

UNIVERSITY OF NEW ORLEANS
DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING

ENCE 4321
HYDROLOGY

COURSE DISCRIPTION:

The hydrologic cycle, runoff relations, unit hydrographs, flood routing, probability in hydrology, hydrologic simulation, and stochastic methods in hydrology.

PREREQUISITES: ENCE 3318 OR ENME 3720 and credit or registration in ENCE 3300.

TEXTBOOK: Bedient, P. B.; and W. C. Huber, "Hydrology and Floodplain Analysis," 3rd Edition, Addison-Wesley Publishing Company, 2002

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LECTURES: 6:00-7:15PM Thursday
Room SC 2068

OFFICE HOURS: 2:00-3:00PM Monday and Wednesday
Or by appointment

GRADING SCHEME:

1. TESTS: 4 @ 25% each. 100%
2. FINAL EXAM: 25%
3. PAPER/PROJECT: --- (Graduate Students Only)
(I will drop your lowest 25%)

GRADING SYSTEM:

- A 90 and up
- B 80 to 89
- C 70 to 79
- D 60 to 69
- F Below 60

Course Objectives

To give the civil engineering student an introduction to the basic principles of hydrology and its use in solving engineering problems related to water resources systems. The use of hydrologic techniques to derive design discharges for water resources projects will be stressed. These techniques include analytical, numerical, and statistical solutions to the basic equations governing hydrologic systems. Specific skills the student should learn include:

1. Understand the dynamic nature of weather systems-watershed processes to apply and solve the mass balance equations (hydrologic equation).
2. Understand the assumptions and limitations of the Rational Method and the Hydrograph Method to solve for estimates of surface runoff.
3. Understand the fundamental of hydrologic and hydraulic Routing methods to apply and solve simple reservoir routing and Muskingum method for streamflow routing.

LECTURE TOPIC OUTLINE

TOPIC	TEXT CHAPTER	CLASSES
Hydrologic Cycle and Hydrologic Budget, Precipitation, Hydrologic Abstractions, Surface Runoff and Stream flow	CHAPTER 1	4
Hydrologic Analysis	CHAPTER 2	4
Routing Techniques	CHAPTER 4	4
Hydrologic Design and Applications	LA DOTD HYDRALIC MANUAL	4
Frequency Analysis	CHAPTER 3	4
Simulations and Synthesis Models	CHAPTER 5	4

Donald Jersilleman

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**ENCE 4321
HYDROLOGY**

TEST 1

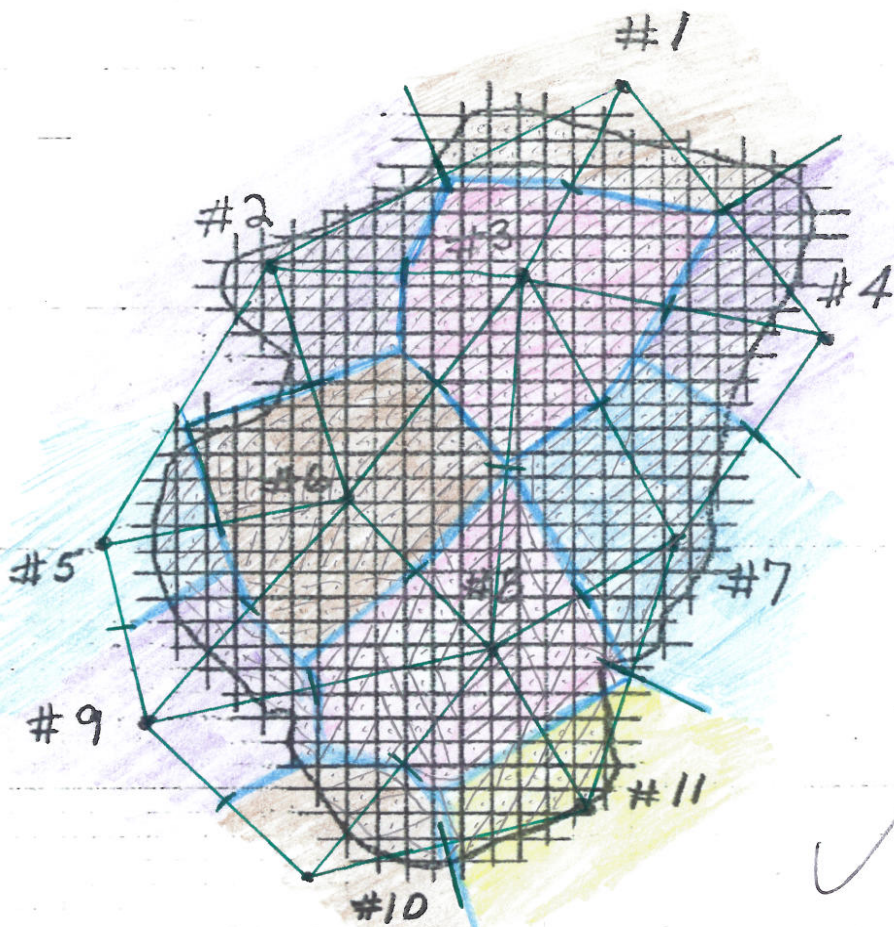
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1. Use the Thiessen Polygon method to determine the Basin Average Precipitation (BAP) for the watershed on the attached sheet. The average annual precipitation at each rain gage is as follows:

Rain Gage Number	Average Annual Precipitation (inches)
1	39.9
2	36.41
3	59.99
4	63.92
5	38.4
6	64.56
7	63.53
8	42.62
9	52.46
10	44.69
11	23.27

52.0" ✓

- 1) 39.9
- 2) 36.14
- 3) 59.99
- 4) 63.92
- 5) 38.4
- 6) 64.56
- 7) 63.53
- 8) 42.62
- 9) 52.46
- 10) 44.69
- 11) 23.27



Total Est.: 465

	Count	Weighted amount ($\frac{\text{count}}{\text{Total}}$)	Weighted amount \times gage reading
Total	451		
1	28	0.0621	2.478
2	37	0.0820	2.96
3	73	0.1619	9.71
4	31	0.0687	4.39
5	12	0.0266	1.021
6	81	0.1796	11.59
7	55	0.122	7.75
8	83	0.184	7.84
9	16	0.0355	1.862
10	13	0.0288	1.287
11	22	0.0488	1.136
		$\Sigma = 1$ ✓	52.0"

$$209.2 \frac{\text{ft}^3}{\text{s}} \left(\frac{0.3048 \text{ m}}{1 \text{ ft}} \right)^3 = 5.92 \frac{\text{m}^3}{\text{s}}$$

2. Compute the streamflow for the following measurement data:

	Distance from bank (feet)	Total Depth, d (feet)	Velocity at 0.2d (ft/sec)	Velocity at 0.8d (ft/sec)
1	0	0	0	0
2	2	2.0	1.0	0.8
3	4	4.0	1.7	1.4
4	6	6.0	2.6	2.1
5	8	8.0	2.9	2.3
6	10	8.0	2.7	2.2
7	12	8.0	2.4	2.0
8	14	6.0	2.3	1.9
9	16	4.0	2.3	2.0
10	18	2.0	1.8	1.5
11	20	0	0	0

Express your answer in cubic feet per second and in cubic meters per second.

	width (ΔW (feet))	mean velocity (ft/s)	Area (ft^2) $\Delta W(D)$	Q (ft^3/s) $[\Delta W(\text{mean vel.})(d)]$
1)	$0 + 1 = 1$	0	0	0
2)	$\frac{2}{2} + \frac{2}{2} = 2$	$\frac{1+0.8}{2} = 0.9$	4	3.6
3)	$\frac{2}{2} + \frac{2}{2} = 2$	1.55	8	12.4
4)	$\frac{2}{2} + \frac{2}{2} = 2$	2.35	12	28.2
5)	$1 + 1 = 2$	2.6	16	41.6
6)	2	2.45	16	39.2
7)	2	2.2	16	35.2
8)	2	2.1	12	25.2
9)	2	2.15	8	17.2
10)	2	1.65	4	6.6
11)	1	0	0	0
Σ			96 ft^2	209.2 $\text{ft}^3/\text{s} = 5.92 \frac{\text{m}^3}{\text{s}}$

$$\frac{\text{Acre}}{43560 \text{ ft}^2}$$

3. It is desired to fill a reservoir for a hydropower project by means of a 30 inch RCP (reinforced concrete pipe). The attached tailwater rating curve gives the flow through the pipe for various reservoir levels. If the volume in the reservoir between elevations 55 and 60 feet NGVD is 100 acre-ft, how long will it take to raise the level from 55 to 60 feet NGVD? Use the average pipe discharge between these elevations in your calculations, BUT USE 1 FOOT INCREMENTS. Give your answer in hours.

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$$Q_{60ft} = 58 \text{ cfs} \quad Q_{55ft} = 64 \text{ cfs} \quad Q_{\text{Avg}} = \frac{64 + 58}{2} = 61 \text{ cfs}$$

$$Q_{56ft} = 63 \text{ cfs} \quad Q_{57ft} = 62 \text{ cfs} \quad Q_{58ft} = 61 \text{ cfs} \quad Q_{59ft} = 59 \text{ cfs}$$

100 acre-ft (43560 ft²/acre)

$$Q_{55-56} = \frac{64 + 63}{2} = 63.5 \quad Q_{56-57} = \frac{62 + 63}{2} = 62.5 \quad Q_{57-58} = \frac{61 + 62}{2} = 61.5$$

$$Q_{58-59} = \frac{61 + 59}{2} = 60 \quad Q_{59-60} = \frac{59 + 58}{2} = 58.5$$

$$\frac{4356000 \text{ ft}^3}{63.5 \text{ cfs} + 62.5 \text{ cfs} + 61.5 \text{ cfs} + 60 \text{ cfs} + 58.5 \text{ cfs}} = 14235 \text{ s} \rightarrow 14235 \left(\frac{\text{hr}}{3600 \text{ s}} \right) = 3.95 \text{ hr}$$

$$X = \frac{4356000}{63.5} + \frac{X}{62.5} + \frac{X}{61.5} + \frac{X}{60} + \frac{X}{58.5}$$

$$= 685984 + 69696 + 70829 + 72600 + 74462 = 356185 \left(\frac{\text{hr}}{3600 \text{ s}} \right) = 98.9 \text{ hr}$$

$$X = \frac{4356000}{5} = 871200$$

$$X \left(\frac{1}{63.5} + \frac{1}{62.5} + \frac{1}{61.5} + \frac{1}{60} + \frac{1}{58.5} \right) =$$

$$871200 (0.091769) = 71237 \left(\frac{\text{hr}}{3600} \right) =$$

19.79 hr.

$$\frac{63.5 + 62.5 + 61.5 + 60 + 58.5}{5} = 61.2$$

$$\frac{4356000}{61.2} = 71176.5 \left(\frac{\text{hr}}{3600} \right) = 19.77 \text{ hr}$$

right for
wrong reason

4. Rain falls at an average intensity of 0.4 in/hr over a 600 acre area for 3 days.

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- Determine the average rate of rainfall in cfs;
- Determine the 3-day volume of rainfall in acre-ft; and
- Determine the 3-day volume of rainfall in inches of equivalent depth over the 600 acre area.

$$a) \quad 0.4 \frac{\text{in}}{\text{hr}} \left(\frac{\text{ft}}{12 \text{ in}} \right) \left(\frac{\text{hr}}{3600 \text{ s}} \right) (600 \text{ acre}) \left(\frac{43560 \text{ ft}^2}{\text{acre}} \right) = \boxed{242 \text{ ft}^3/\text{s}}$$

$$b) \quad 0.4 \frac{\text{in}}{\text{hr}} \left(\frac{24 \text{ hr}}{1 \text{ d}} \right) (3 \text{ d}) \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) (600 \text{ acre}) = \boxed{1440 \text{ acre-ft}}$$

$$c) \quad 1440 \text{ acre-ft} \left(\frac{1}{600 \text{ acre}} \right) \left(\frac{12 \text{ in}}{\text{ft}} \right) = \boxed{28.8 \text{ in}}$$

$$1 \text{ gallon} = 0.133680556 \text{ ft}^3$$

$$120000000 \text{ gal} \left(\frac{0.133680556 \text{ ft}^3}{1 \text{ gal}} \right) = 1.6041667 \times 10^7 \text{ ft}^3$$

5. 16 A flow of 120 mgd is to be developed from a 175 mi² watershed. At the flow line, ^P the area's reservoir is estimated to cover 4200 acres. The annual rainfall is 38 inches, the annual runoff is 13 inches, and the annual evaporation is 47 inches. Find the net gain or loss in storage this represents. Calculate the volume of water evaporated in acre-feet.

$$\Delta S = Q_{in} + P_{in} - Q_{out} - E = 13 \text{ in} \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) (175 \text{ mi}^2) \left(\frac{5280 \text{ ft}}{\text{mi}} \right)^2 \left(\frac{1}{\text{yr}} \right) \left(\frac{365.25 \text{ d}}{\text{yr}} \right)$$

$$+ 38 \text{ in/yr} (4200 \text{ acres}) \left(\frac{43560 \text{ ft}^2}{\text{acre}} \right) \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) \left(\frac{365.25 \text{ d}}{\text{yr}} \right) \left(\frac{\text{ft}}{12 \text{ in}} \right)$$

$$- 120 \text{ mgd}$$

$$- 47 \text{ in/yr} (4200 \text{ acres}) \left(\frac{43560 \text{ ft}^2}{\text{acre}} \right) \left(\frac{\text{ft}}{12 \text{ in}} \right) \left(\frac{365.25 \text{ d}}{\text{yr}} \right)$$

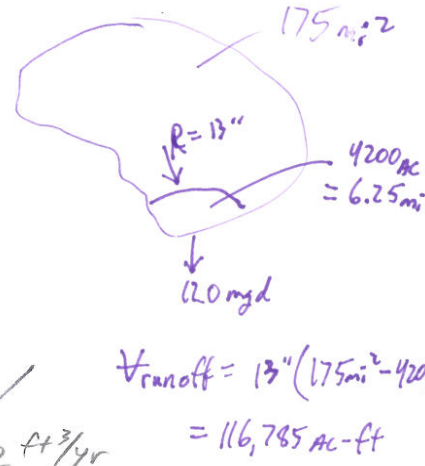
$$\Delta S = + 5,087,082,000$$

$$+ 5,295,280,000 \text{ ft}^3/\text{yr}$$

$$+ 5,793,4800 \text{ ft}^3/\text{yr} \checkmark$$

$$- 1.6041667 \times 10^7 \text{ ft}^3/\text{d} \left(\frac{365.25 \text{ d}}{\text{yr}} \right) = 5,859,218,872 \text{ ft}^3/\text{yr}$$

$$- 716,562,000 \text{ ft}^3/\text{yr} \checkmark$$



$$\Delta S = -1.23256607 \times 10^9 \text{ ft}^3/\text{yr}$$

$$- 905,219,876 \quad \text{OK}$$

If reservoir is subtracted from water shed

$$\Delta S = +57934800$$

$$- 5859218872$$

$$- 716562000$$

$$+ 13 \text{ in/yr} \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) \left[175 \text{ mi}^2 \left(\frac{5280^2 \text{ ft}^2}{\text{mi}^2} \right) - 4200 \text{ acres} \left(\frac{43560 \text{ ft}^2}{\text{acre}} \right) \right] = 5.087082 \times 10^9 \text{ ft}^3/\text{yr}$$

$$\Delta S = -1.43076407 \times 10^9 \text{ ft}^3/\text{yr}$$

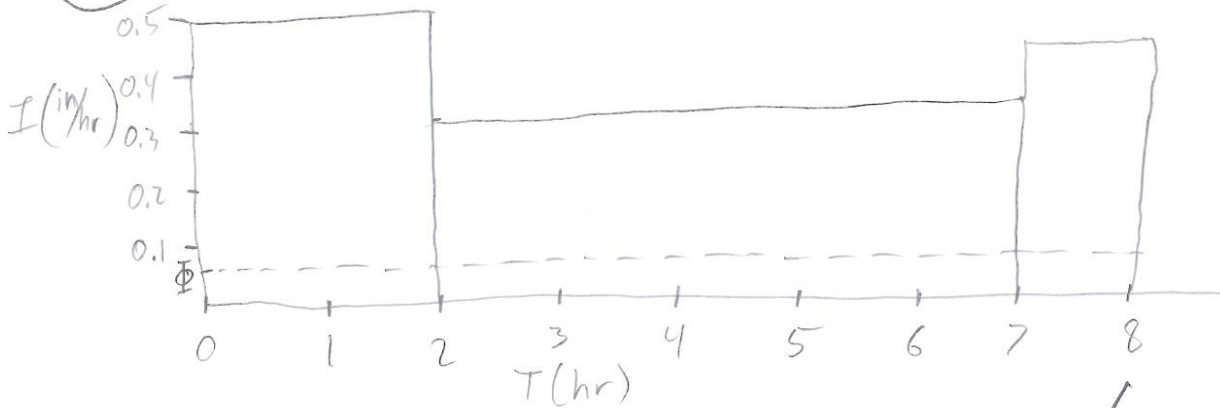
$$E = 47 \text{ in/yr} (4200 \text{ acres}) \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) = 16450 \frac{\text{acre-ft}}{\text{yr}} \checkmark$$

0.5 → 1 to 2
 0.3 → 2 to 7
 0.4 → 8

6.

A watershed covering 600 acres had a storm in which rain fell at a rate of 0.5 in./hr for the first 2 hr, 0.3 in./hr for the next 5 hr, and 0.4 in./hr for the following hour. Determine (a) the total depth of rain that fell during the storm in inches over the watershed and the volume of rain that fell, in acre-feet; and (b) the Φ index for the storm if the runoff is 116 acre-feet.

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$$0.5 \frac{\text{in}}{\text{hr}} (2 \text{ hr}) + 0.3 \frac{\text{in}}{\text{hr}} (5 \text{ hr}) + 0.4 \frac{\text{in}}{\text{hr}} (1 \text{ hr}) = 2.9 \text{ in}$$

$$2.9 \text{ in} (600 \text{ acres}) = 1740 \text{ acre-} \frac{\text{in}}{\text{ft}} \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) = 145 \text{ acre-ft}$$

$$2(0.5 - \phi) + 5(0.3 - \phi) + (0.4 - \phi) = \left(\frac{116 \text{ acre-ft}}{600 \text{ acre}} \right) \left(\frac{12 \text{ in}}{1 \text{ ft}} \right) = 2.32 \text{ in}$$

- Try $\phi = 0.2 = 1.3 \text{ in}$ X
- Try $\phi = 0.1 = 2.1 \text{ in}$ X
- " $\phi = 0.15 = 1.7 \text{ in}$ X
- " $\phi = 0.07 = 2.34 \text{ in}$ X
- " $\phi = 0.08 = 2.26 \text{ in}$ X
- " $\phi = 0.075 = 2.3 \text{ in}$ X
- " $\phi = 0.0747 = 2.3024 \text{ in}$ X
- " $\phi = 0.0741 = 2.308 \text{ in}$ X
- " $\phi = 0.073 = 2.316 \text{ in}$ X
- " $\phi = 0.072 = 2.324 \text{ in}$ X
- " $\phi = 0.0725 = 2.32 \text{ in}$ ✓

$$\phi = 0.0725 \text{ inches}$$

Exel
1-9

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TEST 2

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1. An ephemeral drainage basin has a drainage area of 91.53 mi². The accompanying hydrologic data were obtained for a storm on June 23. The antecedent moisture conditions are considered to be normal.

TIME	Discharge (cfs)		
	JUNE 23	JUNE 24	JUNE 25
0100-June 23		2860 2567	525 471
0200		3370 3025	450 404
0300		3585 3218	375 337
0400		3800 3411	300 269
0500		3575 3209	225 202
0600		3350 3007	150 135
0700		3215 2886	115 103
0800		3080 2765	80 72
0900		3020 2711	40 36
1000		2960 2657	0 0
1100		2810 2522	
1200		2660 2388	
1300		2480 2226	
1400		2300 2065	
1500		2175 1952	
1600	0 0	2050 1840	
1700	100 89.8	1725 1549	
1800	150 134.6	1400 1257	
1900	275 246.9	1235 1109	
2000	550 493.7	1070 961	
2100	975 875	960 862	
2200	1400 1256.7	850 763	
2300	1875 1683	725 650	
2400	2350 2110	600 539	

TIME	RAINFALL ON JUNE 23 (in.)
1600	0
1700	.25
1800	.50
1900	3.00
2000	2.00
2100	0

a) Determine the Φ index (in./hr) for the basin.

1.943" ✓

b) Derive the unit hydrograph ordinates (cfs).

see front page

c) What is the duration of the unit hydrograph?

2 hr. ✓

d) What is the lag time T_p ?

11.45 hr.

9.5

-2

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2. Use the following 2-hr unit hydrograph to determine a 5-hr unit hydrograph.

Time (hrs)	UH (cfs)	5 hr UH ordinate
0	0	0
1	55	22
2	220	88
3	330	154
4	200	168
5	150	214
6	100	192
7	0	126
8		60
9		46
10		0



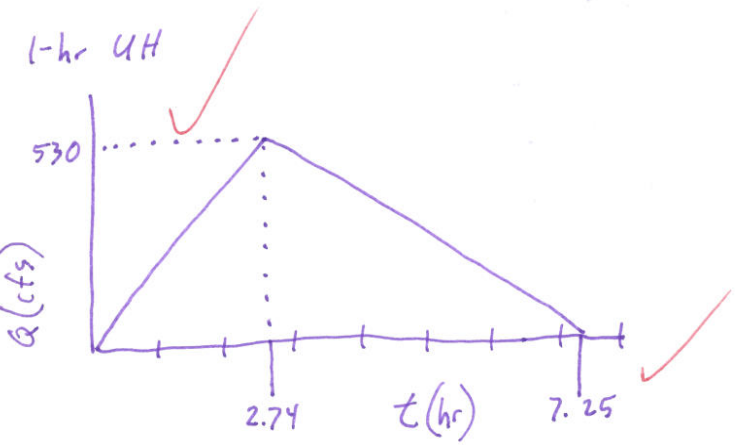
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3. A highway embankment is to be constructed just downstream of a watershed described as follows:

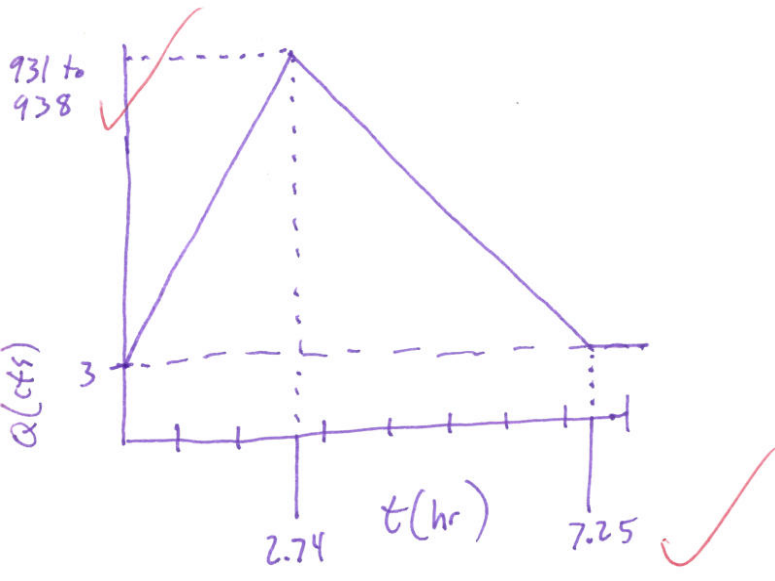
D.A. = 3.0 sq. mi.
length = 2.8 mi.
slope = 1.0%
land use: med. density res (1/4 acre lots)
soils: 70% D and 30% C

Curve # = 86
S = 1.628 in
L = 14784 ft
t_p = 2.24 hr
T_R = 2.74 hr
Q_p = 529.9 cfs
V₀₁ = 1920 ac-in
B = 4.51 hr

- a) Derive a 1 hour unit hydrograph for this basin by the SCS triangular method.
- b) Derive a design hydrograph for this basin assuming it will have 1.75 inches of runoff. Use 1 cfs per sq mi for base flow.



