

6.1. Land use and population density data are given for several cities in the table on the following page. Estimate the percent imperviousness for each city using two methods: (1) imperviousness as a function of population density and (2) weighted imperviousness as a function of land use. Assume that undeveloped land is the same as "open" in Table 6.2, and "other" is 50% institutional and 50% open. (Data sources: Heaney et al., 1977; Sullivan et al., 1978.)

Solution using following assumptions about imperviousness (Table 6.2).

Land Use	Imperviousness (%)
Residential	30
Commercial	81
Industrial	40
Other	17.5 = 0.5 * institutional + 0.5 * open
Undeveloped	5 = open

City	Urbanized Area (1000-ac)	Popul. Dens. (pers/ac)	Land Use (Percent).....					Percent Impervious		
			Res.	Com.	Ind.	Other*	Undev.*	Total	By Pop. Dens.	By Land Use
Boston, MA	425	6.24	38.2	5.6	9.7	11.9	34.6	100	25.9	23.7
Trenton, NJ	42	6.59	39.3	5.8	10.0	12.3	32.6	100	26.6	24.3
Tallahassee, FL	19	4.06	29.1	4.3	7.4	9.1	50.1	100	20.7	19.3
Houston, TX	345	4.86	33.0	4.8	8.3	10.2	43.7	100	22.8	21.1
Chicago, IL	626	9.13	46.0	6.8	11.7	14.3	21.2	100	31.4	27.6
Denver, CO	188	5.58	35.8	5.3	9.1	11.2	38.6	100	24.4	22.6
San Francisco, CA	436	6.86	40.2	5.9	10.2	12.5	31.2	100	27.2	24.7
Windsor, ON	26	7.63	38.0	6.0	10.0	18.7	27.3	100	28.7	24.9

\*"Other" = recreational, schools and colleges, cemeteries.

\*High "undeveloped" results from definition of urbanized area which includes population densities as low as 1 pers/ac.

6.2. A small, 2-ha, mostly impervious urban catchment has an average slope of 1.5% and the following average Horton infiltration parameters:  $f_0 = 4$  mm/hr,  $f_c = 1$  mm/hr,  $k = 2.2$  hr<sup>-1</sup>. (Infiltration occurs through cracks in the paving.) Consider the rainfall hyetograph in Fig. P6.2.

- a) Determine the depression storage using Fig. 6.3.
- b) Determine the time of beginning of runoff.
- c) Calculate and sketch the hyetograph of rainfall excess. Use the same time intervals as for the rainfall hyetograph, and average the first nonzero rainfall excess over the whole time interval.
- d) Determine the runoff coefficient.
- e) Compute the volume of runoff, in m<sup>3</sup>.

a. First, calculate the depression storage. Use Fig. 6.3 or the equation on the figure.

$$DS = 0.0303 S^{(-0.49)} \text{ inches.}$$

$$DS = 0.0303 1.5^{-0.49} = 0.024840 \text{ inches} = 0.630945 \text{ mm.}$$

b. Time to start of runoff:

Volume of first rainfall increment = 6 mm/hr x 1/12 hr = 0.5 mm.  
(Entire first rainfall increment contained by depression storage.)

$$\text{Unused depression storage} = 0.630945 - 0.5 \text{ mm} = 0.130945 \text{ mm.}$$

$$\text{Time to start of runoff} = 1/12 \text{ hr} + 0.130945 \text{ mm} / 20 \text{ mm/hr} = 0.089880 \text{ hrs} = 5.392835 \text{ minutes.}$$

See table for plot calculations.

For plot, start infiltration at end of depression storage. Can plot average rainfall excess during second interval starting at 5.393 min or as average over total 5-min interval.

$$\text{Average} = 16.23 * (10 - 5.393) / 5 = 14.96 \text{ mm/hr.}$$

# Calculations for hyetograph

(Use Lotus 1-2-3 spreadsheet)

6.2 cont'd

fo 4 mm/hr  
fc 1 mm/hr  
k -2.2 1/hr

Avg. over  
Whole  
2nd  
Interval

Time (min)	Intensity (mm/hr)	Time (min)	Intensity (mm/hr)	Horton Infil. (mm/hr)	Avg. Horton Infil. (mm/hr)	Avg. Rainfall Excess (mm/hr)	Avg. Rainfall Excess (mm/hr)
0-5	6	0	6				
5-10	20	5	6				
10-15	16	5	20				14.96
15-20	8	5.392835	20	4.00	3.77	16.23	14.96
20-25	10	10	20	3.53	3.77	16.23	14.96
25-30	14	10	16	3.53	3.32	12.68	12.68
30-35	11	15	16	3.11	3.32	12.68	12.68
35-40	7	15	8	3.11	2.93	5.07	5.07
40-45	9	20	8	2.76	2.93	5.07	5.07
45-50	4	20	10	2.76	2.61	7.39	7.39
		25	10	2.46	2.61	7.39	7.39
Sum	105	25	14	2.46	2.34	11.66	11.66
		30	14	2.22	2.34	11.66	11.66
		30	11	2.22	2.12	8.88	8.88
		35	11	2.01	2.12	8.88	8.88
		35	7	2.01	1.93	5.07	5.07
		40	7	1.84	1.93	5.07	5.07
		40	9	1.84	1.77	7.23	7.23
		45	9	1.70	1.77	7.23	7.23
		45	4	1.70	1.64	2.36	2.36
		50	4	1.58	1.64	2.36	2.36
		50	0	1.58			
		5.392835	0				
		5.392835	20				

## c. Runoff Coefficient

Rainfall volume = sum of ordinates (mm/hr) x 1/12 hr =  
8.75 mm

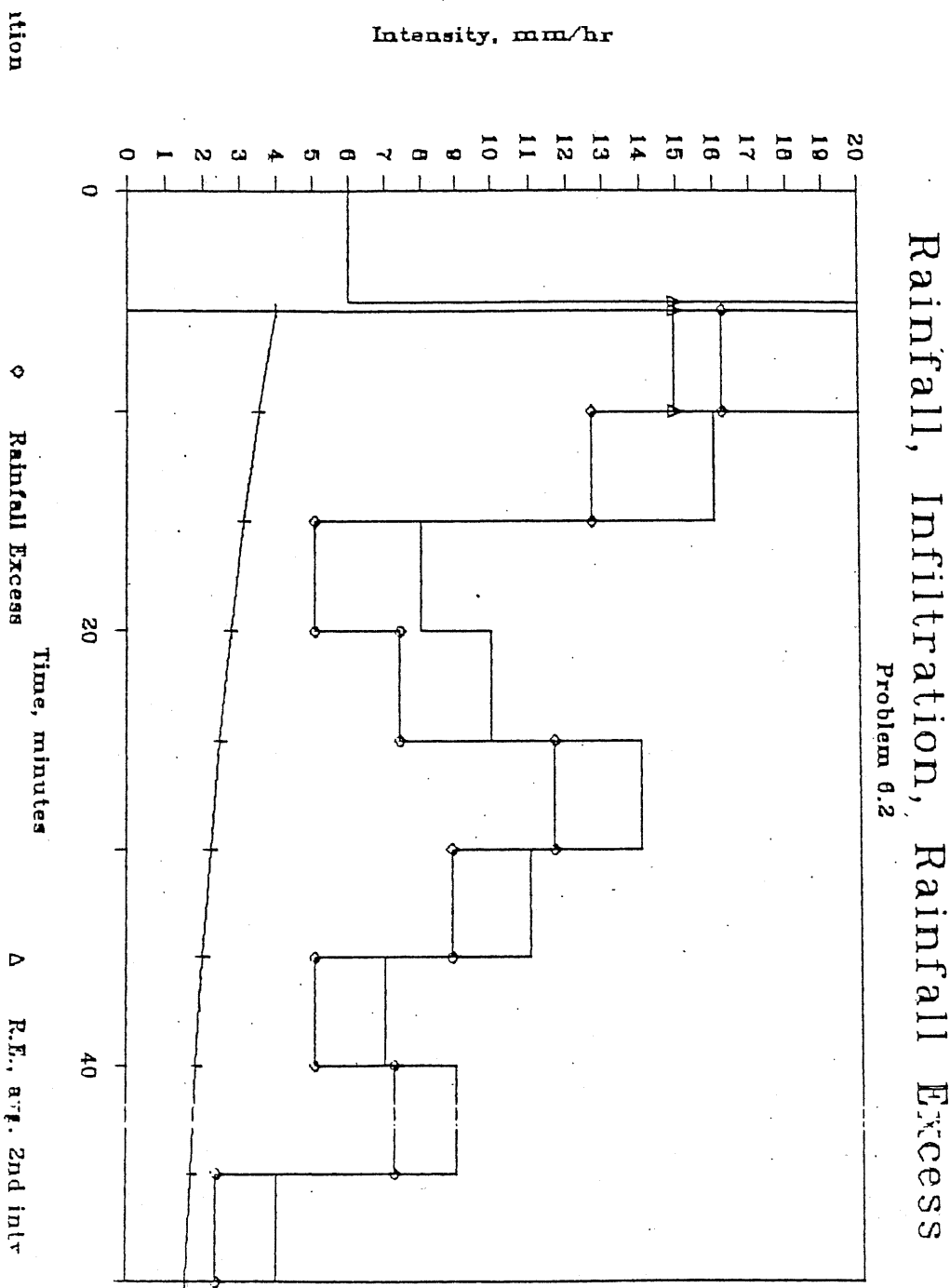
Losses = Depression storage plus infiltration =  
0.63 mm + integral of Horton from 0 to 50-5.393 min =  
0.63 mm + integral from 0 to 0.743 hr =  
0.63 mm + 1.840685 = 2.470685 mm

Rainfall excess = Rainfall - losses = 6.279314 mm

Runoff coefficient = Rainfall Excess / Rainfall = 0.718

d. Volume of runoff 6.2 con'd

Volume = Rainfall Excess x Area =  
6.279314 mm x 2 ha x 1/1000 m/mm x 10000 sq m/ha =  
125.6 cubic meters



- 6.3. Using the runoff volume found in Problem 6.2, determine the depth of storage in a retention basin that has vertical walls and a base area of 500 m<sup>2</sup>.

Depth of storage in retention basin

$$\text{Depth} = \text{Volume} / \text{Area} = 125.6 \text{ cu m} / 500 \text{ sq. m} = 0.2512 \text{ meters}$$

- 6.4. For the hyetograph of hourly rainfall values shown in Fig. P6.4, determine the number of events corresponding to minimum inter-event times (MITs) of 0, 1, 2, 3, 4, and 5 hr. What MIT is needed to have the entire 40-hr sequence treated as one event?

No. of events is tabulated below for various MIT values. When MIT = 7, the entire rainfall sequence becomes one event.

Hour	Rain (0.01 in)	MIT (hrs)	No. Events
0	5	0	20
1	7	1	8
2	1	2	7
3	1	3	6
4		4	2
5	1	5	2
6			
7		6	2
8		7	1
9	4		
10			
11			
12			
13	6		
14	1		
15			
16			
17			
18			
19			
20			
21	1		
22	12		
23	8		
24	4		

6.4 cont'd

25	
26	
27	
28	2
29	
30	
31	1
32	9
33	6
34	2
35	
36	
37	
38	2
39	3
40	8

6.5 The EPA SYNOP program (Environmental Protection Agency, 1976; Hydroscience, 1979; SWMM Rain block – Huber and Dickinson, 1988) has been run for hourly rainfall data for Houston for the period 1948–1979. A minimum interevent time of 16 hr was used to separate independent storm events, giving the following results:

Rank	Date Mo/Dy/Yr	Volume inches	Duration hours	log volume
1	6/11/73	11.55	52	1.06258
2	6/24/60	11.33	63	1.05423
3	10/11/70	7.15	17	0.85431
4	10/14/57	6.78	40	0.83123
5	11/12/61	6.59	22	0.81889
6	7/14/49	6.33	53	0.80140
7	6/3/62	5.73	25	0.75815
8	6/20/63	5.69	11	0.75511
9	4/14/66	5.49	17	0.73957
10	9/9/71	5.45	29	0.73640
11	9/4/73	5.28	41	0.72263
12	7/9/61	5.22	77	0.71767
13	4/14/73	4.81	47	0.68215
14	10/22/70	4.80	21	0.68124
15	10/6/49	4.60	46	0.66276
16	7/29/54	4.55	24	0.65801
17	9/19/67	4.43	47	0.64640
18	8/24/67	4.38	27	0.64147
19	3/20/72	4.22	7	0.62531
20	10/31/74	4.10	34	0.61278
21	5/12/72	3.98	4	0.59988
22	5/21/70	3.90	10	0.59106
23	6/17/68	3.86	8	0.58659
24	8/2/71	3.84	46	0.58433
25	5/15/70	3.74	28	0.57287
26	7/7/73	3.70	11	0.56820
27	12/10/63	3.60	94	0.55630
count		27	27	27
average		5.37	33.37	0.7082
stdev		2.014935	22.1848	0.13248
cv		0.374936	0.664805	0.187065
skew		2.150456	0.938132	1.354677

Part a: Determine magnitudes for T = 2, 5, 10, 25 and 50 yrs using LP3 plot.

