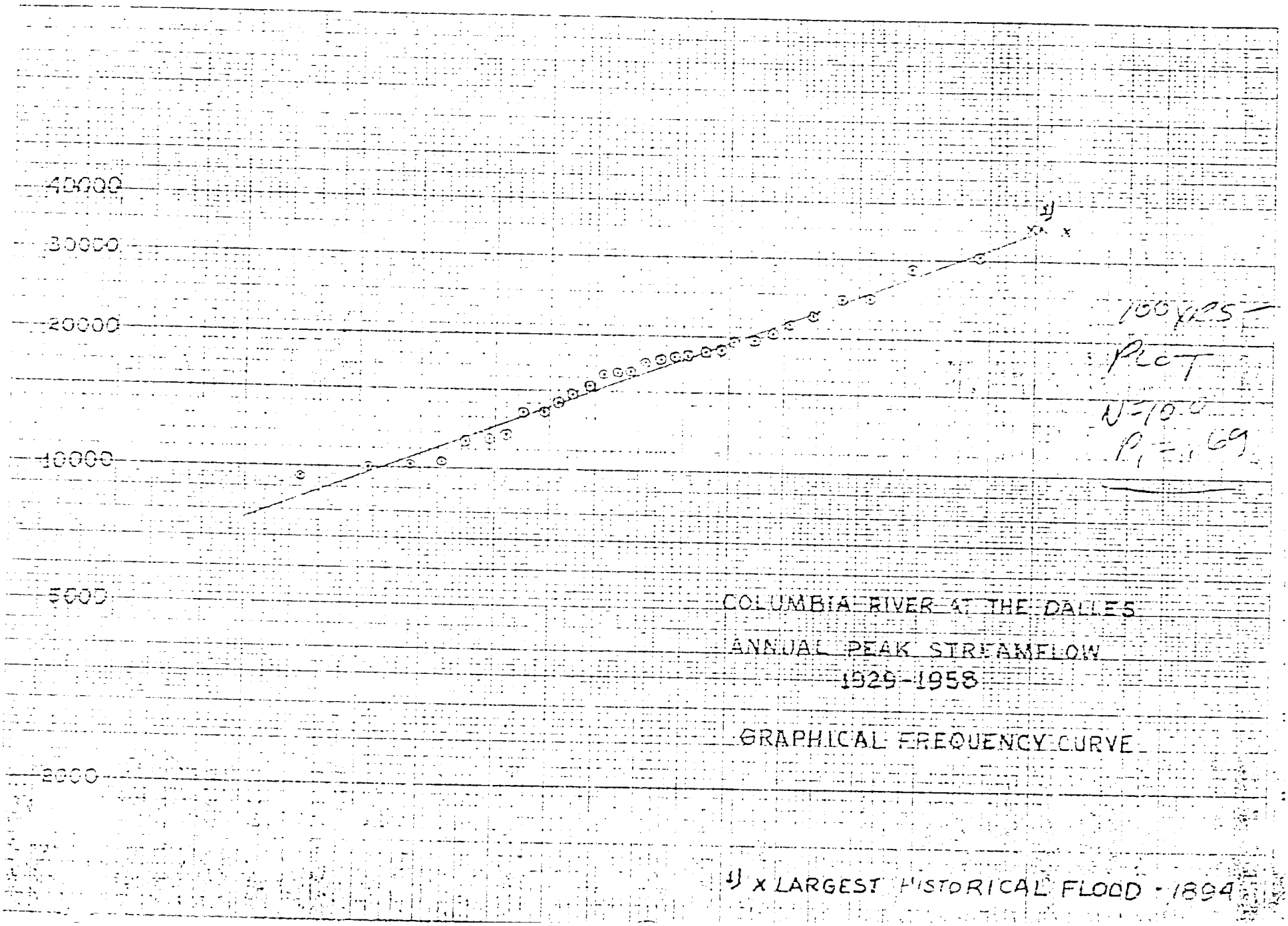
(3) $(X - \mu) \geq + 4 = \frac{4}{2.06} = 1.94$; $P = .026$

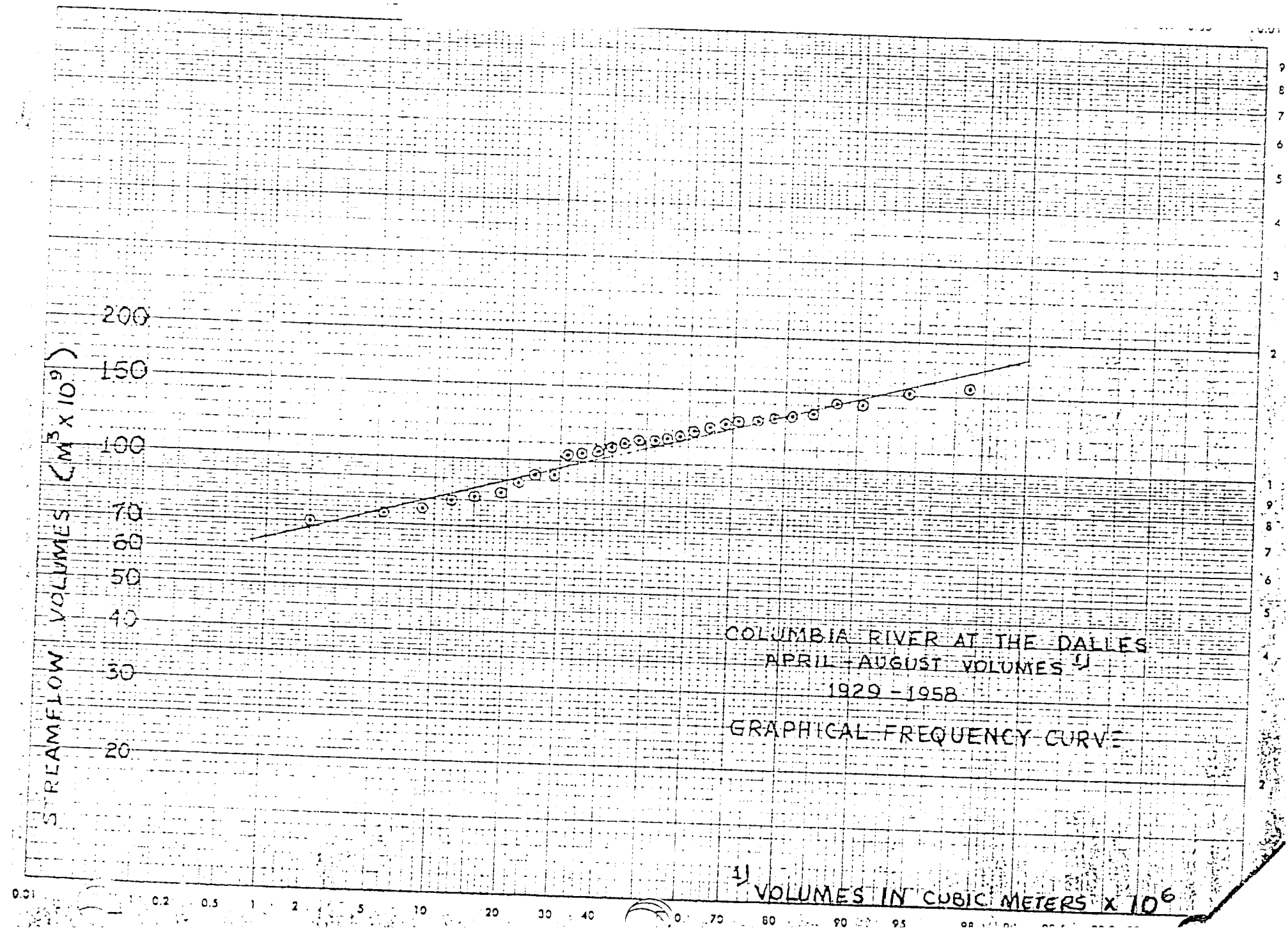
E. Graphical Frequency Analysis.

