Chapter 4 ANSWER KEY, 5th ed.

4.1 How does river routing differ from reservoir routing?

ANSWER:

River routing differs from reservoir routing in that storage in a river reach of length L depends on more than just outflow. Because storage in a river reach is a function of whether states are rising and falling, storage in river routing is a function of both inflow and outflow.

4.2 Compare and contrast hydrologic and hydraulic routing methods.

ANSWER:

Hydrologic routing involves the balancing of inflow, outflow and volume storage through the use of the continuity equation. A second relationship, the storage-discharge relation, is also required between outflow rate and storage in the system. For Hydraulic routing, both the continuity equation and the momentum equation are solved simultaneously rather than through the use of an empirical storage-discharge relation.

4.3 What are the assumptions made for Kinematic Wave Routing? As a result of these assumptions, what are the shortcomings of the method?

ANSWER:

There is an assumption of uniform flow, meaning that there is a balance between gravitational and frictional forces in the channel. This assumption cannot be justified for very flat slopes. This method cannot be applied when there are backwater effects, upstream movement of tides and storm surges, flood waves in channels of very flat slopes and abrupt waves caused by sudden releases from reservoirs or dam failures,

4.4 How does the method in Problem 4.20 compare with the kinematic wave routing method described in this chapter? What hydraulic conditions are necessary for kinematic wave assumptions to be valid for overland flow? What are advantages and disadvantages of using kinematic channel routing compared with solving the Saint Venant equations?

ANSWER:

The method of Problem 4.20 is similar to kinematic wave routing except that values are evaluated at either end of a channel reach instead of being evaluated in a grid along the channel. For overland flow, the kinematic wave number is greater than 10 for small depths and $Fr < 2$. Kinematic wave routing is easier than dynamic wave routing but may not be as accurate for cases of steep slope or for long rivers with backwater effects.

4.5 What is a rating curve? How does a loop rating curve occur?

ANSWER:

A rating curve is a plot of water level (stage) vs. discharge. If flow cannot be approximated as uniform, discharge will be greater during the rising stage of a flood hydrograph than during the falling stage, leading to a loop rating curve.

4.6 An inflow hydrograph is measured for a cross section of a stream. Compute the outflow hydrograph at a point five miles downstream using the Muskingum method. Assume K = 12 hr, x = 0.2, and that outflow equals inflow initially. Plot the inflow and outflow hydrographs.

Time (hr)	Inflow (cfs)
6	50
12	75
18	150
24	450
30	1000
36	840
42	750
48	600
54	300
60	100
66	50

ANSWER:

We have K = 12 hr, x = 0.2, and Δt = 6 hr. Substituting these into Eqs. 4–7 to 4–10 we get:

$D = K - Kx + 0.50\Delta t$

$= 12 - 12(0.2) + 0.50\,(6)$

$= 12.6$

$C_o = (-Kx + 0.5\Delta t) / D$

$= (-(12)(0.2) + 0.5(6)) / 12.6$

$= 0.0476$

$C_1 = (Kx + 0.5\Delta t) / D$

$= ((12)(0.2) + 0.5(6)) / 12.6$

$= 0.4286$

$C_2 = (K - Kx - 0.5\Delta t) / D$

$= (12 - (12)(0.2) - 0.5(6)) / 12.6$

$= 0.5238$

4.6 cont'

Check: $C_0 + C_1 + C_2 = 1.0000$

$$0.0476 + 0.4286 + 0.5238 = 1.0000$$

Substitute the coefficients into Eq. (4-6):