The plot is shown on worksheet "LP3Chart"  
The plot was prepared on worksheet "ProbPlot"

Part b:

From the plot:

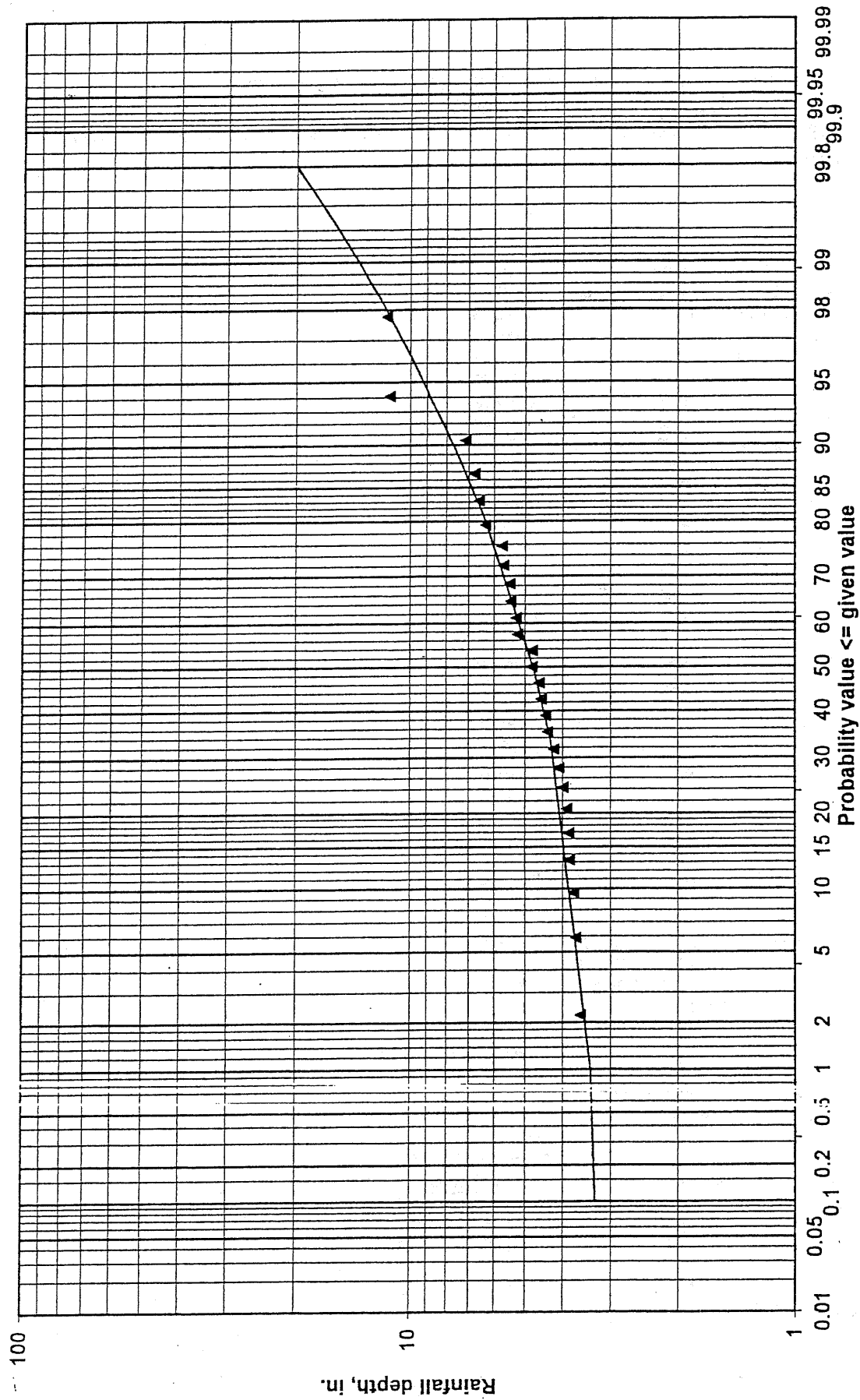
T	F Depth (in.)	
2	0.5	4.8
5	0.8	6.4
10	0.9	7.7
25	0.96	9.7
50	0.98	11.6
100	0.99	13.7

24-hr IDF, Fig. 1.8

i, in./hr	Depth, in.
0.19	4.6
0.26	6.2
0.305	7.3
0.37	8.9
0.40	9.6
0.47	11.3

6.5 continued...

Houston Storms, 1948 - 1979





6.6. Using the IDF curves for Houston shown in Fig. 1.8 and the SCS hyetograph distribution given in Table 6.7, prepare a 25-yr SCS type II design storm for Houston. Plot the hyetograph of hourly values.

**Problem 6.6. Solution:**

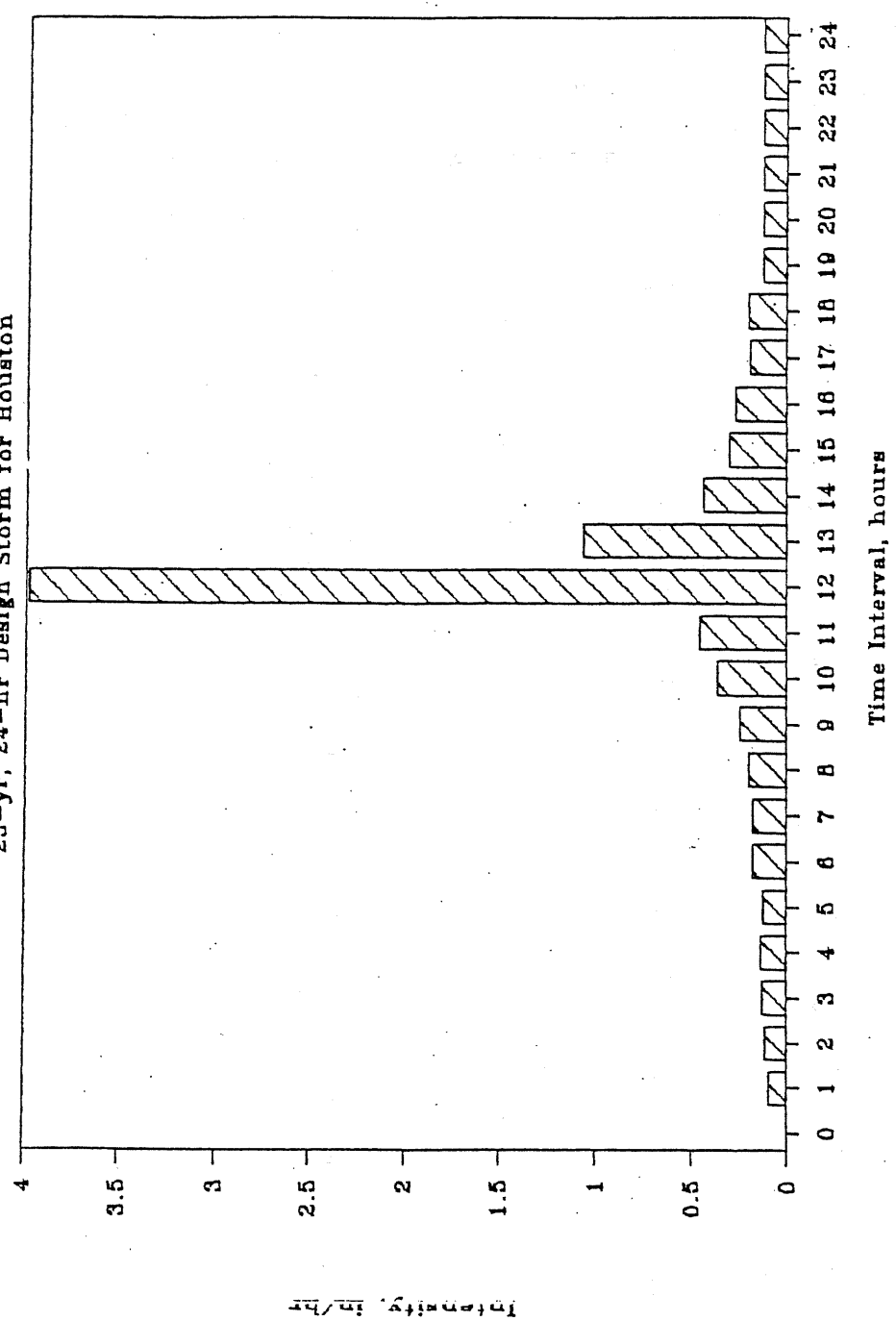
From Fig. 1.8, the 25-yr, 24-hr average intensity is 0.39 in/hr. Thus, the 25-yr, 24-hr depth is  $42 \times 0.39 = 9.36$  inches. The design storm is prepared in the table below:

Total depth = 9.36 inches.

Hour	Percent Total Depth	Cum. Depth in.	Avg. Inten. in/hr
0	0.00	0.00	
1	1.00	0.09	0.09
2	2.20	0.21	0.11
3	3.55	0.33	0.13
4	4.91	0.46	0.13
5	6.20	0.58	0.12
6	8.10	0.76	0.18
7	10.00	0.94	0.18
8	12.10	1.13	0.20
9	14.70	1.38	0.24
10	18.60	1.74	0.37
11	23.50	2.20	0.46
12	66.00	6.18	3.98
13	77.40	7.24	1.07
14	82.10	7.68	0.44
15	85.30	7.98	0.30
16	88.10	8.25	0.26
17	90.10	8.43	0.19
18	92.20	8.63	0.20
19	93.50	8.75	0.12
20	94.80	8.87	0.12
21	96.10	8.99	0.12
22	97.40	9.12	0.12
23	98.70	9.24	0.12
24	100.00	9.36	0.12

6.6 continued...

Problem 6.6. SCS Type II Distribution  
25-yr, 24-hr Design Storm for Houston



6.7. Consider a circular pipe of diameter 1 m and a trapezoidal channel of maximum depth 1 m. The trapezoidal channel has a bottom width of 1 m and side slopes (vertical/horizontal) of 0.25.

- For a Manning roughness of 0.020 and slope of 0.008 for each channel, calculate the water velocity under uniform flow at depths of 0.25, 0.50 and 0.75 m. (Note: Appendix E contains a program for uniform flow computations.)
- Calculate the wave speeds (in downstream direction) in each channel for depths of 0.25, 0.50, and 0.75 m.
- For each wave speed and each channel, calculate the travel time over a length of 300 m.

Will have to calculate the cross-sectional area, hydraulic radius and surface width for all shapes.

For this problem, all units are in meters and/or seconds.

Circular pipe:

Theta = (radians) = subtended angle (> pi for > half full).

r = radius

d = depth

w = surface width

A = area

P = wetted perimeter

R = hydraulic radius

Theta =  $2 \cdot \arccos[(r-d)/r]$

w =  $2r \sin(\text{Theta}/2)$

A =  $r^2/2 [\text{Theta} - \sin(\text{Theta})]$

P = r Theta

R = A/P

V =  $1/n R^{2/3} S^{1/2}$  (Use 1/n for metric)

C =  $V + \sqrt{g A/w}$

r      0.5  
n      0.02  
S      0.008  
g      9.8

d	Theta	w	A	P	R	V	C
0.25	2.094395	0.866025	0.153546	1.047197	0.146625	1.243527	2.561684
0.5	3.141592	1	0.392699	1.570796	0.25	1.774768	3.736515
0.75	4.188790	0.866025	0.631851	2.094395	0.301687	2.011651	4.685614

Trapezoidal channel:

6.7 cont'd

$b$  = bottom width

$ss$  = side slope

$w = b + 2 d/ss$

$A = (b+w)d/2$

$P = b + 2 \{d \sqrt{1+(1/ss)^2}\}$

$b$  1  
 $ss$  0.25  
 $fact$  4.123105

$d$	$w$	$A$	$P$	$R$	$V$	$C$
0.25	3	0.5 3.061552	0.163315	1.336186	2.614206	
0.5	5	1.5 5.123105	0.292791	1.971909	3.686552	
0.75	7	3 7.184658	0.417556	2.498379	4.547769	

Travel times:

$L$  = channel length

$t = L/C$

$L$  300

$d$	Circular		Trapezoidal	
	$C$	$t$	$C$	$t$
0.25	2.561684	117.1104	2.614206	114.7575
0.5	3.736515	80.28871	3.686552	81.37684
0.75	4.685614	64.02575	4.547769	65.96640

5.68 ac

- 6.8. A planned 5.43-ac subdivision is sketched in Fig. P6.8. The soils are generally sandy, and the only runoff will occur from the directly connected (i.e., hydraulically effective) impervious street and driveway surfaces shown in the figure. (Only the  $20 \times 30$  ft portion of each driveway that drains to the street is shown in the figure.) The street is 30 ft wide and the cul-de-sac has a radius of 30 ft. For storm drainage, the plan is to let the stormwater run along the street gutters in lieu of installing a pipe. For purposes of this problem, the entire drainage system can be treated as overland flow. The street slopes from an elevation of 165 ft at the center of the cul-de-sac to 160 ft at the entrance to the subdivision and has a Manning roughness of 0.016.