WATER YEAR	ANNUAL PEAK STREAMFLOW (CU.M/SEC)	PEAK FLOWS RANKED IN ORDER OF MAGNITUDE (CU.M/SEC)	PLOTTING POSITION (Table I-1, N=30)
1929	14190	30530	2.3
30	10140	27520	5.6
31	9570	24580	8.9
32	17840	24470	12.2
33	21440	22260	15.4
34	13422	21440	18.7
35	14750	20390	22.0
36	16310	19820	25.3
37	11720	19650	28.6
38	18460	18690	31.9
39	11980	18460	35.2
1940	11640	18120	38.5
41	10000	18090	41.8
42	13510	17840	45.1
43	16590	17610	48.4
44	10340	16590	51.6
45	15350	16420	54.9
46	17610	16310	58.2
47	16420	15350	61.5
48	30530	14750	64.8
49	19820	14190	68.1
1950	24470	13510	71.4
51	19650	13422	74.7
52	18120	11980	78.0
53	20390	11720	81.3
54	18090	11640	84.6
55	18690	10340	87.8
56	27520	10140	91.1
57	24580	10000	94.4
1958	22260	9570	97.7

GRAPHICAL FREQUENCY SOLUTION

WATER YEAR	APRIL-AUG. VOLUME ($m^3 X 10^9$)	APRIL-AUG. VOLUMES RANKED ACCORDING TO ORDER OF MAGNITUDE ($m^3 X 10^9$)	PLOTTING POSITIONS (%) (Table I-1, N=30)
1929	80	154	2.3
30	81	152	5.6
31	74	141	8.9
32	122	141	12.2
33	130	134	15.4
34	109	132	18.7
35	104	130	22.0
36	106	128	25.3
37	84	127	28.6
38	120	125	31.9
39	93	122	35.2
1940	89	120	38.5
41	76	117	41.8
42	105	116	45.1
43	134	115	48.4
44	70	113	51.6
45	93	111	54.9
46	125	109	58.2
47	116	106	61.5
48	152	105	64.8
49	111	104	68.1
1950	141	93	71.4
51	132	93	74.7
52	127	89	78.0
53	117	84	81.3
54	141	81	84.6
55	113	80	87.8
56	154	76	91.1
57	128	74	94.4
1958	115	70	97.7


40000
30000
20000
10000
5000
2000
1)
100 YRS
PLOT
N=70.0
P1=.69
COLUMBIA RIVER AT THE DALLES
ANNUAL PEAK STREAMFLOW
1929-1958
GRAPHICAL FREQUENCY CURVE
1) X LARGEST HISTORICAL FLOOD - 1894


STREAMFLOW VOLUMES ($M^3 \times 10^9$)
200
150
100
70
60
50
40
30
20
COLUMBIA RIVER AT THE DALLES
APRIL + AUGUST VOLUMES 1)
1929 - 1958
GRAPHICAL FREQUENCY CURVE
1) VOLUMES IN CUBIC METERS x 10^6
0.01
0.2
0.5
1
2
5
10
20
30
40
70
80
90
95

F. Solution - Frequency Study

Before Clearcutting (1962-1970)

Ranking	Control: Annual Peak ($m^3 sec^{-1} km^{-2}$)	Control: Log	Plotting Position	Clearcut: Annual Peak ($m^3 sec^{-1} km^{-2}$)	Clearcut: Log	Plotting Position
1	.155	-.807	7.43	.121	-.917	7.43
2	.147	-.833	18.1	.100	-1.00	18.1
3	.067	-1.174	28.7	.091	-1.041	28.7
4	.032	-1.495	39.4	.082	-1.086	39.4
5	.026	-1.585	50.0	.064	-1.194	50.0
6	.018	-1.745	60.6	.047	-1.328	60.6
7	.017	-1.770	71.3	.042	-1.377	71.3
8	.013	-1.886	81.9	.030	-1.523	81.9
9	.010	-2.00	92.6	.017	-1.770	92.6
	$\bar{X}$ =	-1.477		$\bar{X}$ =	-1.248	
	s =	.442		s =	.442	

After Clearcutting (1971-1979)