N.T.S



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TEST 2

→ 2551709952 ft²

1. An ephemeral drainage basin has a drainage area of 91.53 mi². The accompanying hydrologic data were obtained for a storm on June 23. The antecedent moisture conditions are considered to be normal.

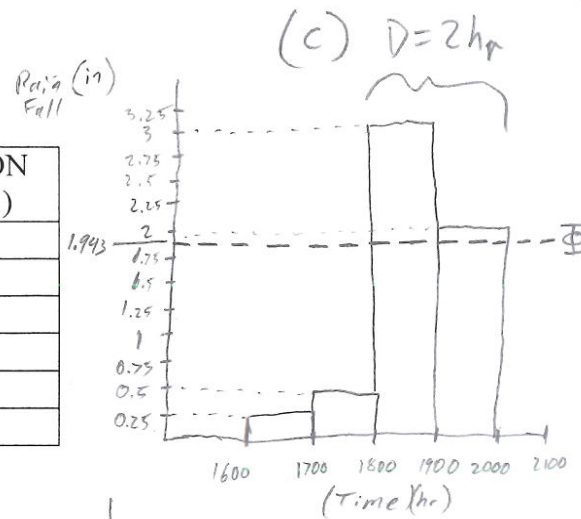
TIME	Discharge (cfs)		
	JUNE 23	JUNE 24	JUNE 25
0100-June 23		2860 2567	525 471
0200		3370 3025	450 404
0300		3585 3218	375 337
0400		3800 3411	300 269
0500		3575 3209	225 202
0600		3350 3007	150 135
0700		3215 2886	115 103
0800		3080 2765	80 72
0900		3020 2711	40 36
1000		2960 2657	0 0
1100		2810 2522	
1200		2660 2388	
1300		2480 2226	
1400		2300 2085	
1500		2175 1952	
1600	0 0	2050 1840	
1700	100 89.8	1725 1549	
1800	150 134.6	1400 1257	
1900	275 246.9	1235 1109	
2000	550 493.7	1070 961	
2100	975 875	960 862	
2200	1400 1256.7	850 763	
2300	1875 1693	725 650	
2400	2350 2110	600 539	
		Σ discharge	65790 cfs (3600s)

= 236844,000 ft³

$t_p = T_R - \text{Center of mass}$
 $= 12 - 0.55 = 11.45 \text{ hr}$

$\left(\frac{\text{mi}}{5280 \text{ ft}}\right)^3 = 0.001609016952 \text{ mi}^3$

TIME	RAINFALL ON JUNE 23 (in.)
1600	0
1700	.25
1800	.50
1900	3.00
2000	2.00
2100	0



- a) Determine the Φ index (in./hr) for the basin.

$\Phi = 1.943''$

- b) Derive the unit hydrograph ordinates (cfs).

see Front page

- c) What is the duration of the unit hydrograph?

2hr

- d) What is the lag time T_p ?

11.45 hr

$$\frac{\text{runoff}}{\text{watershed}} = \frac{236844000 \text{ ft}^3}{2551709952 \text{ ft}^2} = 0.9282 \text{ ft} = 1.1138''$$

$$\begin{aligned} \text{vol. rainfall} &= \text{rainfall [in]} (\text{Area}) \\ &= (0.25 + 0.5 + 3 + 2) (91.53 \text{ mi}^2) \\ &= \left(\frac{1 \text{ ft}}{12''}\right) \left(\frac{5280 \text{ ft}}{\text{mi}}\right)^2 \\ &= 1,222,694,352 \text{ ft}^3 \\ \text{vol. rainfall} - \text{vol. discharge} &= 985,850,352 \text{ ft}^3 \end{aligned}$$

$$(0.25 - \Phi) + (0.5 - \Phi) + (3 - \Phi) + (2 - \Phi) = 1.114''$$

$$T_2 \Phi = 1 \rightarrow 1.75, = 1.5 \rightarrow -0.25 = 1.3 \rightarrow 0.55 = 1.09 \rightarrow 1.39 = 1.1 \rightarrow 1.35$$

$$= 1.2 \rightarrow 0.95 = 1.18 \rightarrow 1.03 = 1.17 \rightarrow 1.07 = 1.11 \rightarrow 1.31 = 1.14 \rightarrow 1.19 = 1.15 \rightarrow 1.15 = 1.16 \rightarrow 1.11 = 1.155 = 1.13$$

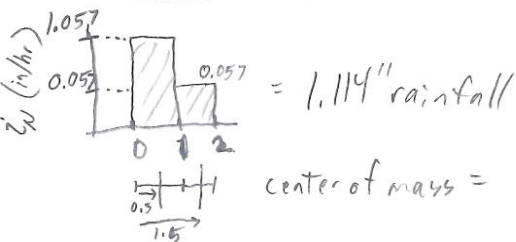
$$= 1.158 \rightarrow 1.118 = 1.1585 \rightarrow 1.116 = 1.1587 \rightarrow 1.1152$$

$$(3 - \Phi) + (2 - \Phi) = 1.114''$$

$$T_{0.5} \Phi = 1 \rightarrow 3 = 2 \rightarrow 1 = 1.9 \rightarrow 1.2 = 1.95 \rightarrow 1.1 = 1.94 \rightarrow 1.12 = 1.945 \rightarrow 1.11$$

$$= 1.944 \rightarrow 1.112 = 1.943 \rightarrow 1.114$$

(a) $\Phi = 1.943''/\text{hr}$



$$\frac{D}{D'} = \frac{2}{5} = 0.4$$

2. Use the following 2-hr unit hydrograph to determine a 5-hr unit hydrograph.

Sum

T	S-Curve Ordinate	S-Curve 5hr Lag	Difference	5-hr UH ordinate	2 hr - UH					Sum	
					Time (hrs)	UH (cfs)					
0	0		0	0	0					0	
1	55		55	22	1	55				55	
2	220		220	88	2	220	0			220	
3	385		385	154	3	330	55			385	
4	420		420	168	4	200	220	0		420	
5	535	0	535	214	5	150	330	55		535	
6	535	55	480	192	6	100	200	220	0	520	
7	535	220	315	126	7	0	150	330	55	535	
8	535	385	150	60	8		100	200	220	0	520
9	535	420	115	46	9		0	150	330	55	535
10	535	535	0	0	10			100	200	220	
								150	330		
								100	200		

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3. A highway embankment is to be constructed just downstream of a watershed described as follows:

D.A. = 3.0 sq. mi.
 Hydraulic length = 2.8 mi.
 slope = 1.0%
 land use: med. density res (1/4 acre lots)
 soils: 70% D and 30% C

P. 99
 Runoff curve Numbers
 38% impervious C=83 D=87
 62% moderate C=74 D=80

- a) Derive a 1 hour unit hydrograph for this basin by the SCS triangular method.
 b) Derive a design hydrograph for this basin assuming it will have 1.75 inches of runoff. Use 1 cfs per sq mi for base flow.

weighted Curve # : $C' = 83(0.38) + 74(0.62) = 77.42$, $D' = 87(0.38) + 80(0.62) = 82.66$

$C'' = C'(0.30) = 23.2$, $D'' = D'(0.70) = 57.86$

weighted Curve # = $C'' + D'' = 81.1$

weighted Curve = $83(30\%) + 87(70\%) = 85.8 = 86$

$S = \frac{1000}{86} - 10 = 1.628$, $L = 2.8 \text{ mi} \left(\frac{5280 \text{ ft}}{\text{mi}} \right) = 14784 \text{ ft}$

$t_p = \frac{14784^{0.8} (1.628)^{0.7}}{1900} = 2.24 \text{ hr.}$

$T_R = \frac{1}{2} + t_p = 2.74 \text{ hr.}$

$Q_p = \frac{484 (3 \text{ mi}^2) \left(\frac{5280 \text{ ft}}{\text{mi}} \right)^2}{2.74 \text{ hr}} [\text{cfs}] = 529.9 \text{ cfs}$, $\text{Vol} = 3 \text{ mi}^2 \left(\frac{5280 \text{ ft}}{\text{mi}} \right)^2 \left(\frac{1 \text{ ac}}{43560 \text{ ft}^2} \right) (1.75) = 1920 \text{ ac-in}$

$B = \left(\text{Vol} - \frac{Q_p T_R}{2} \right) \frac{2}{Q_p} = \left[1920 \text{ ac-in} - \left(\frac{529.9 \text{ cfs} (2.74 \text{ hr})}{2} \right) \right] \left(\frac{2}{529.9 \text{ cfs}} \right) = 4.51 \text{ hr.}$

$\text{TR Vol} = 1.75 \text{ in} \left(\frac{1 \text{ in}}{12 \text{ ft}} \right) (3 \text{ mi}^2) \left(\frac{5280 \text{ ft}}{\text{mi}} \right)^2 = 12,196,800 \text{ ft}^3$

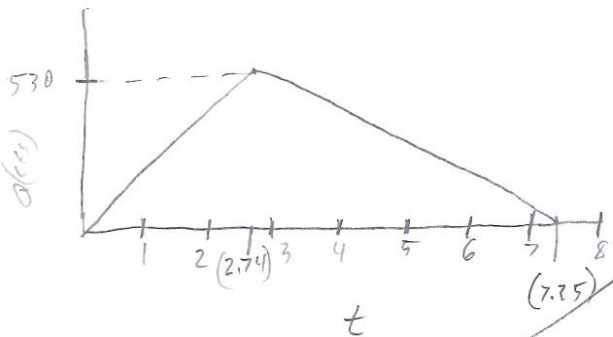
$Q_p = 4H Q_p (1.75) = 928 \text{ cfs}$

check $12,196,800 = Q_p (0.5) (2.74 \text{ hr}) (3600 \frac{\text{s}}{\text{hr}}) + Q_p (0.5) (4.51 \text{ hr}) (3600 \frac{\text{s}}{\text{hr}})$

$Q_p = 935 \text{ cfs}$

baseflow = $\frac{1 \text{ cfs}}{\text{mi}^2} (3 \text{ mi}^2) = 3 \text{ cfs}$

931 - 938



Anne Heath

509-813-9831

can get flow data from
NSGS

Stevens 717-8152
donte 343-0263
moss

Hydro

log pearson

Q (discharge)
for return period
(T_r)

$$Y_T = \bar{y} + k_T' S_y$$

↖ standard deviation of the log transformed data
↗ mean of the log transform data

skew = 0 = normal distributed

C_s = Skew Coefficient

If skew of log transform data
= 0 = log normal distributed

Example: Spring Creek (near Henmora, LA)

Y _r	Q	Y = log ₁₀ X (X=Q)
1971	1900	3.279
72	2240	3.350
3	3090	3.490
4	2000	3.01
5	1680	3.225
6	970	2.987
7	836	2.922
8	900	2.954
9	3220	3.508
80	2860	3.471
1	286	2.456
2	746	2.873
3	5070	3.705
4	3420	3.534
1985	4490	3.652

$$\sum y = 48.707$$

$$\sum y^2 = 159.831$$

$$\sum y^3 = 529.465$$

$$\bar{y} = \frac{\sum y}{n} = \frac{48.707}{15} = 3.247$$

$$S_y^2 (\text{Variance}) = \frac{n(\sum y^2) - (\sum y)^2}{n(n-1)}$$

$$= \frac{15(159.831) - (48.707)^2}{15(14)}$$

$$= 0.119$$

$$S_y (\text{S.D.}) = \sqrt{S_y^2} = 0.346$$

$$g_y (\text{SKEW}) = \frac{n^2(\sum y^3) - 3n(\sum y)(\sum y^2) + 2(\sum y)^3}{n(n-1)(n-2) S_y^3}$$

$$= \frac{15^2(529.465) - 3(15)(48.707)(159.831) + 2(48.707)^3}{15(14)(13)(0.346^3)}$$

$$= -0.78 \rightarrow \text{use } (-0.8) \text{ in table get } k \text{ then make}$$

Recurrence Interval	K	Q
100 yr	6.733	7024
50	6.606	6348
10	6.166	4471
2	0.172	1963

Insert
Log k
3.947
⋮
⋮

HYDRO FEB 22 ①

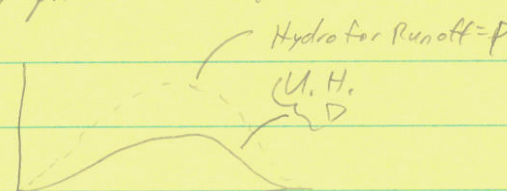
Hydrograph $\xrightarrow{\text{Base Flow}}$ Run-off Hydrograph \rightarrow Vol. run off $\xrightarrow{\text{Direct}}$



Vol. w/ drainage area \rightarrow Hydrograph \rightarrow ϕ index & Duration

Vol / Drain Area = Vol in inches $\xrightarrow{\text{Direct runoff}}$

Unit Hydrograph for duration D
Drainage Area



Runoff $P > 1$ mul. ea. ordinate by P to give hydrograph for duration D

self weight drawings

- 10°
- 60°
- 40°
- 30°

Rational Method $Q_p = CiA$
 i in/hr



D
 $T_c =$ time of concentration

D is wanted for Peak Discharge is when D.A. fully contributes to run off

Synthetic UH Methods

Snyder Method $t_p = C_e (LL_c)^{0.3}$ $L =$ length of main stream from outlet divide

$L_c =$ length of main stream from outlet to point nearest centroid of watershed

$C_e = 0.4$ in mountains to 8.0 Gulf coast

peak discharge at unit Hydro.

$Q_p = 640 C_p A / t_p$
(cfs)

L 0.4 \rightarrow 0.8 (larger values here associate w/ greater value of C_e)

Time base $T_b = 3 + \frac{t_p}{8}$

Hydro march 3 ①

Peak Discharge

Wainage Area

1) Rational Method

$\emptyset < D.A. < 200 \text{ ac.}$

2) SCS Curve Number Method

$200 < D.A. < 2000$

3) USGS Region Method

$2000 < D.A.$

1) Most Engr. say Rational Method =

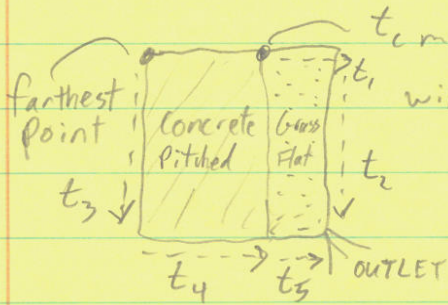
$Q = C I A$
cfs unitless in/hr ac

not correct, correct = $Q = K C I A$ conversion factor = 1,008 to make cfs

C = runoff coefficient

use nomograph to get t_c , to get D , use code to get f
use D, f to get K

T_c = time of concentration (time from outlet to farthest part of watershed)



t_c may be here

will need to use things like Mannings eqn. to find out

$t_{c1} = t_1 + t_2$

$t_{c2} = t_3 + t_4 + t_5$

2) Pick Region (LA has 3)

IDF curve Intensity Duration Frequency curve

$\hookrightarrow D = T_c$

same?

Precipitation - Loss = Runoff $\rightarrow P - E - ET - I = R$

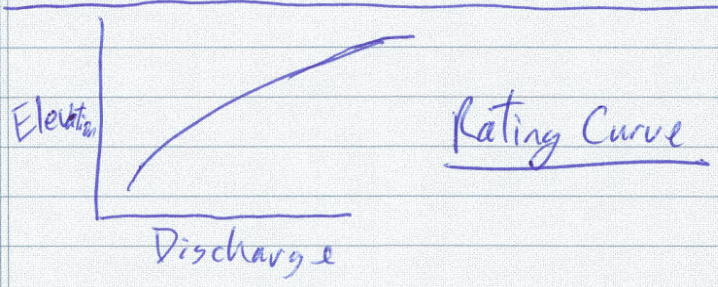
★
test material

Ex: In 2 mo, catchment in Pentypidd^{area} of Wales is expected to receive 254mm of rain w/ expected Evapo-Transpiration (ET) \approx 85mm & that loss to groundwater storage of 20mm. The evap. from a class A pan test for the reservoir was 50mm (this is the equivalent lake evap.)
- what is expected change in storage of a reservoir that has a storage area = 5 sq km (5 km²) if the total catchment area = 70 km²

Runoff = $P - ET - I = 254\text{mm} - 85\text{mm} - 20\text{mm} = 149\text{mm} = \underline{0.149\text{m}}$
 Runoff area = $70\text{km}^2 - 5\text{km}^2 = \underline{65\text{km}^2}$
 Runoff vol = $65\text{km}^2 (0.149\text{m}) \left(\frac{1000\text{m}}{\text{km}}\right)^2 = \underline{9,685,000\text{m}^3}$
 Evap (from lake) = $50\text{mm} \left(\frac{\text{m}}{1000\text{mm}}\right) (5\text{km}^2) \left(\frac{1000\text{m}}{\text{km}}\right)^2 = \underline{250,000\text{m}^3}$
 Precip (on lake) = $254\text{mm} \left(\frac{\text{m}}{1000\text{mm}}\right) (5\text{km}^2) \left(\frac{1000\text{m}}{\text{km}}\right)^2 = \underline{1,270,000\text{m}^3}$

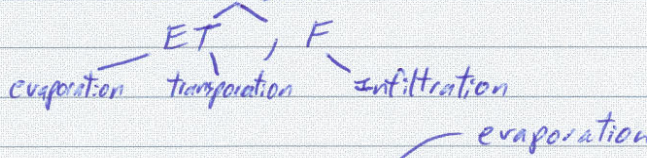
change in storage = $\Delta S = P + R - E = 1270000\text{m}^3 + 9685000\text{m}^3 - 250000\text{m}^3$
 $= \underline{+10,705,000\text{m}^3}$ or $\frac{\Delta S}{A} = \frac{\Delta S}{5\text{km}^2 \left(\frac{1000\text{m}}{\text{km}}\right)^2} = \underline{+2.14\text{m}}$

Basin Avg. Precipitation (BAP)



Hydro Jan 25 ①

$$(P) \text{ Precipitation} - \text{Losses} = (R) \text{ Run off}$$



$$\text{Pan Coefficient} = \frac{E_{\text{lake/reservoir}}}{E_{\text{pan}}} \quad (0.64 \rightarrow 0.81, \text{ U.S. avg} = 0.70)$$



National/State Climate centers have info avail. from tests

Partial pressure (actual vapor pressure)

Saturation vapor pressure (e_s): partial pressure of water vapor in saturated air @ spec. temp.

Relative Humidity %: gravimetric ratio of water vapor in unit space to the water vapor in moist space can hold @ that temp

$$H = \frac{100e}{e_s}$$

Horton's Infiltration eqn.