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$$

$$Q_2 = 0.0476\, I_2 + 0.4286\, I_1 + 0.5238\, Q_1$$

For t = 6 hr,

$$Q_1 = I_1 = 50 \text{ cfs}$$

For t = 12 hr,

$$Q_2 = (0.0476)(75) + (0.4286)(50) + (0.5238)(50) = 51.19 \text{ cfs}$$

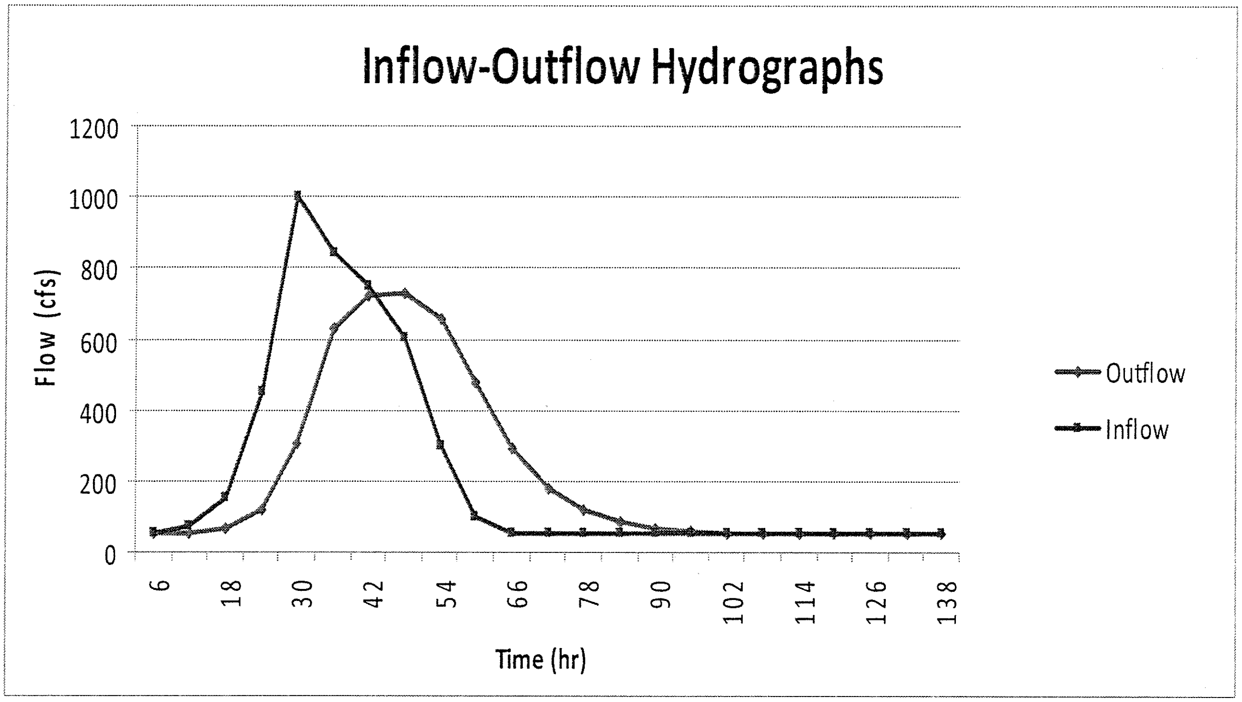
For t = 18 hr,

$$Q_2 = (0.0476)(150) + (0.4286)(75) + (0.5238)(51.19) = 66.10 \text{ cfs}$$

Continue the procedure until the inflow and outflow curves flatten out.

Time (hr)	I_2 (cfs)	I_1 (cfs)	Q_1 (cfs)	Q_2 (cfs)
6	50	0	0	50.00
12	75	50	50	51.19
18	150	75	51	66.10
24	450	150	66	120.33
30	1000	450	120	303.50
36	840	1000	304	627.56
42	750	840	628	724.44
48	600	750	724	729.47
54	300	600	729	653.54
60	100	300	654	475.66
66	50	100	476	294.39
72	50	50	294	178.01
78	50	50	178	117.05
84	50	50	117	85.12
90	50	50	85	68.40
96	50	50	68	59.64
102	50	50	60	55.05
108	50	50	55	52.64
114	50	50	53	51.38
120	50	50	51	50.73
126	50	50	51	50.38
132	50	50	50	50.20
138	50	50	50	50.10

4.6 cont'


Inflow-Outflow Hydrographs
1200
1000
800
600
400
200
0
Flow (cfs)
6
18
30
42
54
66
78
90
102
114
126
138
Time (hr)
Outflow
Inflow

4.7 Using the Muskingum method, route the following inflow hydrograph assuming (a) $K = 4$ hr, $x = 0.1$, and (b) $K = 2$ hr, $x = 0.3$. Plot the inflow and outflow hydrographs for each case assuming initial outflow equals initial inflow.

Time (hr)	Inflow (m^3/s)
0	0
2	5
4	25
6	50
8	35
10	21
12	13
14	7.5
16	2.5
18	0

ANSWER:

(a) $K = 4\text{hr}, x = 0.1$

$$D = K - Kx + 0.5\Delta t = 4 - (4)(0.1) + (0.5)(2) = 4.60$$

$$C_0 = (-Kx + 0.5\Delta t) / D = (-4(0.1) + 0.5(2)) / 4.6 = 0.13$$

$$C_1 = (Kx + 0.5\Delta t) / D = (4(0.1) + 0.5(2)) / 4.6 = 0.30$$

$$C_2 = (K - Kx - 0.5\Delta t) / D = (4 - 4(0.1) - 0.5(2)) / 4.6 = 0.57$$

Find outflows using these coefficients and the equations below

$$O_2 = C_0I_2 + C_1I_1 + C_2O_1 = (0.13)(5) + (0.3)(0) + (0.57)(0) = 0.7$$

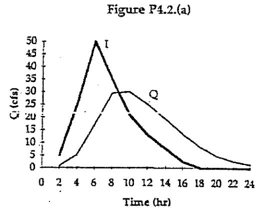
Assuming: $O_1 = I_1 = O$

Similarly, $O_3 = C_0I_3 + C_1I_2 + C_2O_2$

The table and figure below summarize the results of the process.