		Log			Log	
1	.354	-.451	7.43	.617	-.210	7.43
2	.071	-1.148	18.1	.196	-.708	18.1
3	.067	-1.174	28.7	.183	-.738	28.7
4	.064	-1.194	39.4	.175	-.757	39.4
5	.057	-1.244	50.0	.127	-.896	50.0
6	.021	-1.495	60.6	.115	-.939	60.6
7	.021	-1.678	71.3	.077	-1.114	71.3
8	.018	-1.745	81.9	.038	-1.420	81.9
9	.010	-2.00	92.6	.012	-1.921	92.6
	$\bar{X}$ =	-1.348		$\bar{X}$ =	-.967	
	s =	0.450		s =	0.485	

Solution

After Clearcutting (1971-1978)

Ranking	Control Annual Peak	Control Log	Control Plotting Position	Clearcut Annual Peak	Clearcut Log	Clearcut Plotting Position
1	.071	-1.148	8.30	.196	-.708	8.3
2	.067	-1.174	20.2	.183	-.738	20.2
3	.064	-1.194	32.1	.175	-.757	32.1
4	.057	-1.244	44.0	.127	-.896	44.0
5	.032	-1.495	56.0	.115	-.939	56.0
6	.021	-1.678	67.9	.077	-1.114	67.9
7	.018	-1.745	79.8	.038	-1.420	79.8
8	.010	-2.00	91.7	.012	-1.921	91.7
		$\bar{X}$ = -1.460			$\bar{X}$ = -1.062	
		s = 0.320			s = 0.420	

Analytical Frequency Curve Computations

Before Clearcutting

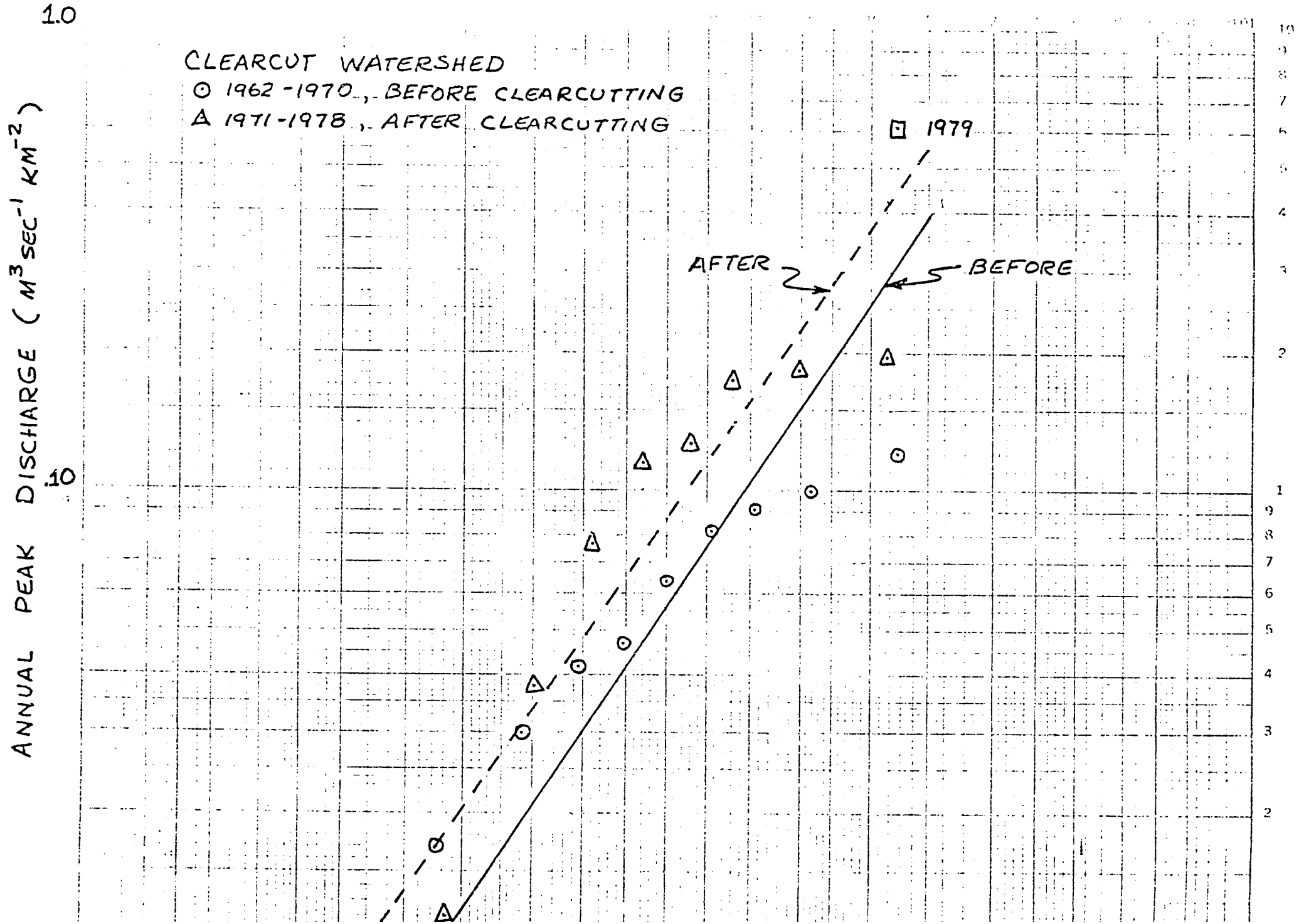
			Control		
P_∞	1.0	5.0	10.0	50	90
K(g=0)	2.33	1.64	1.28	0	-1.28
K· s	1.030	0.725	.566	0	-.566
Log Q = K· + $\overline{X}$	-.447	-.752	-.911	-1.477	-2.043
Q	.357	0.177	.123	0.033	.01
P_N (N-1=8)	2.9	7.9	13.1	50.0	86.9

