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)... Res.	Com.	Ind.	Other*	Undev.#	Total	Percent Impervious 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^{-1}. (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^3.

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

DS = 0.0303 S^(-0.49) inches.

DS = 0.0303 1.5^-0.49 = 0.024840 inches = 0.630945 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.)

Unused depression storage =0.630945 - 0.5mm =0.130945 mm.

Time to start of runoff = 1/12 hr + 0.130945 mm / 20 mm/hr = 0.089880 hrs = 5.392835 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.
Average = 16.23*(10-5.393)/5 = 14.96 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

Time (min)	Intensity (mm/hr)
0-5	6
5-10	20
10-15	16
15-20	8
20-25	10
25-30	14
30-35	11
35-40	7
40-45	9
45-50	4
Sum	105

Time (min)	Intensity (mm/hr)	Horton Infil. (mm/hr)	Avg. Horton Infil. (mm/hr)	Avg. Rainfall Excess (mm/hr)	Avg. over Whole 2nd Interval Avg. Rainfall Excess (mm/hr)
0	6				
5	6				
5	20				14.96
5.392835	20	4.00	3.77	16.23	14.96
10	20	3.53	3.77	16.23	14.96
10	16	3.53	3.32	12.68	12.68
15	16	3.11	3.32	12.68	12.68
15	8	3.11	2.93	5.07	5.07
20	8	2.76	2.93	5.07	5.07
20	10	2.76	2.61	7.39	7.39
25	10	2.46	2.61	7.39	7.39
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

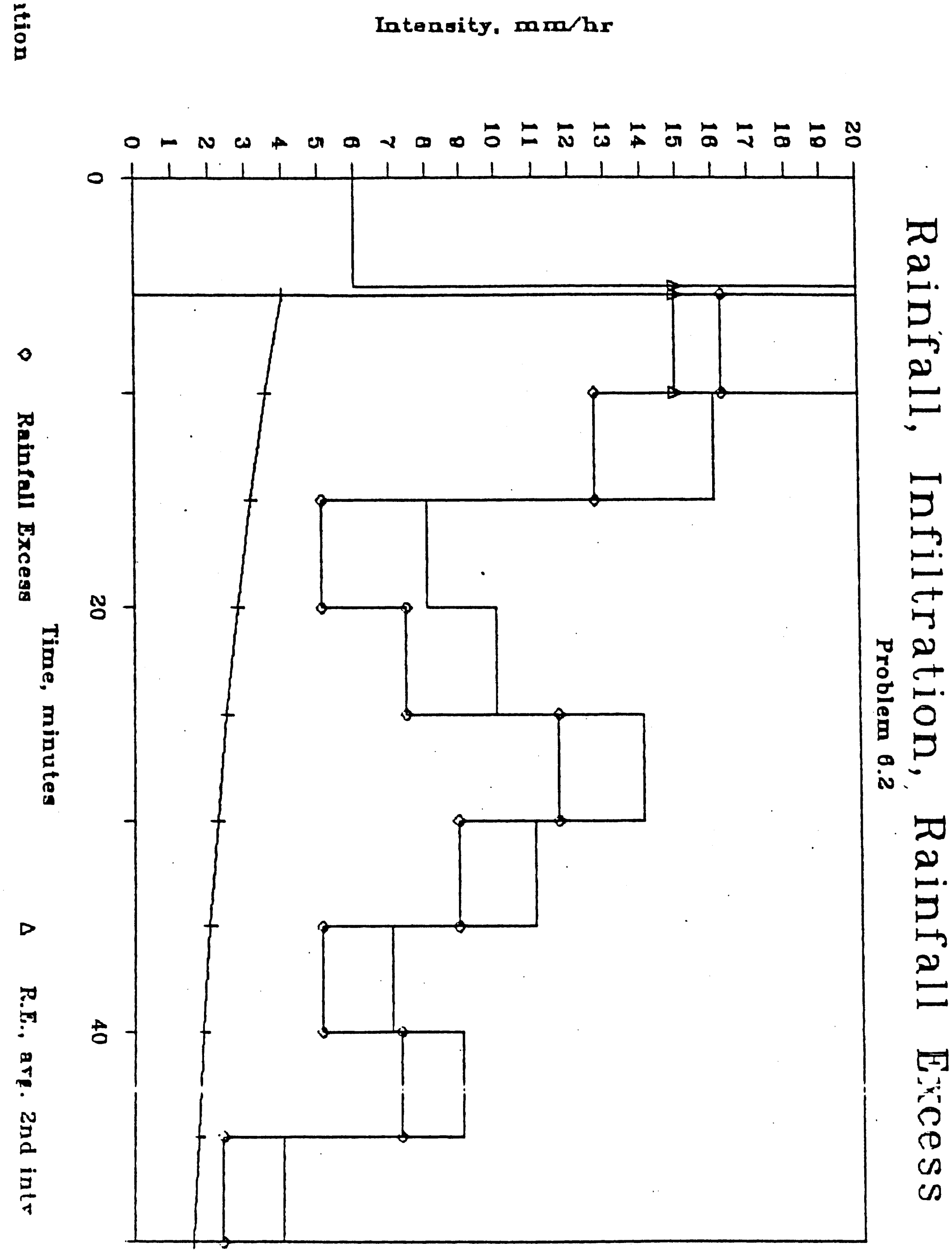
Losses = Depression storage plus infiltration =
0.63 mm + integral of Horton from 0 to 50-5.393 min =
0.63 mm + integral from 0 to 0.743 hr =
0.63 mm + 1.840685 = 2.470685 mm

