Chapter 3

Urban Water Hydrology

Solution of Problems

1- Results of a practice for determining the Horton infiltration capacity in the exponential form are tabulated in Table 3.37. Determine the infiltration capacity exponential equation.

Table 3.37 The recorded filtration capacity data of Problem 1

Time (hr)	0.25	0.5	0.75	1.00	1.25	1.50	1.75	2.00
f_p (cm/hr)	5.60	3.20	2.10	1.50	1.20	1.10	1.00	1.00

Solution:

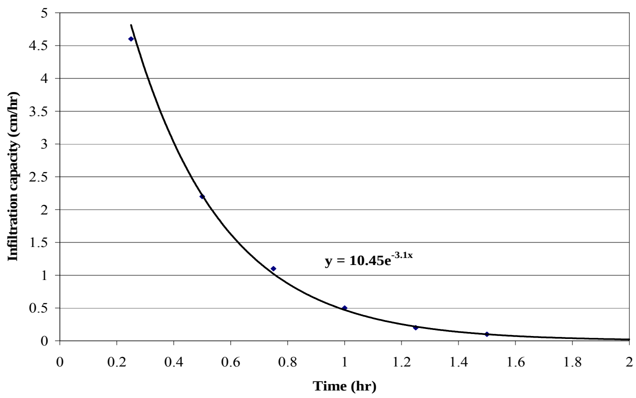
The exponential Horton infiltration capacity is formulated as follows:

$$f_p = f_f + (f_0 - f_f)e^{-mt}$$

There are three unknown parameters in this equation including initial infiltration capacity, final infiltration capacity and decay constant, which should be determined based on the available data.

Since after 1.75 hr, the infiltration rate does not change and remain constant at 1.00 cm/hr, it can be concluded that final infiltration capacity is equal to 1.00 cm/hr. It should be noted that since in the beginning of the rainfall the infiltration capacity decreases very fast, the 5.60 cm/hr filtration capacity which is recorded at 0.25 hr after start of rainfall, could be considered as initial infiltration capacity.

Excel software is employed to fit an exponential curve to the observed data. Because of the limitations of this software in curve fitting the final infiltration capacity (1cm/hr) is subtracted from data before curve fitting. As two last values become equal to zero, they are omitted in curve fitting process. The fitted curve and the corresponding equation are shown in the following Figure.



Therefore the exponential Horton infiltration capacity could be written as follows:

$$f_p = 1.0 + 10.45e^{-3.1t}$$

Based on this equation m is 3.1 and the initial infiltration capacity is equal to 10.45-1.00=9.45cm/hr.

2. Given an initial infiltration capacity f_0 of 3.0 cm/hr and a time constant m of 0.29 1/hr, derive an infiltration capacity versus time curve if the ultimate infiltration capacity is 0.55 cm/hr. For the first 10 hours, estimate the total volume of water infiltrated over the watershed.

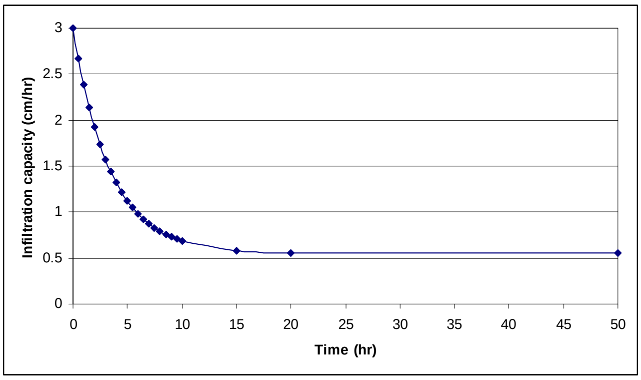
Solution:

Based on the given data, the exponential Horton infiltration capacity equation is written as follows:

$$f_p = 0.55 + 2.45e^{-0.29t}$$

The infiltration capacities in different time steps after start of rain are given in the following table and figure. As it can be seen after 50 hr, the soil reaches its final infiltration capacity (0.55 cm/hr).

Time (hr)	*fp* (cm/hr)	Time (hr)	*fp* (cm/hr)
0	3	6	0.98
0.5	2.67	6.5	0.92
1	2.38	7	0.87
1.5	2.14	7.5	0.83
2	1.92	8	0.79
2.5	1.74	8.5	0.76
3	1.58	9	0.73
3.5	1.44	9.5	0.71
4	1.32	10	0.68
4.5	1.21	15	0.58
5	1.12	20	0.56
5.5	1.05	50	0.550001



The total infiltrated water into the soil in first 10 hours is obtained as follows:

$$F_{0-10} = \int_{t=0}^{10} f_p dt = \int_{t=0}^{10} 0.55 + 2.45e^{-0.29t} dt = 0.55t - \frac{2.45}{0.29} e^{-0.29t} \Big|_{t=0}^{t=10}$$

$$= (0.55 \times 10 - 8.45e^{-0.29\times10}) - (0.55\times 0 - 8.45e^{-0.29\times0}) = 5.04 + 8.45 = 13.49\ cm$$

3. Estimate the time of concentration for shallow turbulent sheet flow on a 500 m section of asphalt roadway at a slope of 8% and manning roughness equal to 0.012. Assume a 25-yr design frequency and the IDF curve of Figure 3.34. Assume the road soil belongs to group A.

Solution:

At first γ is estimated for turbulent flow as follows:

$$\gamma = \frac{\sqrt{S}}{n} = \frac{\sqrt{0.08}}{0.012} = 23.57\ m^{1/3}s^{-1}$$

For determination of time of concentration, the excess rainfall should be estimated which is dependent to the time of concentration. Therefore an iterative procedure should be followed to find the time of concentration. The initial estimation of time of concentration is 0.1 hr or 360 s. The intensity of a 0.1 hr rainfall with 25-year design frequency is determined from Figure 3.34 as 5.4 cm/hr. From table 3.16 the runoff coefficient for street with slop of 8%, soil group A and design frequency less than 25 year, runoff coefficient is determined as 0.72. Therefore the excess rainfall is $5.4 \times 0.72 = 3.888\ cm/hr$. Since k is equal to 5/3, the time of concentration for 100 m of the road is calculated as follows:

$$t_c = \left(\frac{L}{\gamma\, i_e^{k-1}} \right)^{1/k} = \left(\frac{500}{23.57\,(3.888 \times \frac{0.01}{3600})^{5/3-1}} \right)^{1/(5/3)} = 606.2\ s$$

The obtained value of time of concentration is considered as rainfall duration and calculations are repeated. This process is continued until the time of concentration in two consequent iterations remain constant. The calculations are summarized in the following table. In 10th iteration the time of concentration becomes approximately constant and the iterative process stops. Therefore the time of concentration of the considered section of the road is equal to 643 s.

Iteration No.	rainfall duration (s)	Rainfall intensity (cm/hr)	Excess rainfall (cm/hr)	Time of concentration (s)
1	360	5.4	3.888	606.173
2	606.173	4.9	3.528	630.196
3	630.196	4.85	3.492	632.787
4	632.787	4.8	3.456	635.415
5	635.415	4.76	3.427	637.546
6	637.546	4.72	3.398	639.702
7	639.702	4.69	3.377	641.335
8	641.335	4.67	3.362	642.432
9	642.433	4.66	3.355	642.984
10	642.984	4.65	3.348	643.536

4. The mass curve of a rainfall of 100min duration is given in Table 3.38. a) If the catchment has an initial loss of 0.6 cm and Φ index of 0.6 cm/hr, calculate the total surface runoff from the catchment. b) If the direct runoff of catchment is 2 cm, determine the Φ index for the basin.