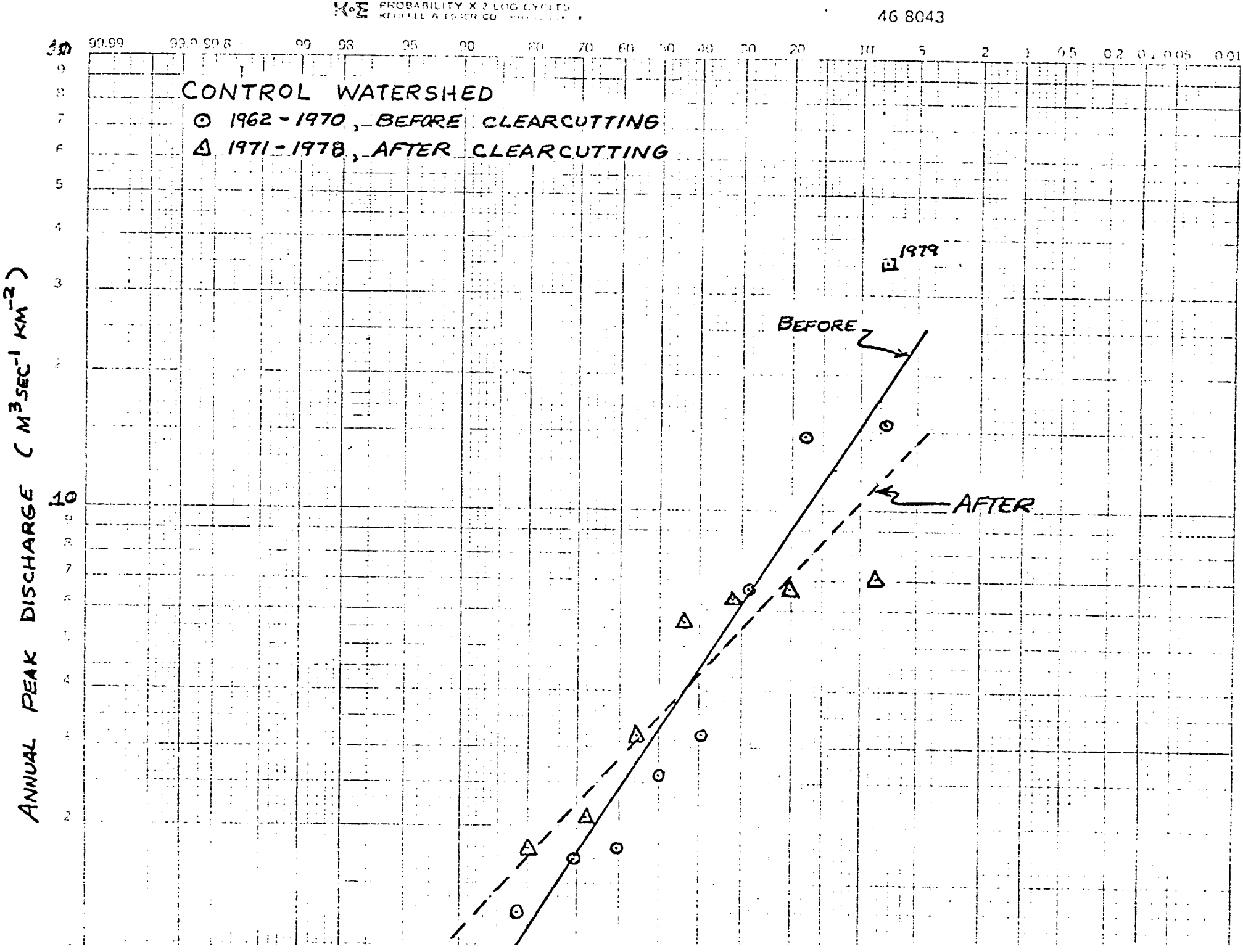
			Treatment		
P_∞	1.0	5.0	10.0	50	90
K(g=0)	2.33	1.64	1.28	0	-1.28
K· s	1.030	.725	.566	0	-.566
Log Q = K· + $\overline{X}$	-.218	-.523	-.682	-1.248	-1.814
Q	.605	.300	.208	0.056	.015
P_N (N-1=8)	2.9	7.9	13.1	50.0	86.9

After Clearcutting (1971-1978)

			Control		
P_∞	1.0	5.0	10.0	50	90
K(g=0)	2.33	1.64	1.28	0	-1.28
K· s	.746	.525	.410	0	-.410
Log Q = K· + $\overline{X}$	-.714	-.935	-1.050	-1.460	-1.870
Q	.193	.116	.089	.035	.013
P_N (N-1=7)	3.2	8.3	13.5	50	86.5

			Clearcut		
P_∞	1.0	5.0	10.0	50.0	90.0
K(g=0)	2.33	1.64	1.28	0	-1.28
K· s	.979	.689	.538	0	-.538
Log Q = K· + $\overline{X}$	-.083	-.373	-.524	-1.062	-1.600
Q	.826	.424	.299	.087	.025
P_N	3.2	8.3	13.5	50	86.5


CLEARCUT WATERSHED
⊙ 1962-1970, BEFORE CLEARCUTTING
△ 1971-1978, AFTER CLEARCUTTING
⊡ 1979
AFTER
BEFORE
ANNUAL PEAK DISCHARGE ($M^3 SEC^{-1} KM^{-2}$)
1.0
.10


K+E PROBABILITY X 3 LOG CYCLES
KEUFFEL & ESSER CO.
46 8043
CONTROL WATERSHED
⊙ 1962 - 1970, BEFORE CLEARCUTTING
△ 1971 - 1978, AFTER CLEARCUTTING
1979
BEFORE
AFTER
ANNUAL PEAK DISCHARGE (M³SEC⁻¹ KM⁻²)