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

convert to month not s
 $f_c \div \text{Area}$ to give inches/mo

* ϕ Index

$$1 \text{ acre} = 43560 \text{ ft}^2 \quad (I)$$

Problem 1.1: Area_{Lake} = 525 acres, $T = 1 \text{ mo.}$, Inflow = 30 cfs, Outflow = 27 cfs,
 (GW) Seepage = 1.5", $P = 4.25"$, $E = 6"$ $P + I - O - \text{GW} - E = \Delta S$
 Find change in storage (ΔS) = $4.25" + (30 - 27) \text{ cfs} \left(\frac{12 \text{ in}}{\text{ft}} \right)$
 $\Delta S = 0.84"$ = 36.75 acre-ft

Hydro Febl SID ①

1 Acre = 43560 ft²

Test 1 Phi index rating curve polygon

Chap 2

quantity

Q_p

Rational Method

Drainage < 200 acres

SCS CN method

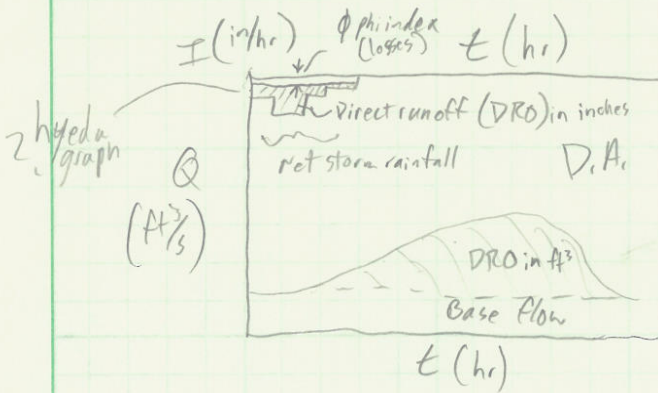
200 acres < D.A. < 2000 acres

USGS-regional method

D.A. > 2000 Acres

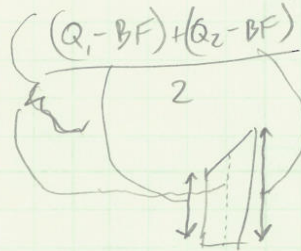
Q/t discharge w/respect to time

Route Q/t



hydrograph

use Remont sums to get area



Separate out base flow

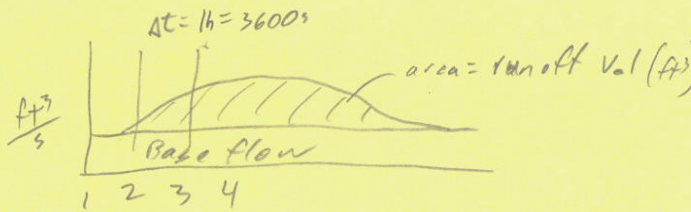
$N = A^{0.2}$

drainage area = A

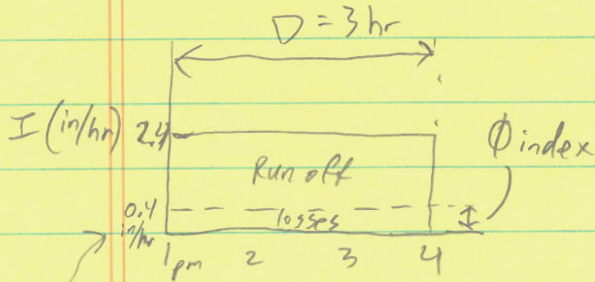
D = Duration = time of precip. excess producing

Hydro Feb 8 510 ①

w.s. A = 421.5 acres



Hye to graph



Hydrograph

Time (hr)	Total discharge (cfs)
1 pm	30
2	30
3	200
4	370
5	540
6	625
7	455
8	285
9	200
10	115
11	30
12 Am	30

Base flow

Rainfall Hydrograph (cfs)

subtract 30	Cumulative runoff (ft³)
0	0
170	$(\frac{0+170}{2})(1h)(3600) = 306000$
340	$\frac{0+170}{2}(3600) = 306000$
510	$\frac{170+340}{2}(3600) = 918000$
595	
425	1530000
255	1989000
170	1836000
85	1224000
0	765000
0	459000
0	153000
0	0

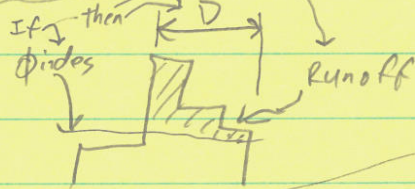
$$\begin{aligned} \Sigma \text{ runoff vol.} &= 9,180,000 \text{ ft}^3 \\ &= \frac{9,180,000 \text{ ft}^3}{421.5 \text{ acres}} \left(\frac{1 \text{ acre}}{43,560 \text{ ft}^2} \right) \\ &= \left(\frac{12 \text{ in}}{1 \text{ ft}} \right) = 6 \text{ inches} \end{aligned}$$

$$P = 2.4 \text{ in/hr} (3 \text{ hr}) = 7.2 \text{ in}$$

$$P - L = R \rightarrow 7.2 \text{ in} - L = 6 \text{ in} \rightarrow L = 1.2 \text{ in}$$

$$\text{intensity of loss} = \frac{1.2 \text{ in}}{3 \text{ hr}} = 0.4 \text{ in/hr}$$

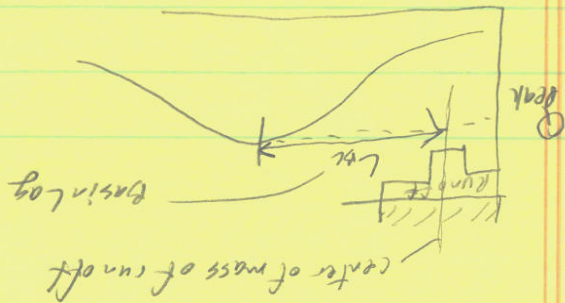
If Hyetograph looked like:



Unit Hydrograph for D = 3 hr.

take cum. runoff & make = lin. i. need to ± Runoff - Base flow

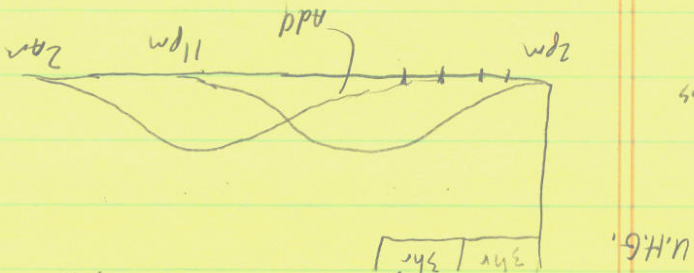
no. want rainfall Hydrograph for 4" runoff D = 3hr	need to ± Runoff - Base flow
0 — % = 0	— 30
0 — % = 0	— 30
170 — 17% = 28.3	— 143.2
340 — 34% = 56.7	— 256.8
— 85	— 370
— 99.2	— 426.6
— 70.8	— 313.2
— 42.5	— 200
— 28.3	— 143.2
— 14.2	— 86.8
— 0	— 30
— 0	— 30



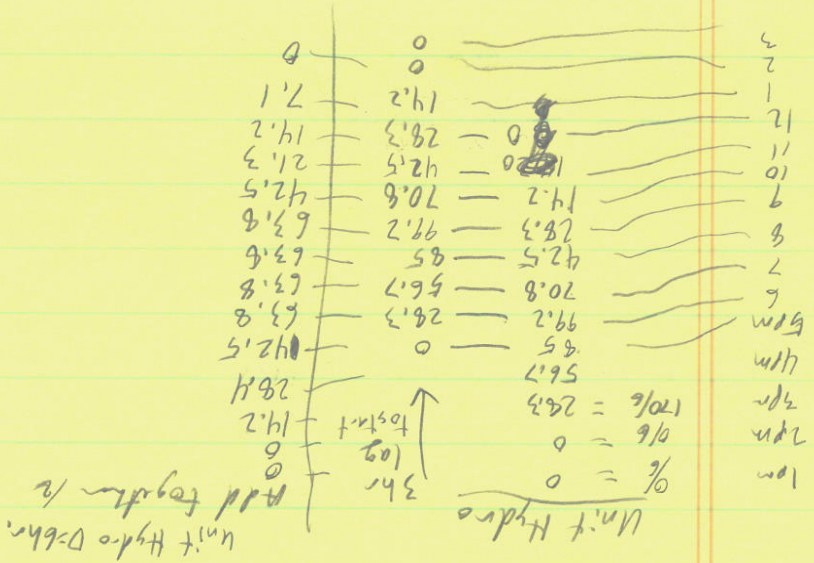
②

Different Duration storms
 U.H. ($D=3\text{ hr.}$) want for a 6hr.
 "Lag" the storm

add another 3hr



Using #s of previous of base



March 24

probability density (pdf) $P(X=a) = \#$

$$\int_{-\infty}^{\infty} f(x) dx = 1$$

$$\sum P(x) = 1$$

cumulative distribution (CDF) $P(X \leq a) = \#$

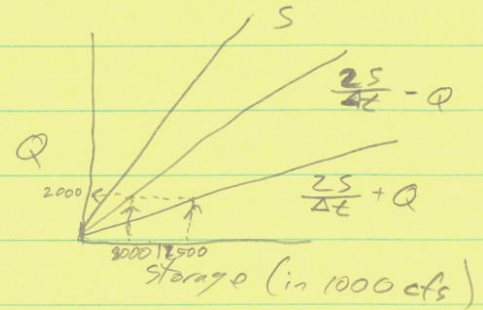
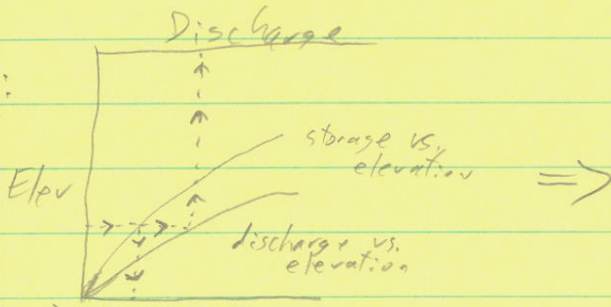
$$0 \leq P(X \leq a) \leq 1$$

$$P(A) = \frac{\#A}{\#\Omega}$$

Water Year Oct 1 \rightarrow Sep 30
ie 2004 \rightarrow ie 2005 } so, Dec. 2004 is in
2005 water year

Hydro March 17 ① chap. 4

Ex. given:



Formula

$$I_1 + I_2 + \left(\frac{2S}{\Delta t} - Q\right)_1 = \left(\frac{2S}{\Delta t} + Q\right)_2$$

time 1 time 2 In Flow Hydrograph

$\Delta t = 12$ hr
convert to sec.

Date	Hr.	I (cfs)	$\frac{2S}{\Delta t} - Q$ (cfs)	$\frac{2S}{\Delta t} + Q$ (cfs)	Q (cfs)
3/8	noon	2000	8500	12500	2000 (out = inflow)
	midnight	2800	8900	13300	2200
3/9	noon	4000	10900	15706	2400
	midnight	5200		20100	3200
3/10	noon	6000			
	midnight	5700			

I_1	3/8	Noon	2000	8500	12500	2000 (b/c $Q_1 = I_1$)
I_2		mid	2800	8900	13300	2200
I_3	3/9	noon	4000			
I_4		mid	5200			
I_5	3/10	noon	6000			
I_6		mid	5700			

Using 2nd chart @ $Q=2000$ Find

Using as x-axis on 2nd chart Find

Using as y-axis on 2nd chart Find



Ex 3.8 $T_R = 20 \text{ yr.}$ $\rightarrow p = \frac{1}{20} = 0.05$ Project complete 3 yr.

Binomial Dist. $B(n, p) = B(3, 0.05)$

$$\text{Prob. failure is exactly once in 3 yr.} \rightarrow p(x=1) = \binom{3}{1} p^1 (1-p)^2 \\ = 3(0.05)(0.95^2) = \underline{0.135}$$

Prob. of occurrence in any 1 yr.

$$(1-p) = 0.95 \quad (\text{prob of not occurring in any one yr})$$

$$(1-p)^n = 0.95^3 \quad (\text{" " " " in 3 consecutive yrs})$$

$$1 - (1-p)^n = 1 - 0.95^3 \quad (\text{" " @ least one occurrence in 3 yrs.}) \\ = 0.143 = \text{Risk}$$

$$\text{Reliability} = 1 - \text{Risk} = 0.857$$

Ex 3.9 110 independent storm events avg duration = 5.3 hr.

$$\text{Year} = 8760 \text{ hrs.}$$

$$8760 - (110(5.3)) = \text{non-raining time}$$

$$t = \frac{8760 - (110(5.3))}{110} = 74.3 \text{ hr} = \frac{1}{\lambda}$$

(avg. time between storm events)

$$\lambda = 0.0135 \text{ hrs}^{-1} \quad p(t \geq 96) = 1 - F(96)$$

$$= e^{-\lambda t}$$

$$= e^{-0.0135(96)}$$

$$= 0.27 \text{ or } 27\%$$



$$p(= \# \text{ exact}) = \emptyset$$

any distribution an exact value $p = \emptyset$

$$p(t \leq 12) = F(x \leq a) = \frac{\text{area under curve} = \emptyset}{F(12)}$$

$$= 1 - e^{-\lambda t}$$

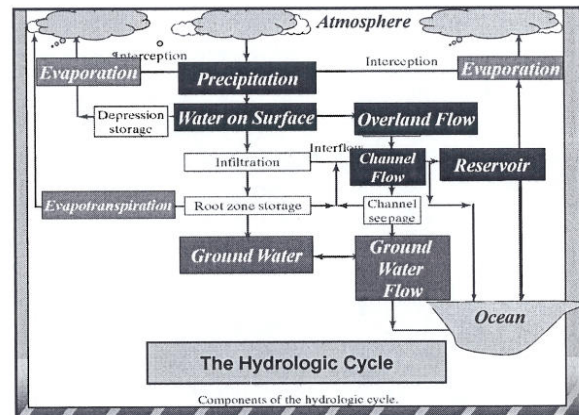
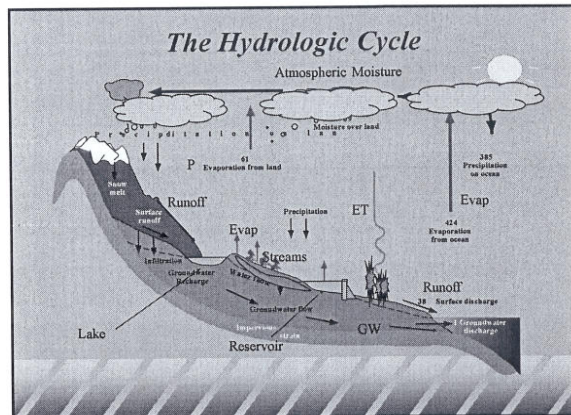
$$= 1 - e^{-0.0135(12)} = 0.15 \text{ or } 15\%$$

Introduction to Surface Water Hydrology

Philip B. Bedient
Rice University

Major Hydrologic Processes

- ◆ Precipitation (measured by radar or rain gage)
- ◆ Evaporation or ET (loss to atmosphere)
- ◆ Infiltration (loss to subsurface soils)
- ◆ Overland flow (sheet flow toward nearest stream)
- ◆ Streamflow (measured flow at stream gage)
- ◆ Ground water flow and well mechanics
- ◆ Water quality and contaminant transport (S & GW)

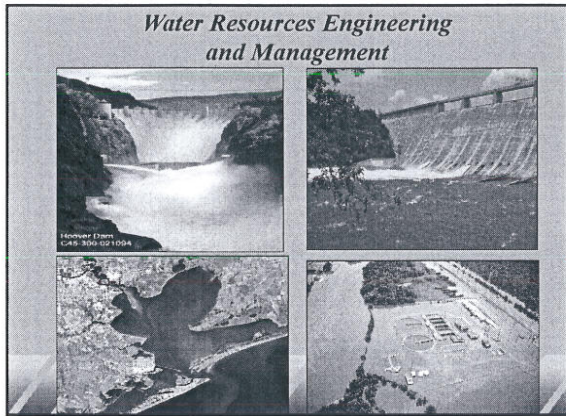


History of Hydrology - 1800s

- ◆ Chezy Channel Formula in the 1780s
- ◆ Open channel flow experiments - 1800s
- ◆ US Army Corps of Eng established (1802)
- ◆ Darcy and Dupuit laws of ground water - 1850s
- ◆ USGS first measured Miss River flow in 1888
- ◆ Manning's Eqn - Open Channel Flow - 1889
- ◆ U.S. Weather Bureau 1891 (NWS)
- ◆ Major Hurricane at Galveston - 1900 (8000 dead)

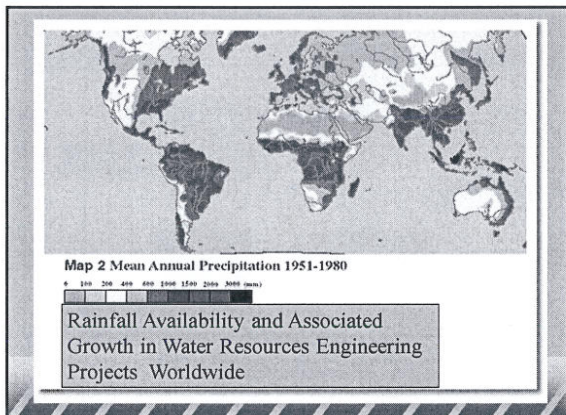
History of Hydrology - 1900s

- ◆ Early 1900s saw great expansion of water supply and flood control dams in the western U.S. - in response to Dust Bowl and the Great Depression of the 1920s & 30s
- ◆ U.S. Dept of Agriculture began many hydrologic studies
- ◆ Sherman UH and Horton infiltration theory - mid 1930s
- ◆ U.S. Army Corps of Engineers (1930s) - large projects
- ◆ Major Hurricane at Florida - over 2000 deaths
- ◆ Penman (1948) - complete theory of evaporation



Recent History of Hydrology

- ◆ Great urban expansion in 1950s and 60s - led to demand for better water supply and prediction (after WW II)
- ◆ EPA formed in 1970 with a mission to clean up the rivers and lakes of America - beginning of environmental science and engineering as we know it today
- ◆ USGS and EPA actively involved in large-scale sampling programs at the national level - (Major Rivers & Lakes)
- ◆ EPA funded development of computer models to address major water quality issues in streams and lakes, and estuarine bays.



Major Computer Advances

- ◆ Stanford watershed Model of 1966 - first digital code
- ◆ US Army Corps of Engineers Hydrologic Engineering Center (HEC) models - 1970s to the present
- ◆ HEC-HMS and HEC-RAS (1990s release)
- ◆ EPA in 1969 - Storm Water Mgt Model (SWMM)
- ◆ USDA and others developed codes in mid 1970s
- ◆ EPA currently supports a suite of advanced models for analyzing water quality in streams and lakes
- ◆ Development of FEMA (1970s) - floodplain mapping and the federal flood insurance program - HEC models

The Watershed or Basin

- ◆ Area of land that drains to a single outlet and is separated from other watersheds by a drainage divide.
- ◆ Rainfall that falls in a watershed will generate runoff to that watershed outlet.
- ◆ Topographic elevation is used to define a watershed boundary (land survey or LIDAR)
- ◆ Scale is a big issue for analysis

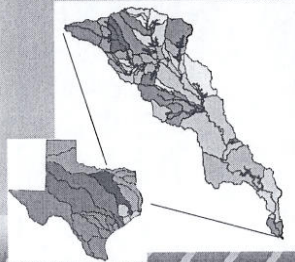
Watershed Characteristics

- ❖ Size
- ❖ Slope
- ❖ Shape
- ❖ Soil type
- ❖ Storage capacity

1 mile

Trinity River Basin DEM Large Basin Scale

Discrete Space Representation



Continuous Space Representation



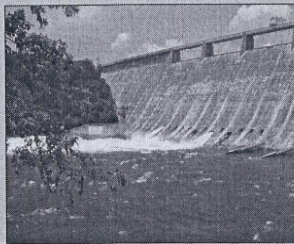
Guadalupe River - Large Basin

- ◆ The Guadalupe River was one of the earliest-explored rivers in Texas
- ◆ Named for Our Lady of Guadalupe by Spanish explorer Alonzo de Leon in 1689
- ◆ Major water supply and recreational river for the hill country near San Marcos - drains to coast
- ◆ Largely spring fed



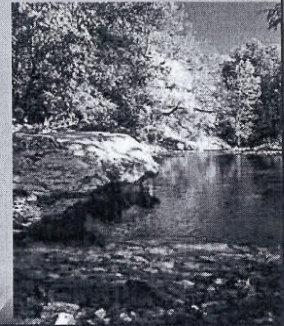
Mansfield Dam - medium scale

- ◆ Mansfield Dam sits across a canyon at Marshall Ford on the Colorado River west of Austin, Texas
- ◆ Built from 1937 to 1941
- ◆ Named in 1941 in honor of U.S. Representative J.J. Mansfield
- ◆ Created a 50 mile long lake that is hundreds of feet deep in lower end

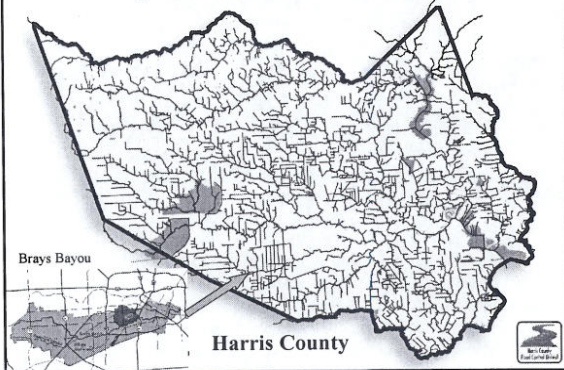


Onion Creek near Austin, TX

- ◆ Limestone area
- ◆ Intense rainfalls
- ◆ Aquifer recharge
- ◆ Very steep slopes
- ◆ High flows
- ◆ Clear water

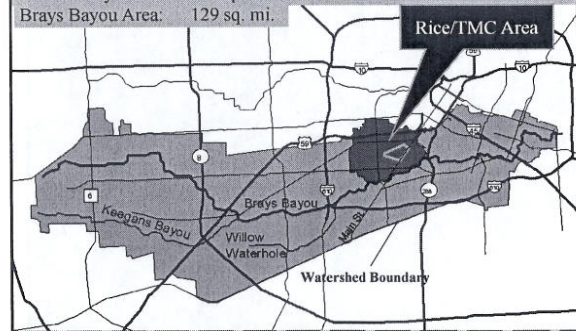


Open Channel Network

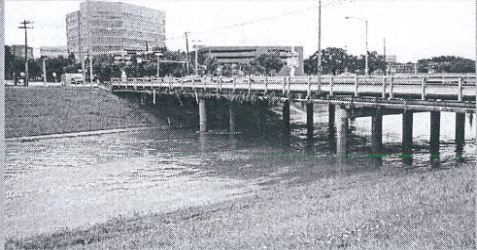


Brays Bayou - small watershed

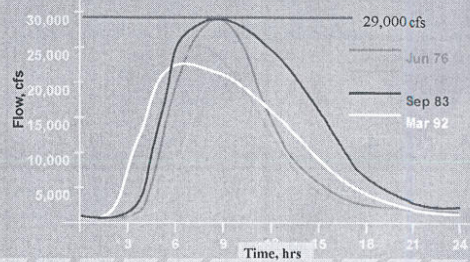
Harris Gully Area: 4.5 sq. mi.
Brays Bayou Area: 129 sq. mi.



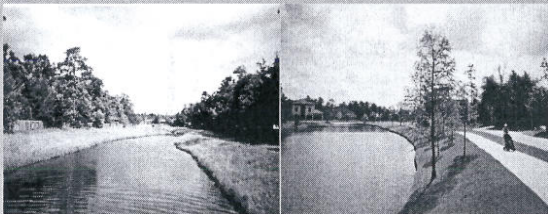
Main St. bridge over Brays Bayou - moderate flow rate



Measured Flow at Main St Gage



The Woodlands - small urban scale



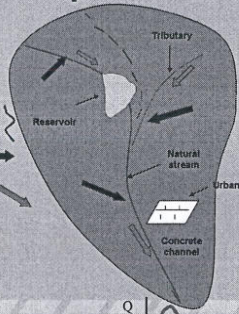
The Woodlands planners wanted to design the ultimate community to handle a 100-year storm. In doing this, they attempted to minimize any changes to the natural floodplain as development expanded.

The Watershed Response - Hydrograph

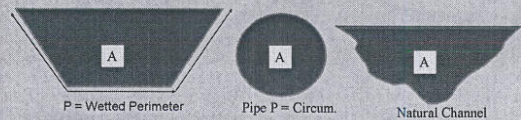
- ❖ As rain falls over a watershed area, a certain portion will infiltrate the soil. Some water will evaporate to atmosphere.
- ❖ Rainfall that does not infiltrate or evaporate is available as overland flow and runs off to the nearest stream.
- ❖ Smaller tributaries or streams then begin to flow and contribute their load to the main channel at confluences.
- ❖ As accumulation continues, the streamflow rises to a maximum (peak flow) and a flood wave moves downstream through the main channel.
- ❖ The flow eventually recedes or subsides as all areas drain out.