Table 3.38 Mass curve of rainfall of Problem 4

Time from start of rainfall (min)	0	20	40	60	80	100
Cumulative rainfall (cm)	0	0.5	1.2	2.6	3.3	3.5

Solution:

a) At first the rainfall depth in each time step is determined as given in the following table. Then the initial loss of 0.6 cm is subtracted from them and results are given in forth row of table. Runoff intensities are determined as: rainfall intensities – Φ index (sixth row). Total runoff is estimated as follows:

$$Total\ runoff = \left(\frac{3.6}{60}\right) \times 20 + \left(\frac{1.5}{60}\right) \times 20 = 1.7\ cm$$

Time from start of rainfall (min)	0	20	40	60	80	100
Cumulative rainfall (cm)	0	0.5	1.2	2.6	3.3	3.5
Rainfall depth (cm)	0	0.5	0.7	1.4	0.7	0.2
Rainfall depth by subtracting initial loss (cm)	0	0	0.1	1.4	0.7	0.2
Rainfall intensities (cm/hr)	0	0	0.3	4.2	2.1	0.6
Runoff intensities (cm/hr)	0	0	0	3.6	1.5	0

b) Infiltration depth is equal to: Rainfall volume-Runoff volume=3.5-2=1.5 cm

Therefore Φ index is obtained as $\frac{1.5cm}{100\min} \times \frac{60\min}{1hr} = 0.9\ cm/hr$

5. The data presented in Table 3.39 is the annual maximum series (Q_p) and the percentage of impervious area (I) for an urbanized watershed for the period from 1930 through 1977. Adjust the flood series to eventual development of 50%. Estimate the effect on the estimated 2-, 10-, 25-, and 100-yr floods.

Solution:

Table 3.39 Data of Problem 5

Year	I (%)	Q_P (cfs)	Year	I (%)	Q_P (cfs)	Year	I (%)	Q_P (cfs)
1930	21	1870	1946	35	1600	1962	45	2560
1931	21	1530	1947	37	3810	1963	45	2215
1932	22	1120	1948	39	2670	1964	45	2210
1933	22	1850	1949	41	758	1965	45	3730
1934	23	4890	1950	43	1630	1966	45	3520
1935	23	2280	1951	45	1620	1967	45	3550
1936	24	1700	1952	45	3811	1968	45	3480
1937	24	2470	1953	45	3140	1969	45	3980
1938	25	5010	1954	45	2410	1970	45	3430
1939	25	2480	1955	45	1890	1971	45	4040
1940	26	1280	1956	45	4550	1972	46	2000
1941	27	2080	1957	45	3090	1973	46	4450
1942	29	2320	1958	45	4830	1974	46	4330

1943	30	4480	1959	45	3170	1975	46	6000
1944	32	1860	1960	45	1710	1976	46	1820
1945	33	2220	1961	45	1480	1977	46	1770

The flood records are adjusted for desired condition of the impervious cover. As the return period for some of the previous events is modified from the measured record, the iterative process is required. These changes in the rank of the events, changes the exceedence probabilities. Since the adjustment factor is dependent on the exceedence probabilities, the adjustment factor also changes. The rank of the events did not change after the third adjustment, thus the procedure of adjustment is stopped. The iteration procedure is tabulated in the following tables.

Initial Ranking and Discharge.

Measured Series					Ordered Data			
Year	Urbanization (%)	Annual Peak (m^3/s)	Rank	Exceed. Prob.	Rank	Annual Peak (m^3/s)	Year	Exceed. Prob.
1930	21	1870	34	0.69	1	6000	1975	0.02
1931	21	1530	44	0.90	2	5010	1938	0.04
1932	22	1120	47	0.96	3	4890	1934	0.06
1933	22	1850	36	0.73	4	4830	1958	0.08
1934	23	4890	3	0.06	5	4550	1956	0.10
1935	23	2280	27	0.55	6	4480	1943	0.12
1936	24	1700	40	0.82	7	4450	1973	0.14
1937	24	2470	24	0.49	8	4330	1974	0.16
1938	25	5010	2	0.04	9	4040	1971	0.18
1939	25	2480	23	0.47	10	3980	1969	0.20
1940	26	1280	46	0.94	11	3811	1952	0.22
1941	27	2080	31	0.63	12	3810	1947	0.24
1942	29	2320	26	0.53	13	3730	1965	0.27
1943	30	4480	6	0.12	14	3550	1967	0.29
1944	32	1860	35	0.71	15	3520	1966	0.31
1945	33	2220	28	0.57	16	3480	1968	0.33
1946	35	1600	43	0.88	17	3430	1970	0.35
1947	37	3810	12	0.24	18	3170	1959	0.37
1948	39	2670	21	0.43	19	3140	1953	0.39
1949	41	758	48	0.98	20	3090	1957	0.41

1950	43	1630	41	0.84	21	2670	1948	0.43
1951	45	1620	42	0.86	22	2560	1962	0.45
1952	45	3811	11	0.22	23	2480	1939	0.47
1953	45	3140	19	0.39	24	2470	1937	0.49
1954	45	2410	25	0.51	25	2410	1954	0.51
1955	45	1890	33	0.67	26	2320	1942	0.53
1956	45	4550	5	0.10	27	2280	1935	0.55
1957	45	3090	20	0.41	28	2220	1945	0.57
1958	45	4830	4	0.08	29	2215	1963	0.59
1959	45	3170	18	0.37	30	2210	1964	0.61
1960	45	1710	39	0.80	31	2080	1941	0.63
1961	45	1480	45	0.92	32	2000	1972	0.65
1962	45	2560	22	0.45	33	1890	1955	0.67
1963	45	2215	29	0.59	34	1870	1930	0.69
1964	45	2210	30	0.61	35	1860	1944	0.71
1965	45	3730	13	0.27	36	1850	1933	0.73
1966	45	3520	15	0.31	37	1820	1976	0.76
1967	45	3550	14	0.29	38	1770	1977	0.78
1968	45	3480	16	0.33	39	1710	1960	0.80
1969	45	3980	10	0.20	40	1700	1936	0.82
1970	45	3430	17	0.35	41	1630	1950	0.84
1971	45	4040	9	0.18	42	1620	1951	0.86
1972	46	2000	32	0.65	43	1600	1946	0.88
1973	46	4450	7	0.14	44	1530	1931	0.90
1974	46	4330	8	0.16	45	1480	1961	0.92
1975	46	6000	1	0.02	46	1280	1940	0.94
1976	46	1820	37	0.76	47	1120	1932	0.96
1977	46	1770	38	0.78	48	758	1949	0.98

1st Iteration

Year	Urbanization (%)	Measured Peak (m^3/s)	Correction Factor			Adjusted Series	
			Exist.	Ultimate	Peak	Rank	Exceed. Prob.
1930	21	1870	1.4362	1.85979	2421.53	30	0.61224
1931	21	1530	1.36409	1.72208	1931.53	38	0.77551
1932	22	1120	1.33245	1.63704	1376.02	47	0.95918