SOLUTION PROBLEM II

WATER BUDGET EXERCISE

Table II-A. WATER BUDGET EXERCISE - AFTER CLEARCUTTING

	Year 1												Year 2								
	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN
	mm																				
1. Average rainfall	56	103	133	90	100	48	27	4	31	42	36	12	50	120	140	105	90	95	65	20	46
2. Initial soil moisture	56	57	140	273	279	279	279	248	181	181	181	181	181	181	279	279	279	279	279	279	210
3. Total available moisture	114	160	273	371	379	327	306	252	212	223	217	193	231	301	419	384	369	374	344	299	256
4. Potential ET	57	20	0	0	3	13	58	89	127	173	157	107	57	20	0	0	3	13	58	89	127
5. Actual ET	57	20	0	0	3	13	58	71	31	42	36	12	50	20	0	0	3	13	58	89	75
6. Remaining avail. moist.	57	140	273	371	376	314	248	181	181	181	181	181	181	281	419	384	366	361	286	210	181
7. Final soil moisture	57	140	273	279*	279	279	248	181	181	181	181	181	181	279	279	279	279	279	279	210	181
8. Runoff	0	0	0	92	97	35	0	0	0	0	0	0	0	2	140	105	87	82	7	0	0

* Note: Just as with the "before clearcutting" condition, the same total water deficit in the soil must be satisfied initially; once satisfied, the soil water which can be depleted by ET is 98 mm, therefore, 181 mm of soil water can no longer be depleted by ET and becomes the minimum soil water content.

Assumptions:

1. After timber harvesting (for the 21-mo. period) soil water can be depleted only to a depth of 0.6 m which corresponds to (0.6 m x 164 mm/m) = 98 mm available water for soil profile. This is due to restricted root uptake of water by remaining vegetation which is largely herbaceous and brush spp.
2. Groundwater "leakage" out of or into the system is negligible.
3. Rainfall and soil moisture data are representative of areal averages for the watershed.
4. Infiltration rates were not sufficiently reduced, i.e., stormflow volumes remained a function only of soil water storage capacity.

Factors important in determining "availability" of increased water yield:

1. Distance from clearcut area(s) to downstream reservoir, i.e., evaporative losses, transpiration losses by ripanan vegetation.
2. Channel characteristics - transmission losses.
3. Timing of the increased water yield, i.e., if the increased streamflow occurs during periods when reservoirs are full, the additional water supplies will not be retained.

Water Yield Before Clearcut = 466 mm x 10 m^3 mm^{-1} ha^{-1} x 190 ha = 885,400 m^3

Water Yield After Clearcut = 647 mm x 10 m^3 mm^{-1} ha^{-1} x 190 ha = 1,229,300 m^3

$\frac{(1,229,300 - 885,400)}{\ldots}$ x 100 = 39% increase

PROBLEM SET III - Solution

B. Mean Basin Precipitation

Total rainfall at each station:

Dale	= 67
Ford	= 115
Grayson	= 36
Bear	= 63
Casper	= 48
Eileen	= 84
North	= 102

The storm isohyetal pattern and the Thiessen Polygons are shown on page 2.

Thiessen Polygon Method

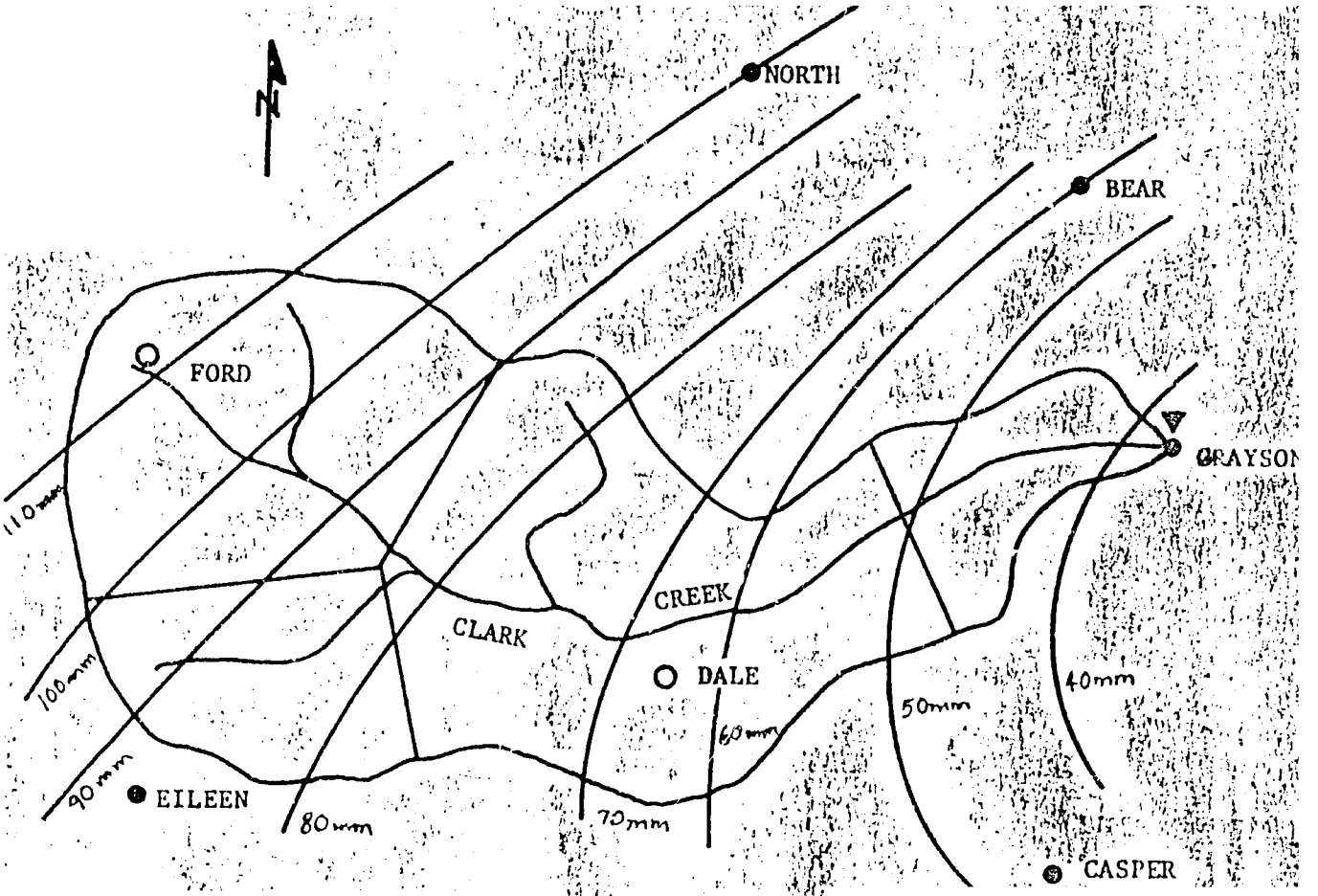
Station	Depth at Station	Area Within Polygon	Volume
Grayson	36	30	1080
Dale	67	115	7700
Eileen	84	36	3020
Ford	115	80	9200
			21000