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^2.

```
Depth of storage in retention basin

Depth = Volume / Area =
   125.6 cu m /          500 sq. m =     0.2512 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)
0	5
1	7
2	1
3	1
4	
5	1
6	
7	
8	
9	4
10	
11	
12	
13	6
14	1
15	
16	
17	
18	
19	
20	
21	1
22	12
23	8
24	4

MIT (hrs)	No. Events
0	20
1	8
2	7
3	6
4	2
5	2
6	2
7	1

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"

From the plot:

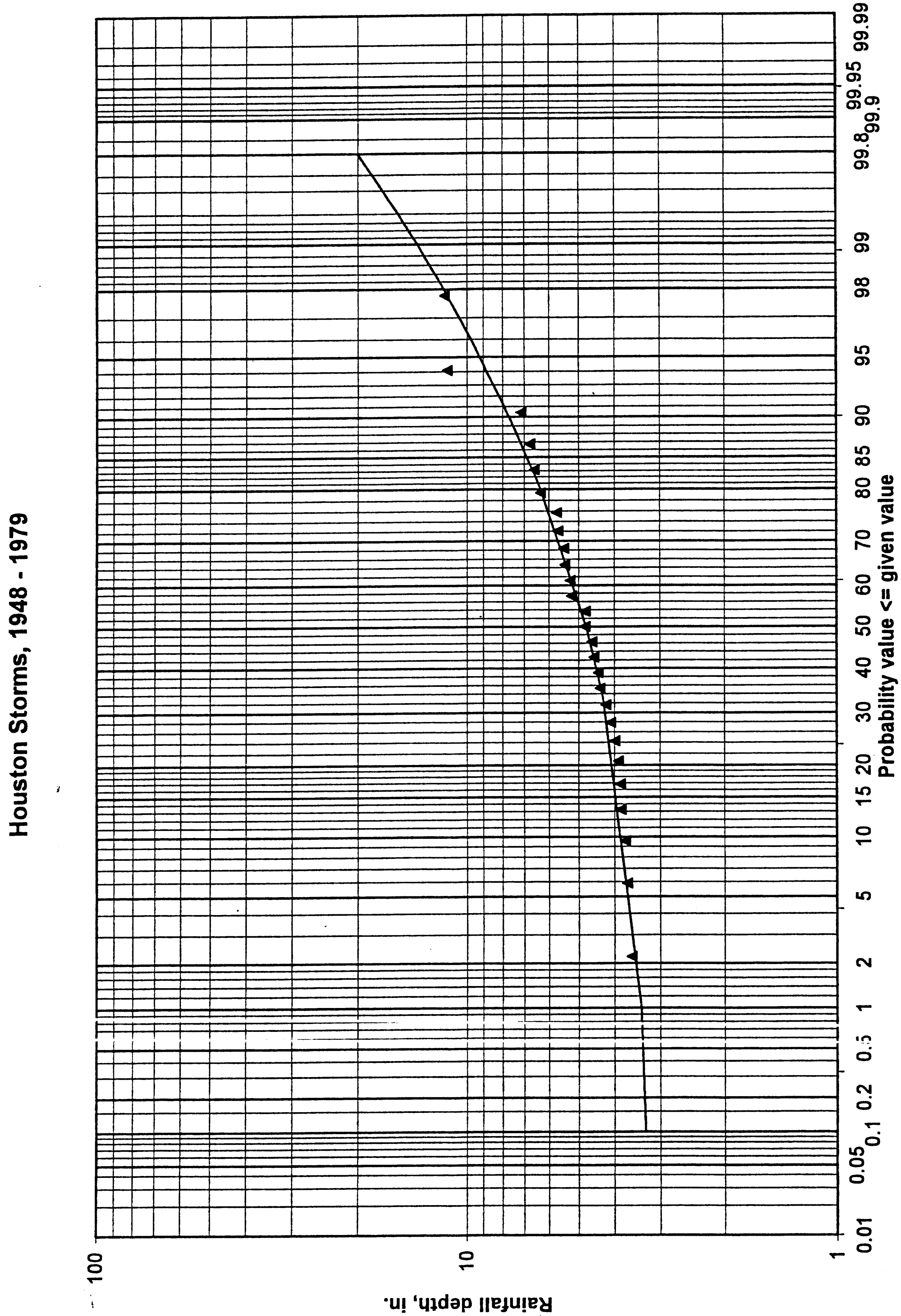
T	F	Depth (in.)
2	0.5	4.8
5	0.8	6.4
10	0.9	7.7
25	0.96	9.7
50	0.98	11.6
100	0.99	13.7

Part b:

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...



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.

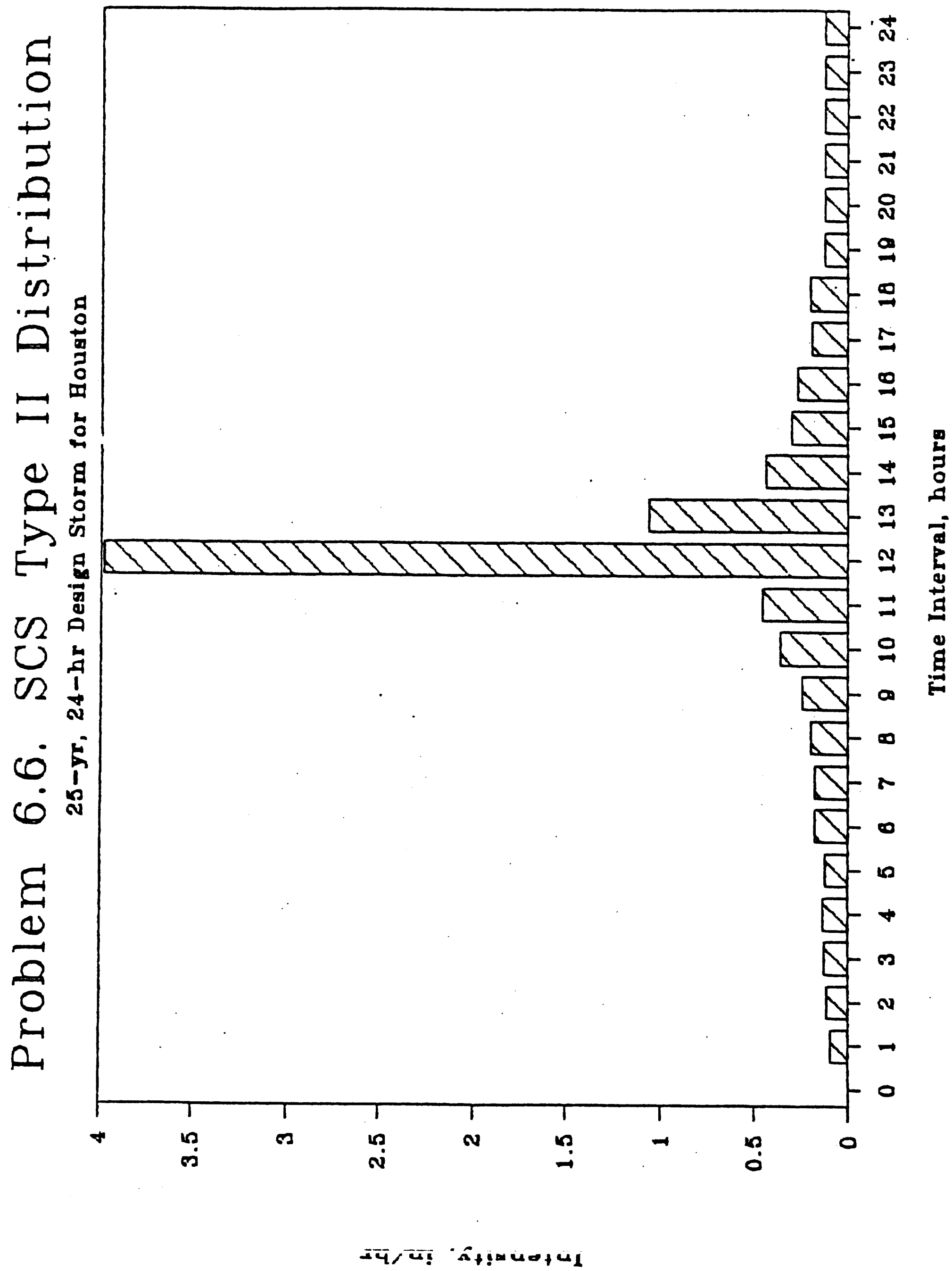
Problem 6.6. Solution:

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 x 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...



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.

a) 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.)

b) Calculate the wave speeds (in downstream direction) in each channel for depths of 0.25, 0.50, and 0.75 m.

c) 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*arccos[(r-d)/r]
w = 2r sin(Theta/2)
A = r^2/2 [Theta - sin(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

	Circular		Trapezoidal	
d	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

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 × 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.

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

	Circular		Trapezoidal	
d	C	t	C	t
0.25	2.561684	117.1104	2.614206	114.7575
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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.

6.8 cont'd

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

a) Estimate the directly connected impervious area (ac).
b) Estimate the maximum drainage length along the directly connected impervious area (ft).