Watershed Response

- ❖ Precipitation over the area ↓
- ❖ Portion Infiltrates the soil ↓
- ❖ Portion Evaporates or ET back
- ❖ Remainder - Overland Flow →
- ❖ Overland flow - Channel flow →
- ❖ Final Hydrograph at Outlet



Manning's Equation - Compute Peak Flow



$$Q = \frac{1.49}{n} AR^{2/3} \sqrt{S}$$

- Q = Flowrate, cfs
- n = Manning's Roughness Coefficient (ranges from 0.015 - 0.15)
- S = Slope of channel in longitudinal direction
- R = A/P, the hydraulic radius, where
 - A = Cross-sectional Area of Flow (area of trapezoid or flow area)
 - P = Wetted Perimeter (perimeter in contact with water)

Manning's Equation Open Channels

$$Q = \frac{1.49}{n} AR^{2/3} \sqrt{S}$$

Q = Flowrate, cfs

n = Manning's Roughness Coefficient

S = Longitudinal Slope of Channel or Pipe, ft/ft

R = A/P, where

A = Cross-sectional Area of Flow

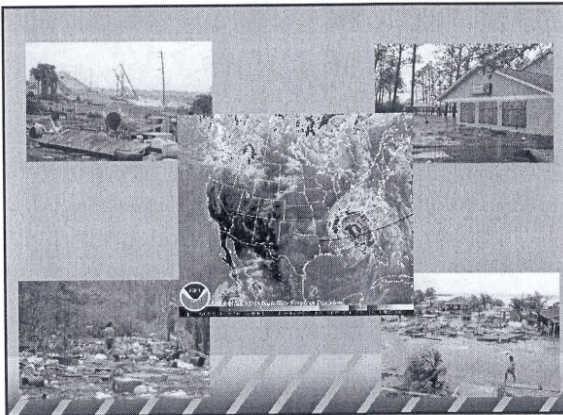
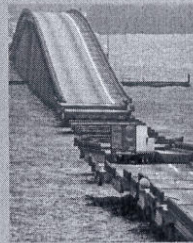
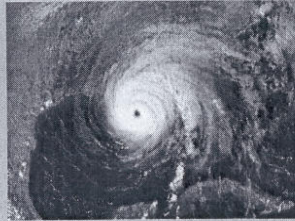
P = Wetted Perimeter

Problems in Hydrology

- ◆ Extreme weather and rainfall variation
- ◆ Streamflow and major flood devastation
- ◆ River routing and hydraulic conditions
- ◆ Overall water supply - local and global scales
- ◆ Flow and hydraulics in pipes, streams and channels
- ◆ Flood control and drought measures
- ◆ Watershed management for urban development

- ◆ Ivan spawned tornadoes from northern Florida into Georgia and Alabama with 22 deaths reported in Florida.
- ◆ Waves as high as 50 feet were measured 75 miles south of Dauphin Island from 130-150 mph winds.
- ◆ Ivan's devastating march across the tropics after it formed was precisely predicted because a big ridge of high pressure steered it as predictably as rails carry a train
- ◆ Created a potential disaster zone of more than 350 miles across.

Hurricane Ivan Sep 04



Technology has Revolutionized the Field of Hydrology


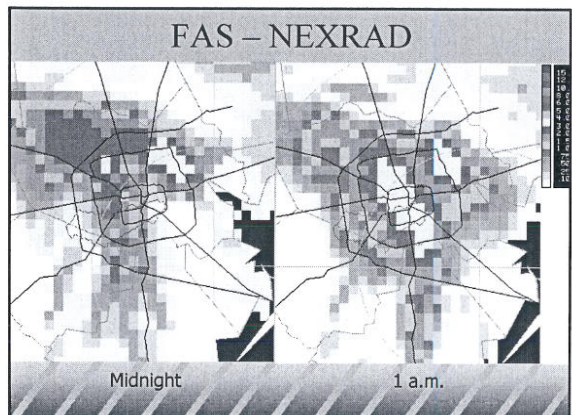
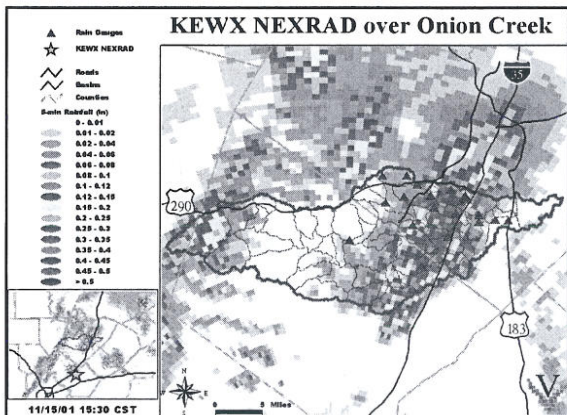
- ◆ High Speed Digital Computation
- ◆ Geographical Information Systems (GIS)
- ◆ Large Hydrologic and Meteorologic Databases
- ◆ GPS and LIDAR methods for ground surveys
- ◆ RADAR rainfall estimates from NEXRAD
- ◆ Advanced forecasting tools for severe weather and flood Alert

QuickTime™ and a
TIFF (LZW) decompressor
are needed to see this picture.

LiDAR Light Detection & Ranging

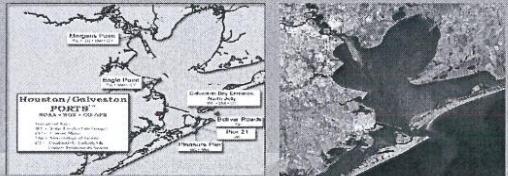
RADAR Rainfall Estimates

- ◆ NEXRAD provides real-time data on a ~16 km² (6 mi²) grid
- ◆ Equivalent to about 21 rain gages in Brays Bayou watershed
- ◆ Each estimate represents an average rainfall amount over the entire 4 x 4 km² area
- ◆ NEXRAD rainfall estimates compare well with point rain gage measurements ($r^2 \sim 0.9$)

Geographic Information Systems

Digital data organized with spatial analysis tools from GIS
Datasets linked to map locations

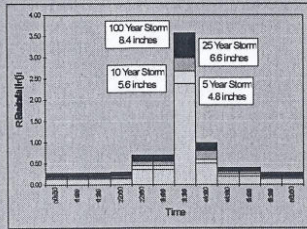


Hydrologic Theory

- ◆ One of the principal objectives in hydrology is to transform rainfall that has fallen over a watershed area into flows to be expected in the receiving stream.
- ◆ Losses must be considered such as infiltration or evaporation (long-term)
- ◆ Watershed characteristics are important

Design Rainfalls

- ◆ Design Storm from
 - HCFC and NWS
- ◆ Based on Statistical Analysis of Data
- ◆ 5, 10, 25, 50, 100 Year Events
- ◆ Various Durations



A Note on Units

- ◆ Rainfall volume is normally measured in inches or cm
- ◆ Rainfall rate or intensity in inches/hr or cm/hr
- ◆ Infiltration is measured in inches/hr or cm/hr
- ◆ Evaporation is measured in inches or in/hr (cm/hr)
- ◆ Streamflow is measured in cfs or m³/s
- ◆ One acre-ft of volume is 43,560 ft³ of water
- ◆ 1 ac-inch/hr is approx. equal to 1.008 cfs
- ◆ Ground water flows are measured as ft³/day or m³/day

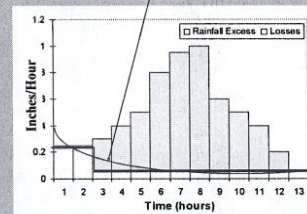
Unit Hydrograph Theory

- ◆ The unit hydrograph represents the basin response to 1 inch (1 cm) of uniform net rainfall for a specified duration.
- ◆ Linear method originally devised in 1932.
- ◆ Works best for relatively small subareas - in the range of 1 to 10 sq miles.
- ◆ Several computational methods exist.

Loss Rate Method:

Initial & Uniform or Horton Method

- ◆ Initial Amount Lost to Infiltration (in)
- Soil is Saturated.
- ◆ Uniform Loss at a Constant Rate (in/hr)

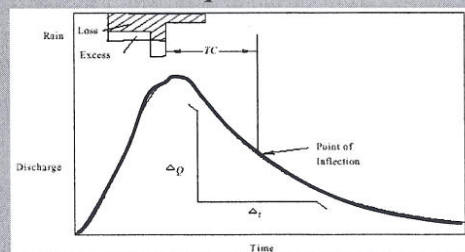


Example: Initial Loss = 0.5 in, Uniform Loss = 0.05 in/hr

Unit Hydrograph Method

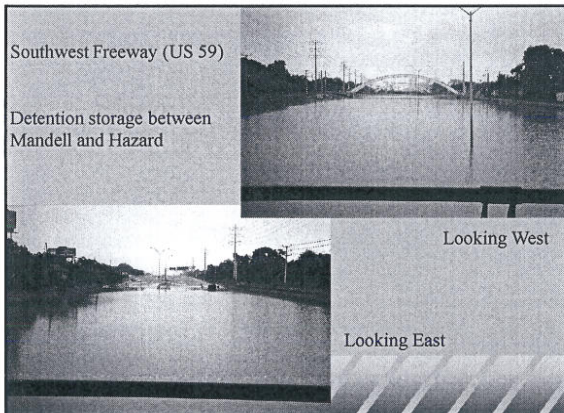
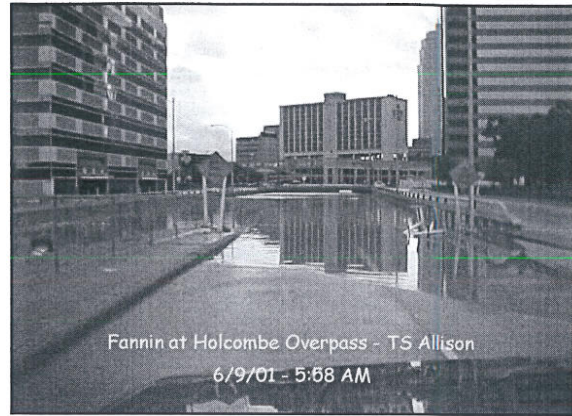
- ◆ Snyder's Method (1938)
- ◆ Clark TC & R Method (1945)
- ◆ Nash (1958) and Gray (1962)
- ◆ SCS Method (1964)
- ◆ Espey-Winslow (1968)
- ◆ Instantaneous UH

Clark Unit Hydrograph (UH) Computation



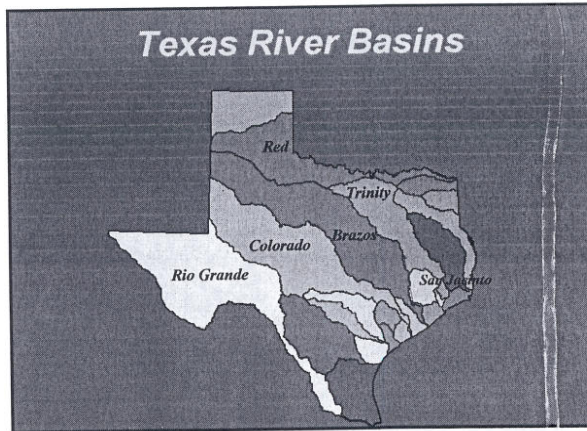
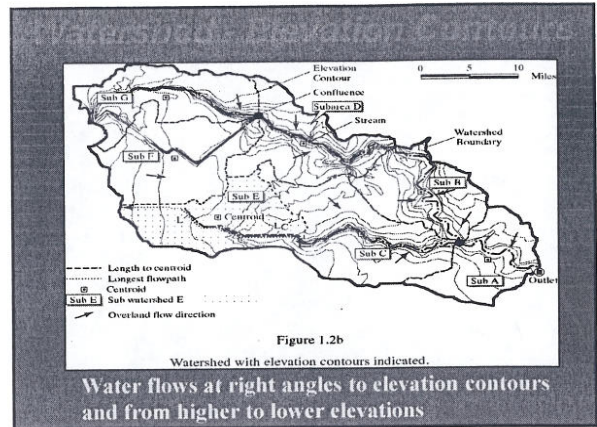
Major Causes of Flooding (Excess Water that Inundates)

- ◆ Highly Developed (urbanized) Area
- ◆ Intensity and Duration of Rainfall
- ◆ Flat Topography with Little Storage
- ◆ Poor Building Practices in floodprone areas
- ◆ No replacement of lost storage as area grows



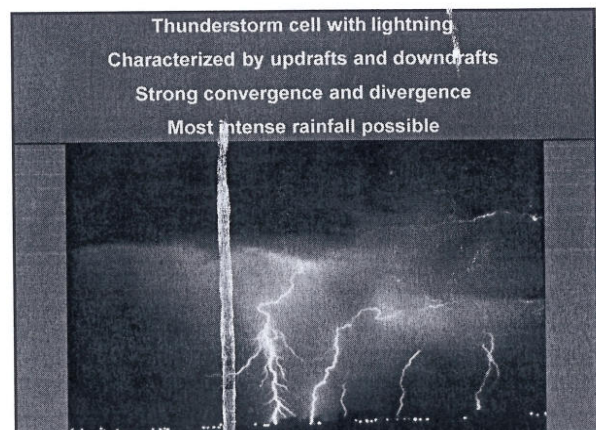
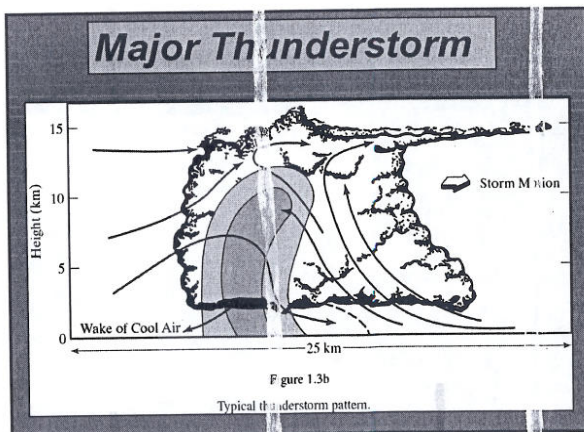
Introduction to Rainfall & Streamflow

Philip B. Bedient



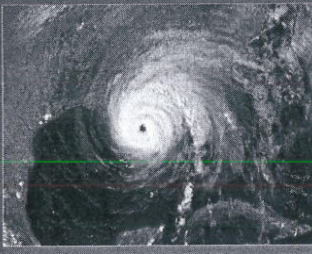
I. Mechanisms of Rainfall

- Convective Storms - Radiational
- Low Pressure Systems - tropical storms and hurricanes
- Frontal Systems - Cold or Warm
- Dew and Fog
- Hail and Ice Storms
- Condensation



Hurricane Ivan September, 2004

- Ivan spawned tornadoes from Florida into Alabama with deaths reported near Panama City and Tallahassee.
- Waves as high as 50 feet were measured 75 miles south of Dauphin Island. Ivan steadily moved northward through Alabama.
- Hurricane Ivan's devastating march was precisely predicted because a ridge of high pressure around the Bahamas steered it
- The combination of slow speed and no clear zones of low and high pressure created the potential disaster zone of more than 350 miles across.



Formation of Precipitation

- Source of moisture
- Lifting mechanism (orographic or heating)
- Phase change from vapor to water - Energy
- Small nuclei or dust for droplet formation
- Droplets must grow as they fall to earth

Lifting Mechanisms

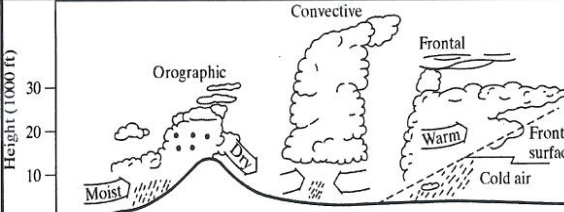
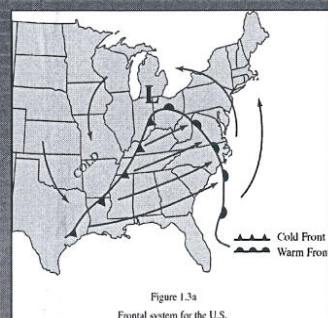


Figure 1.4
Precipitation lifting mechanisms.

Fronts and Low Pressure



- Cold/Warm Front
- Lifting/Condensation
- High and Low Pres
- Rainfall Zone
- Circulation Issues
- Main weather makers

Figure 1.3a
Frontal system for the U.S.

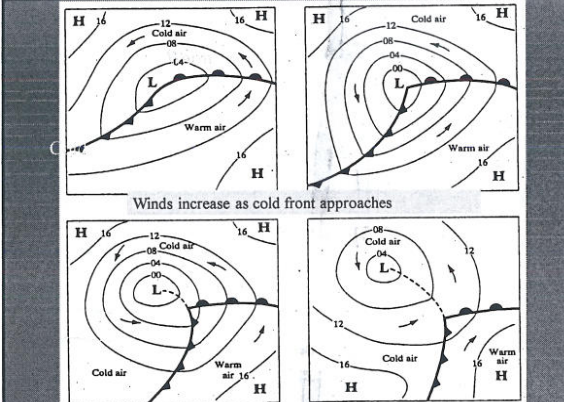
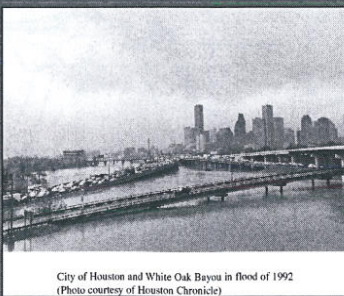


Figure 5.9 Development of a wave cyclone in the Northern Hemisphere.

Major Storm Damages

- Intense Rainfalls
- 8 to 15 inches
- Severe Flooding
- Billion\$ damages
- Lost productivity



City of Houston and White Oak Bayou in flood of 1992
(Photo courtesy of Houston Chronicle)

Hurricane Andrew -1992

- Formed in the Atlantic
- Moved to Florida coast
- Winds in excess of 150
- Major damage to Florida
- Moved over Gulf and strengthened and hit LA
- Most damaging until 2005

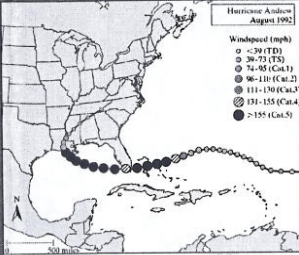
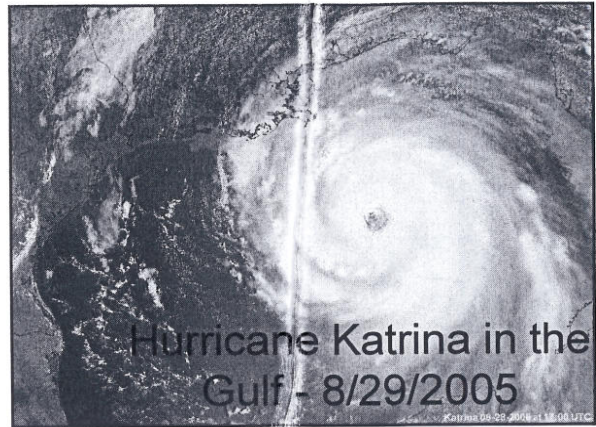
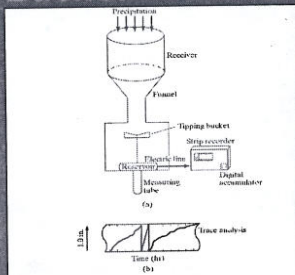


Figure 1.3c
Hurricane Andrew over Florida in August, 1992.



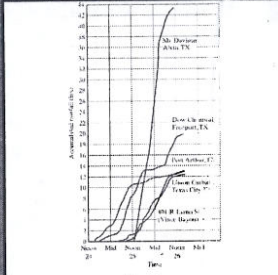
Measuring Rainfall - Tipping Bucket



- Recording gage
- Collector and Funnel
- Bucket and Recorder
- Accurate to .01 ft
- Telemetry- computer
- HCOEM website

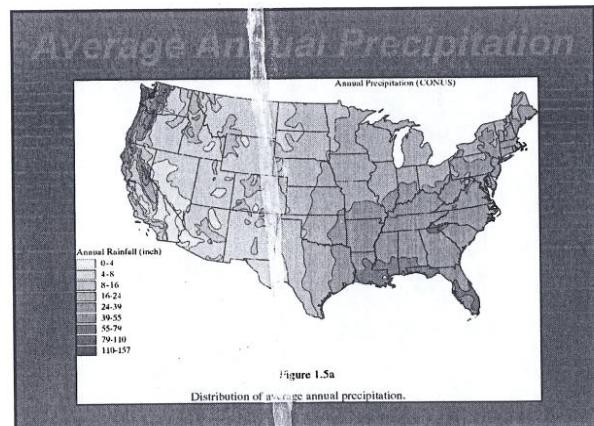
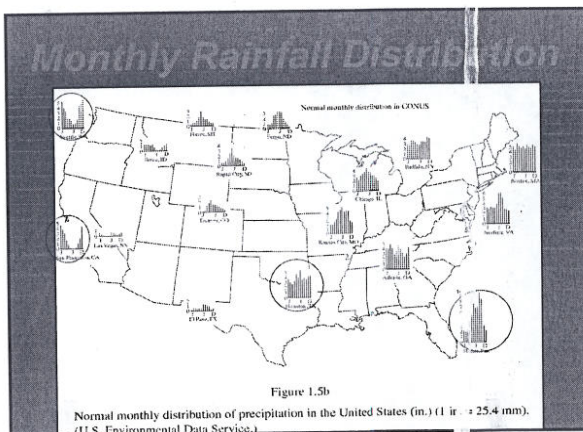
Figure 1.7
Recording tipping bucket gage. Trace returns to zero after each inch of rainfall. The slope of the trace indicates intensity. (in./hr.)

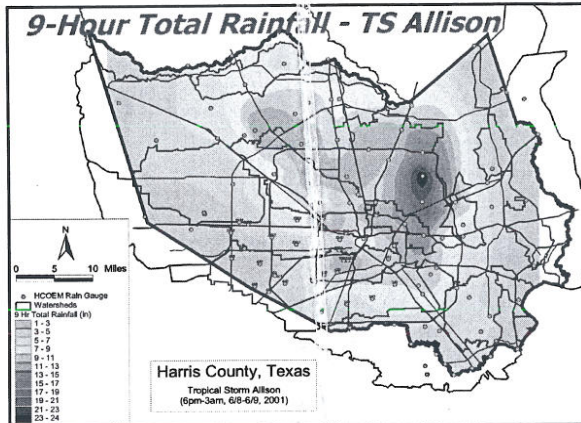
Largest One Day U.S. Total Rainfall



- Alvin, Texas
- "43 inches in 24 hours"
- Measured in one gage
- Associated with T.S. Claudette in July 1979
- Texas accounts for 12 world rainfall records

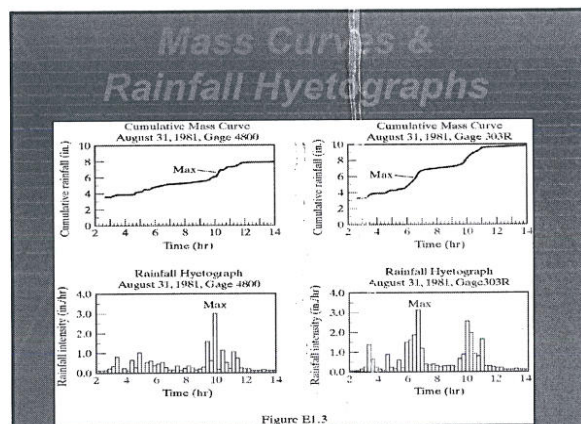
Figure 1.6a
Storm event near Houston, Texas. (a) Accumulated rainfall, July 24-26, 1979.