1933	22	1850	1.44041	1.8381	2360.76	31	0.63265
1934	23	4890	1.67434	2.22996	6512.71	2	0.04082
1935	23	2280	1.50604	1.92811	2918.98	23	0.46939
1936	24	1700	1.43907	1.78859	2112.88	35	0.71429
1937	24	2470	1.53437	1.95586	3148.51	21	0.42857
1938	25	5010	1.736	2.26545	6537.97	1	0.02041
1939	25	2480	1.5598	1.96511	3124.42	22	0.44898
1940	26	1280	1.40124	1.67254	1527.83	46	0.93878
1941	27	2080	1.54715	1.8901	2541.06	28	0.57143
1942	29	2320	1.61229	1.93739	2787.8	25	0.5102
1943	30	4480	1.77451	2.16107	5455.92	4	0.08163
1944	32	1860	1.59656	1.84913	2154.24	34	0.69388
1945	33	2220	1.66456	1.91877	2559.04	27	0.55102
1946	35	1600	1.56123	1.74142	1784.67	41	0.83673
1947	37	3810	1.85469	2.07587	4264.34	9	0.18367
1948	39	2670	1.81557	1.98373	2917.29	24	0.4898
1949	41	758	1.50055	1.58217	799.23	48	0.97959
1950	43	1630	1.68862	1.77421	1712.61	43	0.87755
1951	45	1620	1.69794	1.75861	1677.88	44	0.89796
1952	45	3811	2.00445	2.08786	3969.6	12	0.2449
1953	45	3140	1.92517	2.0027	3266.46	19	0.38776
1954	45	2410	1.87297	1.94663	2504.78	29	0.59184
1955	45	1890	1.80176	1.87014	1961.72	37	0.7551
1956	45	4550	2.0906	2.18041	4745.46	6	0.12245
1957	45	3090	1.91629	1.99316	3213.96	20	0.40816
1958	45	4830	2.11146	2.20282	5038.98	5	0.10204
1959	45	3170	1.9342	2.0124	3298.17	18	0.36735
1960	45	1710	1.73833	1.802	1772.63	42	0.85714
1961	45	1480	1.64308	1.69968	1530.98	45	0.91837
1962	45	2560	1.89881	1.97439	2661.89	26	0.53061
1963	45	2215	1.83825	1.90934	2300.65	32	0.65306
1964	45	2210	1.82937	1.89979	2295.07	33	0.67347
1965	45	3730	1.9826	2.0644	3883.89	13	0.26531
1966	45	3520	1.96241	2.04271	3664.03	15	0.30612
1967	45	3550	1.97234	2.05337	3695.85	14	0.28571
1968	45	3480	1.95278	2.03236	3621.82	16	0.32653
1969	45	3980	2.01621	2.1005	4146.39	11	0.22449
1970	45	3430	1.94339	2.02227	3569.23	17	0.34694

1971	45	4040	2.02869	2.11391	4209.7	10	0.20408
1972	46	2000	1.8231	1.88022	2062.67	36	0.73469
1973	46	4450	2.07173	2.14388	4604.97	7	0.14286
1974	46	4330	2.05702	2.12828	4480	8	0.16327
1975	46	6000	2.23812	2.32032	6220.36	3	0.06122
1976	46	1820	1.77255	1.82663	1875.52	39	0.79592
1977	46	1770	1.76124	1.81463	1823.66	40	0.81633

2nd Iteration

Year	Urbanization (%)	Measured Peak (m^3/s)	Correction Factor			Adjusted Series	
			Exist.	Ultimate	Peak	Rank	Exceed. Prob.
1930	21	1870	1.45714	1.89979	2438.06	30	0.61224
1931	21	1530	1.41255	1.81463	1965.51	37	0.7551
1932	22	1120	1.33245	1.63704	1376.02	47	0.95918
1933	22	1850	1.46834	1.8901	2381.39	31	0.63265
1934	23	4890	1.69413	2.26545	6539.08	2	0.04082
1935	23	2280	1.52667	1.96511	2934.79	23	0.46939
1936	24	1700	1.47356	1.84913	2133.27	35	0.71429
1937	24	2470	1.55025	1.98373	3160.67	21	0.42857
1938	25	5010	1.76819	2.32032	6574.42	1	0.02041
1939	25	2480	1.56524	1.97439	3128.26	22	0.44898
1940	26	1280	1.40124	1.67254	1527.83	46	0.93878
1941	27	2080	1.56487	1.91877	2550.39	28	0.57143
1942	29	2320	1.61834	1.94663	2790.63	25	0.5102
1943	30	4480	1.80229	2.20282	5475.59	4	0.08163
1944	32	1860	1.60412	1.85979	2156.45	34	0.69388
1945	33	2220	1.67135	1.92811	2561.04	27	0.55102
1946	35	1600	1.58607	1.77421	1789.79	41	0.83673
1947	37	3810	1.88513	2.11391	4272.39	9	0.18367
1948	39	2670	1.79227	1.95586	2913.71	24	0.4898
1949	41	758	1.50055	1.58217	799.23	48	0.97959
1950	43	1630	1.65922	1.74142	1710.76	43	0.87755
1951	45	1620	1.66394	1.72208	1676.61	44	0.89796
1952	45	3811	1.99328	2.07587	3968.9	12	0.2449
1953	45	3140	1.92517	2.0027	3266.46	19	0.38776

1954	45	2410	1.83825	1.90934	2503.19	29	0.59184
1955	45	1890	1.76126	1.82663	1960.14	38	0.77551
1956	45	4550	2.0726	2.16107	4744.23	6	0.12245
1957	45	3090	1.91629	1.99316	3213.96	20	0.40816
1958	45	4830	2.0906	2.18041	5037.49	5	0.10204
1959	45	3170	1.9342	2.0124	3298.17	18	0.36735
1960	45	1710	1.69794	1.75861	1771.1	42	0.85714
1961	45	1480	1.64308	1.69968	1530.98	45	0.91837
1962	45	2560	1.86437	1.93739	2660.26	26	0.53061
1963	45	2215	1.81115	1.88022	2299.47	32	0.65306
1964	45	2210	1.80176	1.87014	2293.86	33	0.67347
1965	45	3730	1.9826	2.0644	3883.89	13	0.26531
1966	45	3520	1.96241	2.04271	3664.03	15	0.30612
1967	45	3550	1.97234	2.05337	3695.85	14	0.28571
1968	45	3480	1.95278	2.03236	3621.82	16	0.32653
1969	45	3980	2.00445	2.08786	4145.63	11	0.22449
1970	45	3430	1.94339	2.02227	3569.23	17	0.34694
1971	45	4040	2.01621	2.1005	4208.89	10	0.20408
1972	46	2000	1.78337	1.8381	2061.38	36	0.73469
1973	46	4450	2.07173	2.14388	4604.97	7	0.14286
1974	46	4330	2.05702	2.12828	4480	8	0.16327
1975	46	6000	2.1529	2.22996	6214.74	3	0.06122
1976	46	1820	1.74933	1.802	1874.8	39	0.79592
1977	46	1770	1.73668	1.78859	1822.9	40	0.81633

3th Iteration

Year	Urbanization (%)	Measured Peak (m^3/s)	Correction Factor			Adjusted Series	
			Exist.	Ultimate	Peak	Rank	Exceed. Prob.
1930	21	1870	1.45714	1.89979	2438.06	30	0.61224
1931	21	1530	1.41883	1.82663	1969.74	37	0.7551
1932	22	1120	1.33245	1.63704	1376.02	47	0.95918
1933	22	1850	1.46834	1.8901	2381.39	31	0.63265
1934	23	4890	1.69413	2.26545	6539.08	2	0.04082
1935	23	2280	1.52667	1.96511	2934.79	23	0.46939
1936	24	1700	1.47356	1.84913	2133.27	35	0.71429