c) Determine the kinematic wave parameters α and m.
d) 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 x 30 + ~pi r^2 + 11 x (20x30) = 21400 sq ft = 0.492 ac.

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

c. Alpha = 1.49 S^1/2 / n

S approximately = (165-160) / 420 ft = 5/420 = 0.0119.

Alpha = 1.49 sqrt(0.0119) / 0.016 = 10.16 ft^1/3 / sec

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

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

tc = {L/[alpha i^(m-1)]}^(1/m)   Use consistent units!

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

Calculations on Lotus spreadsheet.

Trial tr (min)	IDF i (in/hr)	IDF i (ft/sec)	Calc tc (sec)	Calc tc (min)	Qp (cfs)
10	5.714	0.000132	353.0	5.883	2.831
5	6.957	0.000161	326.3	5.439	3.446
5.45	6.823	0.000157	328.8	5.480	3.380
5.5	6.809	0.000157	329.1	5.485	3.373
5.483	6.813	0.000157	329.0	5.483	3.375

More than close enough agreement between tr and tc.

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

        S      0.005
        n      0.013

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

a) 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.
b) 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 Ỹ 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

Runoff = -0.20433 + 0.670966*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 Intercpt (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.	

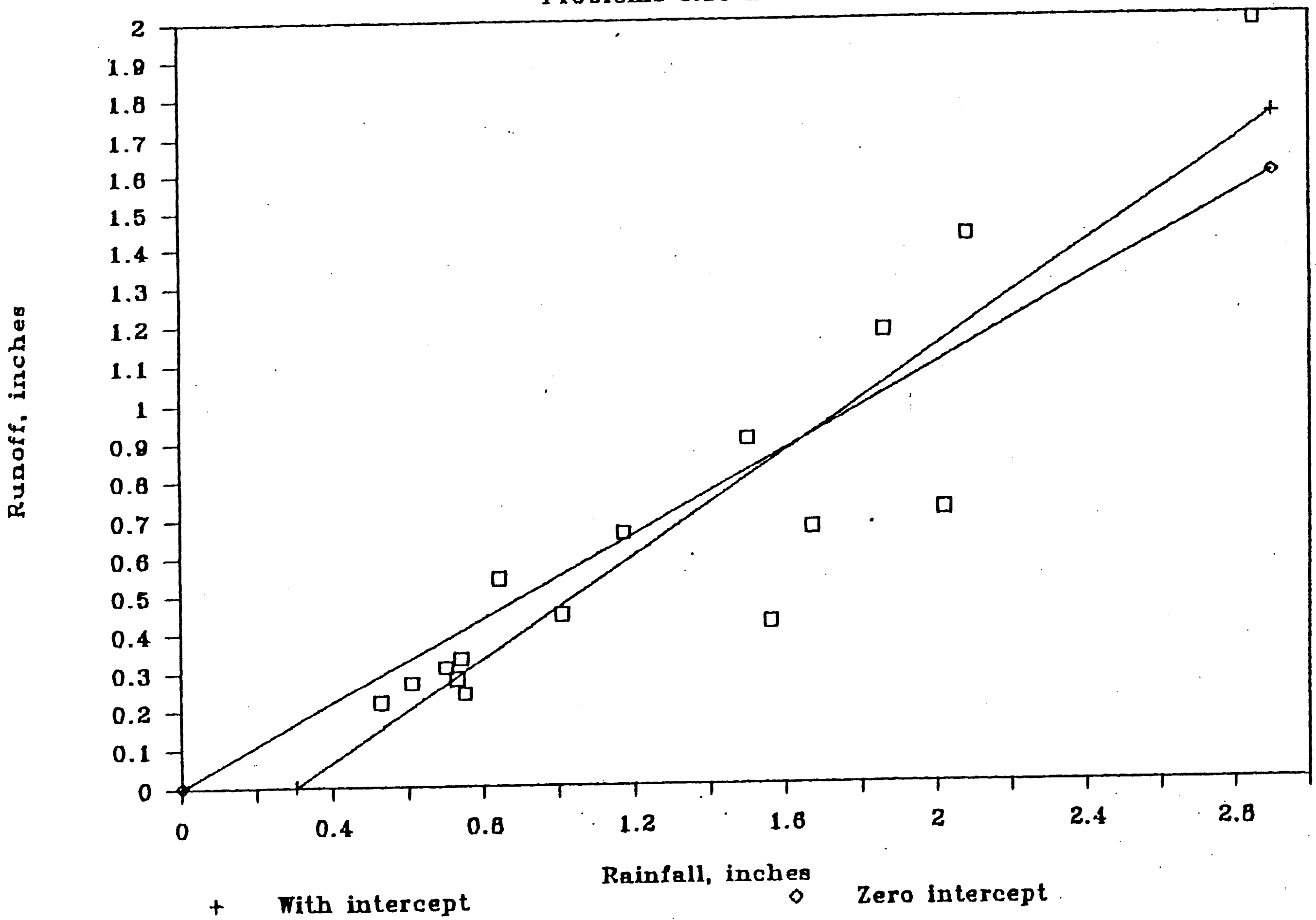
b. Put equation in form Runoff = coef*(Rainfall - DS)

DS = Rainfall at zero runoff = .20433/.670966 = 0.304536 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

a) computing the runoff coefficient for each storm and finding the average;

b) dividing the total runoff for all storms by the total rainfall for all storms;

c) 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 × 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.)