The Hyetograph

- Graph of Rainfall Rate (in/hr) vs Time (hr) at a single gage location
- Usually plotted as a bar chart of gross RF
- Net Rainfall is found by subtracting infiltration
- Integration of Net Rainfall over time = Direct RO Vol (DRO) in inches over a Watershed



Design Rainfalls

- Design Storm from HCFCD and NWS
- Based on Statistical Analysis of Data
- 5, 10, 25, 50, 100 Year Events
- Various Durations of 6 to 24 hours

Six Hour Rainfall

Rainfall Analysis

Center of Mass of rainfall in time
 $T = \sum t_i P_i / \sum P_i$

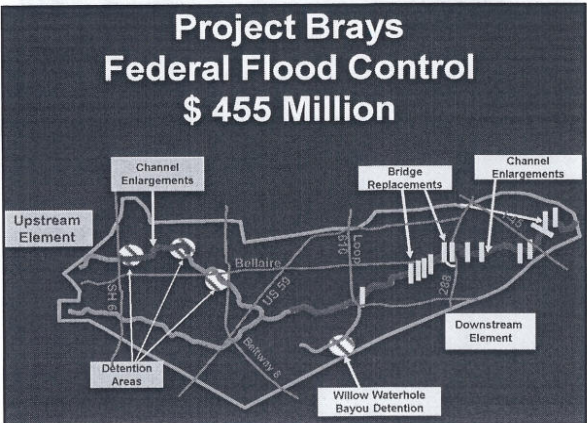
Avg intensity
 $I = \sum P_i / n$

Six Hour Rainfall

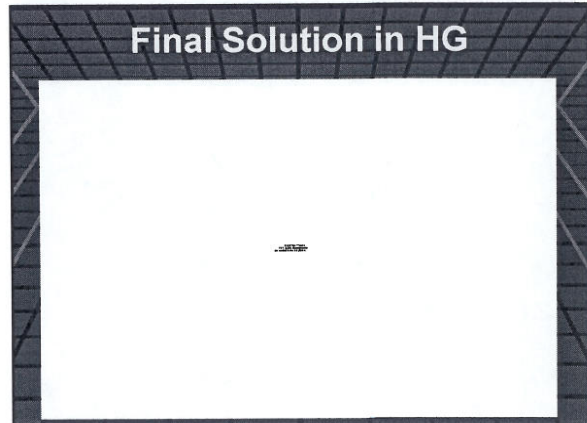
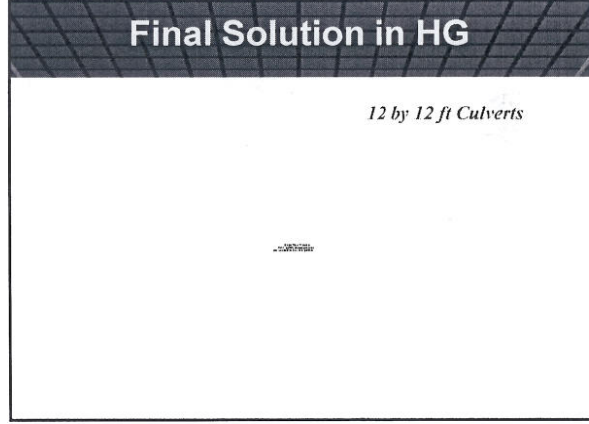
Intensity-Duration-Frequency

- IDF design curves
- All major cities
- Based on NWS data
- Various return periods & durations
- Used for drainage design of pipes & roads
- Used for floodplain designs - watersheds

Figure 1.8
Intensity-duration-frequency curves for Houston, Texas



- ### Final Thoughts...
- The model indicates that under heavy rainfall, serious street flooding results
 - Improve local drainage system
 - Better manage overland flow thru TMC
 - Better use of advanced flood warning
 - Brays Bayou Project – reduces tailwater in the range of 3 - 5 ft in 5 years.



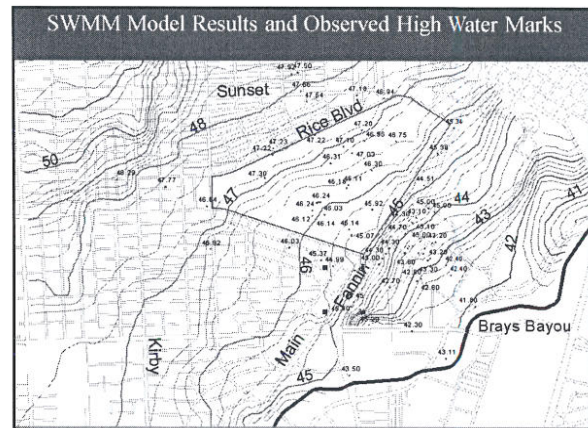
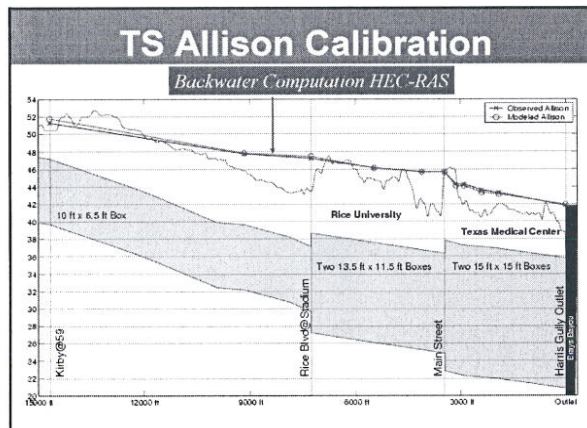
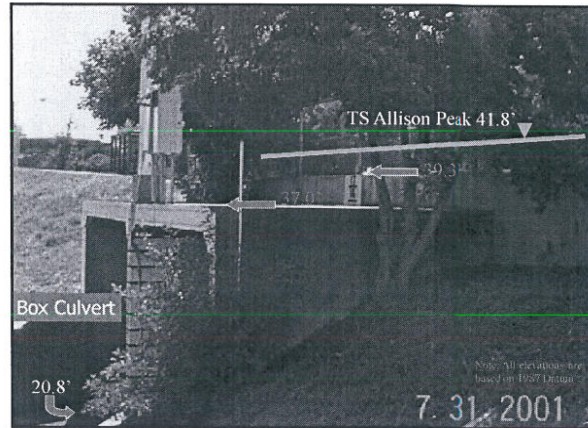
Flood Protection Systems in the Future

Emergency Response - TMC

- Flood Doors
- Flood Gates
- Facility Entrances
- Communications
- Operations
- Training

Modeling the Existing System

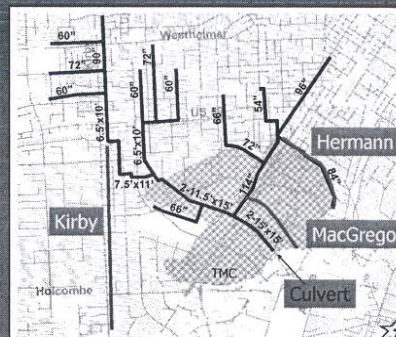
- Enter Major and Minor system structure and connectivity
- Input rainfall data from Allison
- Input tailwater data from Allison
- Run SWMM using Allison input data
- Compare Allison results to observed high water marks near Rice and TMC
- Use calibrated model to analyze various alternatives for the system



Mitigation Options – Harris Gully

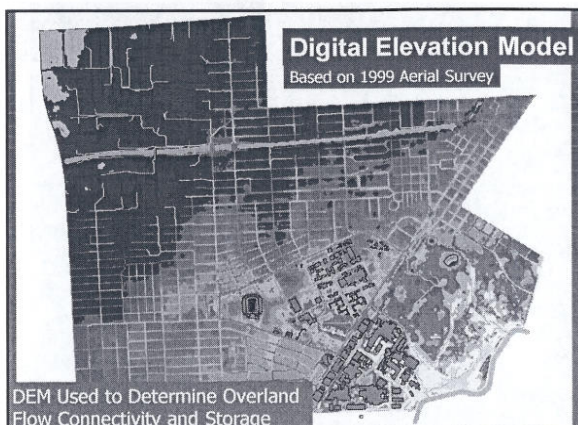
- Use model to evaluate other major culverts
 - Kirby Dr, MacGregor Dr, Hermann Drive
- Evaluate natural overland drainage swale and storage area in the Hermann Park area along the Bayou
- Evaluate all options for Harris Gully including impact of the depressed SW freeway and model their impacts on water levels in TMC area.
- Evaluate effect of Brays Federal Project, which lowers Brays Bayou levels near TMC

Possible Mitigation Alternatives

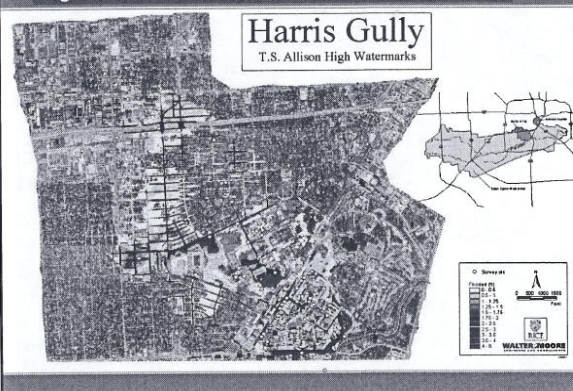


Major System

- All water that cannot flow through the minor system must:
 - Be stored in small or large depressions, or man-made ponds
 - Flow down streets
 - Flow overland as sheet flow
- Major system modeled as storage areas interconnected by weirs.



High Water Inundation in Rice/TMC Basin Area



Tailwater Effects

- High tailwater in Brays reduces the flow capacity of the minor system up to 80%
- When the minor system is full, the major system (streets) starts to fill
- High tailwater will flood some areas near the bayou directly, and cause others to flood due to reduction of flow in the minor system
- *Elevation and Duration of Elevation* are extremely important

Why a Computer Model?

- Analysis tool
 - Why did it flood in the first place?
- Predictive tool
 - How much rain will cause flooding in the future?
- Design tool
 - What can we do to reduce flooding danger?

Rainfall and Stormwater Models

- US Army Corps of Engineers' HEC-HMS
 - Convert rainfall to flows
 - Approximates effects of inlets and leads
- Storm Water Management Model (SWMM)
 - Dynamic hydraulic model of pipes and overland flow and storage
 - Includes tailwater conditions in bayou

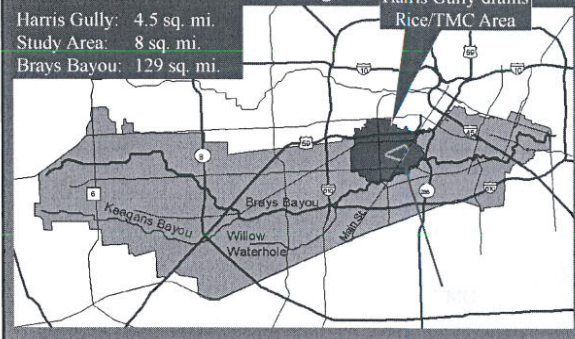
Engineering Research

- This research was funded by:
 - City of Houston
 - Rice University
 - Texas Medical Center
 - Harris County Flood Control District

- Special thanks to Walter P. Moore Associates, Inc. and JF Thompson, Inc. for their assistance on the project

Brays Bayou and Harris Gully

Harris Gully: 4.5 sq. mi.
 Study Area: 8 sq. mi.
 Brays Bayou: 129 sq. mi.

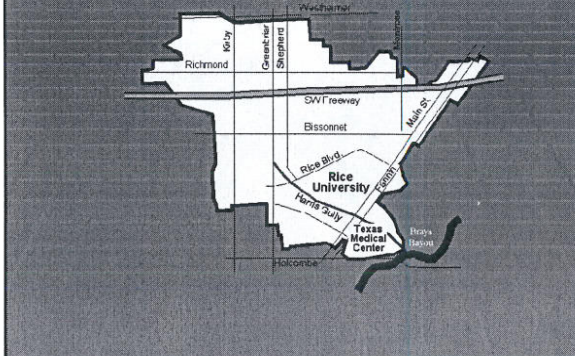


Stormwater Modeling

Four Key Elements

- Rainfall
- Minor Drainage System
 - Pipes, culverts, inlets, leads
- Major Drainage System
 - Streets, sheet flow, open channels, storage
- Tailwater Conditions — Brays Bayou

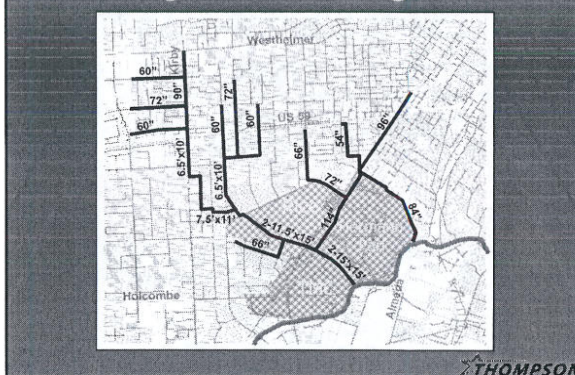
Harris Gully Watershed

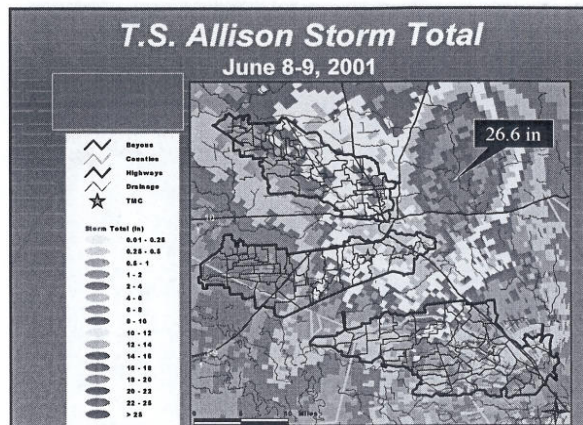
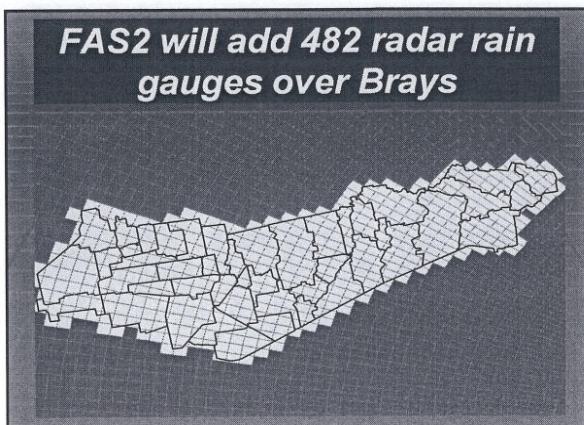


Alumni Drive facing South - 8 am



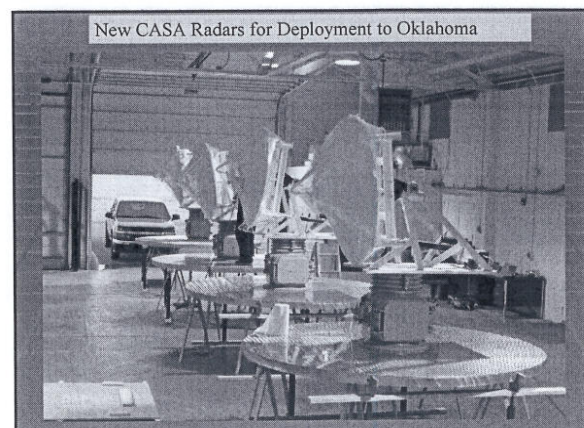
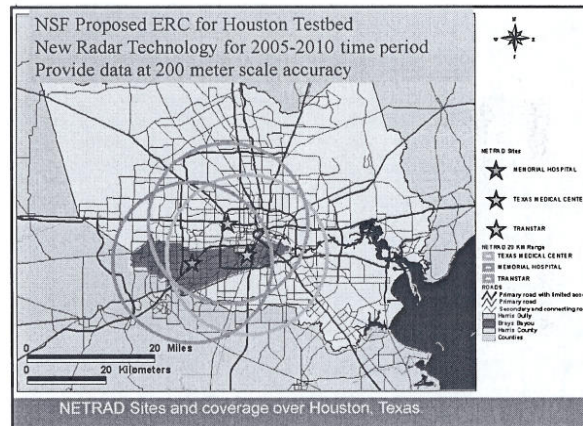
Existing Minor Drainage Network





Prospects for Flood Modeling in the Future

- Forecasting urban streams that respond rapidly to heavy rainfall is difficult.
- Such forecasts can easily underpredict the river stage with little or no lead time
- CASA radars may help revolutionize our ability to see and predict rainfall
- CASA funded as an NSF ERC to several universities including UMass, OU, CSU, Rice



Harris Gully Watershed Analysis

Philip B. Bedient
Civil and Environmental Engineering
Rice University

Weather Radar Systems

- Recently deployed weather radar systems such as NEXRAD offer accurate and reliable precipitation estimation
- Increased sensitivity coupled with improved processing provides high-resolution radar data sets for a variety of applications.
- Provides another source of rainfall information in addition to rain gauges

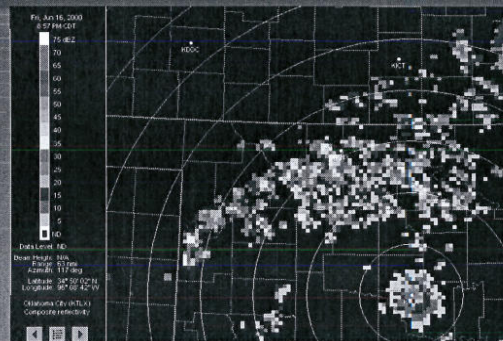
WSR-88D - NEXRAD

- The first operational WSR-88D was installed in May 1990 at Twin Lakes, OK
- 160 + deployed nationwide and overseas.
- Is now being used for much more than weather forecasts.
- Most significant advancement in hydrology in last 20 years!

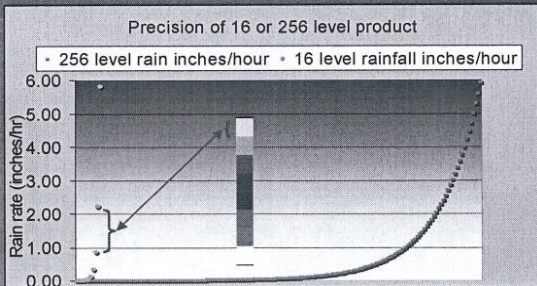
Users of Radar and Meteorological Data

- Real-time access to radar and other meteorological data is now provided to users outside of the NWS
- NEXRAD has spawned a private sector meteorological services industry
- Now other users are beginning to experience the benefits within the hydrologic community

Low Precision 16-level Image

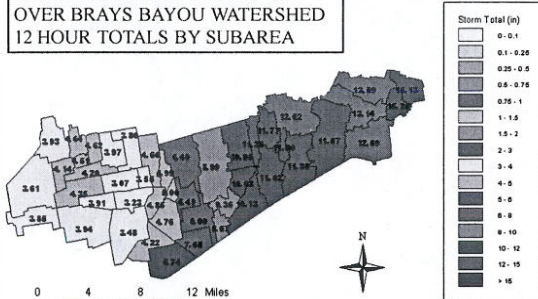


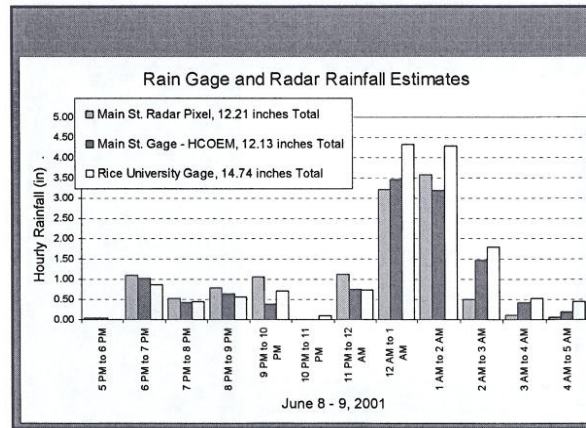
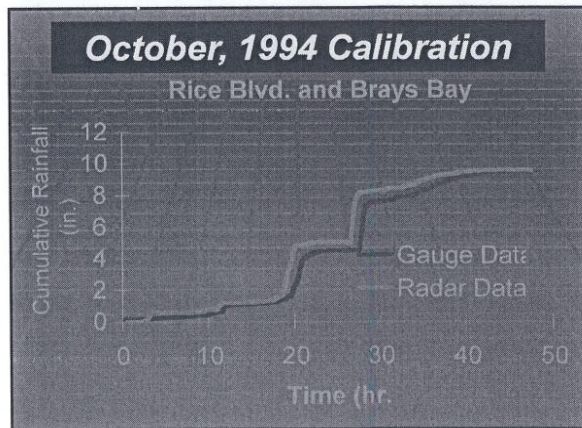
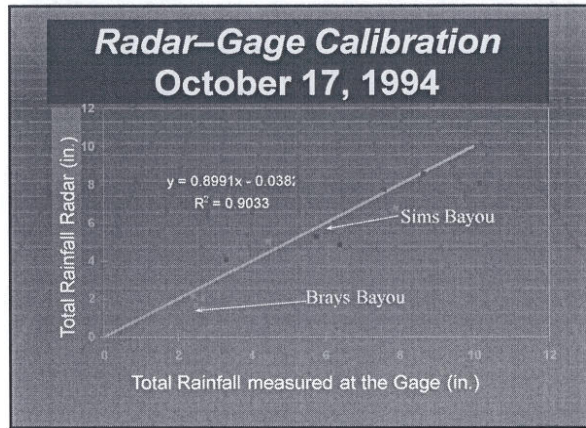
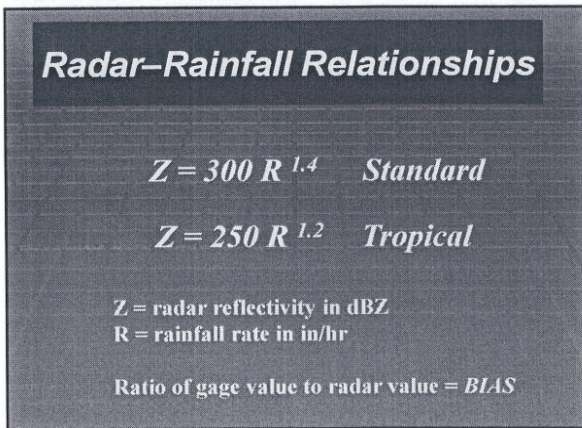
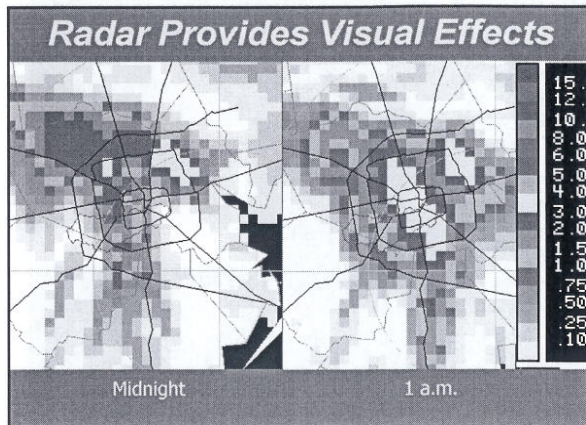
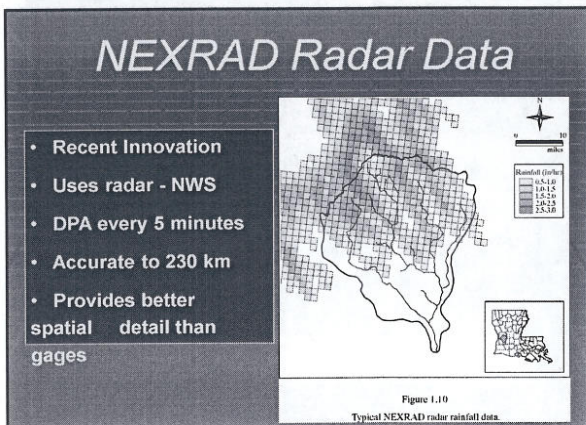
16-level precision image vs. 256-level data



Uncalibrated Subbasin Storm Total for 12-hour period from 6/8 8:00 PM to 6/9 4:00 AM CDT

T.S. ALLISON RADAR RAINFALL OVER BRAYS BAYOU WATERSHED 12 HOUR TOTALS BY SUBAREA





Example of Φ Index

Assume 4.9 in of DRO from a 560 acre Basin

Set up a general Eqn for Φ index

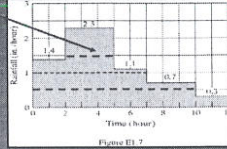
$$2(1.4 - \Phi) + 3(2.3 - \Phi) + 2(1.1 - \Phi) + 3(0.7 - \Phi) + 2(0.3 - \Phi) = 4.9$$

Find Φ by trial and error by assuming a value and solving - try $\Phi = 1.5$ in/hr

And it only accounts for $0.8 \times 3 = 2.4$ in of DRO

$\Phi = 0.5$ in/hr yields 9.0 in of DRO - too much DRO

Try $\Phi = 1.0$ in/hr or $2(.4) + 3(1.3) + 2(.1) = 4.9$ inches



Brays Bayou at Main St Bridge

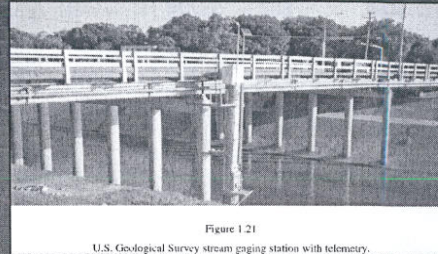


Figure 1.21
U.S. Geological Survey stream gaging station with telemetry.