Basin mean rainfall = 81 millimeters

Combination Method

Station	Mean Depth From Isohyets	Area Within Polygon	Volume
Grayson	45	30	1350
Dale	70	115	8050
Eileen	88	36	3170
Ford	100	80	8000
			21570

Basin mean rainfall = 82 millimeters



CLARK CREEK BASIN

Scale 1:20,000

LEGEND

▼ Stream Gage

● Daily Rainfall Gage

○ Hourly Recording Rainfall Gage

PROBLEM SET III

B. Loss Rates

Problem Solution

Time (hrs)	Dale Precip (mm)	Dale Precip (%)	Basin Mean (mm)*	Basin Mean (mm)	Loss (mm)**	Excess (mm)
2	0					
4	0					
6	6	9.0	7	7	7	
8	22	33	27	20	12	8
10	43	64	53	26	12	14
12	64	95	78	25	12	13
14	67	100	82	4	4	
				82		35

* Dale accumulated % of precipitation times 82 millimeters.

** From successive approximations, the uniform loss rate is 12 millimeters per/2 hr or 6 millimeters/hr.

PROBLEM SET III

C1. Unit Graph Derivation - Solution

Day	Time Ending (hrs)	Rainfall Excess (mm)	Total Flow	Base Flow	Direct Runoff	6-hour Unit Graph Ordinates
			- - - -	cubic meters per	second	- - - -
10-02	0600	0	7.5	7.5	0	0
	0800	8	20.0	7.5	12.5	0.4
	1000	14	50	7.5	42.5	1.2
	1200	13	155	7.5	147.5	4.2
	1400	0	265	7.5	257.5	7.4
	1600		280	7.5	272.5	7.8
	1800		240	7.5	232.5	6.6
	2000		118	8.0	110	3.1
	2200		75	8.0	67	1.9
	2400		53	10.0	43	1.2
	0200		38	10.0	28	.8
	0400		30	10.0	20	.6
	0600		25	10.0	15	.4
	0800		17	10.0	7	.2
	1000		15	10.0	5	0.1
	1200		12	10.0	2	0.1
	1400		10	9.0	1	.0
	1600		9	9	0	0

* Rounded off to nearest .1 cumecs

Direct Runoff Volume = 35 mm

So. U. G. Ordinates = Q : 35 mm

Assuming that the rainfall excess is uniformly distributed, the above ordinates are for a 6-hour unit hydrograph.

2. In order to apply the 6-hour unit hydrograph to the December 1941 flood, the following steps would be necessary:

   a. Determine the rainfall excess that contributed to the December 1941 flood (17 mm, given in the problem).

   b. Select a duration over which a portion or all of the rainfall excess is uniformly distributed. In this case, the rainfall excess of 17 mm was relatively uniform in its distribution over the 4-hour period.

   c. The S-curve method must be applied to convert the 6-hour unit graph to a 4-hour unit graph by the following procedure:

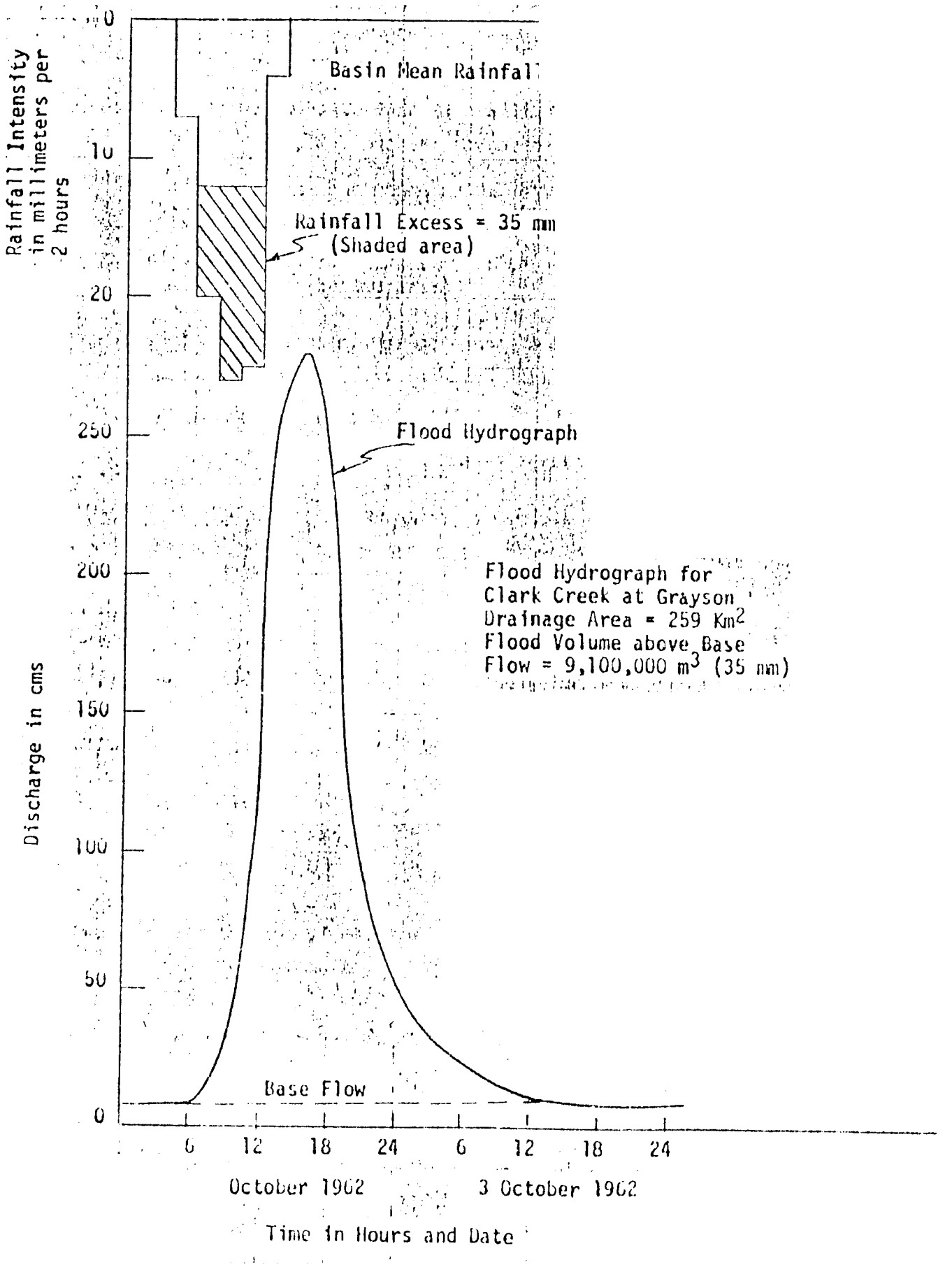
      (1) Develop an S-curve from the 6-hour unit graph, and smooth out the curve as needed.