a) What is the total volume of the pond, in ft^3 and ac-ft?
b) Develop the depth vs. surface area and depth vs. volume curves for the pond. Use 1-ft intervals. Tabulate and plot.
c) 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 x 110 =      16500 square feet
Midpoint area = 135 x 95 =      12825
Bottom area = 120 x 80 =         9600

Vol = Depth/(2*3) * (Asurf + 4 Amid + 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^2)	Volume (ft^3)	Outflow (cfs)	Puls: 2V/dt+O (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	48792	11.34501	173.9850
5	150	110	16500	64530	12.68410	227.7841

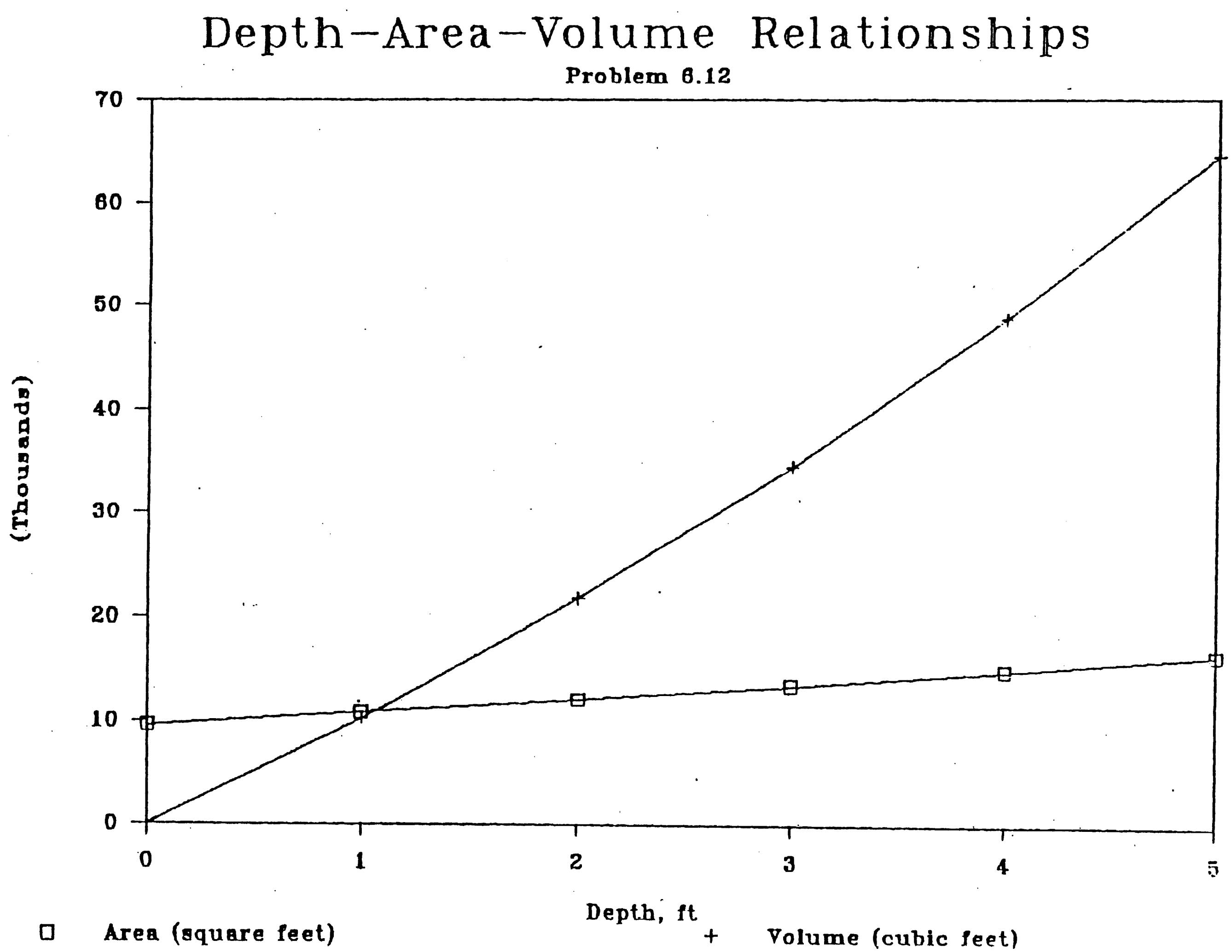
(Close agreement for volume between two integration methods.)

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+O (cfs)	Depth (ft)	Slope: D/Puls	Qo (cfs)	Slope: Qo/Puls	V (ft^3)	Slope: V/Puls
0	0	0.025168	0	0.142767	0	257.1697
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	

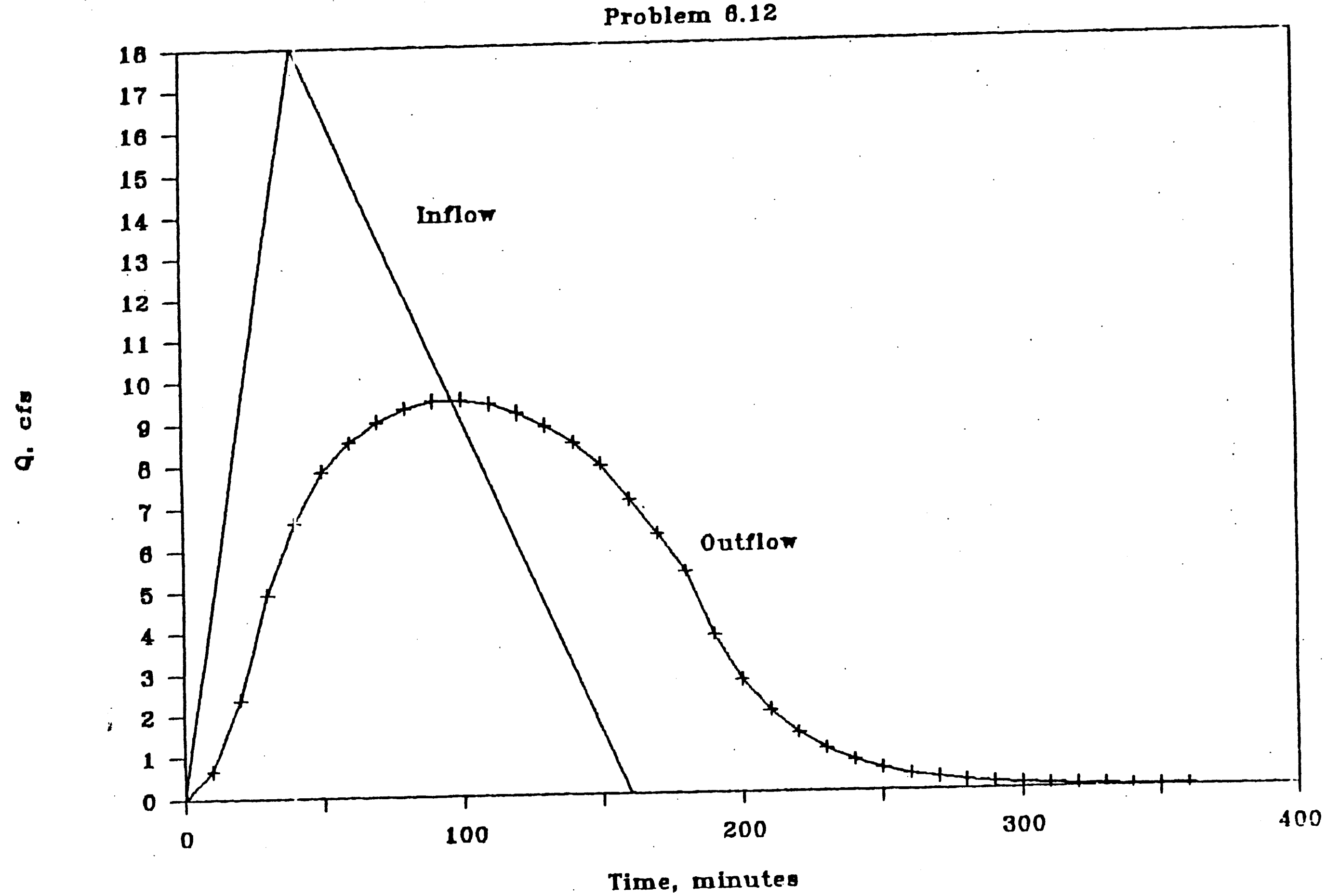
Hydrograph:

t (min)	t (sec)	Qin (cfs)	Puls: LHS (cfs)	Depth (ft)	Qout (cfs)	V (ft^3)
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	8.54	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	9.46	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
70
60
50
40
30
20
10
0
(Thousands)
0
1
2
3
4
5
Depth, ft
□ Area (square feet)
+ Volume (cubic feet)

Puls Routing Through Detention Pond

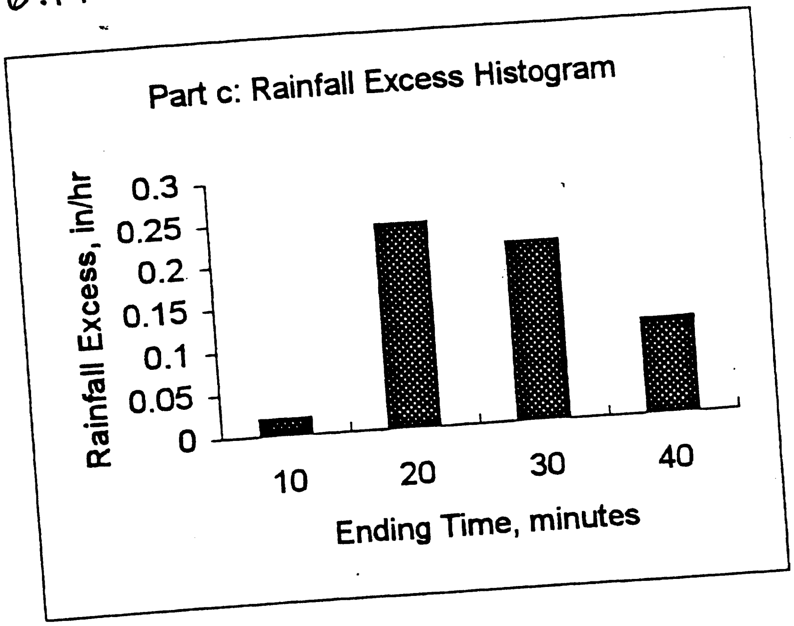
Problem 6.12



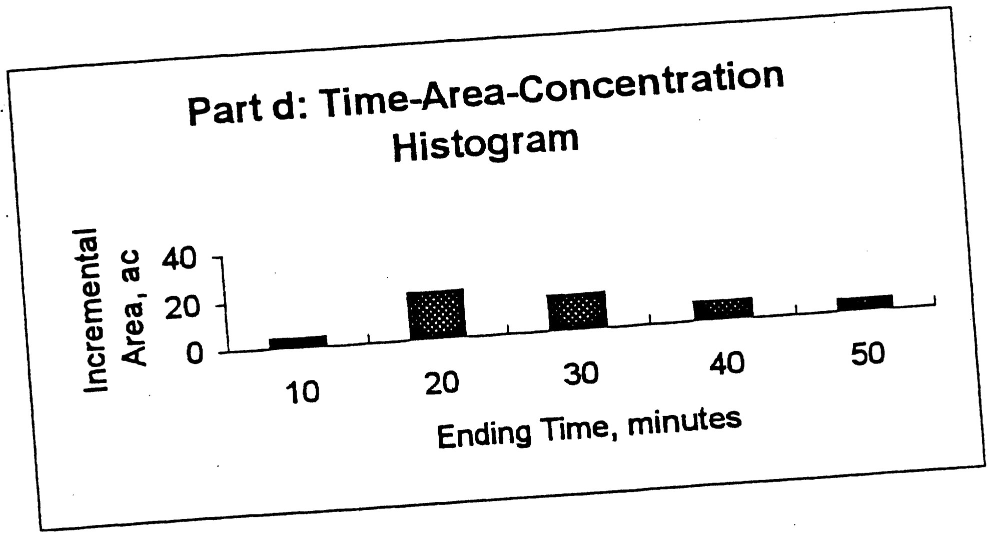
6.13. Use the USGS regression equation (Eq. 6.27) to compute the 5-yr peak flow for a 4-ac Tallahassee catchment that is 95% impervious. Compare your answer with the value computed in Example 6.5. The reason for the large discrepancy is that the regression equations were developed for much larger catchments (the smallest being 0.21 mi²) and probably do not apply to a 4-ac catchment.