Stream Cross-Section for Q

- Measure v at 0.2 and 0.8 of depth
- Average v and multiply by $\Delta W \cdot D$
- Sum up across stream to get total Q

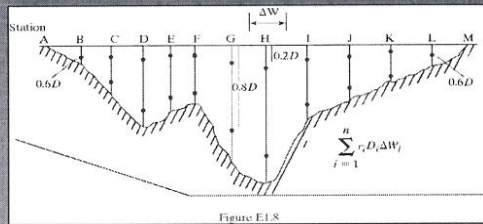


Figure E1.8

Typical Rating Curve for Stream

- Plot of z vs. Q
- Determined from stream measurements of V
- Unique for each stream
- Changes with development
- Available for all USGS gages

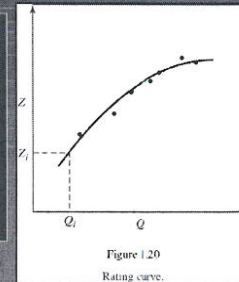


Figure 1.20
Rating curve.

Traditional Flood Alert System

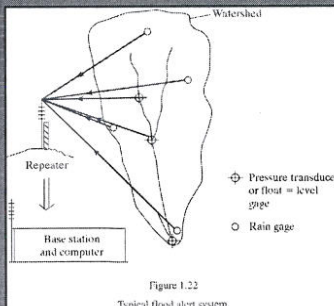


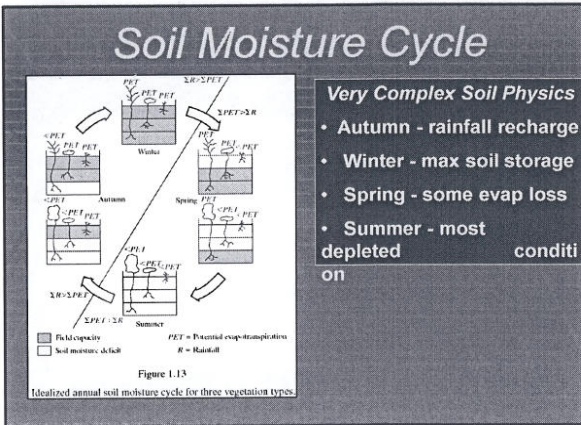
Figure 1.22
Typical flood alert system.

- Use measured rainfall
- Predict hydrologic Response in $x, y,$ and t
- Alert various agencies and emergency mgrs
- Save lives and damages

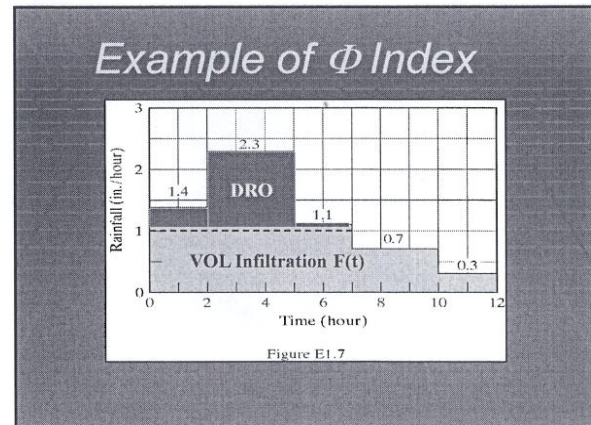
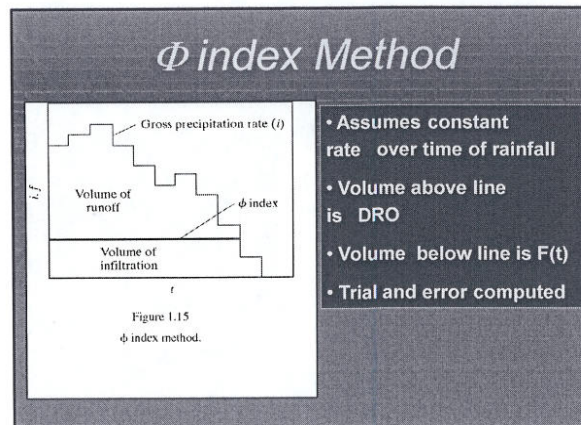
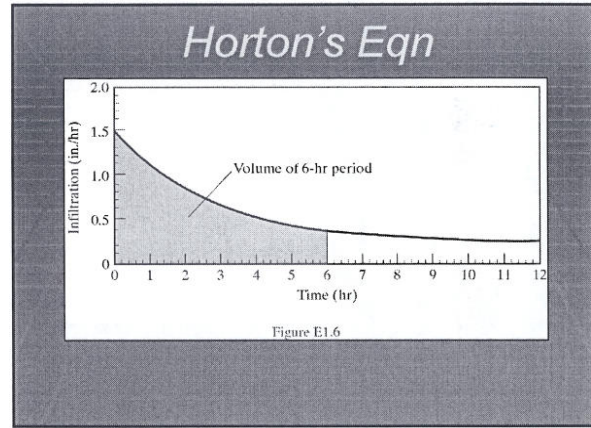
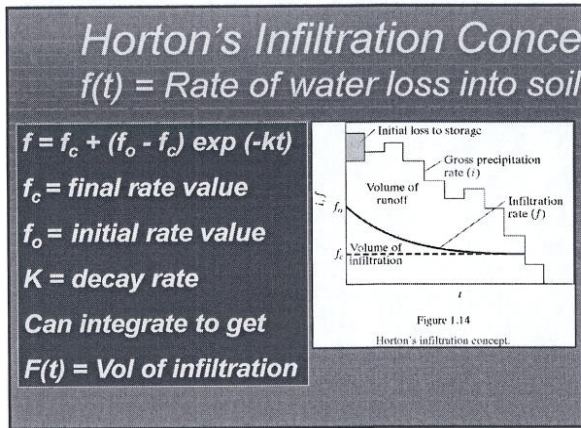
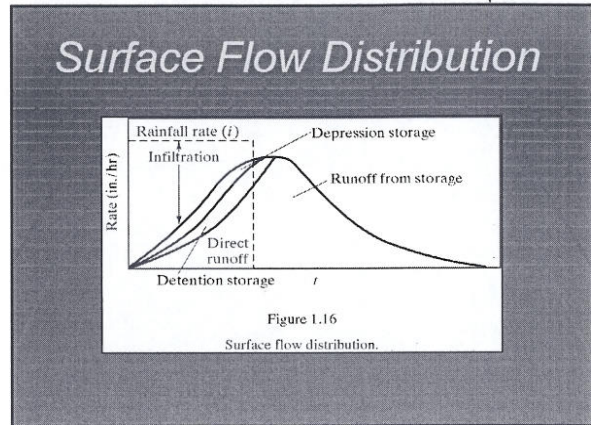
Use of NEXRAD Rainfall for Hydrologic Prediction



Work in Cooperation with
Dr. Baxter Vieux, University of Oklahoma
National Severe Storm Laboratory



- Very Complex Soil Physics**
- Autumn - rainfall recharge
 - Winter - max soil storage
 - Spring - some evap loss
 - Summer - most depleted condition

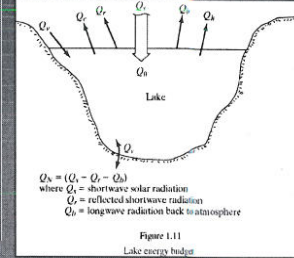


Hydrologic Losses and Radar Measurement

Dr. Philip B. Bedient
Rice University

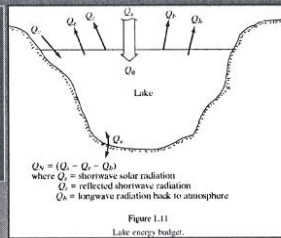
Lake Energy Budget

- Q_e = energy used for evaporation
- Q_h = sensible heat
- Q_g = stored energy
- Q_v = advected energy
- Q_N = net radiation absorbed by water body



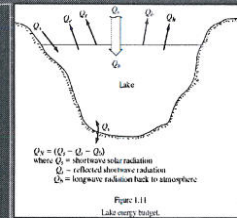
Lake Evaporation

- Function of wind speed, T, and humidity gradient
- Energy source - solar energy
- Mass transfer, energy budget, and pan evaporation
- Penman's combined (1948)



Mass Transfer

- $E = e_s - e_a (a + bu)$
- Where E = evaporation (cm/day)
- e_s = Sat vapor pressure (T)
- e_a = Vapor pres at fixed z
- u = wind speed in m/sec
- a, b = constants



Shallow Lake Evap (Kohler, 1955)

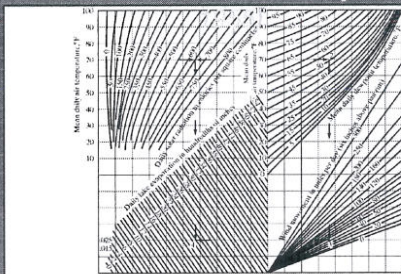


Figure 1.12

Shallow-lake evaporation as a function of solar radiation, air temperature, dew point, and wind movement. Adapted from Kohler et al., 1955.

Evaporation Pans

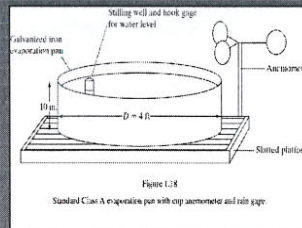
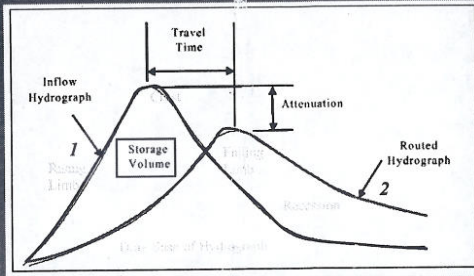


Figure 1.13

Standard Class A evaporation pan with cup anemometer and rain gage.

- Anemometer - wind
- Rain Gage - precip.
- Pan for water - evap
- Level measured daily
- Refilled as necessary

Hydrograph Flood Routing to Next Downstream Location

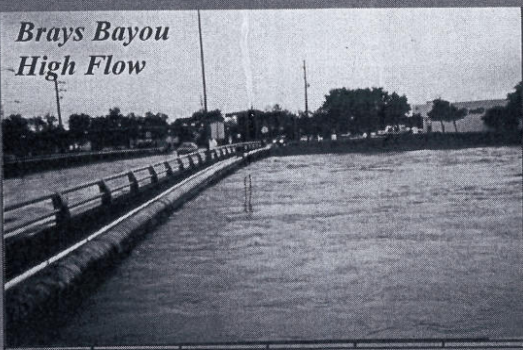


Flood wave is lagged and attenuated as it moves downstream

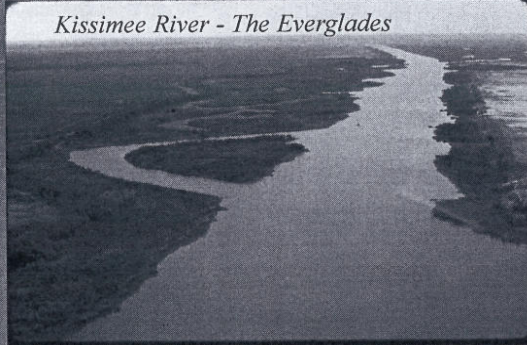
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use needed to see the ppt file.

Excess flows cause major damage in New Orleans

Brays Bayou High Flow



Kissimee River - The Everglades

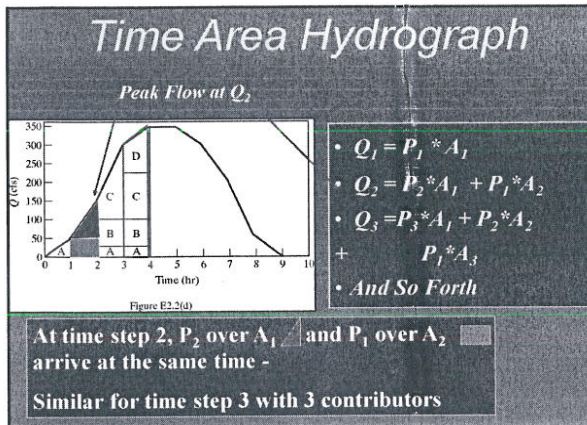


I-45 over Clear Creek - 1979

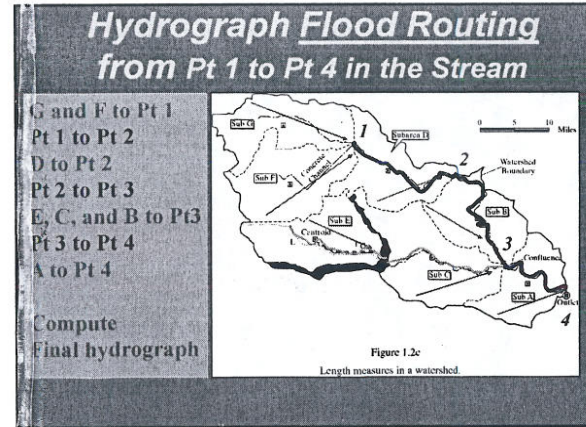
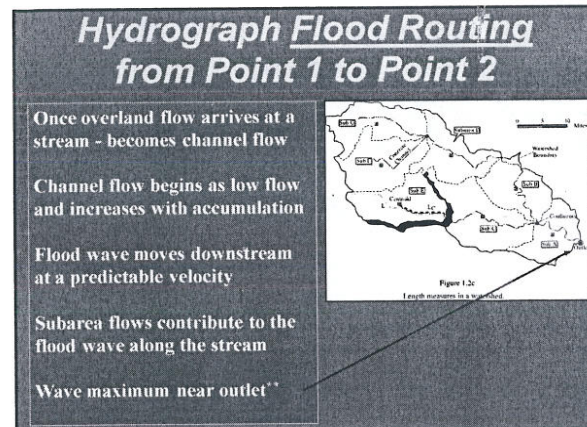
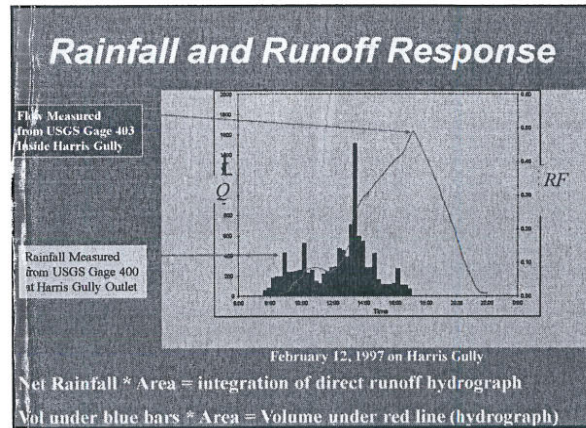
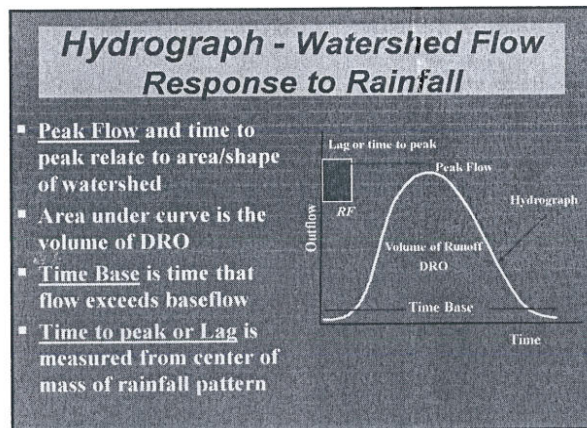


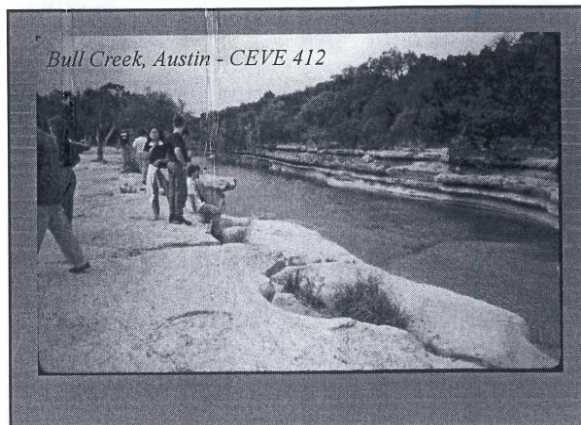
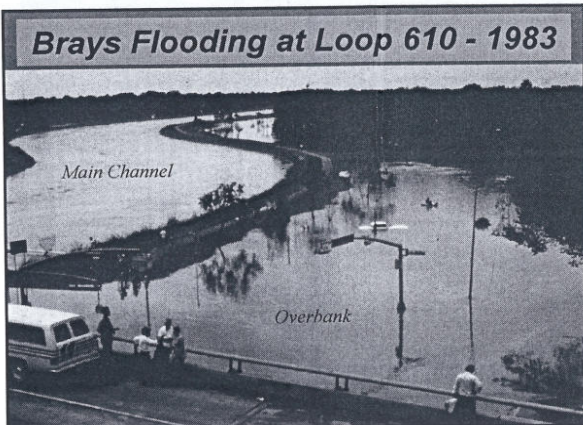
California - Temecula





- ### The Hydrograph
- Graph of discharge vs. time at a single location
 - Rising Limb, Crest Segment, Falling Limb, and Recession
 - Base Flow is usually subtracted to yield DRO
 - Peak gives the maximum flow rate for the event
 - Area under curve yields volume of runoff (inches)





Stream Cross-Section for Q

- Measure V (anemometer) at 0.2 and 0.8 of depth
- Average V and multiply by (Δ width * depth)
- Sum up across stream to get total FLOW
- $Q = \sum (V_i D_i \Delta W_i)$

Figure E1.8

Instantaneous Hydrograph

- Rainfall falls over the basin
- Intensity I reaches the outlet - response based on travel time
- Produces a total storm response hydrograph as shown
- Little delay and no storage
- The above only occurs in small urban basins or parking lots

Unit Hydrograph (UH) Method

- Linear transform method
- Converts complex rainfall to streamflow at outlet
- Produces a total storm hydrograph from given UH
- Used in complex watersheds
- Each subarea is unique
- Storage effects considered

$$Q_n = P_n U_1 + P_{n-1} U_2 + P_{n-2} U_3 + \dots + P_1 U_n$$

Time-Area Method

- Watershed travel times
- Time Area Graph
- Rainfall Intensities
- Add and Lag Method
- Resulting Hydrograph

Figure 2.6
Time-area method.