      (2) Offset the S-curve by 4 hours and tabulate the difference between the ordinates of the original S-curve and those of the offset curve (ΔY).

      (3) The tabulated differences between the S-curves (ΔY) must then be adjusted to 1 mm of direct runoff. Since the 6-hour unit graph corresponds to 1 mm total or .17 mm/hr. the unit graph ordinates for the 1 mm excess in 4 hours are calculated as follows:

$$\begin{aligned} U.G. &= \Delta Y/.17 \text{ mm per hour} * 4 \text{ hours} \\ &= \Delta Y/.68 \text{ mm} \end{aligned}$$

   d. Apply the rainfall excess of 0.68 mm to the 4-hour unit graph ordinates to obtain the direct runoff component of flow (multiply ordinate by .68). Then add the base flow component to obtain the reconstructed hydrograph for the December 1941 flood.


Rainfall Intensity in millimeters per 2 hours
0
10
20
Basin Mean Rainfall
Rainfall Excess = 35 mm
(Shaded area)
Flood Hydrograph
Flood Hydrograph for
Clark Creek at Grayson
Drainage Area = 259 Km2
Flood Volume above Base
Flow = 9,100,000 m3 (35 mm)
Discharge in cms
250
200
150
100
50
0
Base Flow
6
12
18
24
6
12
18
24
October 1962
3 October 1962
Time in Hours and Date

Reconstruction of December 1941 Flood

Derivation of 4-hour U.G. Ordinates by the S-Curve Method

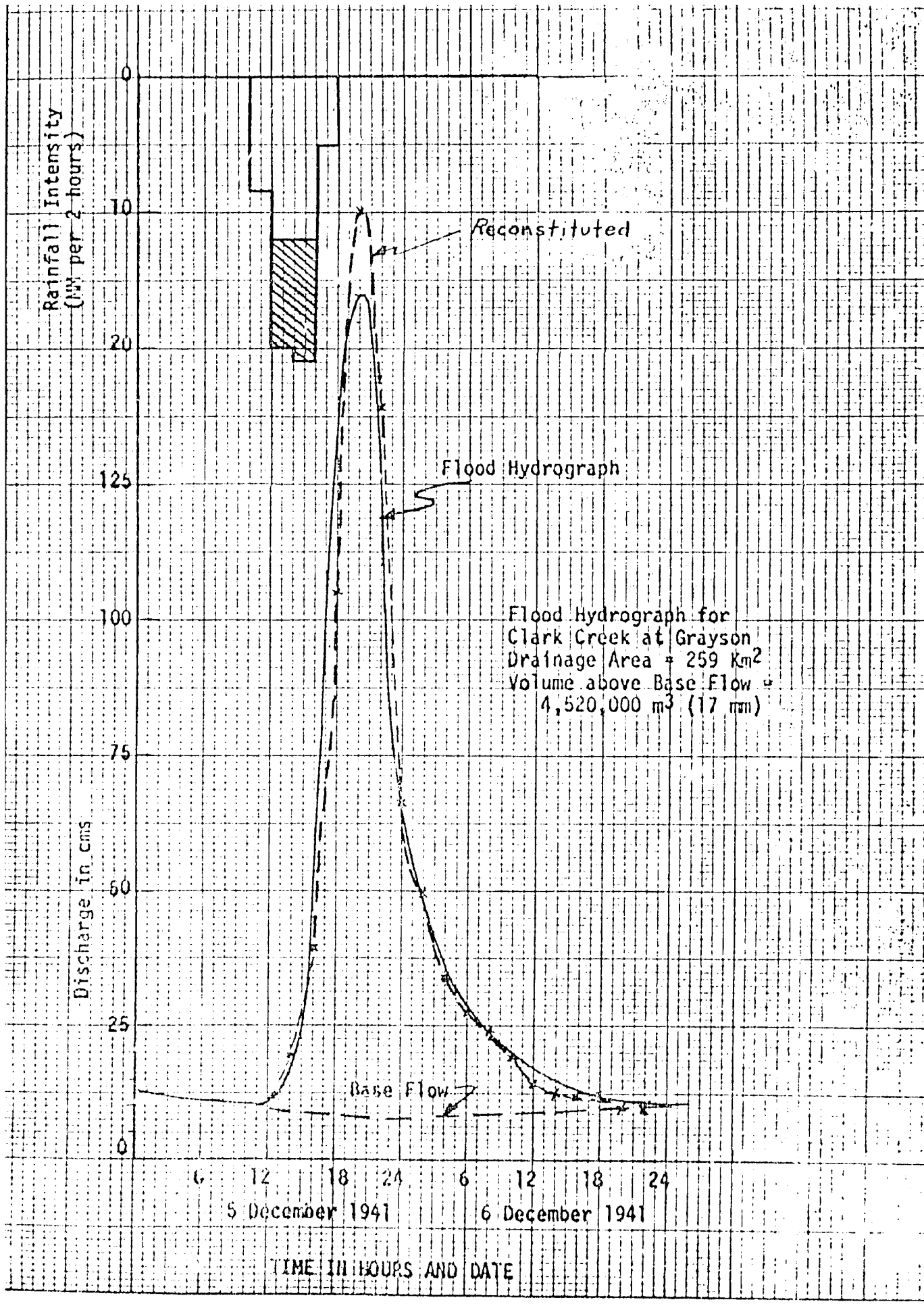
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Time in Hours	6-Hour U.G. Ordinates	S-Hydrograph Additions	Rough S-Curve	Smoothed S-Curve	4-Hour Lagged S-Curve	4-Hour* Unit Hydrograph
	- - - - - - - - - - - -		cubic meters per second		- - - - - - - - - - - -	
0	0		0	0		0
2	.4		[illegible]	.4		.6
4	1.2		1.2	1.2		1.8
6	4.2		4.2	4.2	.4	5.7
8	7.4	.4	7.8	7.8	1.2	9.9
10	7.8	1.2	9.0	9.0	4.2	7.2
12	6.6	4.2	10.8	10.1	7.8	3.4
14	3.1	7.8	10.9	10.7	9.0	2.5
16	1.9	9.0	10.9	11.1	10.1	1.5
18	1.2	10.8	12.0	11.4	10.7	1.1
20	.8	10.9	11.7	11.7	11.1	.9
22	.6	10.9	11.5	11.8	11.4	.6
24	.4	12.0	12.4	11.9	11.7	.3
26	.2	11.7	11.9	11.9	11.8	.2
28	.1	11.5	11.6	12.0	11.9	.2
30	.1	11.4	11.5	12.0	11.9	.2
32	0	11.9	11.9	12.0	12.0	0
34	0	11.6	11.6	12.0	12.0	0
36						