1937	24	2470	1.55025	1.98373	3160.67	21	0.42857
1938	25	5010	1.76819	2.32032	6574.42	1	0.02041
1939	25	2480	1.56524	1.97439	3128.26	22	0.44898
1940	26	1280	1.40124	1.67254	1527.83	46	0.93878
1941	27	2080	1.56487	1.91877	2550.39	28	0.57143
1942	29	2320	1.61834	1.94663	2790.63	25	0.5102
1943	30	4480	1.80229	2.20282	5475.59	4	0.08163
1944	32	1860	1.60412	1.85979	2156.45	34	0.69388
1945	33	2220	1.67135	1.92811	2561.04	27	0.55102
1946	35	1600	1.58607	1.77421	1789.79	41	0.83673
1947	37	3810	1.88513	2.11391	4272.39	9	0.18367
1948	39	2670	1.79227	1.95586	2913.71	24	0.4898
1949	41	758	1.50055	1.58217	799.23	48	0.97959
1950	43	1630	1.65922	1.74142	1710.76	43	0.87755
1951	45	1620	1.66394	1.72208	1676.61	44	0.89796
1952	45	3811	1.99328	2.07587	3968.9	12	0.2449
1953	45	3140	1.92517	2.0027	3266.46	19	0.38776
1954	45	2410	1.83825	1.90934	2503.19	29	0.59184
1955	45	1890	1.75009	1.81463	1959.7	38	0.77551
1956	45	4550	2.0726	2.16107	4744.23	6	0.12245
1957	45	3090	1.91629	1.99316	3213.96	20	0.40816
1958	45	4830	2.0906	2.18041	5037.49	5	0.10204
1959	45	3170	1.9342	2.0124	3298.17	18	0.36735
1960	45	1710	1.69794	1.75861	1771.1	42	0.85714
1961	45	1480	1.64308	1.69968	1530.98	45	0.91837
1962	45	2560	1.86437	1.93739	2660.26	26	0.53061
1963	45	2215	1.81115	1.88022	2299.47	32	0.65306
1964	45	2210	1.80176	1.87014	2293.86	33	0.67347
1965	45	3730	1.9826	2.0644	3883.89	13	0.26531
1966	45	3520	1.96241	2.04271	3664.03	15	0.30612
1967	45	3550	1.97234	2.05337	3695.85	14	0.28571
1968	45	3480	1.95278	2.03236	3621.82	16	0.32653
1969	45	3980	2.00445	2.08786	4145.63	11	0.22449
1970	45	3430	1.94339	2.02227	3569.23	17	0.34694
1971	45	4040	2.01621	2.1005	4208.89	10	0.20408
1972	46	2000	1.78337	1.8381	2061.38	36	0.73469
1973	46	4450	2.07173	2.14388	4604.97	7	0.14286
1974	46	4330	2.05702	2.12828	4480	8	0.16327

1975	46	6000	2.1529	2.22996	6214.74	3	0.06122
1976	46	1820	1.74933	1.802	1874.8	39	0.79592
1977	46	1770	1.73668	1.78859	1822.9	40	0.81633

The mean and standard divergences of the logarithm of adjusted series are 3.49 and 3.13, respectively. The mean increased and the standard deviation decreased after adjustment at it was expected. The mean increased because the earlier events occurred when less impervious cover existed which reduce the peak discharge. The standard deviation decreased because the measured data of peak discharge present both natural variation and variation due to different levels of imperviousness. The adjusted flood frequency curve is generally higher than the curve for the measured series; as higher curve reflects the effect of the higher amount of imperviousness (50%). The adjusted flood frequency curve has also less sleep which reflects the issue that the adjusted series is for a single level of imperviousness.

The percentage increase in the 2-yr, 5-yr, 10-yr, 12-yr, and 100-yr flood magnitudes before and after adjustments are also given in the following table.

Flood discharge before adjustment

Return Period	2	10	25	100
Probability	0.5	0.1	0.04	0.01
Flood Discharge (m^3/s)	2410	4550	5010	6000

Flood Discharge after adjustment

Return Period	2	10	25	100
Probability	0.5	0.1	0.04	0.01
Flood Discharge (m^3/s)	2790	5037	6539	6574

6. Residents in a community at the discharge point of a 614 km^2 watershed believe that recent increase in peak discharge rates is due to deforestation by a logging company that has been occurring in recent years. Analyze the annual maximum discharges (q_p) and an average forest coverage (FC) for the watershed data given in Table 3.40.

Solution:

Table 3.40 Data of Problem 6

Year	q_p (m^3/s)	*FC* (%)	Year	q_p (m^3/s)	*FC* (%)	Year	q_p (m^3/s)	*FC* (%)
1982	8000	53	1987	12200	54	1992	5800	46
1983	8800	56	1988	5700	51	1993	14300	44
1984	7400	57	1989	9400	50	1994	11600	43
1985	6700	58	1990	14200	49	1995	10400	42
1986	11100	55	1991	7600	47			

The homogeneity of data is tested using the Spearman test because of the gradual change in watershed imperviousness. Columns 4 and 5 of the following table present the ranks of the annual maximum discharges and the percent of imperviousness. The rank differences are calculated in column 6. Then the value of R_S is calculated as follows:

$$R_s = 1 - \frac{6\sum_{i=1}^{n} d_i^2}{n^3 - n} = 1 - \frac{6(300)}{14^3 - 14} = 0.341$$

The critical value of R_S is 0.457 for significance level of 0.05, from Table 3.33, which is more than the computed value of R_S. This means that deforestation has had a considerable effect on the annual maximum discharge series.

Year	q_p (m^3/s)	Imperviousness (1-*FC*) (%)	Rank of q_p	Rank of Imperviousness	d (Difference)	d^2
1982	8000	47	9	9	0	0
1983	8800	44	8	12	-4	16
1984	7400	43	11	13	-2	4
1985	6700	42	12	14	-2	4
1986	11100	45	5	11	-6	36
1987	12200	46	3	10	-7	49
1988	5700	49	14	8	6	36
1989	9400	50	7	7	0	0
1990	14200	51	2	6	-4	16
1991	7600	53	10	5	5	25
1992	5800	54	13	4	9	81
1993	14300	56	1	3	-2	4

1994	11600	57	4	2	2	4
1995	10400	58	6	1	5	25

7. Analyze the data of problem 5 to evaluate whether or not the increase in urbanization has been accompanied by an increase in the annual maximum discharge. Apply the Spearman test with both a 1% and 5% level of significance. Discuss the results.

Year	*I* (%)	Q_P (cms)	Year	*I* (%)	Q_P (cms)
1980	19	1870	1995	33	2220
1981	20	1530	1996	35	1600
1982	21	1120	1997	37	3810
1983	22	1850	1998	39	2670
1984	23	1820	1999	41	758
1985	24	1700	2000	42	1630
1986	24	2280	2001	42	1620
1987	24	2470	2002	43	2210
1988	25	5010	2003	44	3730
1989	26	2480	2004	44	3980
1990	26	1280	2005	44	3430
1991	27	2080	2006	45	4040
1992	29	2320	2007	45	2000
1993	30	4480	2008	47	4890
1994	32	1860	2009	48	1770

Solution:

The solution procedure is same as what followed in Problem 6 and results are tabulated in the following table.