Time (hr)	Inflow (cfs)	Outflow (cfs)
0	0	0.0
2	5	0.7
4	25	5.2
6	50	17.0
8	35	29.4
10	21	30.0
12	13	25.1
14	7.5	19.1
16	2.5	13.4
18	0	8.3
20	0	4.7
22	0	2.7
24	0	1.5

Figure P4.2.(a)



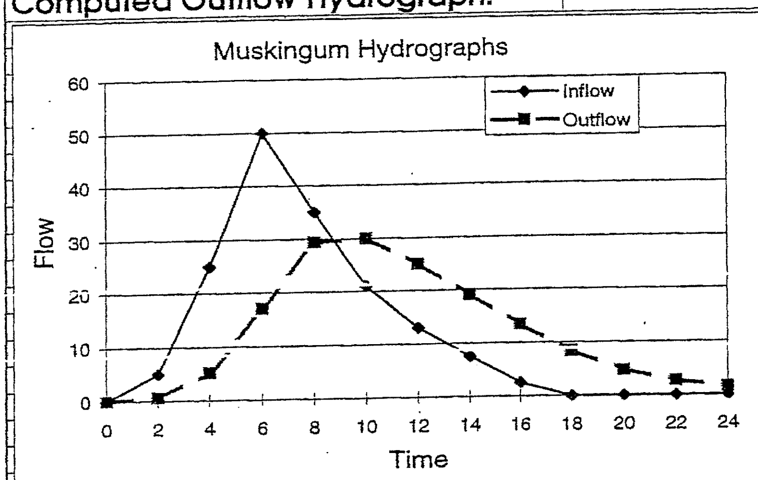
4.8 Develop a Muskingum routing program in Excel, and repeat problem 4.7. Compare the results.

ANSWER:

Part (a)

Inputs:			
	x ==>	0.1	
	K ==>	4	K and dt must have the same units.
	dt ==>	2	
	O1 ==>	0	
Coefficients:			
	C0 =	0.1304	
	C1 =	0.3043	
	C2 =	0.5652	
	D =	4.6	
	C0+C1+C2=	1	Must equal one.
Input Inflow Hydrograph:			
Time	Inflow	Outflow	
0	0	0.00	
2	5	0.65	
4	25	5.15	
6	50	17.04	
8	35	29.42	
10	21	30.02	
12	13	25.05	
14	7.5	19.10	
16	2.5	13.40	
18	0	8.34	
20	0	4.71	
22	0	2.66	
24	0	1.51	

Computed Outflow Hydrograph:



Part (b)

	A	B	C
	Inputs:		
2		x ==>	0.3
3		K ==>	2
4		dt ==>	2
5		O1 ==>	0
6	Coefficients:		
7		C0 =	(((0.5*$C$4) - ($C$3*C2))/$C$10)
8		C1 =	(($C$3*C2 + 0.5*C4)/$C$10)
9		C2 =	((((1-$C$2)*$C$3) - 0.5*C4)/ $C$10)
10		D =	((1-$C$2)*$C$3) + 0.5*$C$4
11		C0+C1+C2=	SUM($C$7:$C$9)
12	Input Inflow Hydrograph:		
13	Time	Inflow	Outflow
14	0	0	$C$5
15	2	5	($C$7*B15) + ($C$8*B14) + ($C$9*C14)
16	4	25	($C$7*B16) + ($C$8*B15) + ($C$9*C15)
17	6	50	($C$7*B17) + ($C$8*B16) + ($C$9*C16)
18	8	35	($C$7*B18) + ($C$8*B17) + ($C$9*C17)
19	10	21	($C$7*B19) + ($C$8*B18) + ($C$9*C18)
20	12	13	($C$7*B20) + ($C$8*B19) + ($C$9*C19)
21	14	7.5	($C$7*B21) + ($C$8*B20) + ($C$9*C20)
22	16	2.5	($C$7*B22) + ($C$8*B21) + ($C$9*C21)
23	18	0	($C$7*B23) + ($C$8*B22) + ($C$9*C22)
24	20	0	($C$7*B24) + ($C$8*B23) + ($C$9*C23)
25	22	0	($C$7*B25) + ($C$8*B24) + ($C$9*C24)
26	24	0	($C$7*B26) + ($C$8*B25) + ($C$9*C25)

4.9 Flood route the given input hydrograph through a linear reservoir ($S = KQ$), given K = 2.5 hr and Δt = 1 hr. Solve one step beyond the peak outflow by developing a simple relation for computing outflow Q_2, as a function of Q_1, and inflows I_1 and I_2. Assume storage and outflow are initially zero. Use the fact that $S = K(xI + (1-x)Q)$ and $I - Q = \Delta S/\Delta t$. (Hint: use the fact that the reservoir is linear to determine x).