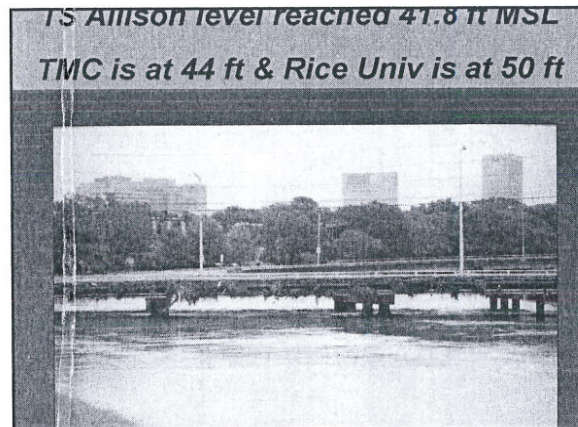
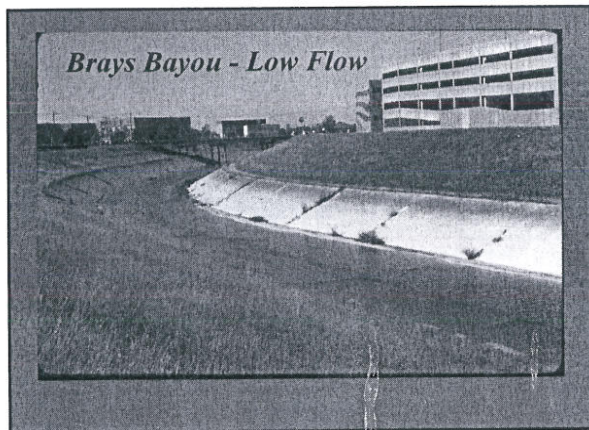
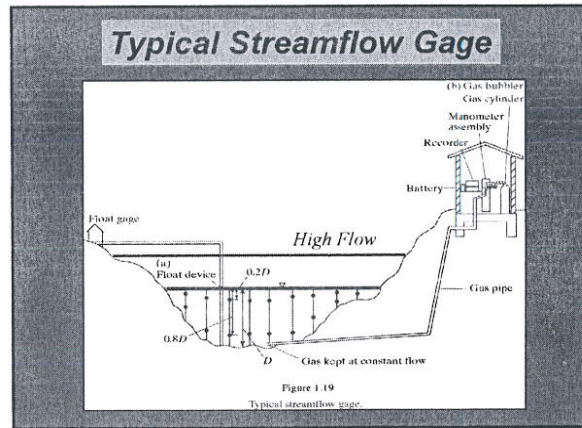
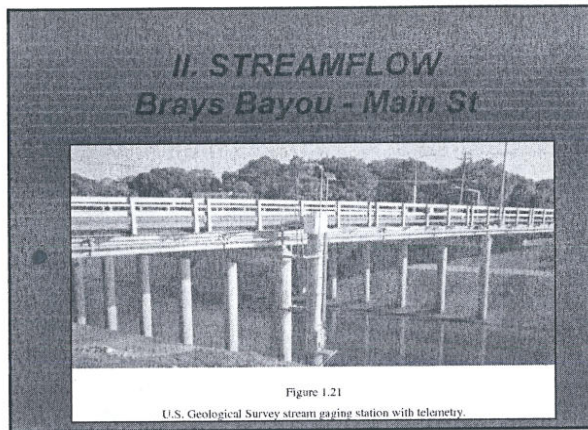
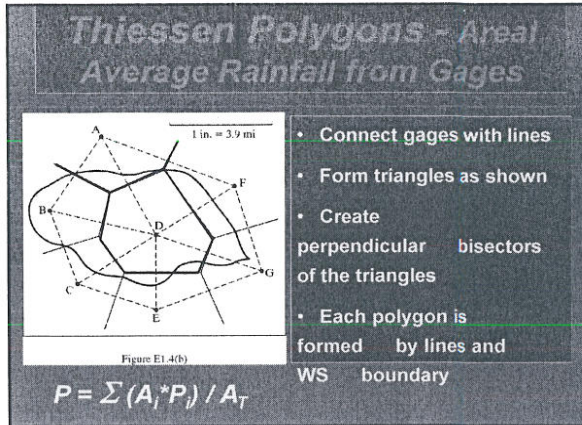
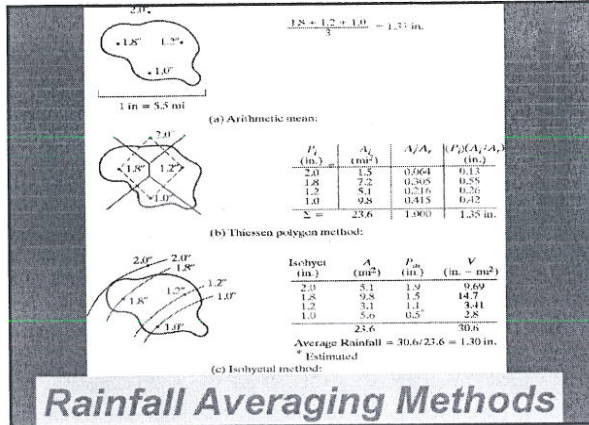


Table 1.1 - Project Design Storm Frequencies for Culverts and Bridges

Design Storm Frequency	15 Years	50 Years	ADT
			> 3000

1.31 Design Criteria

A. Design storm frequency --

When a storm frequency is selected for a particular location, the designer is implying that the estimated effect of a larger storm on property, traffic and the environment do not justify constructing a larger structure at the time. Also, it should be noted that designing for small, frequent storms can result in traffic interruptions and can be costly since expensive repairs and damage to property can be high. The general design storm frequency for a project will be determined as a function of the current average daily traffic (ADT). The project design storm frequencies for culverts and bridges are listed in Table 1.1. The design storm frequencies given in Table 1.1 are considered both typical and minimum. If unusual conditions warrant, a higher frequency may be used. Justification will be required.

A flood hazard evaluation (also referred to as Transmittal 315), as described in Volume 6, Chapter 7, Section 3, Subsection 2, of the Federal Highway Administration's Highway Program Manual will be required for each federal aid project. A major feature of this evaluation is the investigation into the effects of a 100-year frequency flood.

Soil Conservation Service (S.C.S.)
 procedure -
 Reference Number 1
 United States Geological Survey
 (U.S.G.S.) procedure -
 References Numbers 6 and 7
 2000 acres or less
 2000 acres to 3000
 square miles

Table 1.1 - Project Design Storm Frequencies for Culverts and Bridges

	Design Storm Frequency	
	25 Years	50 years
ADT	=< 3000*	> 3000

* A 50 year frequency may be justified at individual sites on a project where:

1. The product of the ADT and the miles of detour which would be required in the event of road closure at the site is greater than 15000.
2. The site is on a primary route for emergency vehicles or community evacuation.
3. Major drainage structures in wetlands (see chapter III).
4. Urban arterial roads and streets.
5. Roads and streets with four or more lanes.
6. Ramps and approach roadways within control of access boundaries of freeways.

A 10 year storm frequency may be used for culverts (not bridges) in rural areas when ADT is less than 400 and the product of the ADT and the miles of detour which would be required in the event of road closure at the site is less than 4000.

B. Methods of computing runoff -

The Department uses two methods to estimate peak runoff rates essential to the hydraulic design of culverts and bridges. The methods vary according to the size of the area contributing runoff to the structure.

Drainage Area	Method and Source
2000 acres or less	Soil Conservation Service (S.C.S.) procedure - Reference Number 1
2000 acres to 3000 square miles	United States Geological Survey (U.S.G.S.) procedure - References Numbers 6 And 7

The United States Geological Survey carries on a continuous program of stream gaging to which the Department of Transportation and Development contributes financially. This stream gaging data (gaging station locations, high water stages, and discharge records) is available to the Department through United States Geological Survey publications and should be used whenever it is applicable to the problem at hand.

1.32 Design Procedure

Design peak rates of runoff are to be based upon conditions which may be expected to exist 20 years in the future. Following discussions establish procedure for estimating peak rates of runoff from ungaged watersheds for use in designing highway drainage structures.

A. Soil Conservation Service (S.C.S.) procedure -

This method is to be used to estimate peak rates of runoff for ungaged rural and urban watersheds of 2000 acres or less. The peak discharge versus time of concentration chart (Figure 1.1) in this Manual was developed by the Louisiana Department of Transportation and Development using Soil Conservation Service method of determining peak rates of runoff. This chart provides a quick and reliable way of estimating peak rates of runoff for a range of factors affecting the peak runoff such as: soil type, land use, cover conditions, rainfall amounts, size of watershed, hydraulic length of watershed, average watershed slope, etc.. 24-hour rainfall with type II rainfall distribution (all areas of U.S. except for parts of Pacific Coast States) and average watershed slopes of between 0.1 and 10 percent were used to develop this chart (Figure 1.1).

The factors affecting peak rate of runoff may be divided into those factors associated with the rainfall and those associated with the watershed.

The rainfall depth (inches), duration, distribution over the duration period, and antecedent rainfall events influence the rate and volume of runoff. 24-hour rainfall maps for 2, 5, 10, 25, 50, and 100 year return intervals were developed for Louisiana from United States Weather Bureau Technical Paper Number 40 and are presented in Figures 1.2 through 1.7. Type II rainfall distribution, which represents all areas of the United States except for parts of Pacific Coast States, is used in peak rate of runoff computations. Peak runoff rate also depends on antecedent moisture condition (AMC) which is determined by the total rainfall in the 5-day period preceding a storm. Average condition (AMC-II) will be used for estimating peak runoff rate in design of highway drainage structures.

The watershed size, shape, slope, soil type and land use all influence the peak rate of runoff. These factors are considered in the development of the equations for time of concentration and runoff. Time of concentration is determined as the flow time from the most remote point in the drainage area to the point under consideration.

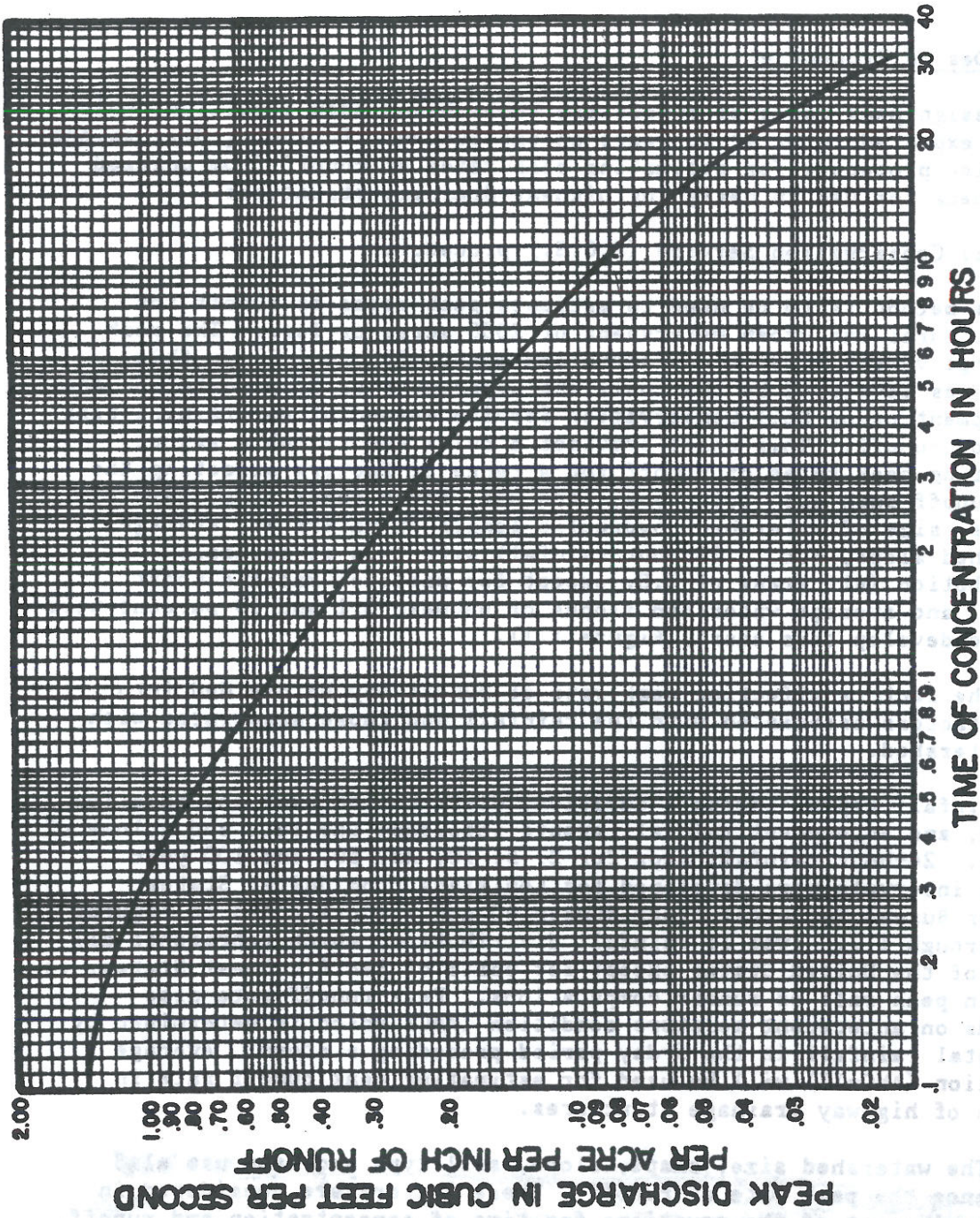


FIGURE I.1 - PEAK DISCHARGE VS. TIME OF CONCENTRATION

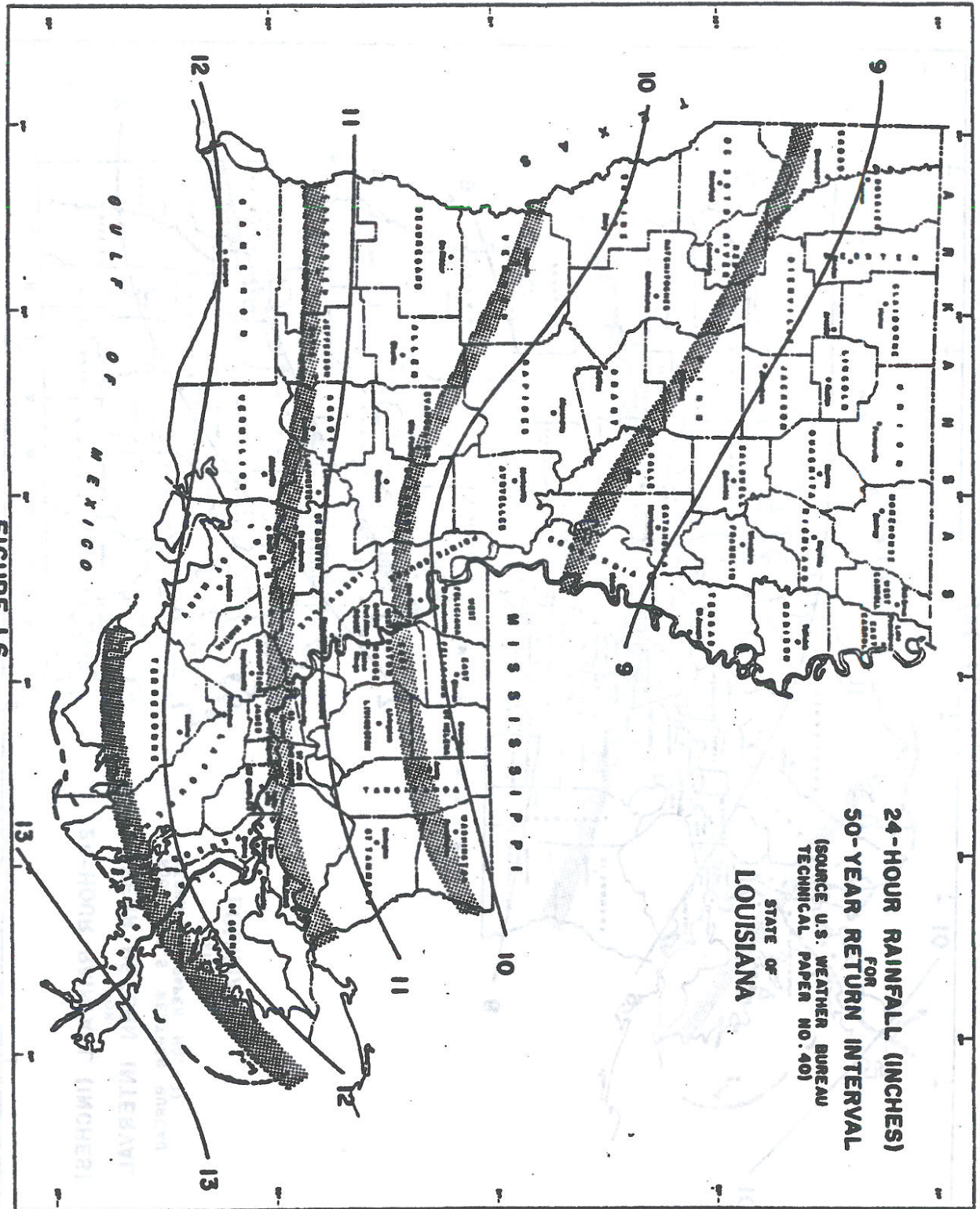


FIGURE I.6

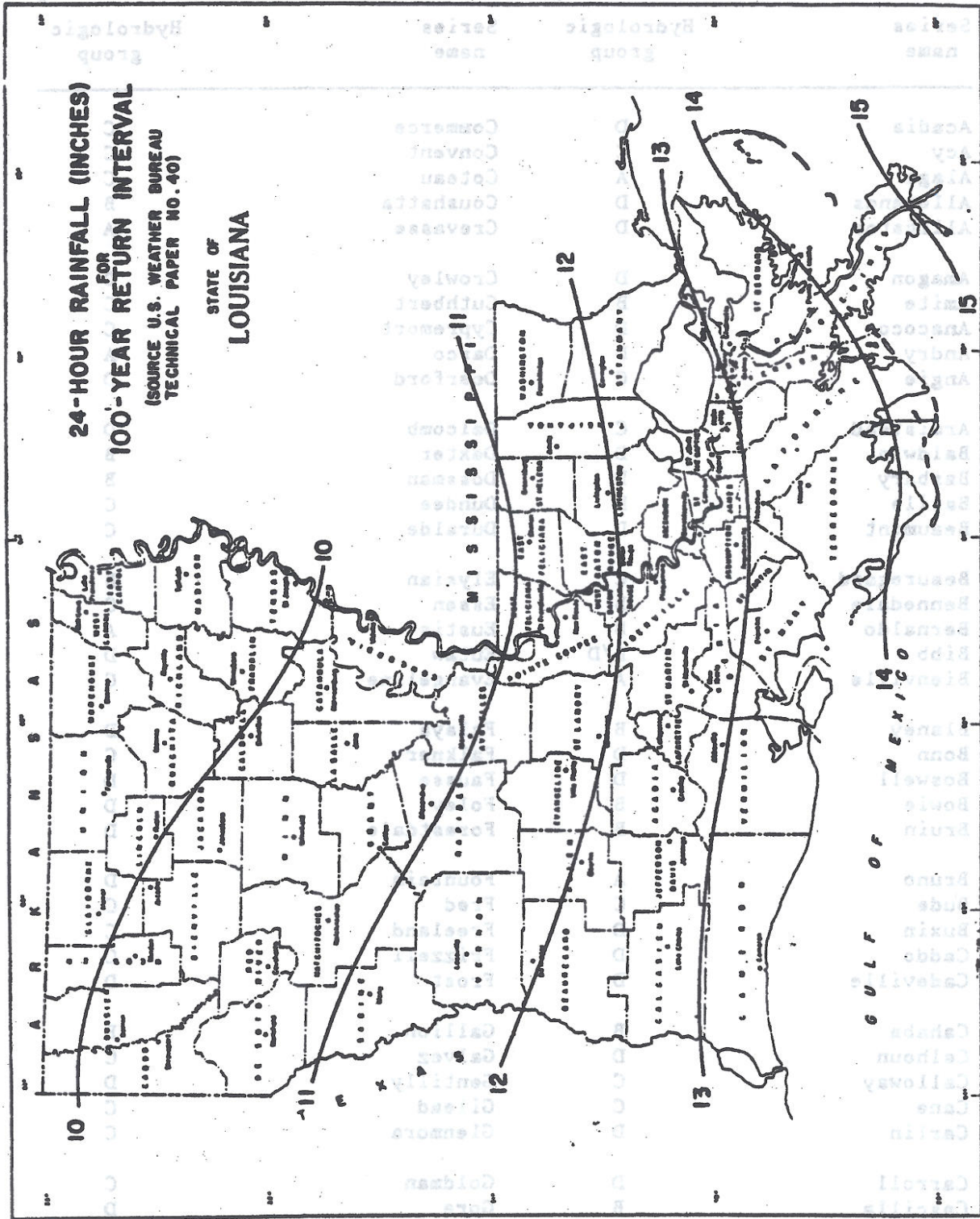


FIGURE I.7

Table 1.2 - Hydrologic Classification of Soils in Louisiana

Series name	Hydrologic group *	Series name	Hydrologic group *
Acadia	D	Commerce	C
Acy	C	Convent	C
Alaga	A	Coteau	C
Allemands	D	Coushatta	B
Alligator	D	Crevasse	A
Amagon	D	Crowley	D
Amite	B	Cuthbert	C
Anacoco	D	Cypremort	C
Andry	D	Darco	A
Angie	C	Deerford	D
Armistead	C	Delcomb	D
Baldwin	D	Dexter	B
Barbary	D	Dossman	B
Basile	D	Dundee	C
Beaumont	D	Duralde	C
Beauregard	C	Elysian	B
Bennedale	B	Essen	C
Bernaldo	B	Eustis	A
Bibb	B/D	Eutaw	D
Bienville	A	Evangeline	C
Blaney	B	Falaya	D
Bonn	D	Falkner	C
Boswell	D	Fausse	D
Bowie	B	Foley	D
Bruin	B	Forestdale	D
Bruno	A	Fountain	D
Bude	C	Fred	C
Buxin	D	Freeland	C
Caddo	D	Frizzell	C
Cadeville	D	Frost	D
Cahaba	B	Gallion	B
Calhoun	D	Galvez	C
Calloway	C	Gentilly	D
Cane	C	Gilead	C
Carlin	D	Glenmora	C
Carroll	D	Goldman	C
Cascilla	B	Gore	D
Caspiana	B	Grenada	C
Chastian	D	Guyton	D
Collins	C	Hannahatchee	C

Table 1.2 - Hydrologic Classification of Soils in Louisiana

Series name	Hydrologic group *	Series name	Hydrologic group *
Harris	D	Mashulaville	B/D
Hebert	C	Maurepas	D
Henry	D	Mayhew	D
Hollywood	D	Mckemie	D
Hortman	D	Mclaurin	B
Houston	D	Memphis	B
Huckabee	A	Mer rouge	B
Iberia	D	Messer	C
Ijam	D	Meth	C
Independence	A	Mhoon	D
Iuka	C	Midland	D
Izagora	C	Miller	D
Jeanerette	D	Moreland	D
Jena	B	Morey	D
Johnston	B/D	Morse	D
Kalmia	B	Mowata	D
Kaufman	D	Muskogee	C
Kenner	D	Myatt	B/D
Kenny	A	Nacogdoches	B
Keo	B	Natchitoches	D
Kirvin	C	Newellton	D
Kisatchie	D	Norfolk	B
Kolin	C	Norwood	B
Kullit	B	Nugent	A
Lafe	D	Ochlockonee	B
Lafitte	D	Okenee	D
Lakeland	A	Oktibbeha	D
Latanier	D	Oliver	C
Leaf	D	Ora	C
Lexington	B	Orangeburg	B
Libuse	C	Osier	B/D
Lintonia	B	Palm beach	A
Loreauville	C	Patoutville	C
Loring	C	Pelhan	B/D
Lucy	A	Perry	D
Lufkin	D	Pheba	C
Luverne	C	Placedo	D
Malbis	B	Pledger	D
Manou	C	Portland	D
Mantachie	C	Prentiss	C

Table 1.2 - Hydrologic Classification of Soils in Louisiana

Series name	Hydrologic group *	Series name	Hydrologic group *
Providence	C	Susquehanna	D
Ragley	D	Tenot	C
Red bay	B	Tensas	D
Rexor	A	Tilden	B
Richland	C	Trebloc	D
Rilla	B	Tunica	D
Robinsonville	B	Una	D
Roebuck	D	Urbo	D
Rosebloom	D	Vacherie	C
Roxana	B	Vaiden	D
Ruston	B	Vacluse	C
Sacul	C	Verdun	D
Savannah	C	Verrett	D
Sawyer	C	Vicksburg	B
Scatlake	D	Vidrine	C
Severn	B	Waller	B/D
Sharkey	D	Waverly	B/D
Shatta	C	Woodtell	D
Shubuta	C	Wrightsville	D
Smithdale	B	Yahola	B
Springfield	D	Zachary	D
Sterlington	B		
Stough	C		
Summerfield	C		
Sumter	C		

* Dual hydrologic soil groups are given for certain wet soils that can be adequately drained. The first letter applies to the drained condition and the second to the undrained from a surface drainage standpoint.