Year	I (%)	Q_P (cms)	Rank of *I*	Rank of Q_p	d (Difference)	d^2
1980	19	1870	18	18	0	0
1981	20	1530	19	27	-8	64
1982	21	1120	20	29	-9	81
1983	22	1850	21	20	1	1
1984	23	1820	20	21	-1	1
1985	24	1700	20	13	7	49
1986	24	2280	19	11	8	64

1988	25	5010	18	1	17	289
1989	26	2480	17	10	7	49
1990	26	1280	17	28	-11	121
1991	27	2080	16	16	0	0
1992	29	2320	15	12	3	9
1993	30	4480	14	3	11	121
1994	32	1860	13	19	-6	36
1995	33	2220	12	14	-2	4
1996	35	1600	11	26	-15	225
1997	37	3810	10	6	4	16
1998	39	2670	9	9	0	0
1999	41	758	8	30	-22	484
2000	42	1630	7	24	-17	289
2001	42	1620	7	25	-18	324
2002	43	2210	6	15	-9	81
2003	44	3730	5	7	-2	4
2004	44	3980	5	5	0	0
2005	44	3430	5	8	-3	9
2006	45	4040	4	4	0	0
2007	45	2000	4	17	-13	169
2008	47	4890	2	2	0	0
2009	48	1770	1	22	-21	441

Then the value of R_s is calculated as follows:

$$R_s = 1 - \frac{6\sum_{i=1}^{n} d_i^2}{n^3 - n} = 1 - \frac{6(2947)}{30^3 - 30} = 0.344$$

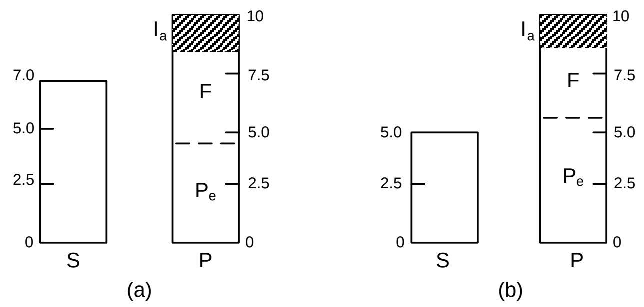
The critical value of R_S is 0.305 for significance level of 0.05, from Table 3.33, which is less than the computed value of Rs. This means that urbanization does not have a considerable effect on the annual maximum discharge series.

Considering a significance level of 0.01, the critical value of R_S is 0.432, from Table 3.33, which is more than the computed value of Rs. This means that urbanization has had a considerable effect on the annual maximum discharge series.

8. Consider two watersheds with different capacities for storage, such as with sandy soil and clay with potential maximum retention of 7 and 5 cm, respectively and also initial abstraction before ponding is 1.5 cm. If a storm of 10 cm during 10 min occurs in both watersheds, determine the percentage of effective rainfall in each watershed.

Solution:

This is shown schematically in the following figure.



Rearranging Equations 3.26 and 3.27 provides the following two equations with two unknowns:

$0 = F - \frac{S}{P - I_a} P_e$ and $P - I_a = F + P_e$

For watershed a we have $0 = F - \left(\frac{7}{8.5}\right) P_e$ and $8.5 = F + P_e$

Solving these two linear simultaneous equations yields P_e = 4.66 cm. and F = 3.84 cm; therefore, $P_e/P = 0.47$.

For the case of watershed b, we have $0 = F - \left(\frac{5}{8.5}\right) P_e$ and $8.5 = F + P_e$. Solving these equations yields P_e= 5.35 cm. and F = 3.15 cm. Therefore, $P_e/P = 0.54$. The results show that the watershed having the greater storage (S) has a smaller protection (P_e/P) of surface runoff. For watershed b the value of F is 0.69 cm less than watershed a, and P_e is 0.69 cm greater.

9. A development project on a small upland watershed is shown in Figure 3.35. The developed portion of the area is 0.7 km^2 in which 21% is an impervious area. The developed area is graded so that runoff is collected in grass-lined swales at the front of the lot and drained in to a paved swale that flows along the side of the main road. Flow from the paved swales passes through a pipe culvert to the upper end of a stream channel. The upper portion of the watershed with a maximum height of 164 m is a forest with B

type soil (CN = 60) and has an area of 0.3 km^2. Estimate the peak runoff of this area if the design return period of the drainage system is 25 years. Use the IDF curve of Problem 3.

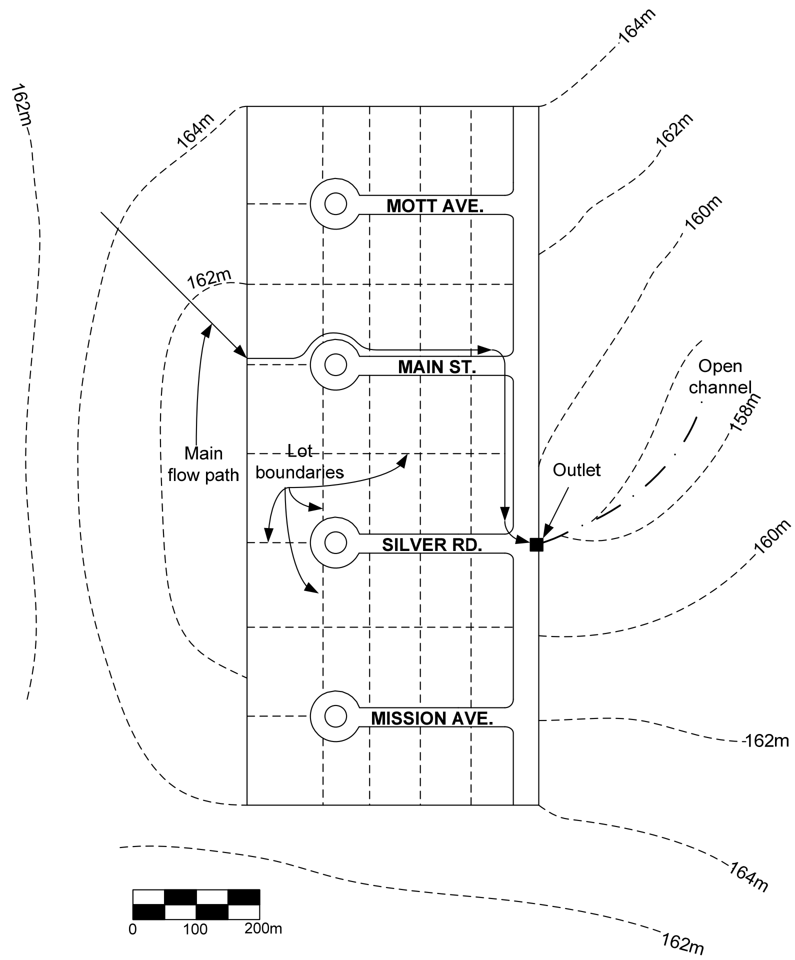


Figure 3.35. A development project on a small upland watershed

Solution:

The main flow path is shown in Figure 3.35 by a series of four arrows. Each arrow in the main flow path represents a different flow regime. The runoff flows overland in the forest, and then enters a grass-lined swale. Computations of the time of concentration are given in the following table. It should be noted that Mannning's roughness coefficients are determined based on Tables 3.21 and 6.2. All the impervious parts of the developed area are connected to the primary drainage system. Thus for B type soil the weighted CN is calculated as follows (CN of impervious area is equal to 98 and the remaining part of the developed area has been considered to have a CN of 61):

For the developed areas CN = $\left[0.21(98)+0.79(61)\right]=68.77$. Equation 3.30 yields the storage equal to 11.5 cm. The storage for the forest portion is 16.9 cm.