Time (hr)	Inflow (cfs)
0	0
1	100
2	250
3	400
4	350
5	300
6	200
7	100
8	50
9	0

The Muskingum eqn is $Q_2 = C_0I_2 + C_1I_1 + C_2Q_1$.

$$C_o = \frac{-Kx + 0.5\Delta t}{D}$$

$$C_1 = \frac{Kx + 0.5\Delta t}{D}$$

$$C_2 = \frac{K - Kx - 0.5\Delta t}{D}$$

$$D = K - Kx + 0.5\Delta t$$

ANSWER:

In order for both $S = KQ$ and $S = K(xI + (1-x)Q)$ to be true, $x = 0$.

$$D = 2.5 - 2.5(0) + 0.5(1) = 3$$

$$C_o = \frac{-2.5(0) + 0.5(1)}{3} = 0.167$$

$$C_1 = \frac{2.5(0) + 0.5(1)}{3} = 0.167$$

4.9 cont'

$$C_2 = \frac{2.5 - 2.5(0) - 0.5(1)}{3} = 0.667$$

Using the Muskingum equation $Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$ and the assumption that $I_o = O_o$. Outflow can be calculated either by hand or using a program such as Excel.

Time (hr)	**Inflow (cfs)**	**Outflow (cfs)**		**Coefficients**
0	0	0.0	k =	2.5
1	100	16.7	x =	0
2	250	69.4	Δt =	1
3	400	154.6	D =	3
4	350	228.1	C_o =	0.167
5	300	260.4	C_1 =	0.167
6	200	256.9	C_2 =	0.667
7	100	221.3		
8	50	172.5		
9	0	123.3		
10	0	82.2		
12	0	54.8		
13	0	36.5		
14	0	24.4		
15	0	16.2		
16	0	10.8		
17	0	7.2		
18	0	4.8		
19	0	3.2		
20	0	2.1		

4.10 A detention pond needs to be designed with a total capacity of 30 ac-in. of storage. The inflow hydrograph for the pond is given in Figure P4-10. Assume that that pond is initially 50% full and outflow will cease when the pond is again 50% full. Graphically determine the peak outflow from the pond assuming a linear rise and fall in the outflow hydrograph. Draw the outflow hydrograph. At what time does outflow cease? (Hint: The outflow hydrograph will peak at the intersection of the inflow and the outflow hydrographs. Also, make use of the fact that the available volume for storage is equal to the area between the two triangular hydrographs and must be equal to 50% of the storage.) In the following figure, the inflow hydrograph is completely accurate, but the outflow hydrograph is merely an approximation.

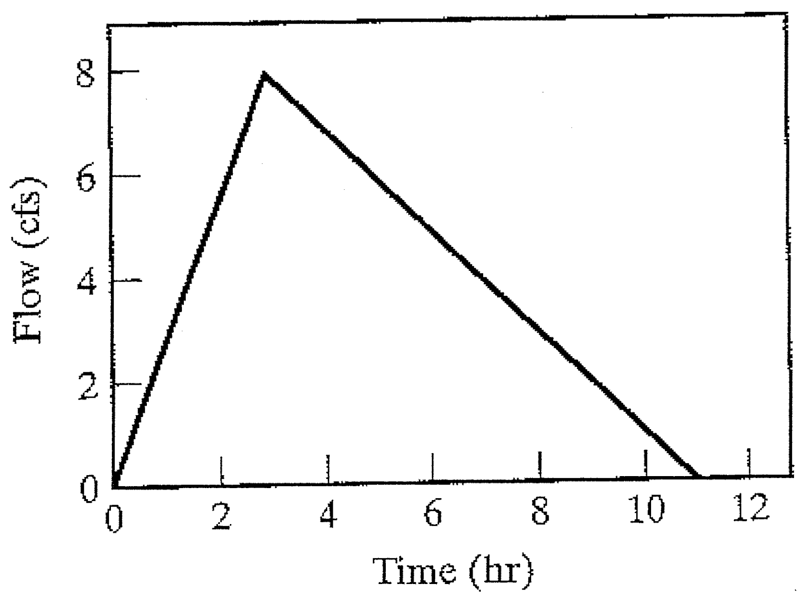


Fig. P4-10

ANSWER:

To find the actual location of point D, we first need to find the area of triangles ABD, ABC, and ADC.

$\Delta ABD = 15$ ac-in. $= 15$ cfs-hr is given

$\Delta ABC = (1/2)(11 \text{ hr})(8 \text{ cfs}) = 44$ cfs-hr

$\Delta ADC = \Delta ABC - \Delta ABD = (44 - 15)$ cfs-hr $= 29$ cfs-hr

Because we know the area of $\Delta ADC = (1/2)(b)(h)$, we can find the true y-coordinate of the intersection of the inflow and outflow hydrographs.