Drainage regulations specify a 5-yr return period design. Local IDF curves can be approximated functionally as

$$i = \frac{a}{b + t_r}$$

where

$i$  = rainfall intensity (in./hr).

$t_r$  = duration (min).

$a, b$  = constants for different return periods.

For this hypothetical location,  $a = 160$  and  $b = 18$  for a return period of 5 yr.

6.8 cont'd

- Estimate the directly connected impervious area (ac).
- Estimate the maximum drainage length along the directly connected impervious area (ft).
- Determine the kinematic wave parameters  $\alpha$  and  $m$ .
- Estimate the 5-yr peak flow at the outlet. Assume that the impervious surface experiences no losses.

a. Area = rectangular street surface + "most" of circular cul de sac + area of 11 driveways.

$$A = 400 \times 30 + \pi r^2 + 11 \times (20 \times 30) = 21400 \text{ sq ft} = 0.492 \text{ ac.}$$

b. From diagram,  $L$  approximately = 465 ft.  
(From outlet to end of top driveway.)

$$c. \text{Alpha} = 1.49 S^{1/2} / n$$

$$S \text{ approximately} = (165 - 160) / 420 \text{ ft} = 5/420 = 0.0119.$$

$$\text{Alpha} = 1.49 \sqrt{0.0119} / 0.016 = 10.16 \text{ ft}^{1/3} / \text{sec}$$

$m = 5/3$  for turbulent flow with Manning equation.

d. Must iterate with time of concentration and IDF curve.

$$t_c = (L / [\text{Alpha } i^{(m-1)}])^{(1/m)} \quad \text{Use consistent units!}$$

alpha	10.16
m	1.666666
L	465
Area	21400
a	160
b	18

Calculations on Lotus spreadsheet.

Trial	IDF	IDF	Calc	Calc	
tr	i	i	t <sub>c</sub>	t <sub>c</sub>	Q <sub>p</sub>
(min)	(in/hr)	(ft/sec)	(sec)	(min)	(cfs)
10	5.714	0.000132	353.0	5.883	2.831
5	6.957	0.000161	326.3	5.438	3.446
3.45	6.823	0.000157	328.8	5.480	3.380
3.5	6.809	0.000157	329.1	5.485	3.373
3.483	6.813	0.000157	329.0	5.483	3.375

More than close enough agreement between tr and t<sub>c</sub>.

6.9. The subdivision of Problem 6.8 drains to the upstream end of a circular pipe that has an  $n$  value of 0.013 and a slope of 0.005 ft/ft. Determine the size of pipe needed to carry the flow from the 5-yr storm. (Note: Standard diameters in the United States start at 12 in. and increase in 6-in. increments to 60 in., then continue to increase in 12-in. increments.)

Flow = 3.375 cfs from problem 6.8.  
Just try various sizes at full-flow.

$$Q = (1.49/n) A R^{2/3} S^{1/2}$$

$$\begin{array}{ll} S & 0.005 \\ n & 0.013 \end{array}$$

D = Diameter

D (ft)	R (ft)	A (sq ft)	Q (cfs)
1	0.25	0.785397	2.526062
1.5	0.375	1.767144	7.447676
2	0.5	3.14159	16.03949

An 18-inch pipe will more than suffice.

6.10. A 14.7-ac multifamily residential catchment in Miami has a total impervious area of 10.4 ac, but it has a hydraulically effective impervious area of only 6.48 ac. The pervious portions of the basin consist of lawns over a Perrine marl, with a very slow infiltration rate. Rainfall and runoff data monitored by the USGS are reported below for 16 storm events. (Data from Hardee et al., 1979.)

EVENT	RAINFALL (in.)	RUNOFF (in.)
1	2.85	1.983
2	1.17	0.657
3	2.08	1.426
4	1.86	1.176
5	1.67	0.668
6	0.53	0.217
7	0.84	0.541
8	1.50	0.900
9	0.70	0.308
10	0.73	0.277
11	2.02	0.712
12	1.56	0.423
13	0.74	0.330
14	0.75	0.238
15	0.61	0.266
16	1.01	0.444

- Determine a linear relationship between runoff and rainfall using linear regression analysis (least squares). Test the significance of the regression and plot the data points and the fitted line.
- What is the value of depression storage for this basin?

Problem 6.10. Solution:

a. Lotus regression results:  
(Regress runoff vs. rainfall.)

Regression Output:

Constant -0.20433  
Std Err of Y Est 0.214283  
R Squared 0.825776  
No. of Observations 16  
Degrees of Freedom 14

X Coefficient(s) 0.670966  
Std Err of Coef. 0.082368

$$\text{Runoff} = -0.20433 + 0.670966 \times \text{Rainfall}$$

The regression is highly significant, since the x-coef. is over 8 standard deviations greater than zero.

See plot for data points and fitted line.

Event	Rainfall (in)	Runoff (in)	Fitted Line (in)	Runoff Coef.	Fitted w. zero Intercept (in)
1	2.85	1.983	1.708	0.696	
2	1.17	0.657	0.581	0.562	
3	2.08	1.426	1.191	0.686	
4	1.86	1.176	1.044	0.632	
5	1.67	0.668	0.916	0.400	
6	0.53	0.217	0.151	0.409	
7	0.84	0.541	0.359	0.644	
8	1.50	0.900	0.802	0.600	
9	0.70	0.308	0.265	0.440	
10	0.73	0.277	0.285	0.379	
11	2.02	0.712	1.151	0.352	
12	1.56	0.423	0.842	0.271	
13	0.74	0.330	0.292	0.446	
14	0.75	0.238	0.299	0.317	
15	0.61	0.266	0.205	0.436	
16	1.01	0.444	0.473	0.440	
	0.305		0.000		
	3		1.809		1.638
	0				0

Sums: 20.62 10.566 0.481917 = average runoff coef.

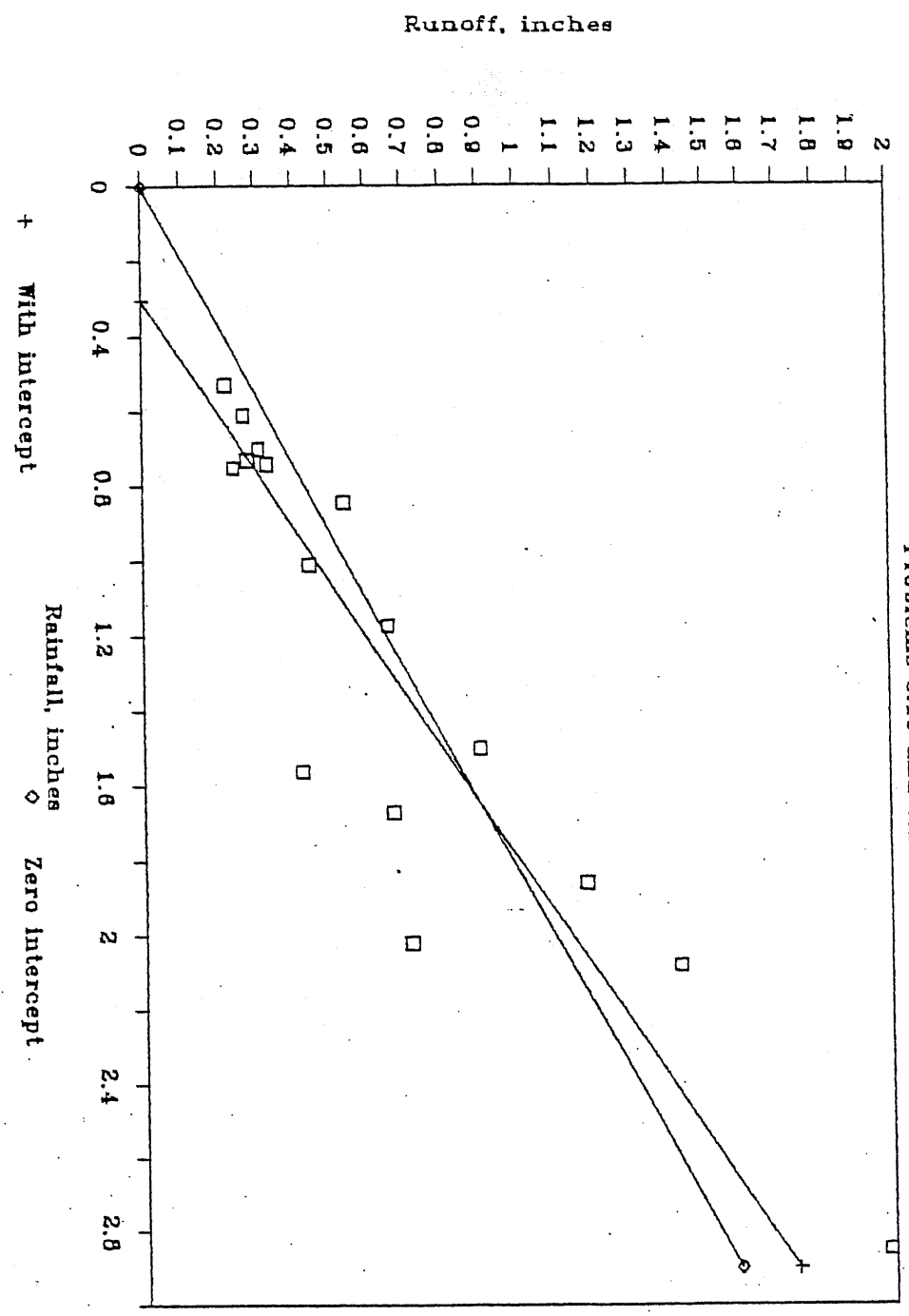
b. Put equation in form  $\text{Runoff} = \text{coef} \times (\text{Rainfall} - \text{DS})$

$$\text{DS} = \text{Rainfall at zero runoff} = .20433 / .670966 = 0.304536 \text{ inches.}$$

This value is clearly seen as the intercept along the rainfall axis.

# Miami Multifamily Residential Catchment

Problems 6.10 and 6.11





- 6.11. Using the data of Problem 6.10, determine the average runoff coefficient by
- computing the runoff coefficient for each storm and finding the average;
  - dividing the total runoff for all storms by the total rainfall for all storms;
  - finding the slope of a runoff vs. rainfall regression line that is "forced" through the origin.

Discuss the computed runoff coefficients in relation to the values of imperviousness and hydraulically effective imperviousness.

Problem 6.11. Solution:

See table of runoff coefficients from problem 6.10.

a. Method 1. Average runoff coefficient = 0.482.

b. Method 2. Average runoff coefficient =  $10.566/20.62 =$

0.512

c. Method 3: Perform regression with zero intercept.

Regression Output:

Constant	0
Std Err of Y Est	0.227807
R Squared	0.789025
No. of Observations	16
Degrees of Freedom	15

X Coefficient(s) 0.544599

Std Err of Coef. 0.039452

Runoff coefficient = slope = 0.546.

This line is also shown in the figure for problem 6.10.

Discussion:

Fraction imperviousness =  $10.4/14.7 =$  0.707482

Fraction hydraul. eff. imperviousness =  $6.48/14.7 =$  0.440816

Runoff coefficient estimates range from 0.48 to 0.54, which are between the two percent imperviousness values. This indicates that the pervious area usually contributes some runoff.