* 4-hour unit hydrograph ordinates = $\Delta Y/IT$; = $\Delta Y/(.167)(4)$
or (column 5 - Column 6) : (.167)(4)

(2) Application of 4-hour U.G. to reconstruct the December 1941 flood.

Date	Time (hrs.)	4-hour U.G. Ordinates (cumecs)	Rainfall Excess (mm)	Direct Runoff* (cumecs)	Base Flow (cumecs)	Total Flow (cumecs)
12-05-41	1200	0	17	0	10	10.0
	1400	.6		10.2	8.8	19.0
	1600	1.8		30.6	8.6	39.2
	1800	5.7		96.9	8.5	105.4
	2000	9.9		168.3	8.5	176.8
	2200	7.2		122.4	7.5	129.9
	2400	3.4		57.8	7.5	65.3
12-06-41	0200	2.5		42.5	7.5	50.0
	0400	1.5		25.5	8.0	33.5
	0600	1.1		18.7	8.8	27.5
	0800	.9		15.3	8.8	24.1
	1000	.6		10.2	8.8	19.0
	1200	.3		5.1	8.8	13.9
	1400	.2		3.4	9.0	12.4
	1600	.2		3.4	9.0	12.4
	1800	.2		3.4	9.0	12.4
	2000	0		0	10.0	10.0
	2200				10.0	10.0
	2400					

* Direct runoff = U.G. ordinates * 17, and are rounded off to the nearest 0.1 cumecs.


Rainfall Intensity
(mm per 2 hours)
0
10
20
Reconstituted
Flood Hydrograph
125
100
75
50
25
0
Discharge in cms
Flood Hydrograph for
Clark Creek at Grayson
Drainage Area = 259 Km2
Volume above Base Flow =
4,520,000 m3 (17 mm)
Base Flow
6
12
18
24
6
12
18
24
5 December 1941
6 December 1941
TIME IN HOURS AND DATE

V. RESERVOIR MANAGEMENT EXERCISE - SOLUTION

1. Demand Curve:

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
Storage[a]	290.2	290.3	290.4	300.5	300.8	300.9	300.8	300.7	300.3	290.2	290.2	290.2
Top of buffer	2890	2890	2890	2890	2890	2890	2890	2890	2890	2890	2890	2890
Total Storage	3180.2	3180.4	3180.4	3190.5	3190.8	3190.8	3190.8	3190.7	3190.3	3180.2	3180.2	3180.2
Elevation[b]	196.0	196.0	196.0	196.05	196.05	196.05	196.05	196.05	196.05	196.05	196.05	196.05

[a] Storages in $m^3 \times 10^6$

[b] Elevation of pool in meters.

2. Matrix

	Water Supply	Irrigation	Hydroelec.	Min. Flow	Flood Control
Water supply	--	MC	CF	CF	CF
Irrigation	MC	--	CO	CF	X
Hydroelec.	CF	CO	--	CO	CF
Min. Flow	CF	CF	CO	--	CF
Flood Control	CF	X	CF	CF	--

CO = complementary
CF = conflicting
MC = moderately conflicting
X = no conflict because of timing difference

3. Present sedimentation rate = 0.9 tons/ha/yr

$$0.9 \text{ tons/ha/yr} \times 200 \text{ ha} = 180.0 \text{ tons/yr}$$
$$180 \text{ tons/yr} \times 10^6 \text{ g/ton} \times 1 \text{ cm}^3/1.6 \text{ g} \times \text{m}^3/10^6 \text{ cm}^3 = 112.5 \text{ m}^3/\text{yr}$$
$$112.5 \text{ m}^3/\text{yr} \times 40 \text{ yr} = 4500 \text{ m}^3 \text{ dead storage}$$

With poor cultivation and overgrazing, we can expect 35 tons/ha/yr, which results in 7000 tons/yr.

$$7000 \text{ tons/yr} = 4375 \text{ m}^3/\text{yr}$$

Therefore, the design dead storage space would be filled with sediment during the second year following construction with the poor land use practices described.