$\Delta ADC = (1/2)(11)(y_d)$

4.10 cont'

$y_d = 5.27$ cfs

Now that we know the peak flow is 5.27 cfs, we can find the time at which it occurs. The equation of the line between points B and C is $y = -x + 11$.

Thus, at $y_d = 5.27$ cfs, $x_d = 5.73$ hr.

The point D is located at (5.73, 5.27).

To determine the time at which outflow ceases (x_e), we make use of the fact that the area of ΔCDE is equal to 15 cfs-hr. Since we know x_d, we can find the areas of ΔCDF and ΔDEF.

$\Delta CDF = (1/2)(b)(h) = (1/2)(11-x_d)(y_d) = (1/2)(11 - 5.73)(5.27) = 13.89$ cfs-hr

$\Delta DEF = \Delta CDF + \Delta CDE = 13.89 + 15 = 28.89$ cfs-hr

$\Delta DEF = 28.89$ cfs-hr $= (1/2)(EF)(5.27)$

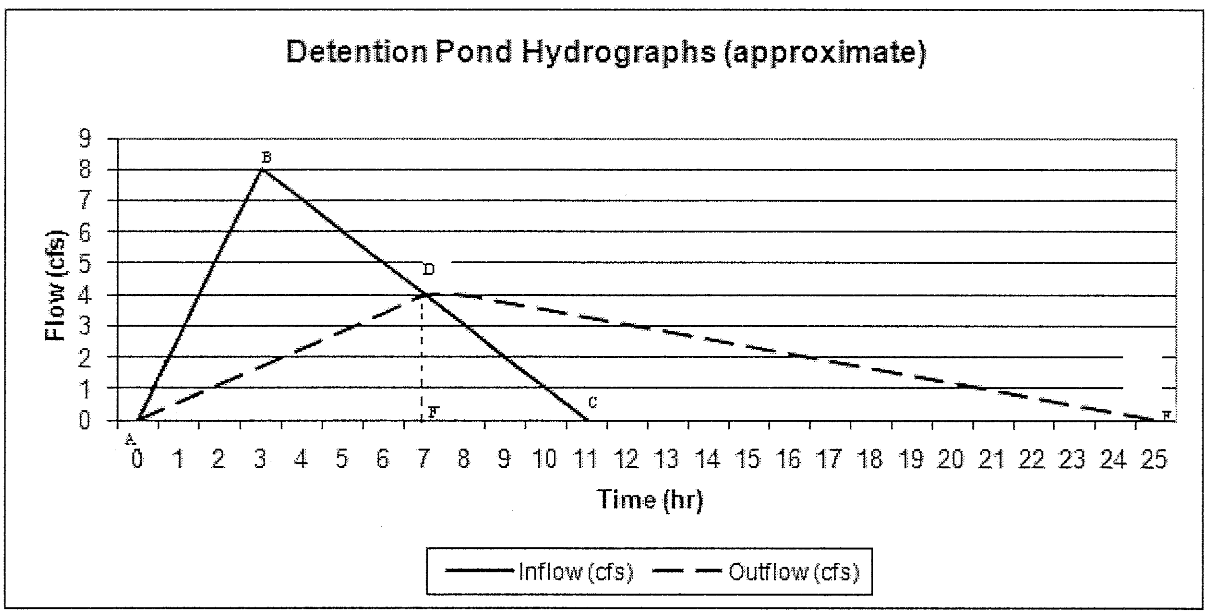
$EF = 10.96$

$x_e = x_d + 10.96 = 16.7$ hr

Therefore, the following coordinates for the outflow hydrograph should change from Fig. P4-10:

old point D (7 hr, 4 cfs) → new point D (5.73 hr, 5.27 cfs)

old point E (24 hr, 0 cfs) → new point E (16.7 hr, 0 cfs)



4.11 An inflow hydrograph is given for a reservoir that has a weir-spillway outflow structure. The flow through the spillway is governed by the equation

$$Q = 3.75 L y^{3/2} \text{ (cfs)},$$

where L is the length of the weir and y is the height of the water above the spillway crest. The storage in the reservoir is governed by

$$S = 300y \text{ (ac-ft)}.$$

Using Δt = 12 hr, L = 15 ft, and $S_0 = Q_0 = O$, route the inflow hydrograph through the reservoir using the storage indication method.