```
Problem 6.13. Solution:

Qp = Cp A^a IA^b

Qp in cfs
A = total area in square miles
IA = percent impervious area

For T = 5 years, from Table 6.25,
      Cp     24.5
       a     0.77
       b    0.943

Qp = 24.5 * (4/640)^.77 * 95^.943 =      36.1 cfs

Value from Example is Qp = 24.7 cfs
```

6.14 continued...



d.)



6.14 continued...

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

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

f & g)

Linear Reservoir Routing Option 1

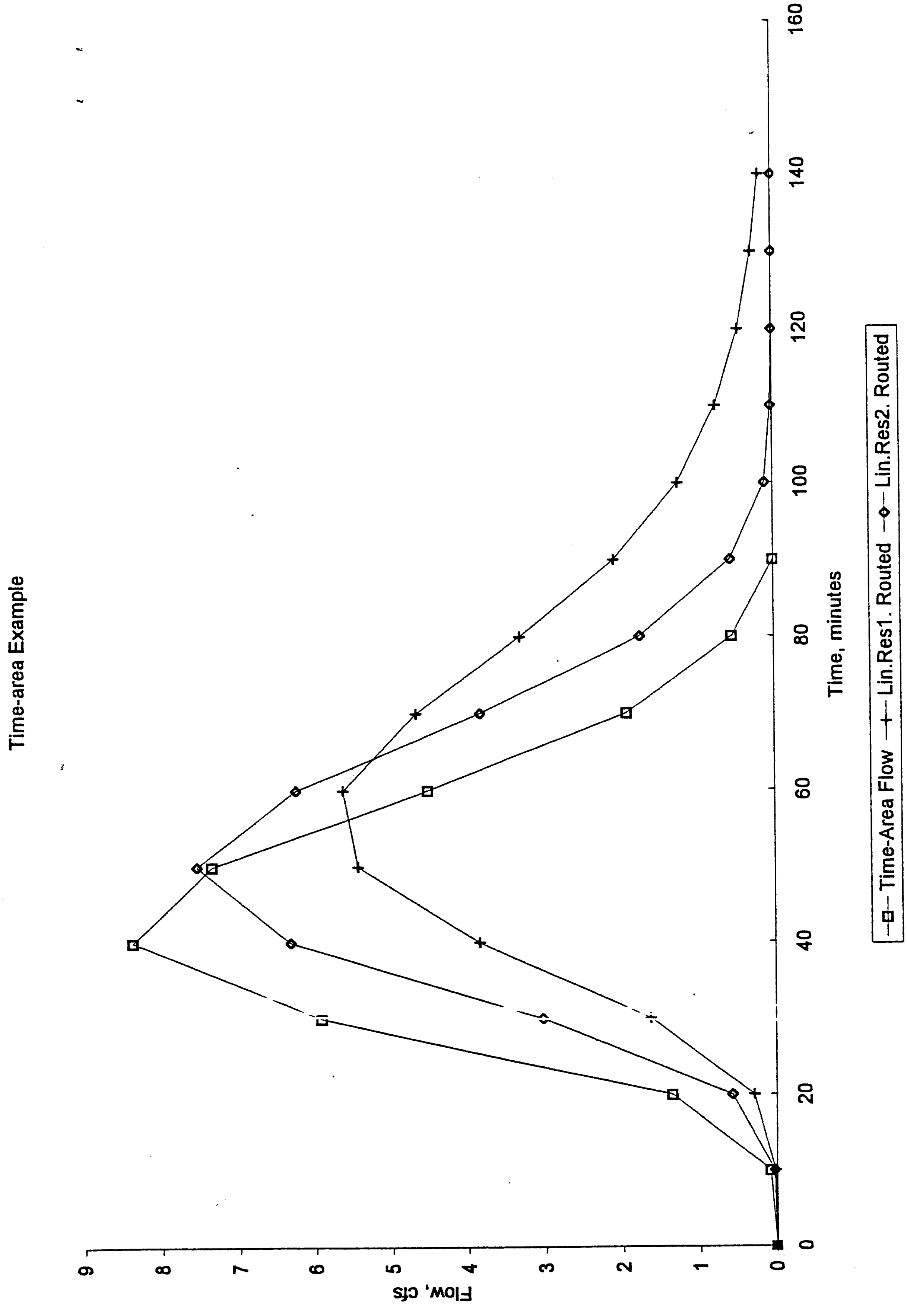
K	20 min
DT	10 min
Muskingum Coefs:	
Denom	25 min
ccc0	0.2
ccc1	0.2
ccc2	0.6

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 min	Time-area Flow cfs	Reservoir1 Routed Flow cfs	Reservoir2 Routed Flow 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

6.14 continued...



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		

a) What is the total area of the catchment?
b) What is the time of concentration of the overall catchment?
c) Perform the indicated time-area routing.
d) The linear reservoir is to be designed (conceptually) such that the peak flow out of the reservoir is only 60% (± 0.5 cfs) of the inflow peak. Using the Muskingum routing method with $x = 0$, experiment with K-values to achieve this result.
e) 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

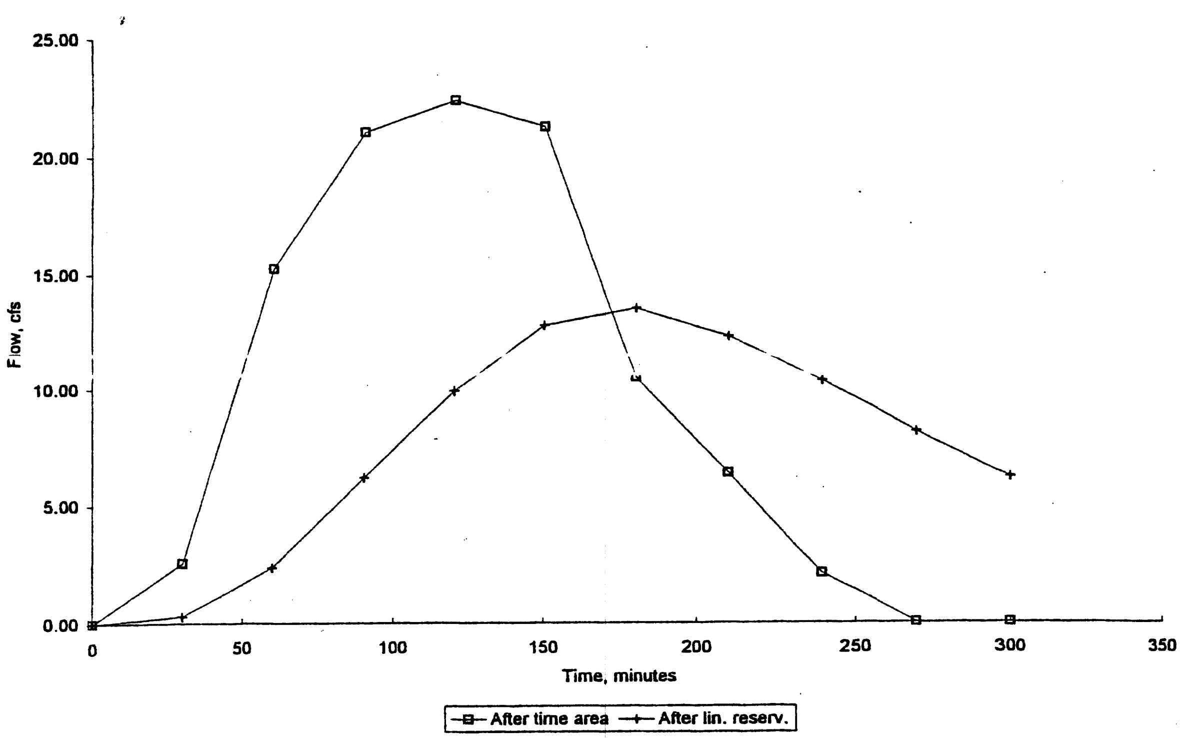
6.15 continued...

Part d: Find K for linear reservoir such that
new peak = 0.6 * old peak, +/- 0.5 cfs
Old peak= 22.34 cfs
Want new-peak = 13.40 or betwee 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. The catchment sketched in Figure P6.16 is to be a major commercial/business area. The underlying soils are known to belong to SCS hydrologic group B. Also on the sketch are isochrones of equal translation time to the outlet. The contributing areas are A1 = 20 ac, A2 = 50 ac, A3 = 15 ac.