A trial and error procedure is followed for estimation of the time of concentration. For calculation of the time of concentration at first an initial value is assumed for effective rainfall intensity and then the time of concentration using Equation 3.5 is calculated. Then the corresponding rainfall intensity with 25-yr return period and duration equal to the time of concentration is estimated from Figure 3.34. Then Equation 3.29 is employed to estimate the runoff intensity. This procedure is followed until the initial estimation of effective rainfall intensity and runoff intensity become the same. A total travel time of 73953 s represents a time of concentration of 20.5 hr. Assuming the unlimited time of rainfall, the peak discharge at the end of the main flow path is calculated as follows:

$$1.34\times10^{-6}\times0.3\times10^{6}+(1.91\times10^{-8}+3.26\times10^{-8}+4.04\times10^{-6})\times0.7\times10^{6}=3.26\ m^3/s$$

Path	Flow Regime	Length	Slope	Manning roughness coefficient	γ	i_e	t_c	i	P	R
		(m)	(m/m)	(-)		(m/s)	(s)	(cm/hr)	(cm)	(m/s)
1	Forest	340	0.007	0.01	8.37	1.34E-06	2063.33	2.36	1.35	1.34E-06
2	Grass swale	480	0.001	0.04	0.79	1.89E-08	57443.59	0.218	3.48	1.91E-08
3	Paved swale	350	0.004	0.015	4.22	3.33E-08	13871.28	0.79	3.04	3.26E-08
4	Pipe	60	0.008	0.015	5.96	4.04E-06	574.56	4.88	0.78	4.04E-06

10. A rainfall hyetograph for a 70-min storm with a total depth of 10.3 cm is shown in Figure 3.36. The depth of direct runoff is 5 cm and the depth of water loss is 5.3 cm. (a) Estimate the Φ index for this basin. (b) Determine the excess rainfall and loss during the rainfall.

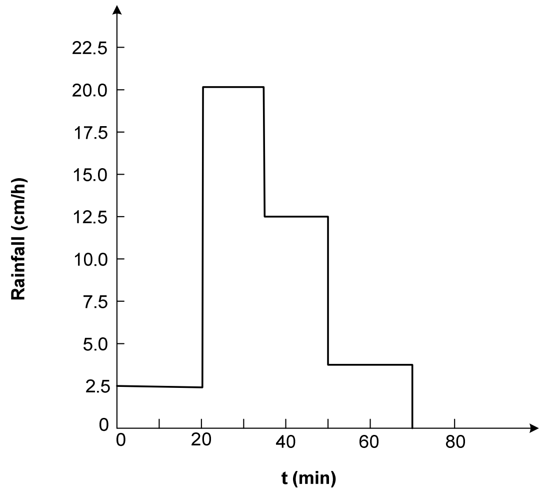


Figure 3.36. The hyetograph of rainfall of Problem 10

Solution:

The depth of rainfall is,

$$P = \frac{1}{60}\left[2.5(20) + 20(15) + 12.5(15) + 4(20)\right] = 10.29\,\text{cm}.$$

The initial estimate of ϕ is $\phi = \frac{P-R}{D} = \frac{10.3-5}{(70/60)} = 4.54$ cm/h. The loss function is calculated as follows:

$$L(t) = \begin{cases} \phi & if\ \phi \le P \\ P & if \phi > P \end{cases}$$

Using the above equation, the loss function is given in column 3 of the following table, and the depths of losses are given in column 4. The total depth of loss is 4.44 cm. Since this is less than the difference between rainfall and runoff, the value of ϕ must be corrected as follows:

$$\Delta\phi = \frac{P-R-V_L}{D_1} = \frac{10.3-5-4.44}{30/60} = 1.72\,\text{cm/h}$$

A value for D_1 of 30 min is used because the initial value of ϕ is bigger than P in the first and last time steps. Thus, the adjusted ϕ is 4.54 + 1.72 = 6.26 cm/h, which is used to compute a revised loss function (column 5). The total loss is now 5.3 cm which equals the depth of total loss. Therefore there is no need for further adjustment. The rainfall-excess intensity, $i_e(t)$, is computed (column 7) as the difference between the rainfall intensity (column 2) and the loss function of column 5. The depths of excess are given in column 8 with a total of 5 cm which equals the depth of direct runoff.

Time period	$i\,(t)$	$L\,(t)$	$V_L\,(t)$	$L\,(t)$	$V_L\,(t)$	$i_e\,(t)$	Excess rainfall
(min)	(cm/h)	(cm/h)	(cm)	(cm/h)	(cm)	(cm/h)	(cm)
0-20	2.5	2.5	0.83	2.5	0.83	0	0
20-35	20	4.54	1.14	6.26	1.57	13.74	3.44
35-50	12.5	4.54	1.14	6.26	1.57	6.24	1.56
50-70	4	4.0	1.33	4	1.33	0	0
Sum			4.44		5.30		5.00

11. Determine the infiltration losses and excess rainfall from the rainfall data given in Table 3.41 on a watershed using the modified Horton equation. The Horton infiltration capacity in the watershed is as follows: $f = 0.2 + 0.4\, e^{-0.6t}$

Table 3.41. The Hyetograph of rainfall of problem 11

Time (h)	*i* (cm/hr)	Time (h)	*i* (cm/hr)
0	1.2	2.00	2.0
0.25	1.2	2.25	1.7
0.05	1.2	2.50	1.7
0.75	1.8	2.75	1.7
1.00	1.8	3.00	0.9
1.25	1.8	3.25	0.9
1.50	2.0	3.50	0.4
1.75	2.0	3.75	0.4
		4.00	0

Solution:

The calculations are summarized in the following table. The F_{e1} values corresponding to t_1 are tabulated in column 4 which for the first time interval F_{e1} = 0. The infiltration capacity in column 5 is calculated by using Equation 3.17 where F_{e1} is used instead of F_e so that the first entry in column 5 is the same as f_0 (the initial infiltration capacity equal to 0.2+0.4=0.6). The smaller value of i and f_p is considered as f (column 6). The entry F_{e2} in column 8 which corresponds to the end of the time interval, is determined using $F_{e2}=F_{e1}+(f - f_f)\Delta t$. By subtracting the infiltration rate from rainfall rate, the rate of effective rainfall i_e, is obtained. The calculations are repeated for all the time intervals in the same manner. It should be noted that the value of F_{e2} in a time step is the F_{e1} for the next step. The total depth of filtered water is equal to $F = \Delta t\, \Sigma f$ = 1.417cm, and the total depth of excess rainfall is equal to 4.258 cm.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
t_1 (hr)	t_2 (hr)	i (cm/hr)	F_{e1} (cm)	f_p (cm/hr)	f (cm/hr)	$(f-f_f)\Delta t$ (cm)	F_{e2} (cm)	i_e (cm/hr)
0	0.25	1.2	0.000	0.600	0.600	0.100	0.100	0.600
0.25	0.50	1.2	0.100	0.540	0.540	0.085	0.185	0.660
0.50	0.75	1.2	0.185	0.489	0.489	0.072	0.257	0.711
0.75	1.00	1.8	0.257	0.446	0.446	0.062	0.319	1.354
1.00	1.25	1.8	0.319	0.409	0.409	0.052	0.371	1.391
1.25	1.50	1.8	0.371	0.378	0.378	0.044	0.415	1.422
1.50	1.75	2.0	0.415	0.351	0.351	0.038	0.453	1.649
1.75	2.00	2.0	0.453	0.328	0.328	0.032	0.485	1.672
2.00	2.25	2.0	0.485	0.309	0.309	0.027	0.512	1.691
2.25	2.50	1.7	0.512	0.293	0.293	0.023	0.535	1.407
2.50	2.75	1.7	0.535	0.279	0.279	0.020	0.555	1.421
2.75	3.00	1.7	0.555	0.267	0.267	0.017	0.572	1.433
3.00	3.25	0.9	0.572	0.257	0.257	0.014	0.586	0.643
3.25	3.50	0.9	0.586	0.248	0.248	0.012	0.598	0.652
3.50	3.75	0.4	0.598	0.241	0.241	0.010	0.608	0.159
3.75	4.00	0.4	0.608	0.235	0.235	0.009	0.617	0.165
		Σ=5.675cm			Σ=1.417cm			Σ=4.258cm

12. A watershed with an area of 230 km^2 is under further development during a 10-year horizon. After finishing the development project the percentage of imperviousness, watershed storage, and basin development factor will change from 25, 4 and 3 to 45, 6 and 9, respectively. Evaluate the effect of the urbanization on the peak discharge of this watershed. Assume that the average slope of watershed is 0.02 m/m. The IDF curve of Problem 3 could be applied in this watershed. The peak discharges from different return periods before urbanization are given in Table 3.42.