Table 1.3 - Runoff Curve Number (CN) for Selected Agricultural, Suburban, and Urban Land Uses (Antecedent Moisture Condition II)

Land Use	Hydrologic Soil Group			
	A	B	C	D
Woods or forest land	36	61	74	80
Pasture or range land	54	70	80	84
Cultivated land	67	76	83	86
Open spaces, lawns, parks, golf courses, cemeteries, etc.				
Good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover of 50% to 75% of the area	49	69	79	84
Residential				
average lot size average % impervious				
1/8 acre or less 65	77	85	90	92
1/4 acre 38	61	75	83	87
1/3 acre 30	57	72	81	86
1/2 acre 25	54	70	80	85
1 acre 20	51	68	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Paved parking lots, roofs, driveways, etc.	95	95	95	95
Paved streets and roads				
streets with curbs and storm sewers	95	95	95	95
roads	76	85	90	92

CN = weighted runoff curve number (future-condition)

The following design procedure is provided to estimate peak rates of runoff for ungaged watersheds.

1. Prepare a drainage map indicating the following for each drainage area.
 - (A) Drainage area in acres.
 - (B) Hydraulic length of watershed in feet.
Hydraulic length of watershed is the distance the runoff must traverse from the most distant portion of the watershed to the point under consideration.
 - (C) Watershed boundaries.
 - (D) Average watershed land slope in percent.
The weighted average slope should be calculated using elevations along the hydraulic length of watershed.
 - (E) Predicted (for 20 years in future) land use.
 - (F) Soil type(s) and hydrologic soil group(s) forming the drainage area. The soil type can be determined from Soil Conservation Service parish soil survey maps. Hydrologic grouping for the various soil types are included in Table 1.2.
2. Determine the runoff curve number (CN) to be assigned to each drainage area by using the information in steps 1-E and 1-F with the curve number guide (see Table 1.3). A weighted curve number (CN) may be required for watersheds having different soil types and/or land use.
3. Use the 24-hour rainfall maps to determine the inches of rainfall associated with the design frequency storm (Figures 1.2 - 1.7).
4. Compute runoff from following equation:

$$R = \frac{\{ P - 0.2 (1000/CN - 10) \}^2}{P + 0.8 (1000/CN - 10)} \quad (1.1)$$

Where

R = Runoff in inches

P = Precipitation in inches (from Figures 1.2-1.7)

CN = Weighted runoff curve number (future-condition)

5. Compute time of concentration from following equation:

$$TC = \frac{L \left(\frac{1000}{CN} - 9 \right)^{0.8}}{1140 (S)^{0.5}} \quad (1.2)$$

Where

TC = Time of concentration in hours

L = Hydraulic length of watershed

CN = Weighted runoff curve number (future-condition)

S = Average watershed land slope in percent
(a minimum slope of 0.1 percent will be used)

6. Determine peak discharge in cubic feet per second per acre per inch of runoff (Q) from Figure 1.1 using the time of concentration (previously computed in step 5).

7. Compute design peak rate of runoff from following equation:

$$Q_p = A R Q \quad (1.3)$$

Where

Q_p = Peak rate of runoff in cubic feet per second

Q = Peak discharge in cubic feet per second per acre per inch of runoff

A = Drainage area in acres

R = Runoff in inches

Modification of peak rate of runoff where ponding or swampy areas occur in the watershed -

Peak rates of runoff as determined by the use of the previously described procedure assume that the topography is such that you have approximately uniform surface flow into channels, drains, and streams. The peak rates of runoff are approximately the same as those obtained by preparing flood hydrographs and valley flood routing to the point of design. The values obtained by this procedure provide reasonable peak discharges for small watersheds except on flat areas where ponds or swampy areas occur in the watershed. Where these conditions occur, a considerable amount of the surface runoff may be retained in temporary storage and the peak rate of runoff should be reduced to reflect this condition. Preparing hydrographs and flood routing through these areas gives the proper reduction in peaks, but for all practical purposes, the use of the adjustment factors in Tables 1.4, 1.5, and 1.6 provides satisfactory approximations to determine this reduction.

Table 1.4 contains adjustment factors to be used when the ponding or swampy areas are located in the path of flow in the vicinity of the design point. Table 1.5 contains adjustment factors to be used when a significant amount of the flow from the complete watershed passes through ponding or swampy areas and these areas are spread throughout the watershed. Table 1.6 contains adjustment factors to be used when a significant amount of the flow from the complete watershed passes through ponding or swampy areas which are located only in the upper reaches of the watershed.

Whenever ponding modification is applicable the factor is applied to the peak runoff rates as follows:

$$Q_{p \text{ mod}} = Q_{p \text{ pon}} \{ \text{Factor} \} \tag{1.4}$$

Where

$Q_{p \text{ mod}}$ - Modified peak rate of runoff in cfs

Q_{p} - Peak rate of runoff in cfs

Factor_{pon} - Ponding adjustment factor

It should be emphasized that sound judgement must be exercised in adjusting peak rates of runoff where ponding or swampy areas occur in a watershed. It cannot be used blindly for the purpose of obtaining smaller peak discharge values.

Table 1.4 - Adjustment Factors Where Ponding and Swampy Areas Occur at Design Point

Ratio D.A./Pond and Swampy Area	Percent Pond and Swamp in D.A.	Return Period (year)					
		2	5	10	25	50	100
500	0.2	.92	.94	.95	.96	.97	.98
200	0.5	.86	.87	.88	.90	.92	.93
100	1.0	.80	.81	.83	.85	.87	.89
50	2.0	.74	.75	.76	.79	.82	.86
40	2.5	.69	.70	.72	.75	.78	.82
30	3.3	.64	.65	.67	.71	.75	.78
20	5.0	.59	.61	.63	.67	.71	.75
15	6.7	.57	.58	.60	.64	.67	.71
10	10.0	.53	.54	.56	.60	.63	.68
5	20.0	.48	.49	.51	.55	.59	.64

Table 1.5 - Adjustment Factors Where Ponding and Swamy Areas Are Spread Throughout Watershed or Occur in Central Parts of Watershed

Ratio D.A./Pond and Swamy Area	Percent Pond and swamy Areas in D.A.	Return Period (year)					
		2	5	10	25	50	100
500	0.2	.94	.95	.96	.97	.98	.99
200	0.5	.88	.89	.90	.91	.92	.94
100	1.0	.83	.84	.86	.87	.88	.90
50	2.0	.78	.79	.81	.83	.85	.87
40	2.5	.73	.74	.76	.78	.81	.84
30	3.3	.69	.70	.71	.74	.77	.81
20	5.0	.65	.66	.68	.72	.75	.78
15	6.7	.62	.63	.65	.69	.72	.75
10	10.0	.58	.59	.61	.65	.68	.71
5	20.0	.53	.54	.56	.60	.63	.68
4	25.0	.50	.51	.53	.57	.61	.66
3	33.3	.47	.49	.50	.54	.59	.64
2	50.0	.45	.47	.48	.52	.57	.62
1	100.0	.43	.45	.46	.50	.55	.61

Table 1.6 - Adjustment Factor Where Ponding and Swamy Areas Are Located in Upper Reaches of Watershed

Ratio D.A./Pond and Swamy Area	Percent Pond and Swamy Areas in D.A.	Return Period (year)					
		2	5	10	25	50	100
500	0.2	.96	.97	.98	.98	.99	.99
200	0.5	.93	.94	.94	.95	.96	.97
100	1.0	.90	.91	.92	.93	.94	.95
50	2.0	.87	.88	.88	.90	.91	.93
40	2.5	.85	.85	.86	.88	.89	.91
30	3.3	.82	.83	.84	.86	.88	.89
20	5.0	.80	.81	.82	.84	.86	.88
15	6.7	.78	.79	.80	.82	.84	.86
10	10.0	.77	.77	.78	.80	.82	.84
5	20.0	.74	.75	.76	.78	.80	.82

Modification of peak rate of runoff for rice fields -

Basic discharges computed for rice field areas will be reduced by a factor to be selected from Table 1.5, assuming a ratio of D.A./pond and swampy area of 5. The factor so selected is to be applied only to that portion of the watershed being used for rice cultivation. In the event that only a portion of the total area is in rice cultivation, a weighted reduction factor (K) will be used.

Modification of peak rate of runoff due to urbanization -

The method of estimating peak runoff rates which has been described in this section was developed primarily from agricultural watersheds data. However, urbanization commonly increases the volume of runoff and the velocity at which water can flow from its point of impact on the watershed to the watershed outlet. The increase in the volume of runoff due to urbanization depends more on the percentage of impervious area whereas the reduction in the watershed lag depends more on the percentage of hydraulic length modified. Lag is the time from the center of mass of excess rainfall to the time to peak.

Figures 1.8 and 1.9 give factors for adjusting peak runoff rates calculated by the method previously described in this section. The factors are applied to the peak runoff rates using future-condition runoff curve numbers as follows:

$$Q_{p \text{ mod}} = Q_p \left\{ \text{Factor}_{\text{imp}} \right\} \left\{ \text{Factor}_{\text{hlm}} \right\} \quad (1.5)$$

Where

$Q_{p \text{ mod}}$ - Modified peak runoff rate due to urbanization in cfs

Q_p - Peak runoff rate for future CN in cfs

$\text{Factor}_{\text{imp}}$ - Factor based on the total percentage of the watershed area which is likely to be impervious (Figure 1.8). The percentage of impervious area in watershed could be determined using United States Geological Survey topographic maps, land use maps, aerial photographs, and field reconnaissance.

$\text{Factor}_{\text{hlm}}$ - Factor based on the percentage of the hydraulic length of the watershed which is likely to be (or has been) modified as a result of urbanization (Figure 1.9). Hydraulic length modification factor is applied when parts or the entire watershed is served by storm sewers or a stream (usually the major water course in the watershed) has been changed over natural conditions by straightening or by enlarging stream capacity.

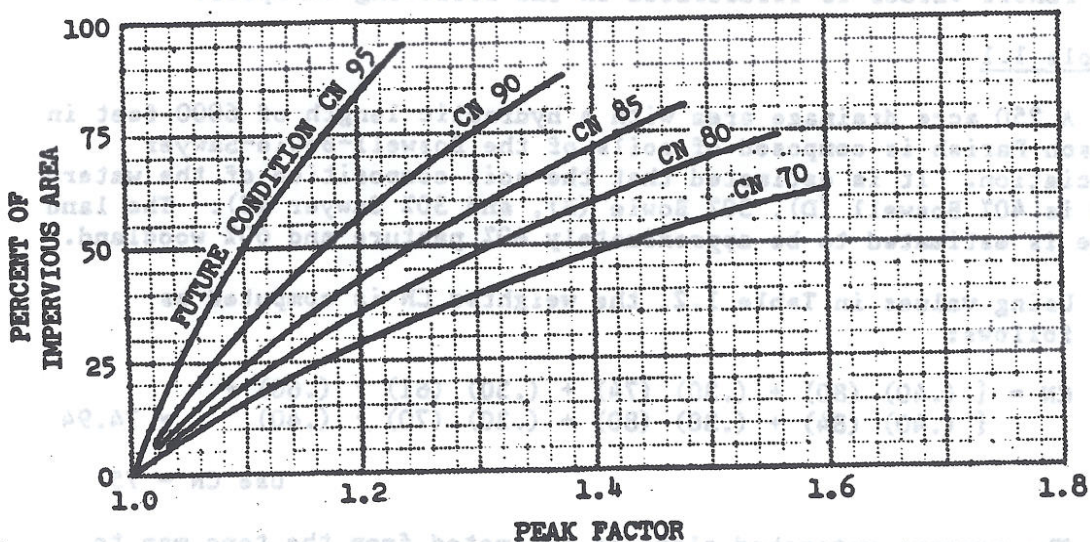


FIGURE 1.8 - Factors for adjusting peak discharges for a given future-condition runoff curve number based on the percentage of impervious area in the watershed.

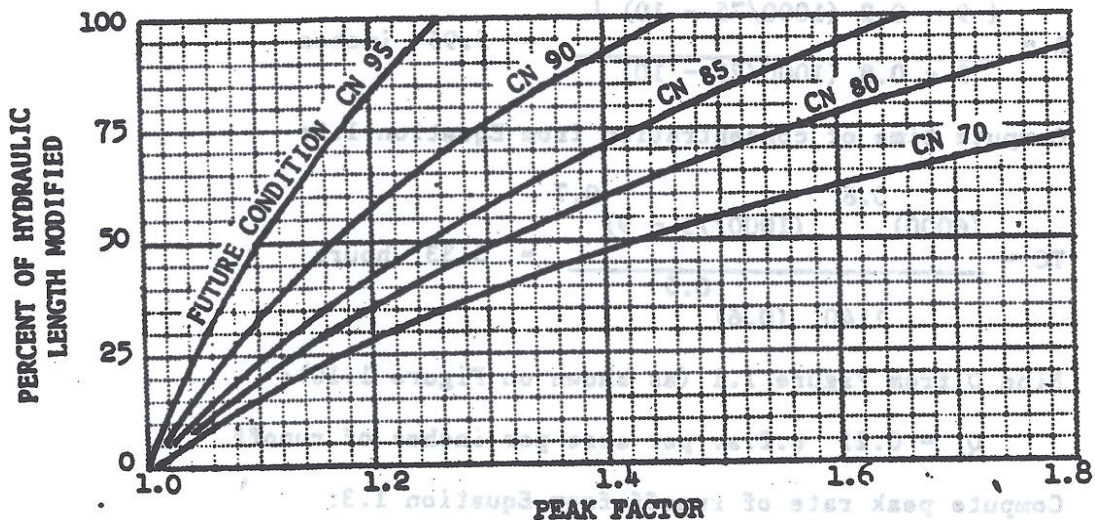


FIGURE 1.9 - Factors for adjusting peak discharges for a given future-condition runoff curve number based on the percentage of hydraulic length modified.

Application of Soil Conservation Service procedure for estimating peak runoff values is illustrated in the following examples.

Example 1.1

A 250 acre drainage area with a hydraulic length of 6000 feet in Jackson Parish is composed of soils of the Boswell-Bowie-Sawyer association. It is estimated that the soil composition of the watershed is 40% Boswell (D), 30% Bowie (B), and 30% Sawyer (C). The land use is estimated to be approximately 40% pasture and 60% woodland.

Using values in Table 1.2, the weighted CN is computed as follows:

$$CN = \left\{ \begin{array}{l} (.40) (80) + (.30) (74) + (.30) (61) \\ (.40) (84) + (.30) (80) + (.30) (70) \end{array} \right\} \left\{ \begin{array}{l} (.60) \\ (.40) \end{array} \right\} = 74.94$$

Use CN = 75

The average watershed slope is estimated from the topo map to be 0.6%. For a 50-year design storm, the 24-hour rainfall is found in Figure 1.6 to be nine inches.

Following the steps outlined previously, the above information is used to compute peak rate of runoff.

Compute runoff from Equation 1.1:

$$R = \frac{\{ 9 - 0.2 (1000/75 - 10) \}^2}{9 + 0.8 (1000/75 - 10)} = 5.95 \text{ inches}$$

Compute time of concentration from Equation 1.2:

$$TC = \frac{(6000)^{0.8} (1000/75 - 9)^{0.7}}{1140 (0.6)^{0.5}} = 3.33 \text{ hours}$$

Find Q from Figure 1.1 (as shown on Figure 1.10):

$$Q = 0.21 \text{ c.f.s. per acre per inches of runoff}$$

Compute peak rate of runoff from Equation 1.3:

$$Q_p = 250 \times 5.95 \times 0.21 = 313 \text{ c.f.s.}$$

Example 1.2:

Assume that in the watershed in Example 1.1, there are swampy and

ponding areas located in the upper reaches of the watershed comprising approximately 10% of the total watershed area. From Table 1.6, the adjustment factor is found to be .82. The adjusted design discharge is:

$$Q_p = (0.82)(313) = 257 \text{ c.f.s.}$$

**PEAK DISCHARGE IN CUBIC FEET PER SECOND
PER ACRE PER INCH OF RUNOFF**

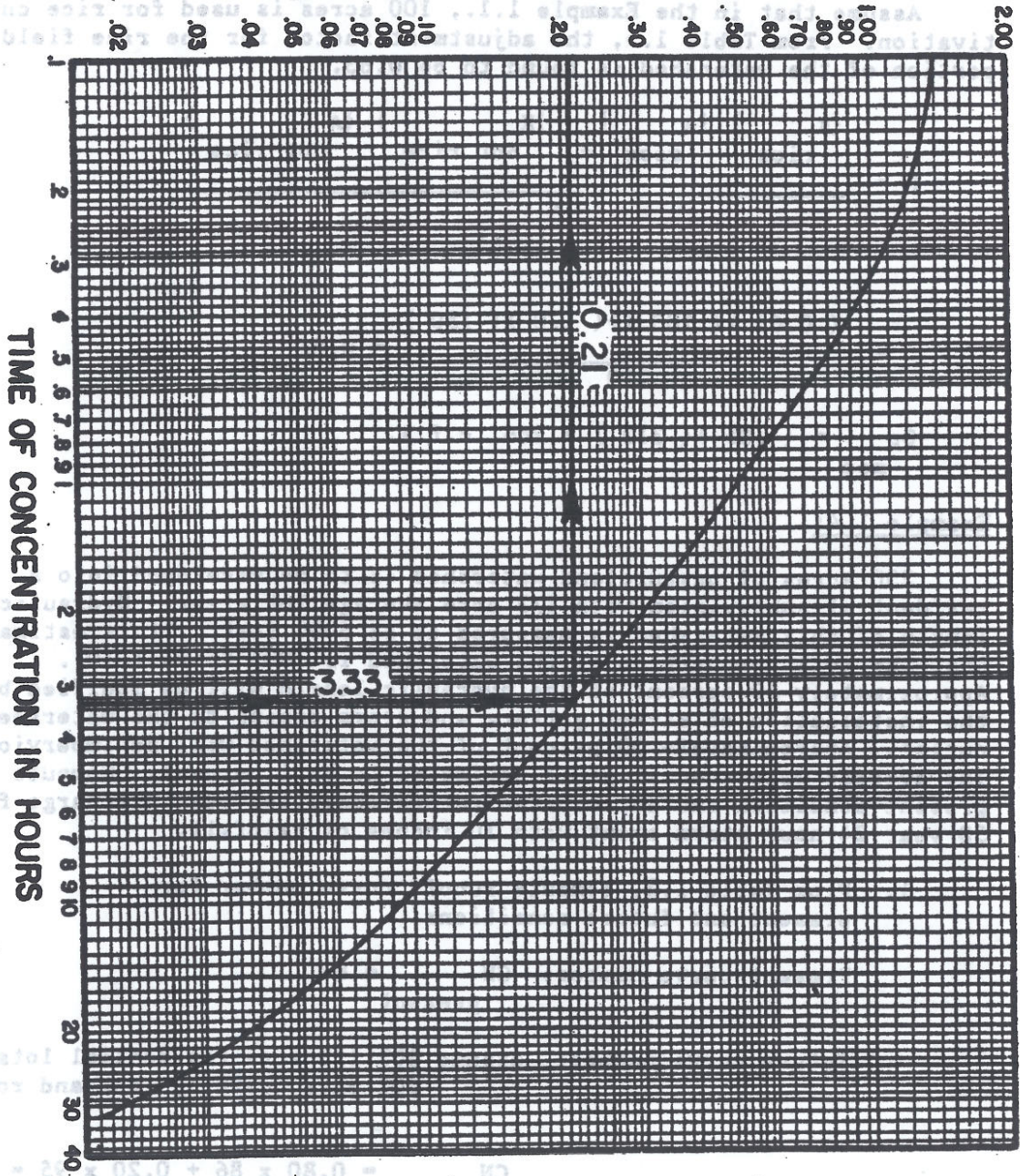


FIGURE 1.10 - EXAMPLE PROBLEM 1.1

ponding areas located in the upper reaches of the watershed comprising approximately 10% of the total watershed area. From Table 1.6, the adjustment factor is found to be .82. The adjusted design discharge is:

$$Q_p \text{ mod} = (.82) (313) = 257 \text{ c.f.s.}$$

Example 1.3:

Assume that in the Example 1.1., 100 acres is used for rice cultivation. From Table 1.5, the adjustment factor for the rice field portion of the watershed is found to be 0.63.

$$K = \frac{\{K_{\text{rice}}\} \{A_{\text{rice}}\} + \{K_{\text{not rice}}\} \{A_{\text{not rice}}\}}{A_{\text{total}}}$$

$$= \frac{(0.63)(100) + (1.0)(150)}{250} = 0.85$$

$$Q_p \text{ mod} = (.85)(313) = 266 \text{ c.f.s.}$$

Example 1.4:

250 Acres of pastureland watershed is to be developed into a residential subdivision with 1/3 acre average lot sizes. Hydraulic length of this watershed is measured to be 6000 feet. It is estimated that the soil composition of the watershed is 100% Boswell (D). Approximately 50 percent of the hydraulic length will be modified by the installation of street gutters and storm drains to the watershed outlet. Approximately 30 percent of the watershed will be impervious. The average watershed slope is estimated to be 1 percent. Compute the present-condition and anticipated future-condition peak discharge for a 50-year 24-hour storm event with 10 inches of rainfall.

1. From Table 1.3, compute runoff curve number for present and future conditions.

$$\text{Present curve number: } CN_{\text{present}} = 84$$

$$\text{Future curve number: assume } 80\% \text{ } 1/3 \text{ acre residential lots} \\ 20\% \text{ paved parking lots and roads}$$

$$CN_{\text{future}} = 0.80 \times 86 + 0.20 \times 95 = 87.8$$

2. Following the same procedures as in Example 1.1, compute the runoff, time of concentration and then peak rate of runoff for present-condition.

$$\text{CN}_{\text{present}} = 84$$

$$R = 8.03 \text{ inches}$$

$$\text{TC} = 1.95 \text{ Hours} \quad \Rightarrow \quad Q = 0.313 \text{ (Figure 1.1)}$$

$$Q_{\text{present}} = 0.31 \times 8.03 \times 250 = 629 \text{ c.f.s.}$$

3. Following the same procedure as in step 2, compute the runoff, time of concentration and then base peak rate of runoff for future-condition.

$$\text{CN}_{\text{future}} = 87.8$$

$$R = 8.51 \text{ inches}$$

$$\text{TC} = 1.70 \text{ Hours} \quad \Rightarrow \quad Q = 0.340 \text{ (Figure 1.1)}$$

$$Q_{\text{p base future}} = 0.345 \times 8.51 \times 250 = 734 \text{ c.f.s.}$$

4. From Figure 1.8, with 30 percent impervious area and future-condition runoff curve number of 87.8, read peak factor = 1.10.
5. From Figure 1.9, with 50 percent of hydraulic length modified and future-condition runoff curve number 87.8, read peak factor = 1.18.
6. The future-condition peak rate of runoff is:
$$734 (1.10)(1.18) = 953 \text{ c.f.s.}$$
7. The effect of this proposed development is to increase the peak discharge from 629 to 953 c.f.s.