Time (hr)	Inflow (cfs)
12	40
24	35
36	37
48	125
60	340
72	575
84	722
96	740
108	673
120	456
132	250
144	140
156	10

Table P4.11

4.11 cont'

ANSWER:

First, we develop a table of $2S_n/\Delta t + Q$ versus Q.

y (ft)	Q (cfs)	S (ac-ft)	S (cfs-hr)	2S/Δt + Q (cfs)
0	0	0	0	0
0.2	5.03	60	720	125
0.4	14.23	120	1440	254
0.6	26.14	180	2160	386
0.8	40.25	240	2880	520
1	56.25	300	3600	656
1.2	73.94	360	4320	794
1.4	93.18	420	5040	933
1.6	113.8	480	5760	1074
1.8	135.8	540	6480	1216
2	159.1	600	7200	1359
2.2	183.6	660	7920	1504
2.4	209.1	720	8640	1649
2.6	235.8	780	9360	1796
2.8	263.6	840	10080	1944
3	292.3	900	10800	2092
3.2	322	960	11520	2242
3.4	352.7	1020	12240	2393
3.6	384.2	1080	12960	2544
3.8	416.7	1140	13680	2697
4	450	1200	14400	2850
4.2	484.2	1260	15120	3004
4.4	519.2	1320	15840	3159
4.6	555	1380	16560	3315
4.8	591.5	1440	17280	3472
5	628.9	1500	18000	3629

4.11 cont'

Next, Eq. 2.13 is solved for the right hand side. This value is used to find a value of Q. For example:

$Q_1 = 0$, $s_I = 0$, $(2S_1/\Delta t - Q_I) = 0$

$(I_1 + I_2) + (2S_1/\Delta t - Q_1) = (2S_2/\Delta t - Q_2)$

$(40 + 35) + (0 - 0) = 75$

For $(2S/\Delta t + Q) = 75$, $Q = 3.02$. The values are tabulated as:

4.11 cont'

Time (hr)	I_i (cfs)	$I_i + I_{i+1}$ (cfs)	$2S_{i+1}/\Delta t + Q_i$ (cfs)	$2S_{i+1}/\Delta t + Q_{i+1}$ (cfs)	Q_i (cfs)
12	40	75	0	75	0
24	35	72	69	141	3.0
36	37	162	129	291	6.0
48	125	465	256	721	17.5
60	340	915	592	1507	64.6
72	575	1297	1139	2436	184.1
84	722	1462	1713	2175	361.6
96	740	1413	2129	3542	522.8
108	673	1129	2325	3454	608.3
120	456	706	2279	2985	587.4
132	250	390	2025	2415	480.0
144	140	150	1701	1851	357.3
156	10	10	1359	1369	246.0
168	0	0	1048	1048	160.8
180	0	0	828	828	110.0
192	0	0	671	671	78.7
204	0	0	555	555	58.1
216	0	0	466	466	44.3
228	0	0	397	397	34.6
240	0				27.3

4.12 A reservoir has a linear *S*-*Q* relationship of

$$S = KQ,$$

where $K = 1.21$ hr. The inflow hydrograph for a storm event is given in the table.

(a) Develop a simple recursive relation using the continuity equation and *S*-*Q* relationship for the linear reservoir [i.e., $aQ_2 = bQ_1 + c\bar{I}$, where *a, b,* and *c* are constants and $\bar{I} = (I_1 + I_2)/2$].

(b) Storage route the hydrograph through the reservoir using $\Delta t = 1$ hr.

(c) Explain why the shape of storage-discharge relations is usually not linear for actual reservoirs.

Time (hr)	Inflow (m^3/s)
0	0
1	100
2	200
3	400
4	300
5	200
6	100
7	50
8	0

ANSWER:

(a) The continuity equation,

$$\text{In} - \text{Out} = \Delta S/\Delta t$$

Can be re-written as:

$$(I_i + I_{i+1})(\Delta t/2) - (O_i + O_{i+1})(\Delta t/2) = \Delta S = S_{i+1} - S_i$$

Substituting for $S = KQ$ and rearranging yields:

$$(I_i + I_{i+1})(\Delta t/2) - (O_i + O_{i+1})(\Delta t/2) = K(O_{i+1} - O_i)$$

$$(K/\Delta t + 0.5)O_{i+1} = \bar{I} + (K/\Delta t - 0.5)O_i$$

(b) For $K = 1.21$ and $\Delta t = 1$hr, the equation from part (a) becomes:

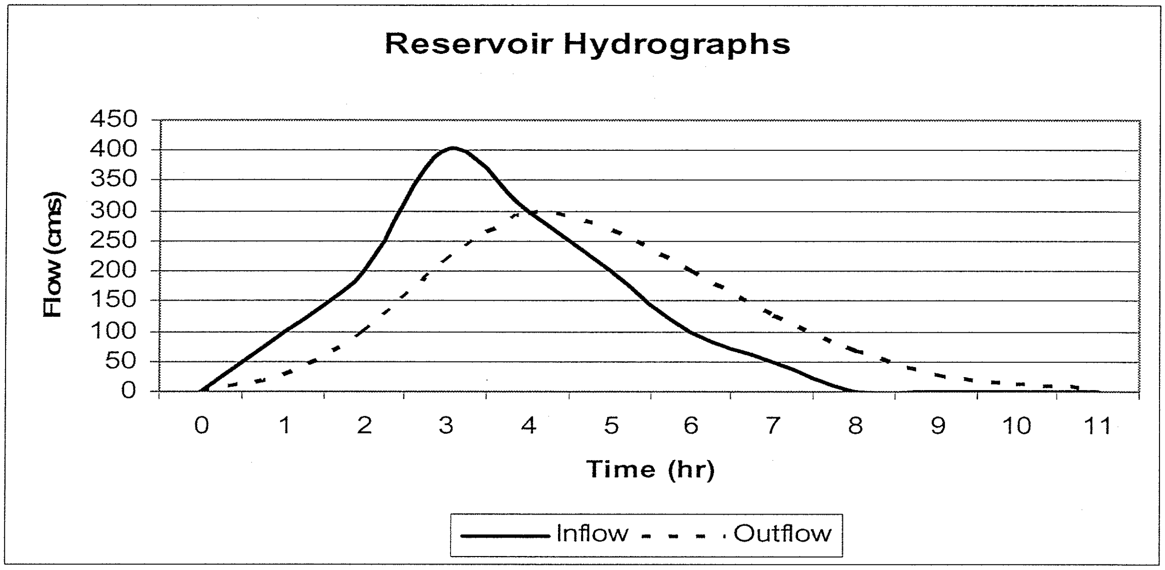
$$(1.71)O_{i+1} = \bar{I} + (0.71)O_i$$

Note: The hydrographs are based on the columns *I* and *Q*, where $Q = O_{i+1}$

4.12 cont'

Time (hrs)	I (cms)	$\bar{I}$ (cms)	$0.71O_i$ (cms)	$1.71O_{i+1}$ (cms)	Q (cms)
0	0				0
		50	0	50.0	
1	100				29.2
		150	20.8	170.8	
2	200				99.9
		300	70.9	370.9	
3	400				216.9
		350	154.0	504.0	
4	300				294.7
		250	209.3	459.3	
5	200				268.6
		150	190.7	340.7	
6	100				199.2
		75	141.5	216.5	
7	50				126.6
		25	89.9	114.9	
8	0				67.2
		0	47.7	47.7	
9	0				27.9
		0	19.8	19.8	
10	0				11.6
		0	8.2	8.2	
11	0				4.8

4.12 cont'



	B	C	D	E	F	G
	Time (hrs)	**I** (cms)	**$\bar{I}$** (cms)	**$0.71O_i$** (cms)	**$1.71O_{i+1}$** (cms)	**Q** (cms)
3	**0**	0				0
4			(C3 + C5) / 2	G3 * 0.71	D4 + E4	
5	**1**	100				F4 / 1.71
6			(C5 + C7) / 2	G5 * 0.71	D6 + E6	
7	**2**	200				F6 / 1.71
z8			(C7 + C9) / 2	G7 * 0.71	D8 + E8	
9	**3**	400				F8 / 1.71
10			(C9 + C11) / 2	G9 * 0.71	D10 + E10	
11	**4**	300				F10 / 1.71
12			(C11 + C13) / 2	G11 * 0.71	D12 + E12	
13	**5**	200				F12 / 1.71
14			(C13 + C15) / 2	G13 * 0.71	D14 + E14	
15	**6**	100				F14 / 1.71
16			(C15 + C17) / 2	G15 * 0.71	D16 + E16	
17	**7**	50				F16 / 1.71
18			(C17 + C19) / 2	G17 * 0.71	D18 + E18	

19	**8**	0				F18 / 1.71
20			(C19 + C21) / 2	G19 * 0.71	D20 + E20	
21	**9**	0				F20 / 1.71
22			(C21 + C23) / 2	G21 * 0.71	D22 + E22	
23	**10**	0				F22 / 1.71
24			(C23 + C25) / 2	G23 * 0.71	D24 + E24	
25	**11**	0				F24 / 1.71

(c) Storage is rarely uniform with depth since few reservoirs are uniform in shape. Most outflow structures have flow relations which are a function of depth raised to a power. Both of these facts lead to a non-linear storage-discharge relation.

4.13 Given the reservoir with a storage-discharge relationship governed by the equation

$$S = KQ^{3/2},$$

route the inflow hydrograph for problem 4.5 using storage routing techniques and a value of $K = 1.21$ for Q in m^3/s and S in m^3/s-hr. Discuss the differences in the outflow hydrograph for this reservoir and for the reservoir of problem 4.12. Use $\Delta t = 1$ hr.