- 6.12. A detention pond has the shape of an inverted truncated pyramid, shown in Fig. P6.12(a). It has a rectangular bottom of dimension  $120 \times 80$  ft, a maximum depth of 5 ft, and uniform side slopes of 3:1 (horizontal:vertical). Hence the dimensions at 5-ft depth are also rectangular, with length 150 ft and width 110 ft. The outlet from the basin behaves as an orifice, with a diameter of 1 ft and a discharge coefficient of 0.9. The opening of the orifice (a pipe draining from the center of the basin) is effectively at a depth of zero (i.e., at the bottom of the pond). (The pond floor would typically slope toward the outlet, but this slope will be ignored in this problem. In addition, the orifice will be assumed to follow its theoretical behavior even at very small depths.)
- What is the total volume of the pond, in  $\text{ft}^3$  and ac-ft?
  - Develop the depth vs. surface area and depth vs. volume curves for the pond. Use 1-ft intervals. Tabulate and plot.
  - A triangular inflow hydrograph is shown in Fig. P6.12(b). Route it through the detention pond. The storage indication method (Puls method) of Section 4.3 is recommended, with a time step of 10 min. Plot the outflow hydrograph on the same graph as the inflow hydrograph. (Note: Appendix E contains a computer program that will perform storage indication routing.)

Problem 6.12. Solution:

a. Volume of truncated pyramid = volume of prismoid. Simpson's rule will give exact answer in this case.

Surface area =  $150 \times 110 = 16500$  square feet  
 Midpoint area =  $135 \times 95 = 12825$   
 Bottom area =  $120 \times 80 = 9600$

Vol =  $\text{Depth} / (2 \times 3) \times (\text{Asurf} + 4 \text{ Amid} + \text{Abot}) = 64500$  cubic feet  
 Acre-feet = cubic feet / 43560 = 1.48 acre-feet

b. Compute areas at each depth and integrate using trapezoidal rule. Include outflow calculation for orifice for use in Puls tables.

Slope 1/3  
 Invslope 3  
 Cd 0.9  
 Dia 1 ft  
 Ao 0.785397 sq. ft  
 dt 600 sec

Depth (ft)	Length (ft)	Width (ft)	Area (ft <sup>2</sup> )	Volume (ft <sup>3</sup> )	Outflow (cfs)	Puls: 2V/dt+Q (cfs)
0	120	80	9600	0	0	0
1	126	86	10836	10218	5.672505	39.73250
2	132	92	12144	21708	8.022134	80.38213
3	138	98	13524	34542	9.825068	124.9650
4	144	104	14976	48772	11.34501	173.9850
5	150	110	16500	64500	12.68410	227.7841

(Close agreement for volume between two integration methods.)

c. Put Puls values in another table for interpolation using Lotus.  
(Lotus needs the slopes to perform linear interpolation using lookup function.)

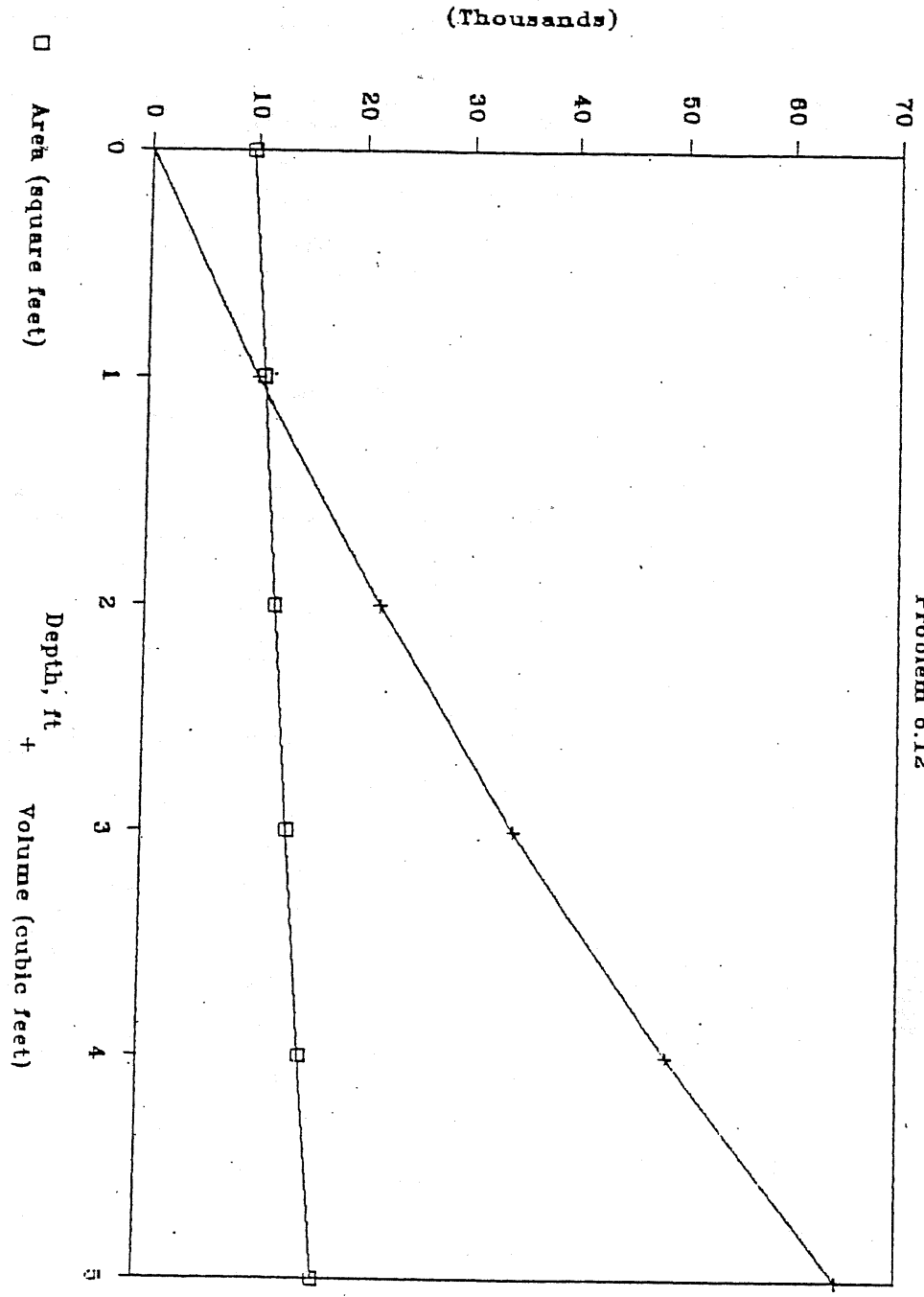
Puls: 2V/dt+0 (cfs)	Depth (ft)	Slope: D/Puls	Qo (cfs)	Slope: Qo/Puls	V (ft <sup>3</sup> )	Slope: V/Puls
0	0	0.025168	0	0.142767	0	257.1497
39.73250	1	0.024600	5.672505	0.057801	10218	282.6594
80.38213	2	0.022430	8.022134	0.040439	21708	287.8680
124.9650	3	0.020399	9.825068	0.031006	34542	290.6980
173.9850	4	0.018587	11.34501	0.024890	48792	292.5327
227.7841	5		12.68410		64530	

#### Hydrographs:

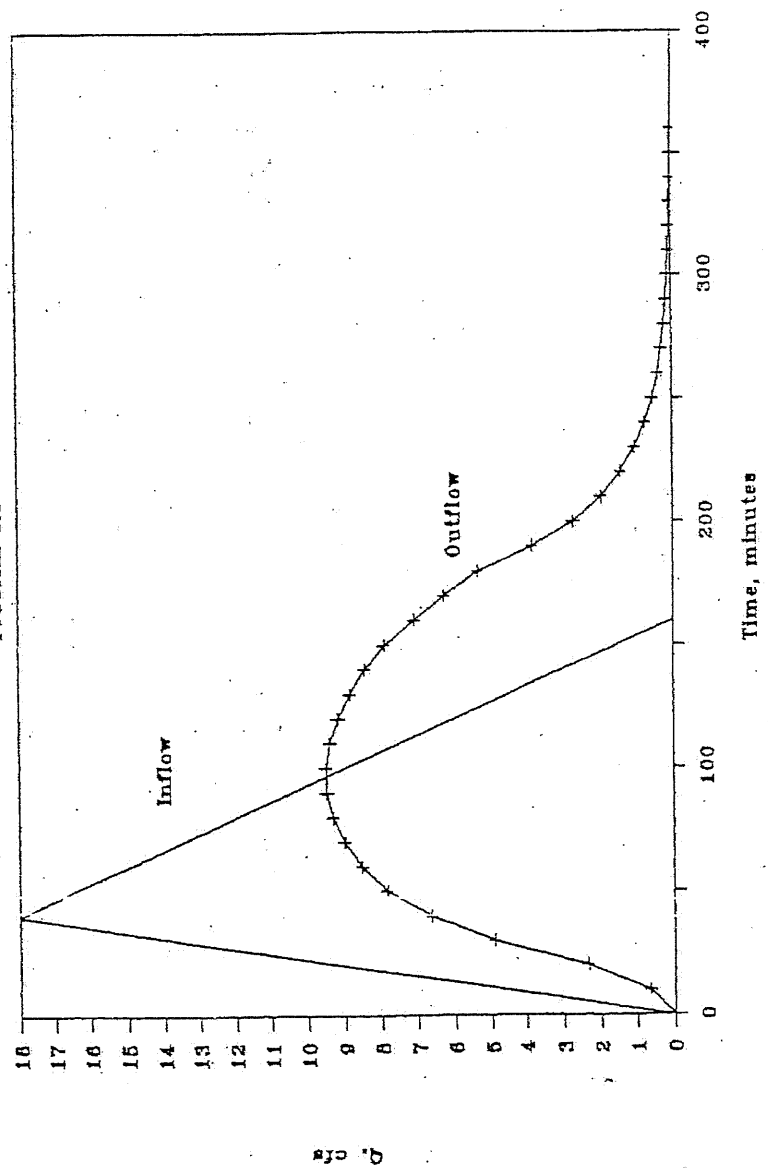
t (min)	t (sec)	Qin (cfs)	Puls: LHS (cfs)	Depth (ft)	Qout (cfs)	V (ft <sup>3</sup> )
0	0	0	0.000	0.000	0.00	0.0
10	600	4.5	4.500	0.113	0.64	1157.3
20	1200	9	16.715	0.421	2.39	4298.6
30	1800	13.5	34.442	0.867	4.92	8857.5
40	2400	18	56.108	1.403	6.62	14846.6
50	3000	16.5	77.370	1.926	7.85	20856.5
60	3600	15	93.174	2.287	9.54	25390.3
70	4200	13.5	104.595	2.543	9.00	28678.1
80	4800	12	112.092	2.711	9.30	30836.3
90	5400	10.5	115.983	2.799	9.44	31956.4
100	6000	9	116.560	2.811	9.49	32122.3
110	6600	7.5	114.089	2.756	9.39	31411.2
120	7200	6	108.819	2.638	9.17	29894.0
130	7800	4.5	100.975	2.462	8.85	27635.9
140	8400	3	90.765	2.233	8.44	24696.8
150	9000	1.5	78.381	1.951	7.91	21142.3
160	9600	0	64.068	1.599	7.08	17096.6
170	10200	0	49.910	1.250	6.26	13094.7
180	10800	0	37.388	0.941	5.34	9615.1
190	11400	0	26.712	0.672	3.81	6869.6
200	12000	0	19.085	0.480	2.72	4908.1
210	12600	0	13.636	0.343	1.95	3506.7
220	13200	0	9.742	0.245	1.39	2505.4
230	13800	0	6.960	0.175	0.99	1790.0
240	14400	0	4.973	0.125	0.71	1278.9
250	15000	0	3.553	0.089	0.51	913.7
260	15600	0	2.539	0.064	0.36	652.8
270	16200	0	1.814	0.046	0.26	466.4
280	16800	0	1.296	0.033	0.19	333.2
290	17400	0	0.926	0.023	0.13	238.1
300	18000	0	0.661	0.017	0.09	170.1
310	18600	0	0.473	0.012	0.07	121.5
320	19200	0	0.338	0.008	0.05	86.8
330	19800	0	0.241	0.006	0.03	62.0
340	20400	0	0.172	0.004	0.02	44.3
350	21000	0	0.123	0.003	0.02	31.7
360	21600	0	0.088	0.002	0.01	22.6

# Depth--Area--Volume Relationships

Problem 6.12



Puls Routing Through Detention Pond  
Problem 6.12



**6.13** Use the USGS regression equation (3-82) to compute the 5-yr peak flow for a 4-ac Tallahassee catchment that is 95% impervious. Compare your answer with the value computed in Example 6.6. The reason for the large discrepancy is that the regression equations were developed for much larger catchments (the smallest being  $0.21 \text{ mi}^2$ )

**ANSWER:**

$$Q_p(T) = C_p A^a I A^b$$

$Q_p$  is in cfs

A = Total area in square miles

IA = Percent Impervious area

For T = 5 yrs from Table 3.9

$$C_p = 24.5$$

$$a = 0.77$$

$$b = 0.943$$

$$Q_p = 24.5 * \left(\frac{4}{640}\right)^{.77} * 95^{.943} = 36.1 \text{ cfs}$$

Value for example is 24.7 cfs

6.14. The time-area-concentration curve for an urban basin is given below. Also provided is a table giving the rainfall hyetograph and losses.