a) Assuming the land surface characteristics are fairly homogeneous, make a hypothetical but plausible sketch of elevation vs. distance upstream from the outlet, (i.e., z vs. x along the main drainage pathway). Explain the basis for your sketch.

b) A half-hour duration storm has average rainfall over 15-min increments of 1.5 and 1.1 in./hr. Use the SCS method to calculate the net runoff (inches) from the total storm. What is the volumetric runoff coefficient?

c) Use the method of Example 2.8b to compute the hyetograph of rainfall excess for the two time steps. That is, distribute the total loss of part b over the two hyetograph increments according to this method.

d) Using the hyetograph of rainfall excess from part c, perform time-area routing using the time-area-concentration data provided in the beginning paragraph and the sketch of Figure P6.16.

Part a:

From Fig. P6.16, the 50 acres of A2 contribute much faster than do the 20 ac of A1 or the 15 ac of A3. The main reason would be slope: the slope of area A2 must be steeper than A1 or A3. Hence, a plausible sketch of elevation, z, vs. distance along main channel, x is shown on worksheet ElevVsDist

Part b:

SCS Soil Group B for Commercial land use has CN= 92

S = 1000/CN - 10 = 0.870 inches

Time min	Intensity in/hr	Depth in.
0 - 15	1.5	0.375
15 - 30	1.1	0.275
		0.65 Sum = total depth = P

Q = (P-0.2S)^2 / (P + 0.8S) = 0.168 inches = total runoff depth

Volumetric runoff coefficient = runoff/rainfall = Q/P = 0.259

Part c: Distribute losses using SCS technique of Example 2.8b to get rainfall excess hyetograph

Initial abstraction, Ia = 0.2 S = 0.174 inches

At each time interval:

infiltration, F = S*(P-Ia)/(P-Ia+S) = S(P-.2S)/(P+0.8S)

Q = P - Ia - F

Remember, F and Q are cumulative values, to indicated time.

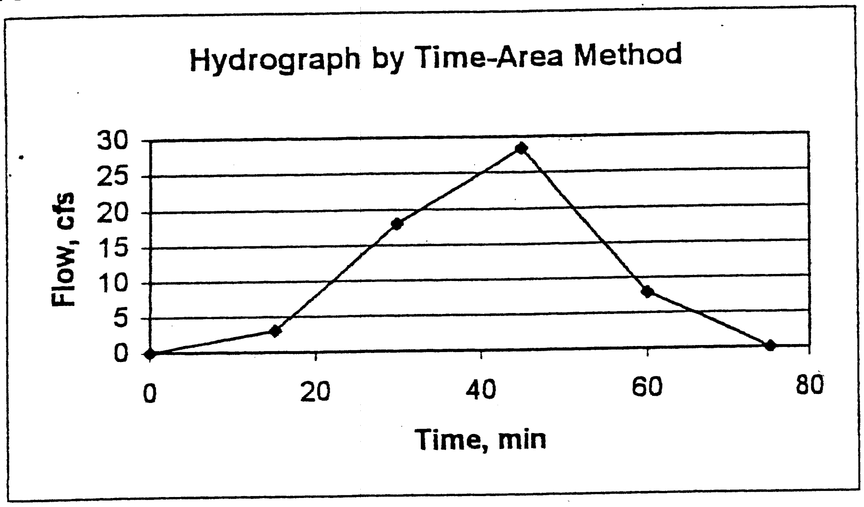
Rainfall excess = incremental-Q/Dt

Time min	Rain Intensity in/hr	Cumulative Depth in.	F in.	Q in.	Incr-Q in.	Rainfall Excess in./hr