Table 3.42. Estimated Peak Discharges for Watershed of Problem 12 in Different Urbanization Conditions

Return Period (yr)	Rural Peaks (m^3/s)
2	32.5
5	49.8
10	67.3
25	86.1
50	104.8
100	150.3
500	210.2

Solution:

Based on the given watershed characteristics and the previous urbanization peak discharges, the parameters of urban peak discharge equations are estimated for the current situation and also after a 10 year horizon. Then the peak discharges for different return periods are estimated using equations of Table 3.18. The intensity of a 2-hr, 2-yr rainfall is estimated using IDF curve of problem 3. The results are tabulated in the following table. The urban peaks are considerably growing by increase of urbanization especially for the smaller return periods. But it is logical that the upper end of the rural flood frequency curve move toward the upper end of the urban curve, because during large storms such as the 500-yr event, the watershed becomes saturated and then reacts almost the same as the impervious watershed.

Estimated peak discharges for watershed of Problem 12 in different urbanization conditions

Return period (yr)	Rural peaks (m^3/s)	Urban peaks (m^3/s)	
		Current situation	After 10-yr horizon
2	32.5	54.04	71.58
5	49.8	77.82	100.70
10	67.3	99.14	126.02
25	86.1	123.70	154.46
50	104.8	146.77	181.09
100	150.3	196.58	242.93
500	210.2	237.82	288.61

13. Calculate the Espey 10-min UH for an urban watershed with an area of 5 km^2, length of 1500 m, percentage of imperviousness of 45%, and a Manning's Roughness coefficient (n) of 0.012. The maximum difference between main water path altitude at the outlet and upstream (*H)* is about 60 m. If during urbanization *H* and *n* decrease to 45m and 0.01, how will the UH of this watershed be affected?

Solution:

First the conveyance coefficient (ϕ) is determined from Figure 3.19. Since n = 0.012, the conveyance coefficient is independent from imperviousness and is estimated to be equal to 0.6. Then Equations 3.33 to 3.42 are employed to obtain the seven points of the hydrograph as follows:

$$S = \frac{60}{(0.8)(1500)} = 0.05$$

$$T_p = \frac{4.1(1500)^{0.23}(0.60)^{1.57}}{(0.05)^{0.25}(45)^{0.18}} = 10.54\,\text{min}$$

$$Q_p = \frac{138.7(5)^{0.96}}{(10.54)^{1.07}} = 52.34\ m^3/s$$

$$T_B = \frac{666.7(5)}{(52.34)^{0.95}} = 77.63\,\text{min}$$

$$W_{50} = \frac{105.1(5)^{0.93}}{(52.34)^{0.92}} = 12.31\,\text{min}$$

$$W_{75} = \frac{45.1(5)^{0.79}}{(52.34)^{0.78}} = 7.34\,\text{min}$$

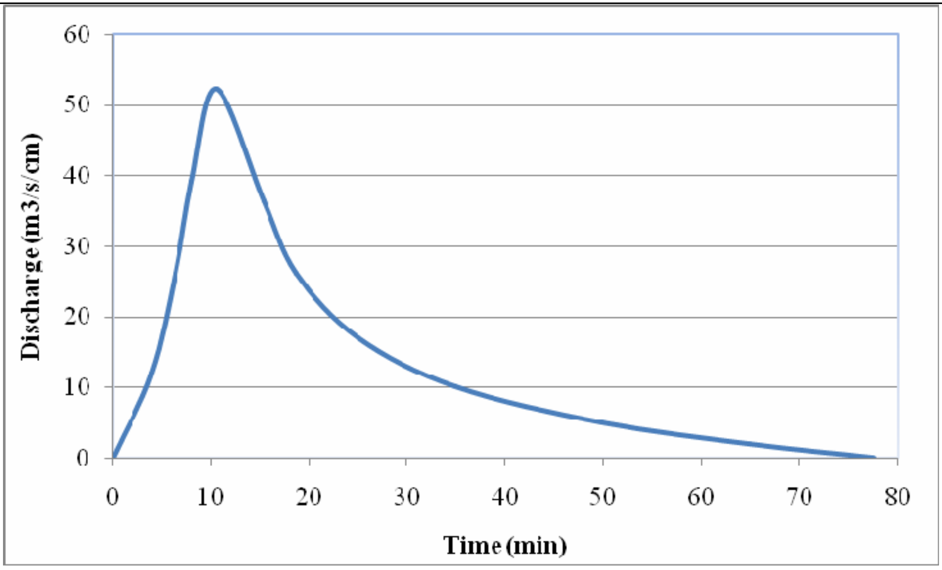
$$t_A = 10.54 - \frac{12.31}{3} = 6.43\,\text{min}$$

$$t_B = 10.54 - \frac{7.34}{3} = 8.10\,\text{min}$$

$$t_E = 10.54 + 2(\frac{7.34}{3}) = 15.43\,\text{min}$$

$$t_F = 10.54 + 2(\frac{12.31}{3}) = 18.74\,\text{min}$$

The discharge at t_A and t_F is $0.50Q_p$ = 26.17 m^3/s/cm and at t_B and t_E, the discharge is $0.75Q_p$ = 39.26 m^3/s/cm. The developed 10-min unit hydrograph of the watershed is depicted in the following figure.



Now the area under the hydrograph is checked to be equal to 1 cm depth of direct runoff. For this purpose the discharges are read from the unit hydrograph at equal time increments and are tabulated in the following table.

Time (min)	Q(m^3/s/cm)	Time (min)	Q(m^3/s/cm)
0	0	44	7
4	12	48	5.5
8	38	52	4.5
12	49	56	3.5
16	30	60	3
20	24	64	2
24	18	68	1.5
28	14.5	72	0.75
32	11.5	76	0.25
36	9.5	80	0
40	8	Sum	242.5

The runoff volume is calculated as follows:

Runoff volume = (242.5 m^3/s)(4 min)(60 s/min) = 58200 m^3

The runoff depth is obtained by dividing the runoff volume by the basin area (5000000 m^2), as follows,

Depth = 58200 m^3/5000000 m^2 = 0.011 m ≈= 1 cm.

As the depth of excess runoff is equal to 1, the hydrograph modification is not needed.

The decrease of n to 0.01 does not cause any change in watershed unit hydrograph but decrease of H to 45m will considerably change the watershed slope and therefore change the ordinates of the hydrograph. The estimated parameters of Espey unit hydrograph are calculated as follows:

$S = 0.038$, $T_p = 11.32\,\text{min}$, $Q_p = 48.46\, m^3/s$, $T_B = 83.51\,\text{min}$, $W_{50} = 13.21\,\text{min}$, $W_{75} = 7.79\,\text{min}$

$t_A = 6.92\,\text{min}$, $t_B = 8.72\,\text{min}$, $t_E = 16.52\,\text{min}$, $t_F = 20.13\,\text{min}$

By comparison of these new values with previous ones, it can be concluded that by the decrease of the watershed slope, the peak discharge volume decreases and is delayed. Furthermore the base time of hydrograph will increase which result in smoother flood hydrograph.