- What is the total area of the catchment?
- What is the time of concentration of the overall catchment?
- Compute and plot the hyetograph of rainfall excess as a bar chart.
- Plot the time-area-concentration curve as a bar chart.
- Use the time-area method to compute the runoff hydrograph, without any additional attenuation.
- Provide additional attenuation by routing the runoff hydrograph through a linear reservoir with time constant  $K = 20$  min. Use the Muskingum method (Chapter 4) to perform the routing, with routing parameter  $x = 0$ . When a time-area hydrograph is routed through a linear reservoir, this is known as a Clark model (1945).
- Repeat part f for a time constant  $K = 7.5$  min. Plot the inflow hydrograph from part e and the attenuated hydrographs from parts f and g on the same chart. Note the difference in attenuation for the two linear reservoirs.

a.) Time-area-concentration histogram is:

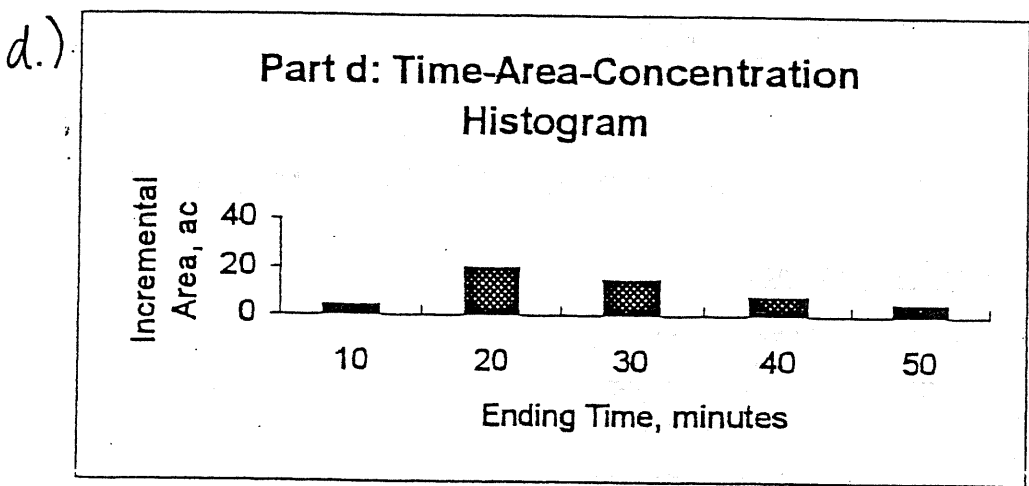
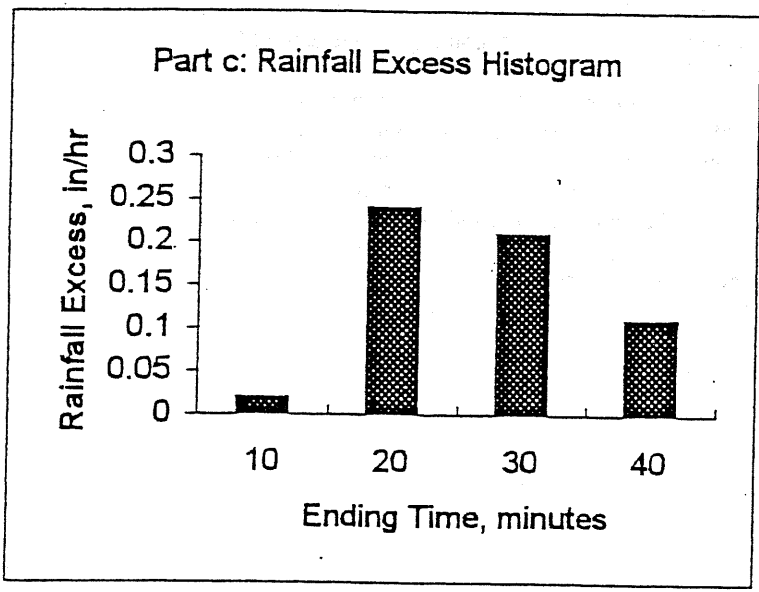
Time	Increm. Area
min	ac
10	4
20	20
30	15
40	8
50	5
	52 Sum = total area (Part a)

b.) Overall time of concentration = largest travel time = 50 min

c.) Rain (constant to indicated time):

Time	Intensity	Losses	Rainfall Excess
min	in/hr	in/hr	in/hr
10	0.10	0.08	0.02
20	0.30	0.06	0.24
30	0.25	0.04	0.21
40	0.15	0.04	0.11

6.14 continued...





6.14 continued...

e) Runoff Computation by Time-Area Method:  
Note: units use approximation 1 ac-in/hr = 1 cfs

Time = Rainfall Excess (in/hr)						Sum
Ending	A = Area (ac)					i*A
min						cfs
0						0.00
10	i: 0.02	0.24	0.21	0.11		0.08
	A: 4					
20	i: 0.02	0.24	0.21	0.11		1.36
	A: 20	4				
30	i: 0.02	0.24	0.21	0.11		5.94
	A: 15	20	4			
40	i: 0.02	0.24	0.21	0.11		8.40
	A: 8	15	20	4		
50	i: 0.02	0.24	0.21	0.11		7.37
	A: 5	8	15	20		
60	i: 0.02	0.24	0.21	0.11		4.53
	A: 5	5	8	15		
70	i: 0.02	0.24	0.21	0.11		1.93
	A: 5		5	8		
80	i: 0.02	0.24	0.21	0.11		0.55
	A: 5			5		
90						0.00

f & g)

Linear Reservoir Routing Option 1

K	20 min
DT	10 min
Muskingum Coefs:	
Denom	25 min
ccc0	0.2
ccc1	0.2
ccc2	0.6

g Option 2

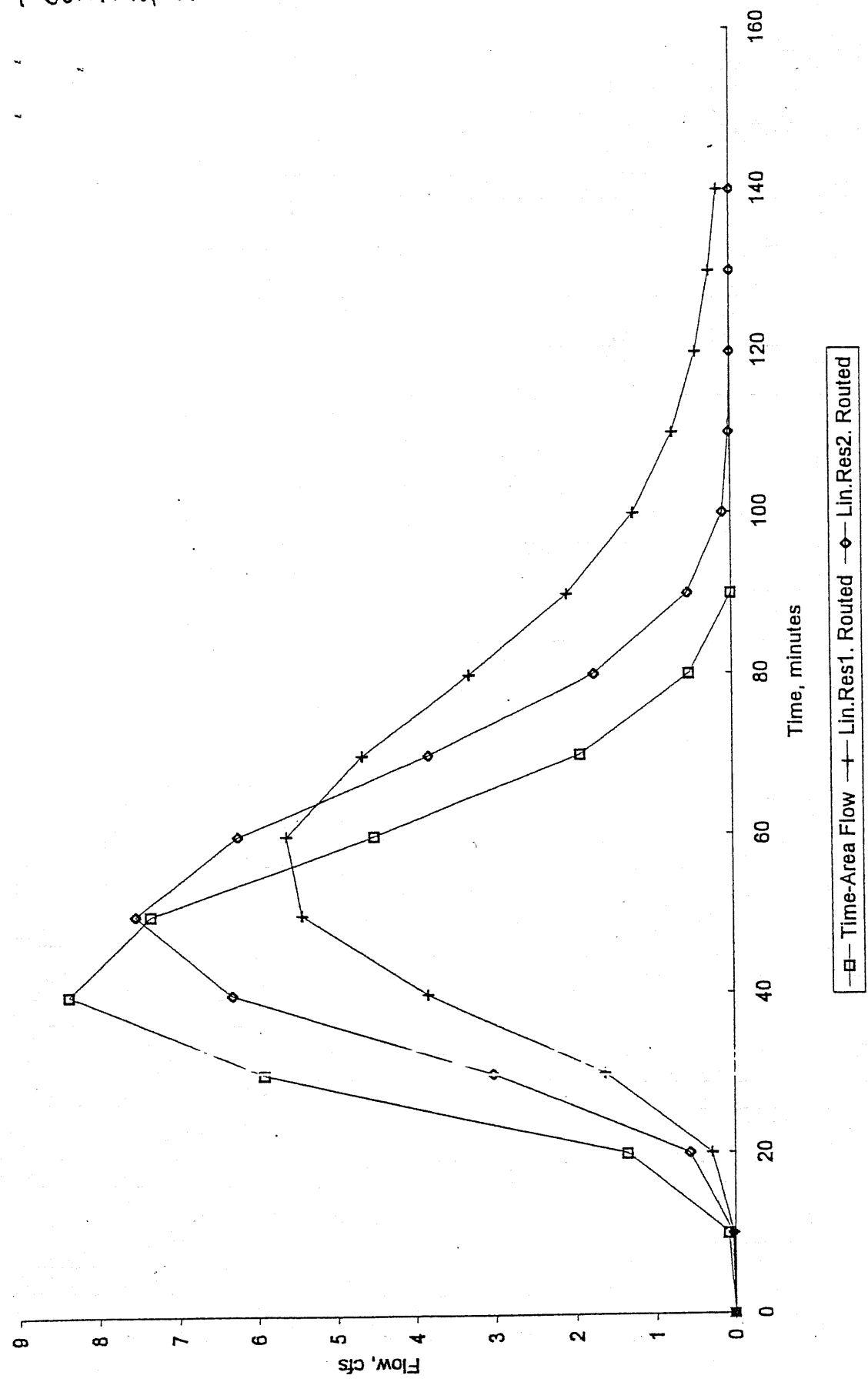
K	7.5 min
DT	10 min
Muskingum Coefs:	
Denom	12.5 min
ccc0	0.4
ccc1	0.4
ccc2	0.2

	Time-area	Reservoir1	Reservoir2
Time	Flow	Routed	Routed
min	cfs	Flow	Flow
		cfs	cfs
0	0	0	0
10	0.080	0.016	0.032
20	1.360	0.298	0.582
30	5.940	1.639	3.036
40	8.400	3.851	6.343
50	7.370	5.465	7.577
60	4.530	5.659	6.275
70	1.930	4.687	3.839
80	0.550	3.308	1.760
90	0	2.095	0.572
100	0	1.257	0.114
110	0	0.754	0.023
120	0	0.453	0.005
130	0	0.272	0.001
140	0	0.163	0.000

6-25

6.14 continued...

Time-area Example



6-26

- 6.15. A catchment is to be simulated using the Clark model, that is, by routing using a time-area method (to produce hydrograph time delays), followed by routing through a linear reservoir (to produce hydrograph attenuation). The time-area and rainfall-excess data are given below:

TIME, min	AREA, ac	TIME, min	RAINFALL EXCESS, in./hr
0-30	8	0-30	0.32
30-60	42	30-60	0.22
60-90	30	60-90	0.27
90-120	11	90-120	0.11
120-150	19		

- What is the total area of the catchment?
- What is the time of concentration of the overall catchment?
- Perform the indicated time-area routing.
- The linear reservoir is to be designed (conceptually) such that the peak flow out of the reservoir is only 60% ( $\pm 0.5$  cfs) of the inflow peak. Using the Muskingum routing method with  $x = 0$ , experiment with K-values to achieve this result.
- Tabulate and plot (on the same chart) the time-area hydrograph ("inflow") and the outflow hydrograph from the linear reservoir identified in part d.

a) Sum = total area = 110

b: Time of concentration = maximum travel time = 150 min.

Part c:

Time min	Area ac	Rainx in/hr	R1* cfs	R2* cfs	R3* cfs	R4* cfs	Sum cfs
0							0.00
30	8	0.32	2.56				2.56
60	42	0.22	13.44	1.76			15.20
90	30	0.27	9.6	9.24	2.16		21.00
120	11	0.11	3.52	6.6	11.34	0.88	22.34
150	19		6.08	2.42	8.1	4.62	21.22
180				4.18	2.97	3.3	10.45
210					5.13	1.21	6.34
240						2.09	2.09
270							0.00

6.15 continued...