0 - 15	1.5	0.375	0.163	0.038	0.038	0.151
15 - 30	1.1	0.65	0.308	0.168	0.131	0.523
					0.168	sum = check = same as total storm calc.

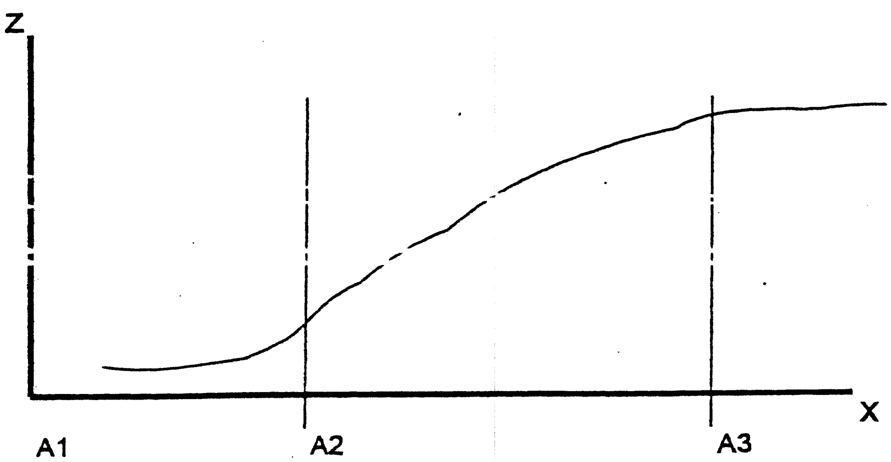
6.16 continued...

Part d:

Time min	Area ac	Rainx in/hr	R1* cfs	R2* cfs	R3* cfs	R4* cfs	Sum cfs
0							0
15	20	0.151	3.0				3.0
30	50	0.523	7.6	10.5			18.0
45	15		2.3	26.1	0		28.4
60			0.0	7.8	0	0	7.8
75			0.0	0	0	0	0



Plausible relationship between elevation and distance along stream length.



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

a) 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.

b) Does this catchment exhibit the characteristics of depression storage or of baseflow?

c) 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 Runoff = C(Precip - DS).

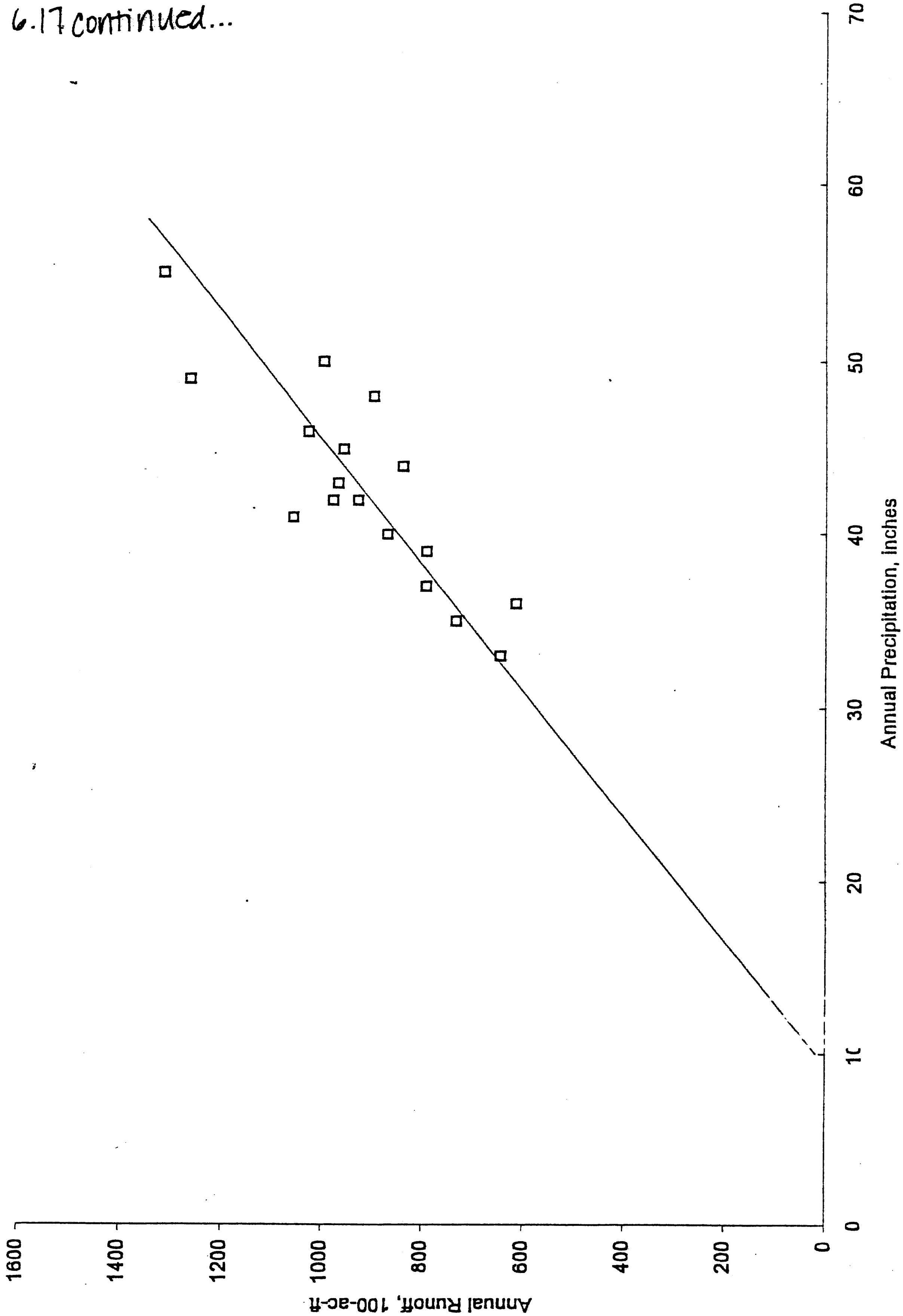
Here, constant = Coef*DS, so DS = 9.36 in. (but not needed for HW answer).

Part c:

X-coefficient = slope = 28.02 1000acft/inch * 12 * 1000 = 336284 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...



6.18 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 *x* runoff depth *x* 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 lb/ac-in = 12.4 lb / (9.6 ac * 0.24 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

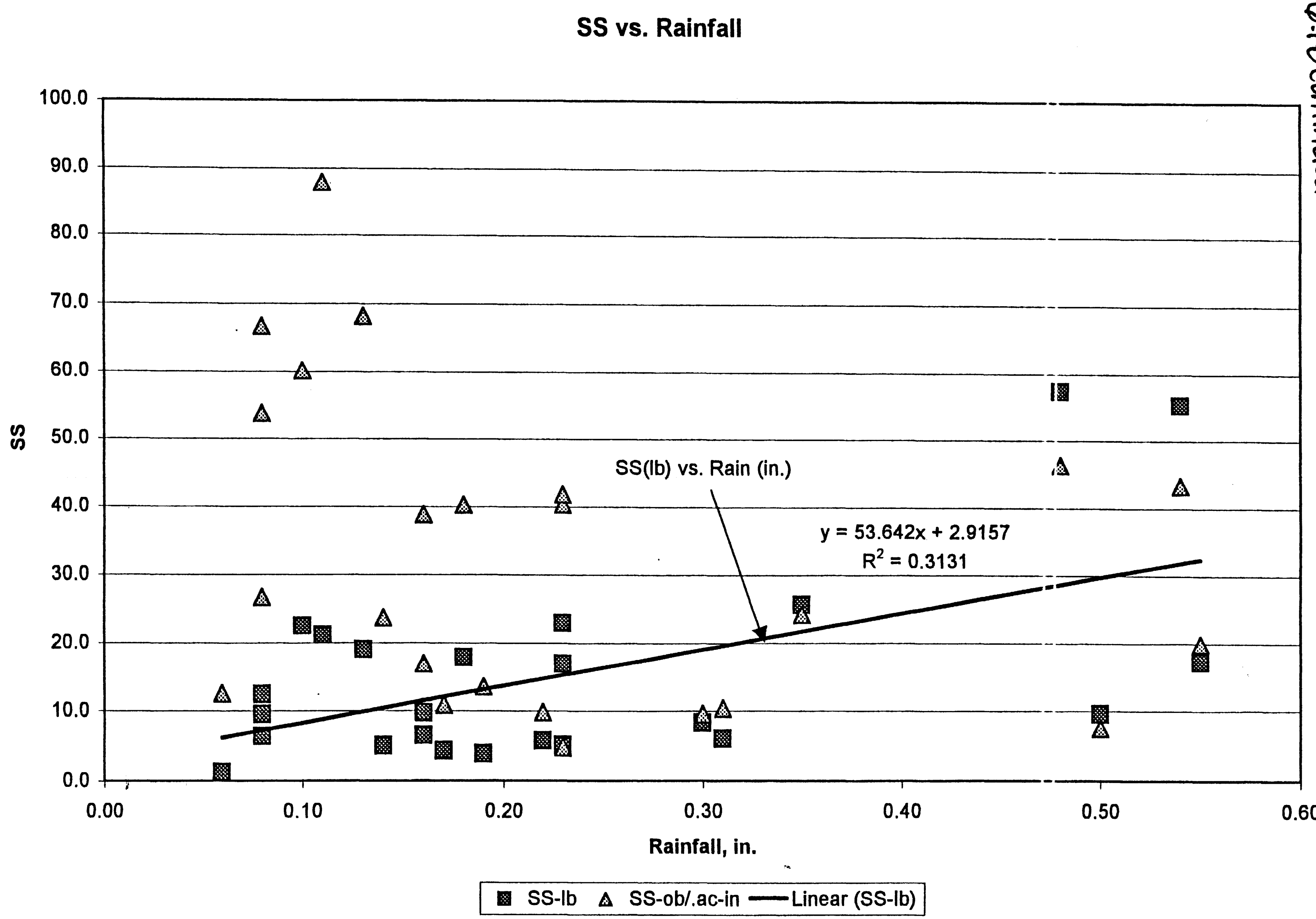
X Coefficient(s)	-43.7143	
Std Err of Coef.	30.95516	
t(n-2)	-1.41218	abs(t(n-2)) < T-table, so NOT significant

Part c:

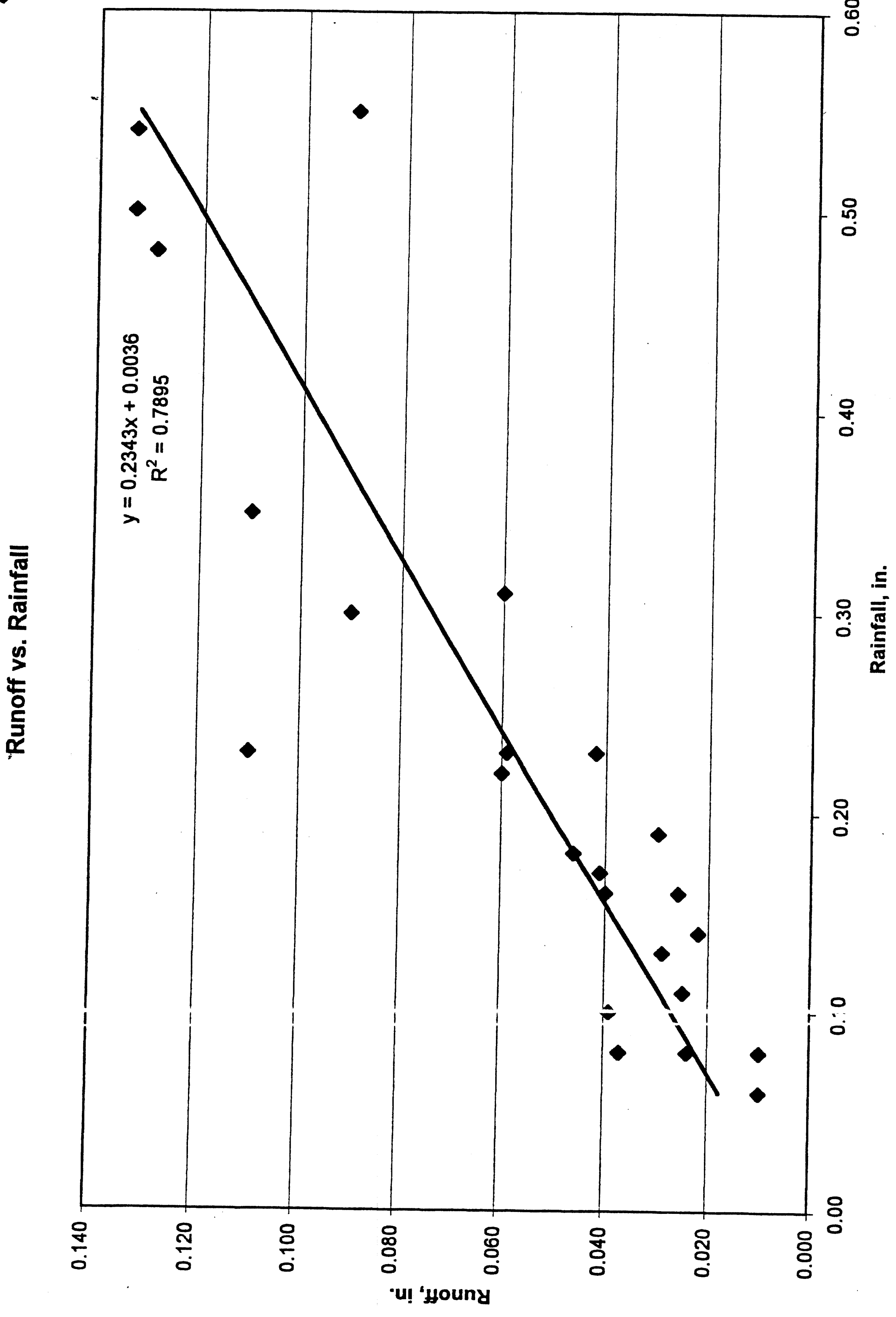
Other possible causative (independent variables) for regression of runoff depth vs. rainfall depth include:

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

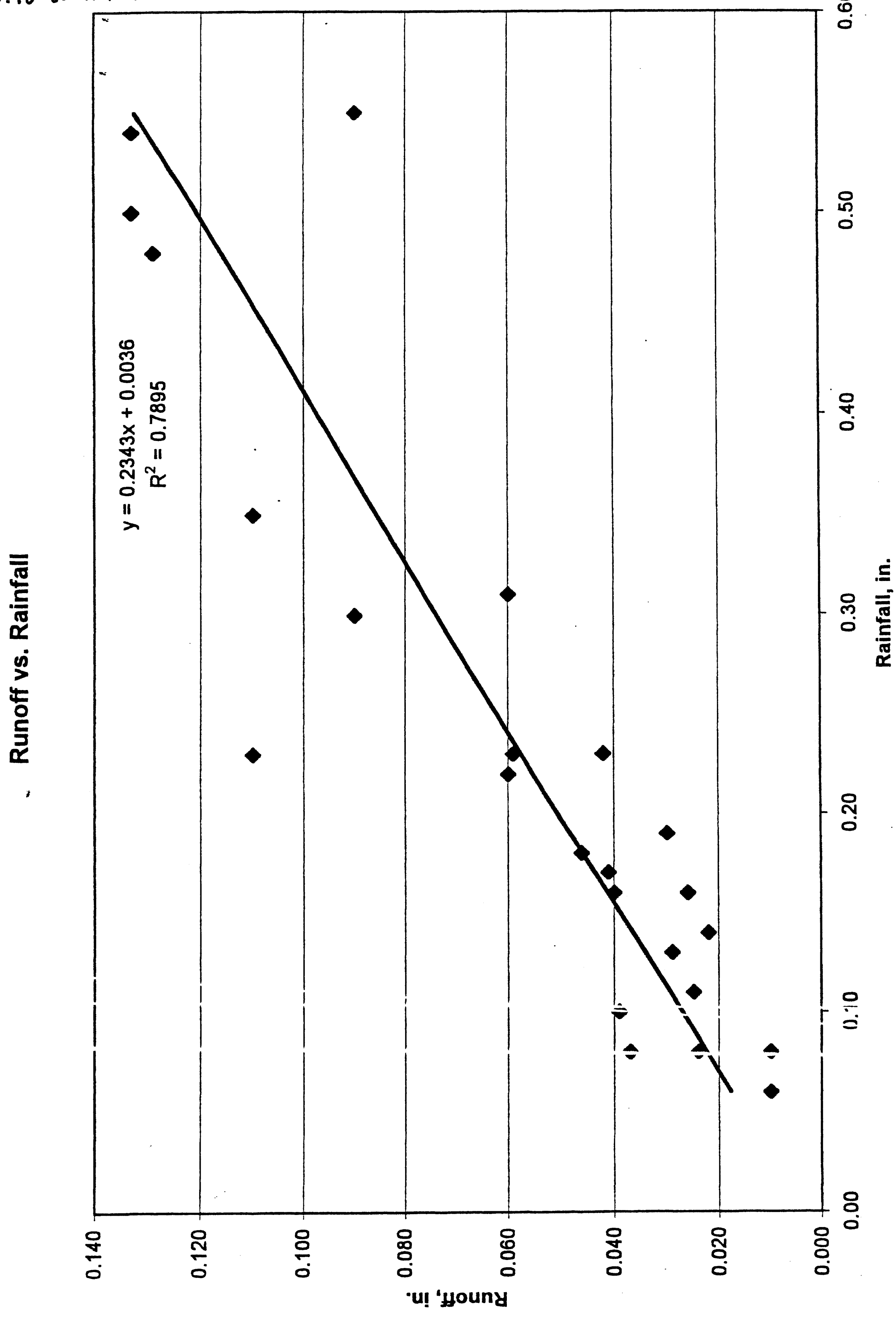
6.18 continued...



6.18 continued



6.18 continued



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.

a) What is the 25-yr outflow, by the rational method? Remember to compute runoff for both the total area and just for the DCIA.