14. Develop the 1 h UH for an urban watershed with an area of 595 km^2 and a time of concentration of 10 h. The time of concentration and area for different regions of this watershed are given in Figure 3.37. Assume a storage coefficient of 45 min.

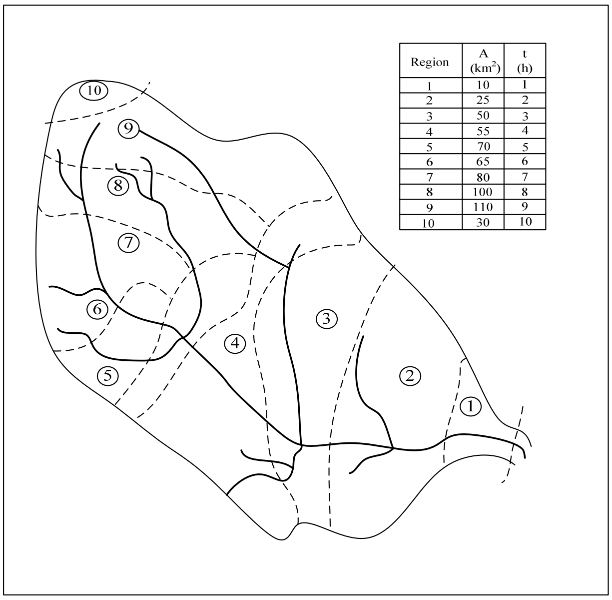


Figure 3.37. The watershed of Problem 14

Solution:

For D =60 min, the rainfall intensity is as i_e = 1cm/3600s = 0.0003 cm/s = 0.000003 m/s. The calculations are summarized in the following table. The time-area curve of this watershed obtained from figure 3.37 is given in Columns 1 and 2 of the following table.

The translation hydrograph presented in Column 3 is calculated by multiplying the rainfall intensity to the corresponding area in the outlet flow at time t. Then C_a and C_b are calculated as follows (using Equation 3.57):

$$C_a = \frac{\Delta t}{R + 0.5\Delta t} = \frac{1}{0.75 + 0.5 \times 1} = 0.8 \text{ and } C_b = 1 - C_a = 1 - 0.8 = 0.2$$

(1)	(2)	(3)	(4)	(5)	(6)	(7)
t (hr)	A_c (km^2)	I (m^3/s)	*IUH* (m^3/s)	*1 hr- lagged IUH*(m^3/s)	*Column 5+ 6* (m^3/s)	*1-hr UH* (m^3/s)
0	0	0	0	0	0.00	0.00
1	10	27.78	13.89	0	13.89	6.94
2	35	97.22	55.56	13.89	69.44	34.72
3	85	236.11	145.83	55.56	201.39	100.69
4	140	388.89	267.36	145.83	413.19	206.60
5	210	583.33	425.35	267.36	692.71	346.35
6	275	763.89	594.62	425.35	1019.97	509.98
7	355	986.11	790.36	594.62	1384.98	692.49
8	455	1263.89	1027.13	790.36	1817.49	908.75
9	565	1569.44	1298.29	1027.13	2325.41	1162.71
10	595	1652.78	1475.53	1298.29	2773.82	1386.91
11		0.00	737.77	1475.53	2213.30	1106.65
12		0.00	368.88	737.77	1106.65	553.32
13		0.00	184.44	368.88	553.32	276.66
14		0.00	92.22	184.44	276.66	138.33
15		0.00	46.11	92.22	138.33	69.17
16		0.00	23.06	46.11	69.17	34.58
17		0.00	11.53	23.06	34.58	17.29
18		0.00	5.76	11.53	17.29	8.65
19		0.00	2.88	5.76	8.65	4.32
20		0.00	1.44	2.88	4.32	2.16
21		0.00	0.72	1.44	2.16	1.08
22		0.00	0.36	0.72	1.08	0.54
23		0.00	0.18	0.36	0.54	0.27
24		0	0.09	0.18	0.27	0.14
25		0	0.05	0.09	0.14	0.07
26		0	0.02	0.05	0.07	0.03
27		0	0.01	0.02	0.03	0.02
28		0	0.01	0.01	0.02	0.01
29		0	0.00	0.01	0.01	0.00
30		0	0.00	0.00	0.00	0.00

The ordinates of IUH are calculated using Equation 3.56 (Column 4). The outflow at time zero is equal to zero. For calculation of the 1-hr unit hydrograph, first the ordinates of two instantaneous unit hydrographs, one of which is lagged 1 hr are summed. The resulting hydrograph results in 2 cm excess rainfall, therefore the results are divided by 2 to develop the 1-hr unit hydrograph of the watershed (column 7).

15. The ordinates of a 4-h unit hydrograph of a catchment are given in Table 3.43. Derive the flood hydrograph in the catchment for the storm figures presented in Table 3.44. The storm loss rate (Φ index)

for the catchment is estimated to be 0.43 cm/h. The base flow at the beginning is 10 m^3/s and will increase by 1.5 m^3/s every 8 h until the end of direct runoff hydrograph.

Table 3.43. The ordinates of 4-h unit hydrograph of problem 15

Time (h)	0	2	4	6	8	10	12	14	16	18	20	22	24	26	28
Q (m^3/s)	0	25	50	85	125	160	185	160	110	60	36	25	16	8	0

Table 3.44. The hyetograph of storm of Problem 15

Time form start of storm (h)	0	4	8	12
Accumulated rainfall (cm)	0	3.5	11.0	16.5

Solution:

The effective rainfall hyetograph is calculated in the following table.

Time interval	0-4	4-8	8-12
Rainfall depth (cm)	3.5	7.5	5
Loss depth (cm)	2.58	2.58	2.58
Effective rainfall (cm)	0.92	4.92	2.42

The direct rainfall hydrograph is calculated in the following table. The ordinates of the unit hydrograph are multiplied by the effective rainfall values successively. The second and third sets of ordinates are advanced by 4 and 8 h respectively and the ordinates at a given time interval are added. The base flow is then added to obtain the flood hydrograph shown in Column 8.

Time (h)	Q (m^3/s)	Unit hydrograph × 0.92	Unit hydrograph × 4.92 (Advanced 4 hr)	Unit hydrograph × 2.42 (Advanced 8 hr)	Ordinates of final hydrograph (m3/s)	Base flow (m3/s)	Ordinates of flood hydrograph (m3/s)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0	0	0.00			0.00	10.00	10.00
2	25	23.00			23.00	10.38	33.38
4	50	46.00	0.00		46.00	10.75	56.75
6	85	78.20	123.00		201.20	11.13	212.33

8	125	115.00	246.00	0.00	361.00	11.50	372.50
10	160	147.20	418.20	60.50	625.90	11.88	637.78
12	185	170.20	615.00	121.00	906.20	12.25	918.45
14	160	147.20	787.20	205.70	1140.10	12.63	1152.73
16	110	101.20	910.20	302.50	1313.90	13.00	1326.90
18	60	55.20	787.20	387.20	1229.60	13.38	1242.98
20	36	33.12	541.20	447.70	1022.02	13.75	1035.77
22	25	23.00	295.20	387.20	705.40	14.13	719.53
24	16	14.72	177.12	266.20	458.04	14.50	472.54
26	8	7.36	123.00	145.20	275.56	14.88	290.44
28	0	0.00	78.72	87.12	165.84	15.25	181.09
30	0	0.00	39.36	60.50	99.86	15.63	115.49
32	0	0.00	0.00	38.72	38.72	16.00	54.72
34	0	0.00	0.00	19.36	19.36	16.38	35.74
36	0	0.00	0.00	0.00	0.00	16.75	16.75