Part d: Find K for linear reservoir such that

new peak =  $0.6 \times \text{old peak}$ ,  $\pm 0.5$  cfs

Old peak = 22.34 cfs

Want new peak = 13.40 or between 12.90 and

13.90 cfs

Experiment with K values to get this.

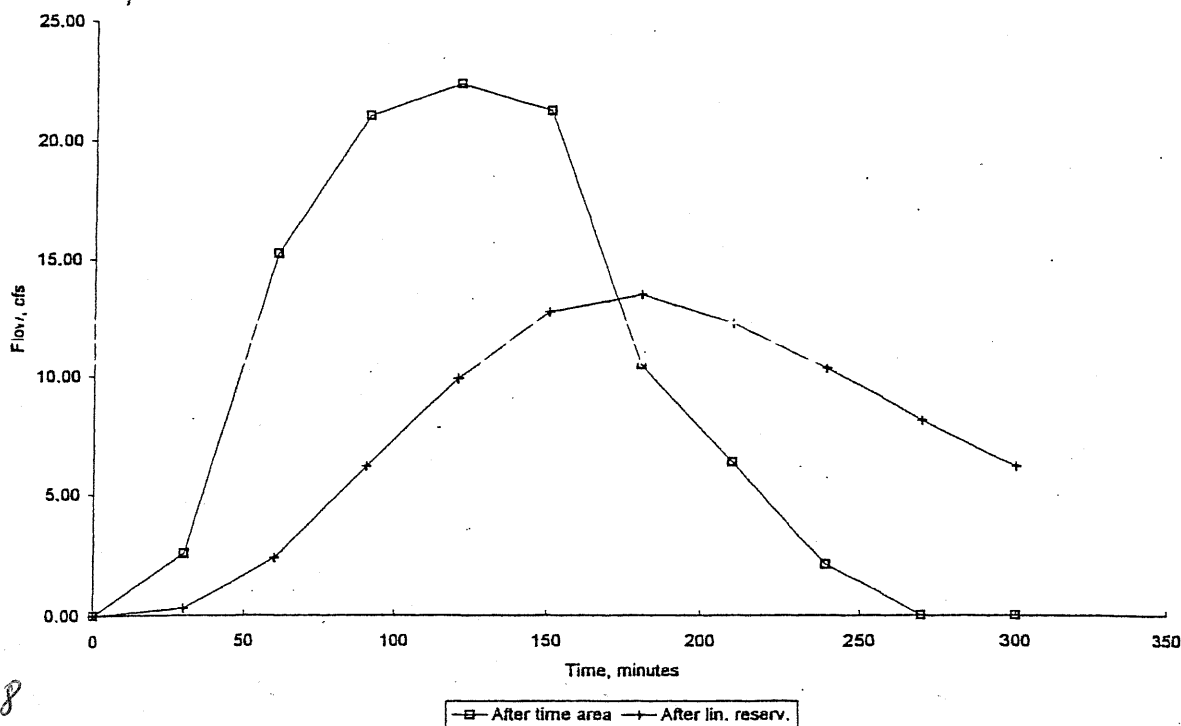
Linear reservoir, K = 45 min

Use Muskingum routing with  $x = 0$ .

	Trial 1	Trial 2	Trial 3	Trial 4
K (min)	45	60	90	110
Dt (min)	30	30	30	30
ccc0	0.25	0.2	0.1429	0.1200
ccc1	0.25	0.2	0.1429	0.1200
ccc2	0.5	0.6	0.7143	0.7600
sum	1	1	1	1

Time min	Inflow cfs	Outflow cfs	Outflow cfs	Outflow cfs	Outflow cfs
0	0.00	0	0	0	0
30	2.56	0.64	0.51	0.37	0.31
60	15.20	4.76	3.86	2.80	2.36
90	21.00	11.43	9.56	7.17	6.14
120	22.34	16.55	14.40	11.31	9.87
150	21.22	19.17	17.35	14.30	12.73
180	10.45	17.50	16.75	14.74	13.47
210	6.34	12.95	13.41	12.93	12.25
240	2.09	8.58	9.73	10.44	10.32
270	0.00	4.81	6.26	7.75	8.10
300	0.00	2.41	3.75	5.54	6.15

Hydrograph from Time-Area Procedure



## 6.16 Compare and contrast the features of SWMM model and HEC-HMS (See Chapter 5)

### ANSWER:

The HEC-HMS software, released by the U.S. Army Corps of Engineers in 1992, is a lumped precipitation-runoff model that can be applied to various geographic terrains to simulate hydrographs from watersheds. HEC-HMS can be coupled with other software for a multitude of applications including floodplain delineations, urbanization studies, reservoir design, and water availability analysis. In terms of floodplain modeling, HEC-HMS is the hydrologic model that simulates flow hydrographs to be used as inputs in other hydraulic models, such as HEC-RAS.

The EPA's Storm Water Management Model (SWMM), first developed in 1971, has been widely used for planning, analysis and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas. One particular version of SWMM, the XP-SWMM (eXPert Stormwater and Wastewater Management Model), is a software package used for dynamic modeling of storm water systems, sanitary systems, and river systems. The program was created by XP Software, but uses the EPA SWMM model as the primary runoff hydrograph simulation method to simulate runoff quantity and quality. The model includes subcatchment areas for runoff modeling and channels and pipes for routing. The XP-SWMM software uses the capabilities of SWMM and combines it with a user friendly interface and the ability to link the 1D SWMM model to a 2D overland flow model. The major advantage of XP-SWMM is the software's capability of combining a 1D river hydraulic model with a 2D rainfall-runoff model to generate floodplain maps. XP-SWMM is approved by FEMA for use in hydraulic and hydrologic applications such as floodplain mapping. The xp2D module available for XP-SWMM allows the user to utilize the 1D river modeling capability of the XP-SWMM software with a 2D TUFLOW-based overland flow model.

6.17. Monitoring data for an Oregon catchment produce the following record of annual precipitation and runoff:

- Determine a linear relationship between runoff and precipitation using linear regression analysis (least squares). Test the significance of the regression and plot the data points and the fitted line.
- Does this catchment exhibit the characteristics of depression storage or of base-flow?
- Do you think this is a large catchment or a smaller one? Assume "large" is an area greater than 100 square miles. Base your assessment on an evaluation of the slope of the fitted line and its units.

a.)		Fitted Line
	10	18.0
	58	1363.2

Check Sum      725    15860

Averages: 42.65    932.94

Regression Output:

Constant	-262.19
Std Err of Y Est	101.08
R Squared	0.73665
No. of Observations	17
Degrees of Freedom	15

X Coefficient(s) 28.0237

Std Err of Coef. 4.3263

r 0.8583

t = x-coef/std er 6.4776 highly significant without  
need even to check table

See plot on separate worksheet

Part b:

Constant is negative ==> negative outflow at zero rain. Thus, line must intersect rainfall axis, and catchment has a depression storage if put in form  $\text{Runoff} = C(\text{Precip} - \text{DS})$ .

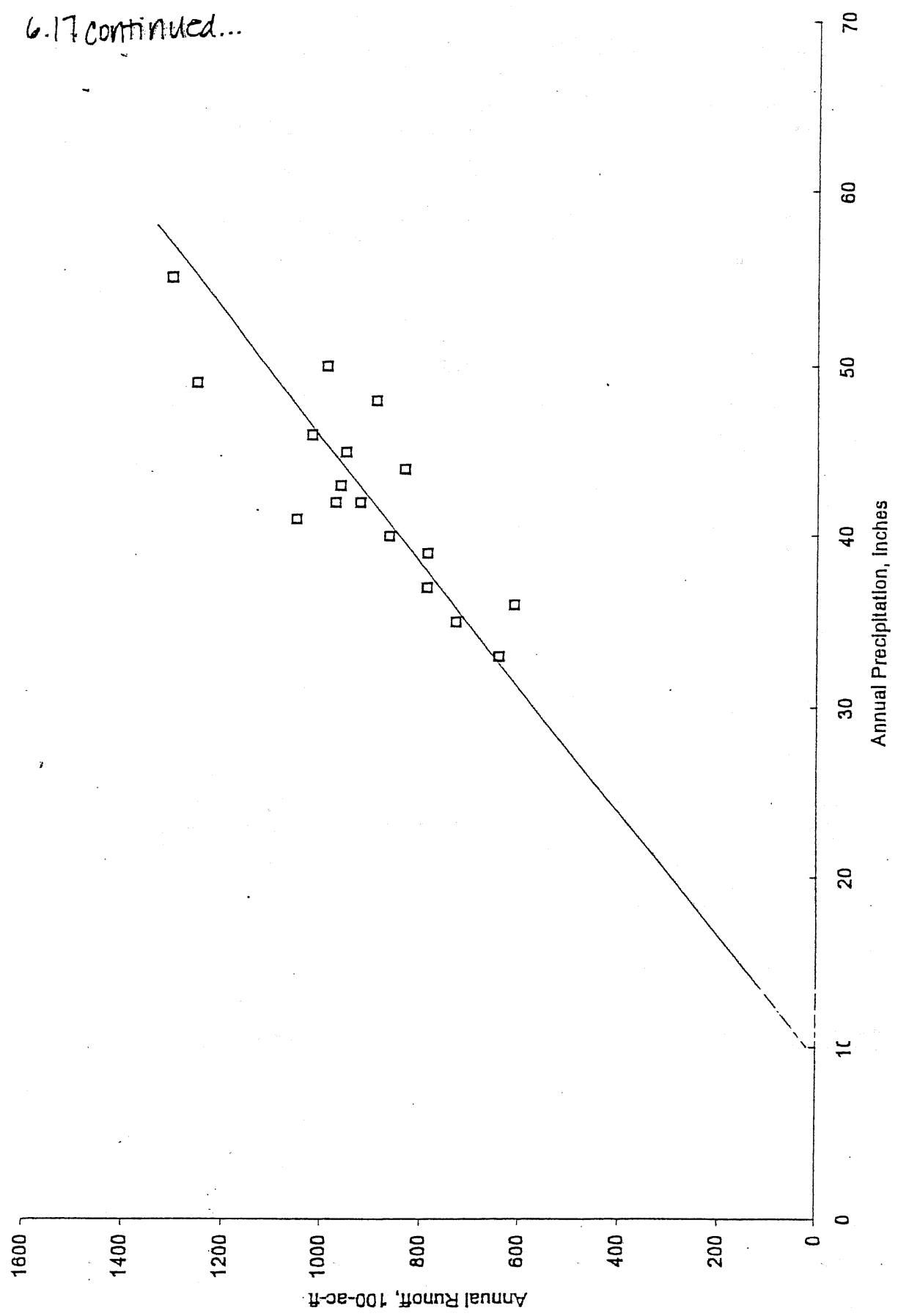
Here, constant = Coef\*DS, so DS = 9.36 in.  
(but not needed for HW answer).

Part c:

X-coefficient = slope = 28.02 1000acft/inch \* 12 \* 1000 =  
336294 acres = 525.445 sq miles

This would be area (Vol = area x depth) for 100% runoff. Real catchment has much lower runoff fraction. Hence, real area >> 525 sq miles.

6.17 continued...



6-31

- 6.18 a) Demonstrate the units conversion computation for the first row in the table (i.e., that 53.8 lb/ac-in. with a runoff of 0.024 in. results in 12.4 lb of SS).
- b) Rainfall is to be considered as an explanatory variable (independent variable) for the prediction of runoff, SS (lb), and SS (lb/ac-in.). Perform the three indicated linear regressions. Test the significance of the regressions at the 95% level ( $\alpha = 5\%$ ). Plot the data points for runoff vs. rainfall on one graph and for both SS values vs. rainfall on one or two other graphs. If the regression is significant, include the predicted straight line.
- Although your software may test the significance automatically, list the "table" T-value that must be exceeded for the regression to be significant. Obtain this value from a statistics book.
- c) What other causative factors (that would vary with each storm) might be included in a multiple linear regression of runoff vs. rainfall (depths)?

Note: This problem illustrates an example of "spurious correlation" for the SS vs. rainfall data. Load is the product of a constant  $\times$  runoff depth  $\times$  concentration. Since runoff is correlated with rainfall, the dependent variable (load) "includes" rainfall as part of its value. Hence, load will always correlate better with rainfall than will concentration.