b) 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.
c) 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

Atotal = 5000 x 3000 ft2=	15000000 ft2 =	344.4 ac
A-DCIA = 2500x1000 ft2 =	2500000 ft2 =	57.4 ac
A-pervious = difference =		287.0 ac
Weighted runoff coefficient, by areas = Cbar =		0.283

25-year intensities for Tallahassee from Fig. 6.5:

Duration min	i in/hr		Q cfs
20	6.5	Q = C-DCIA * i * A-DCIA =	354
120	2.4	Q = C-bar*i*A-total =	234

Using just DCIA gives considerably higher peak flow.

6.20 continued...

Part b:

Tallahassee regression equation is: Q = Cp * A^a * IA^b
IA = percent impervious area = 57.4/344 = 16.667 %
A = 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 * sqrt(slope) * A*R^2/3
slope = 0.01
1.49/n*sqrt(slope) = 6.208

Hydraulic radius, R = D/4
Try different diameters

D ft	R ft	A ft2	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 α 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 = coef0 + coef1/tr + coef2/tr^2 + coef3/tr^3
intensity in in/hr, duration in minutes

coef0	0.2081
coef1	29.8438
coef2	-181.51
coef3	432.8875

tc = [L/alpha-i^m-1]^1/m
m 1.67
Kinematic wave iterations. Total area:

tr min	i in/hr	i ft/s	C*i ft/s	tc min	Q=CiA 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	

6.21 continued...

Part b:
Kinematic wave iterations. Impervious area only.

tr	i	i	C*i	tc	Q=CiA
min	in/hr	ft/s	ft/s	min	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

Stop! 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 = 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 * sqrt(slope), ft^1/3/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 in2
1	3.10
2	3.08
	3.09 average. Use this.

From scale, 1 in2 = 40000 ft2 = 0.918 ac/in2

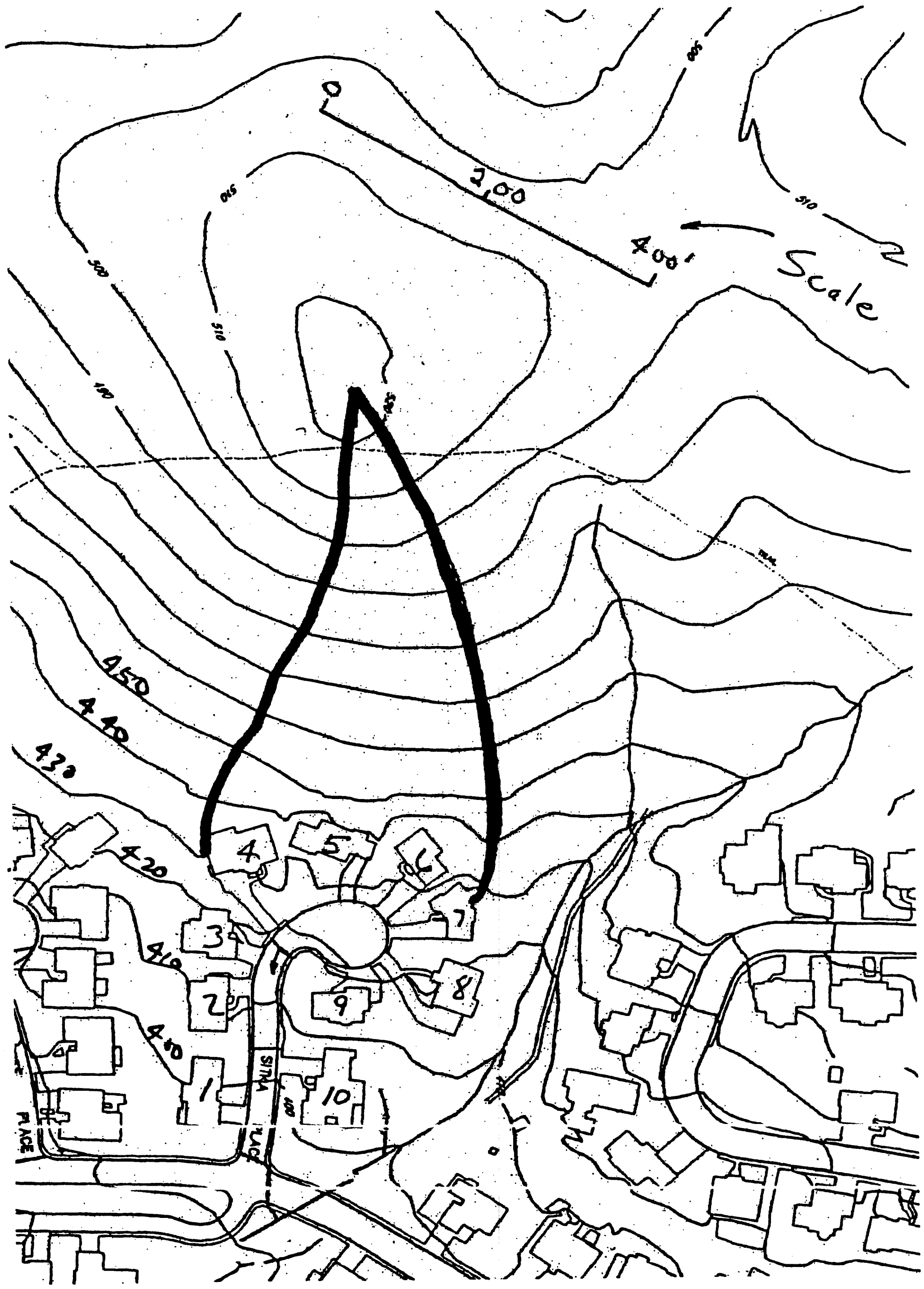
Overall area = 2.84 ac

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

Trial	Area in2
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.