Part a:  $53.8 \text{ lb/ac-in} = 12.4 \text{ lb} / (9.6 \text{ ac} \times 0.24 \text{ in.})$

Rain, runoff, quality data

Rain in	Runoff in	SS lb	SS lb/ac-in	Pred Runoff	Pred SS lb
0.08	0.024	12.4	53.8		
0.08	0.010	6.4	66.7		
0.31	0.060	6.0	10.4		
0.50	0.133	9.6	7.5		
0.19	0.030	3.9	13.5		
0.23	0.110	5.1	4.8		
0.30	0.090	8.3	9.6		
0.22	0.060	5.7	9.9		
0.11	0.025	21.1	87.9		
0.13	0.029	19.0	68.2		
0.16	0.026	9.7	38.9		
0.18	0.046	17.8	40.3		
0.17	0.041	4.3	10.9		
0.48	0.129	57.4	46.4		
0.10	0.039	22.5	60.1		
0.06	0.010	1.2	12.5		
0.14	0.022	5.0	23.7		
0.54	0.133	55.4	43.4		
0.08	0.037	9.5	26.7		
0.55	0.090	17.1	19.8		
0.23	0.059	22.9	40.4		
0.23	0.042	16.9	41.9		
0.16	0.040	6.5	16.9		
0.35	0.110	25.6	24.2		
0				0.0036	2.9157
0.6				0.1442	35.1010
5.58	1.395	369.3	778.4 = check sums		

T-value from table (for  $n-2 = 22$  and  $\alpha = 5\%$ ) = 2.074,



## 6.18 continued...

### Part b:

#### Runoff vs. Rainfall

##### Regression Output:

Constant	0.003642
Std Err of Y Est	0.018656
R Squared	0.789456
No. of Observations	24
Degrees of Freedom	22

X Coefficient(s)	0.234336	
Std Err of Coef.	0.025801	
t(n-2)	9.082457	t(n-2) > T-table, so significant

#### SS (lb) vs. rainfall

##### Regression Output:

Constant	2.915722
Std Err of Y Est	12.24916
R Squared	0.313082
No. of Observations	24
Degrees of Freedom	22

X Coefficient(s)	53.64206	
Std Err of Coef.	16.94017	
t(n-2)	3.16656	t(n-2) > T-table, so significant

#### SS(lb/ac-in) vs. rainfall

##### Regression Output:

Constant	42.5969
Std Err of Y Est	22.38317
R Squared	0.083114
No. of Observations	24
Degrees of Freedom	22

X Coefficient(s)	-43.7143	
Std Err of Coef.	30.95516	
t(n-2)	-1.41218	abs(t(n-2)) < T-table, so NOT significant

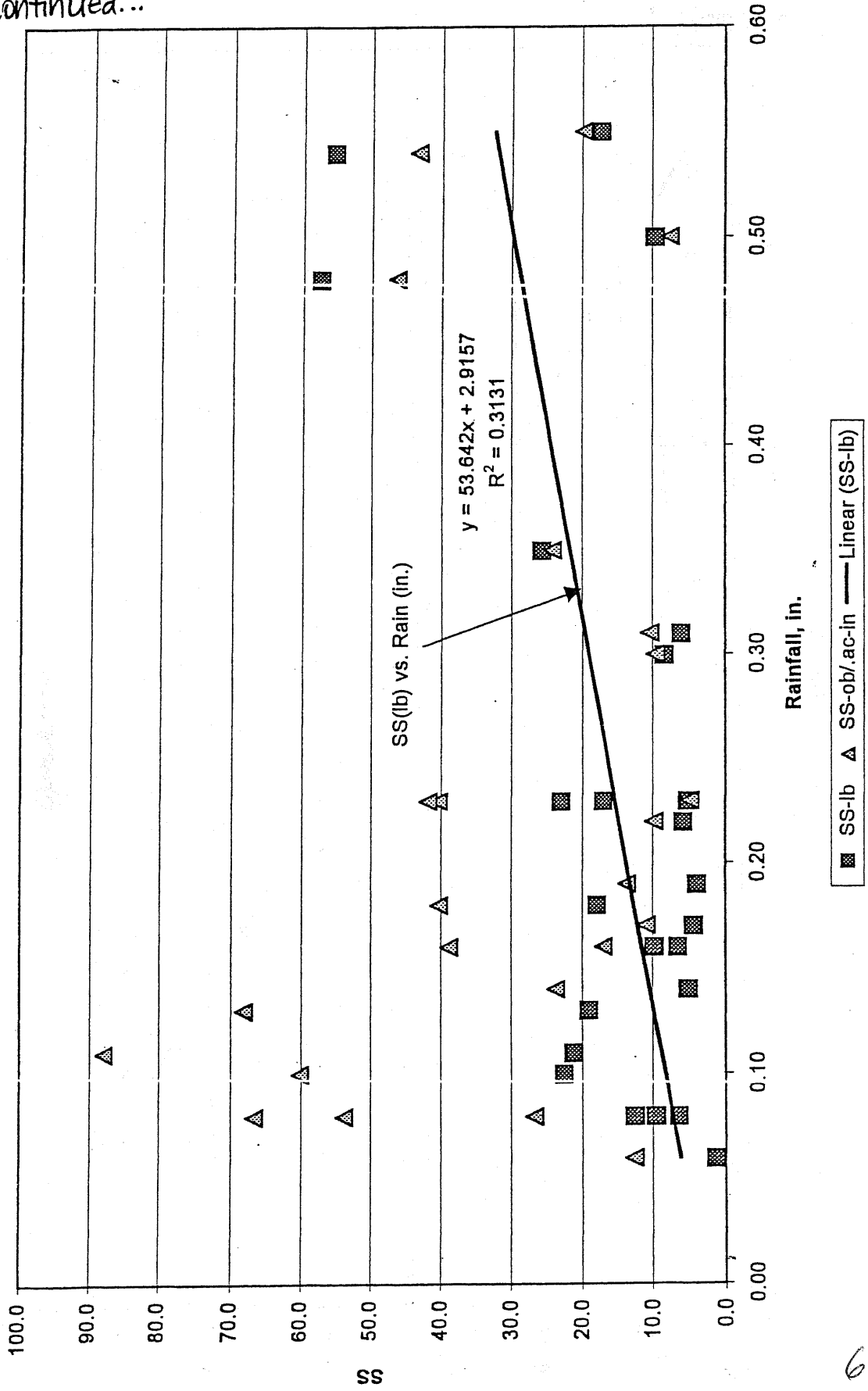
### Part c:

Other possible causative (independent variables) for regression of runoff depth vs. rainfall depth include:

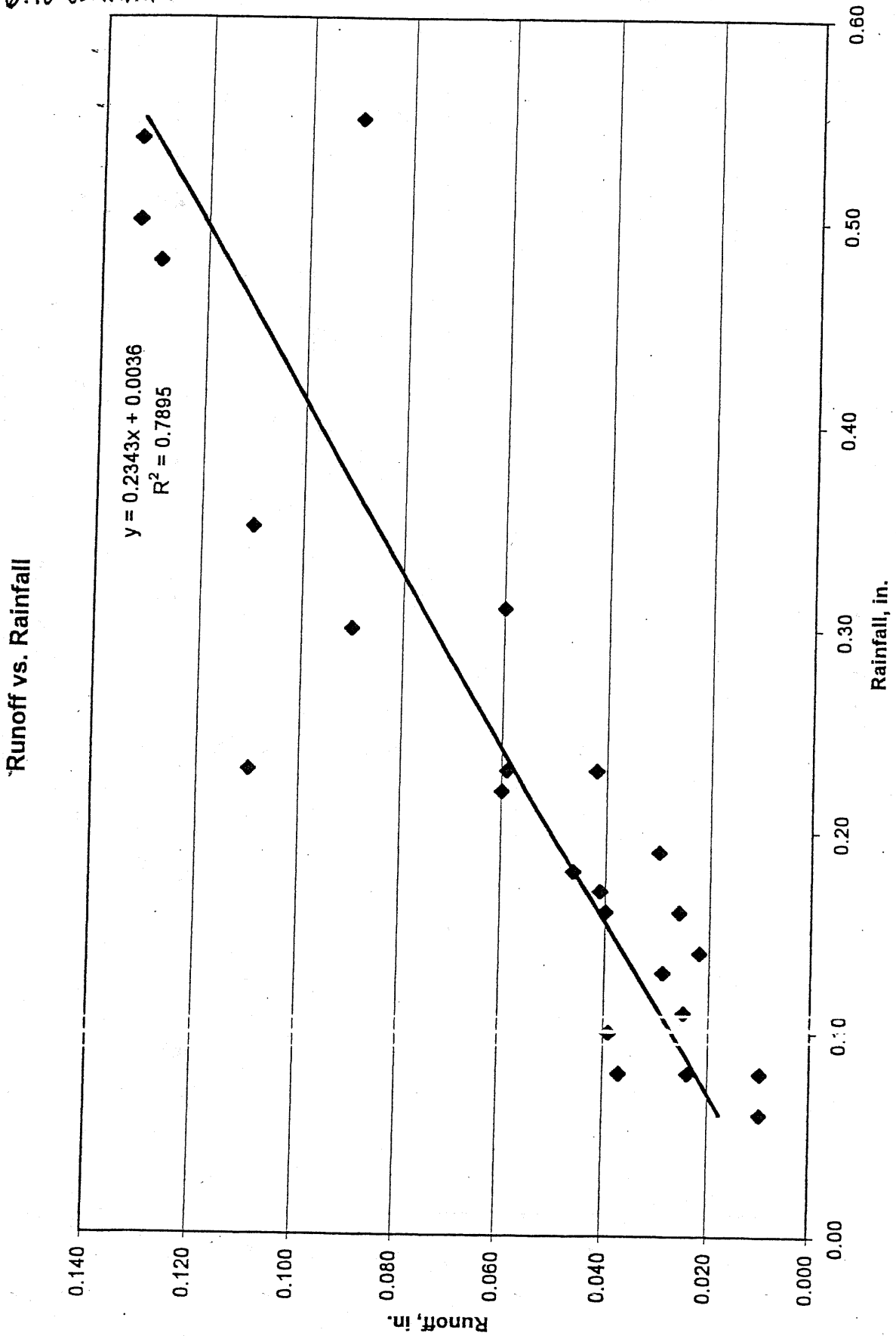
antecedent rainfall	time since last rainfall
temperature	soil moisture
evaporation	etc.

6-18 continued...

SS vs. Rainfall



6.18 continued



6-35

- 6.19. A 6-ac basin is to be developed into 2 ac of commercial development ( $C = 0.95$ ) and 4 ac of park ( $C = 0.2$ ), as sketched in Figure P6.19. Using the tabulated IDF information, what should be the design flow at the inlet?

DURATION, min.	INTENSITY, in./hr
5	10.1
10	8.2
15	7.3
20	6.6
25	6.1
30	5.7

C-comm= 0.95  
C-park= 0.2

A-comm= 2 ac  
A-park= 4 ac  
A-total= 6 ac

C-bar = weighted C by areas = 0.45

Need to check total area and commercial area separately with Rational Method

	Area ac	tc min	i in/hr	C	Q=CiA cfs
Commerc	2	5	10.1	0.95	19.2
Total	6	20	6.6	0.45	17.8

Close, but design for peak flow from commercial area.

6.20 The inner rectangle represents DCIA, consisting of rooftops, streets, sidewalks, driveways, and so on. The dimensions of the inner rectangle are 1000 ft wide by 2500 ft tall. The pervious area behaves like "lawns, sandy soil, average slope." The DCIA has characteristics of streets and roofs. "High-end" values of runoff coefficients should be used.

The inlet time for flow from the far upper end of the catchment (e.g., an upper corner of the pervious area) is 2 hrs. The inlet time just for the DCIA is 20 min.

- What is the 25-yr outflow, by the rational method? Remember to compute runoff for both the total area and just for the DCIA.
- Use the USGS regression equations for Tallahassee to compute the 10-yr peak flow. Observe that this value is close to the rational method estimate for DCIA, from part a.
- A corrugated steel pipe (CSP) culvert ( $n = 0.024$ ) will receive the 25-yr flow from the catchment, using the rational method of part a. If the pipe slope is 1%, what standard U.S. diameter (see problem 6.9) should be used?

**Part a:**

**Rational method:**

$C =$  0.15 lawns, sandy soil, average slope  
 $C =$  0.95 DCIA = directly connected impervious area

$A_{\text{total}} = 5000 \times 3000 \text{ ft}^2 =$  15000000  $\text{ft}^2 =$  344.4 ac

$A_{\text{DCIA}} = 2500 \times 1000 \text{ ft}^2 =$  2500000  $\text{ft}^2 =$  57.4 ac

$A_{\text{pervious}} = \text{difference} =$  287.0 ac

Weighted runoff coefficient, by areas =  $\bar{C}$  = 0.283

25-year intensities for Tallahassee from Fig. 6.5:

Duration	$i$	$Q$
min	in/hr	cfs
20	6.5 $Q = C_{\text{DCIA}} \cdot i \cdot A_{\text{DCIA}} =$	354
120	2.4 $Q = \bar{C} \cdot i \cdot A_{\text{total}} =$	234

Using just DCIA gives considerably higher peak flow.

b. 20 continued...

**Part b:**

Tallahassee regression equation is:  $Q = C_p \cdot A^a \cdot I^b$   
 $IA = \text{percent impervious area} = 57.4/344 = 16.667 \%$   
 $A = \text{area in square miles} = 0.538$

**Coefficients**

T	Cp	a	b	Q, cfs
25	63.2	0.787	0.791	359

**Part c:**

Culvert sizing, use Mannings equation with  $n = 0.024$  for CSP (French, page 125)

$n = 0.024$   
 $Q = 1.49/n \cdot \sqrt{\text{slope}} \cdot A \cdot R^{2/3}$   
 $\text{slope} = 0.01$   
 $1.49/n \cdot \sqrt{\text{slope}} = 6.208$

Hydraulic radius,  $R = D/4$

Try different diameters

D ft	R ft	A ft <sup>2</sup>	Q cfs
7	1.75	38.5	347
8	2	50.3	495

Use diameter = 8 ft

- 6.21. A subdivision in Corvallis, Oregon, is shown in Figure P6.21. Rational method design is performed using the 10-yr return period IDF curve of the Oregon Department of Transportation (1990) for Zone 8, which may be approximated as follows:

$$i = 0.2081 + \frac{29.8438}{t_r} - \frac{184.51}{t_r^2} + \frac{432.8875}{t_r^3}$$

with  $i$  in in./hr and  $t_r$  in minutes. The standard error of estimate is 0.0079 in./hr.

- a) Estimate the peak flow at the outlet from the "curved end portion" of Sitka Place, roughly where a line on the asphalt pavement would connect houses 3 and 8, as shown by the arrow on the figure. Follow these steps and guidelines:

- You must construct an estimate of the tributary area. Make an enlarged photocopy of Figure P6.21 for this purpose.
- Runoff from the impervious areas of houses 4, 5, 6, and 7 contribute to flow at the upper end of the street, but none of the other houses do. Thus, you can assume the downhill edges of houses 4 and 7 and the downstream edge of their driveways define the lower boundary of the basin (apart from the pavement of the cul-de-sac).
- Work uphill from these lower boundaries toward the high point on the hill to define the tributary area, keeping in mind that the boundary should be perpendicular to the contours. This will define a "tear-drop" shaped catchment.
- Estimate the area using a planimeter or by counting squares. Estimate an average slope by dividing the difference in upper and lower elevations by an estimated path length.
- The undeveloped area above the end of the street behaves like a lawn on "heavy soil." Overland flow roughness for this surface could be characterized as for "dense shrubbery and forest litter."
- Use the kinematic wave method to estimate  $t_c$  and hence the peak flow. Assume that the travel time over the impervious area is much less than the travel time over the pervious area. Thus, base kinematic wave parameter  $\alpha$  only on the pervious area characteristics. Do not compute a weighted Manning's roughness.

For map and drainage divide, see "map"

For areas, see "areas"

For parameter estimates, see "parameters"

From other worksheets:

	Area, ac	Length, ft	Up-elev, ft	Low-elev, ft	Slope	n	alpha	C	Wtd C
Total	2.84	700	525	418	0.153	0.4	1.456	0.4	0.52
Impervious	0.615	150	430	418	0.080	0.013	32.418	0.95	

Note,  $T = 10$  yr for Corvallis residential

Use ODOT Zone 8 IDF curves

$$i = \text{coef0} + \text{coef1}/t_r + \text{coef2}/t_r^2 + \text{coef3}/t_r^3$$

intensity in in/hr, duration in minutes

coef0	0.2081
coef1	29.8438
coef2	-181.51
coef3	432.8875

$$t_c = [L/\alpha - i^{m-1}]^{1/m}$$

m 1.67

Kinematic wave iterations. Total area:

$t_r$ min	$i$ in/hr	$i$ ft/s	$C^*$ ft/s	$t_c$ min	$Q = C^*A$ cfs
40	0.85	1.962E-05	1.018E-05	67	
80	0.55	1.282E-05	6.653E-06	80 OK	0.816 Q for total area
90	0.52	1.199E-05	6.223E-06	82 Too easy. Try another value. OK	

6-39

## 6.21 continued...

### Part b:

Kinematic wave iterations. Impervious area only.

$t_r$ min	$i$ in/hr	$i$ ft/s	$C^*i$ ft/s	$t_c$ min	$Q=CiA$ cfs
5	2.38	5.508E-05	5.233E-05	2	
4	3.09	7.149E-05	6.792E-05	1.9	
3	6.02	1.394E-04	1.324E-04	1.5	3.5

~~Slope~~ Approximation for intensity is uncertain and 6 in/hr is high for Corvallis even for 3 minutes.  
Use  $i = 6.02$  in/hr for probably very conservative  $Q$

Check 12-inch pipe:

Upper elevation approx.	418 ft
Lower elevation approx.	395 ft
Length approx. = 1.25 inches =	250 ft
Slope	0.092
$n$	0.013
$d$	1 ft
$A$	0.7853982 sq ft
$Q = \text{Manning eqn.}$	
$Q$	10.8 cfs
	More than handles the flow!

### Part c:

Regarding new development, need to plan for ultimate upstream development when

Pathlengths and kinematic wave parameter  $\alpha$

Scaling from map, total path length from top to bottom is approximately 700 ft

DCIA path length approximately 150 ft

Note, don't include rooftops since they are so steep as to have negligible contribution to travel time

These are all estimates. Do not worry too much about subtleties and minor differences in assumptions.

Slopes

	slope
For total area, slope approx= $(525-418)/700 =$	0.153
For impervious area, slope approx= $(430-418)/150 =$	0.080

Roughness:

	$n$
Total area, "dense shrubbery and forest litter"	0.4
DCIA, asphalt and concrete pavement	0.013

Kinematic wave  $\alpha = 1.49/n * \text{sqrt}(\text{slope})$ ,  $\text{ft}^{1/3}/\text{sec}$

	$\alpha$
Total area	1.46
DCIA	32.4



6.21 continued..

From map, draw drainage divide approximately as shown on map figure

Planimetering results:

Overall area

Trial	Area in <sup>2</sup>
1	3.10
2	3.08

3.09 average. Use this.

From scale, 1 in<sup>2</sup> = 40000 ft<sup>2</sup> = 0.918 ac/in<sup>2</sup>

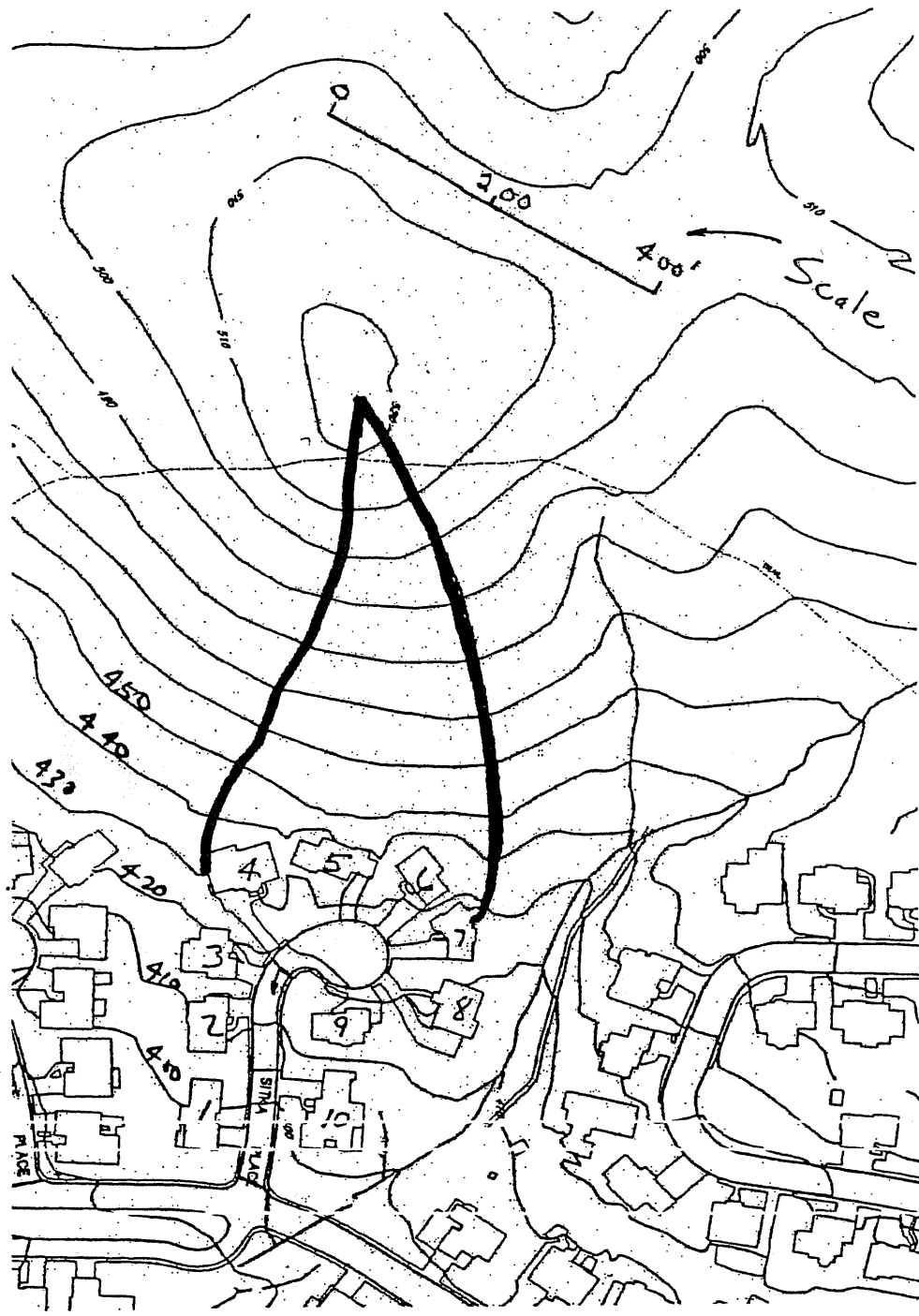
Overall area = 2.84 ac

Impervious area = houses 4,5,6,7 driveways and rooftops plus pavement in cul-de-sac

Trial	Area in <sup>2</sup>
1	0.69
2	0.61
3	0.67
4	0.67 Use

DCIA = 0.615 ac

6.21 continued...



Catchment boundary for drainage to cul-de-sac. Boundary should be constructed so that it is perpendicular to elevation contours.

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**6.22** The Santa Barbara Urban Hydrograph Method (SBUHM) consists of two basic computational steps:

(1) Compute rainfall excesses for each rainfall time step. Rainfall excesses (depth/time) from the pervious and impervious areas are multiplied by their responsive areas and summed to obtain one flow value per time step for the entire catchment (the “instantaneous hydrograph”). Any convenient means may be used to compute rainfall excess, ranging from very simple runoff coefficients with depression storage [Eq. (6-11)], to the SCS curve numbers (Chapter 2), the infiltration equations of Chapter 2. In this method, DCIA is often assumed to have no losses.

(2) The resulting instantaneous hydrograph is then routed through an imaginary linear reservoir to provide delay and attenuation. The linear reservoir has a time constant  $K = t_c$  for the catchment. Routing is performed using the Muskingum method equations (Section 4.2), with Muskingum parameter  $x = 0$  (for linear reservoir) and  $\Delta t =$  time step of instantaneous hydrograph.

Use data from problem 6.15 to perform SBUHM routing. Assume that the rainfall excess applies to the entire catchment area (so for this problem there is no merging of runoff from pervious area and DCIA).

- a) Compute the instantaneous hydrograph at 30-min intervals, in units of  $\text{cfs} \approx \text{ac-in/hr}$ .
- b) Inferring a value for the time of concentration for the overall catchment from the given time-area data, compute Muskingum coefficients  $C_0, C_1, C_2$  with Muskingum  $K = t_c$  and  $x = 0$  for the linear reservoir

- c) Perform the indicated flow routing through the hypothetical reservoir. Plot the inflow “instantaneous” hydrograph and outflow hydrograph on the same graph. Remember that the hydrographs begin with zero flow.
- d) If you have worked problem 6.15, compare the hydrographs by the two methods.

### ANSWER

No solution provided, answer will vary based on inferred time of concentration.