ANSWER:

From the storage equation given,

$$S = KQ^{3/2}\ ,\quad K = 1.21$$

We can obtain the storage-discharge relationship and the storage-indication curve:

For example, take Q = 50 cms:

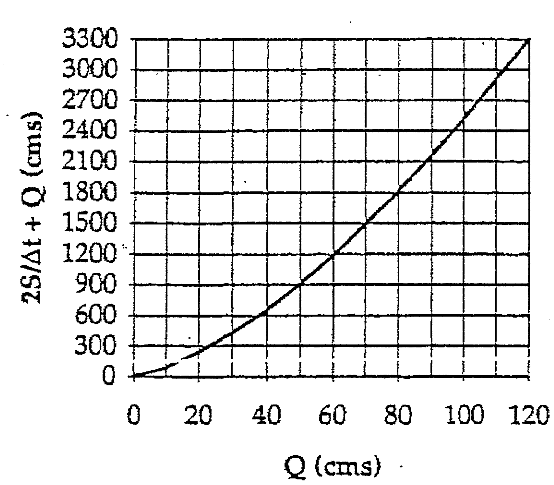
$$S = 1.21\,(50)^{3/2} = (1.21)(353.55) = 427.80 \text{ cms-hr}$$

$$2S/\Delta t + Q = [2\,(427.80) \text{ cms-hr} / 1 \text{ hr}] + 50 \text{ cms} = 906 \text{ cms}$$

The following table and graph show these computations for a range of outflows:

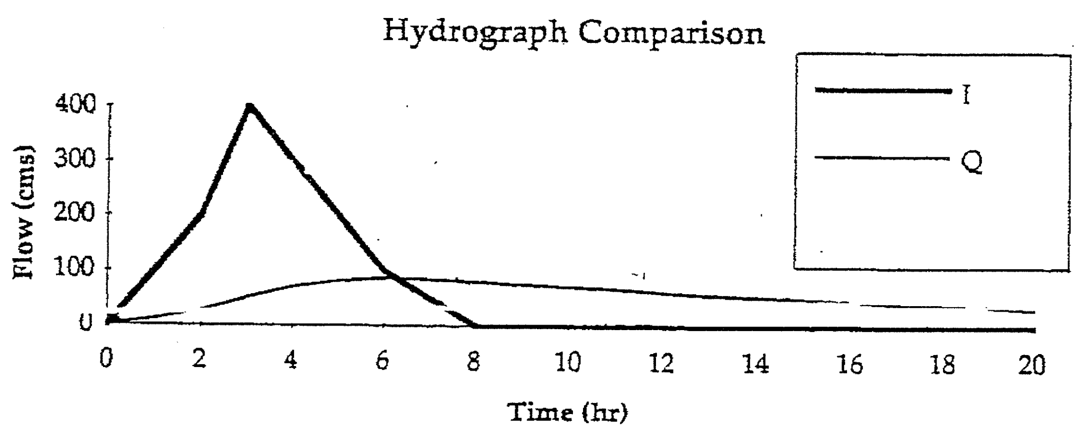
Storage (cms - hr)	Outflow (cms)	2S/Δt + Q (cms)
0.00	0	0
38.26	10	87
108.23	20	236
198.82	30	428
306.11	40	652
427.80	50	906
562.36	60	1185
708.65	70	1487
865.81	80	1812
1033.12	90	2156
1210.00	100	2520
1395.96	110	2902
1590.59	120	3301

Storage-Indication Curve



Then using storage-indication, we obtain a new outflow hydrograph for comparison to problem

Time (hr)	In (cms)	In + I(n+1)	(2Sn/Δt - Qn)	2Sn/Δt + Q(n+1)	Qn (cms)
0	0	100	0		0
1	100	300	78	100	11
2	200	600	324	378	27
3	400	700	822	924	51
4	300	500	1380	1522	71
5	200	300	1716	1880	82
6	100	150	1844	2016	86
7	50	50	1824	1994	85
8	0	0	1710	1874	82
9	0	0	1556	1710	77
10	0	0	1412	1556	72
11	0	0	1276	1412	68
12	0	0	1150	1276	63
13	0	0	1034	1150	58
14	0	0	926	1034	54
15	0	0	824	926	51
16	0	0	730	824	47
17	0	0	646	730	42
18	0	0	566	646	40
19	0	0	492	566	37
20	0	0	426	492	33



There is a drastic reduction in the peak outflow for this S-Q relationship. The Linear reservoir does not reduce the peak by such an amount.

4.14 A storm event occurred on Falls Creek Watershed that produced a rainfall pattern of 5 cm/hr for the first 10 min, 10 cm/hr in the second 10 min, and 5 cm/hr in the next 10 min. The watershed is divided into three subbasins (see Fig. P4–14), with the unit hydrographs given in the following table. Subbasins *A* and *B* had a loss rate of 2.5 cm/hr for the first 10 min and 1.0 cm/hr thereafter. Subbasin *C* had a loss rate of 1.0 cm/hr for the first 10 min and 0 cm/hr thereafter. Using a simple lag routing method (time shift the hydrograph by *K*), determine the storm hydrograph at point 2. Assume a lag time *K* of 30 min.

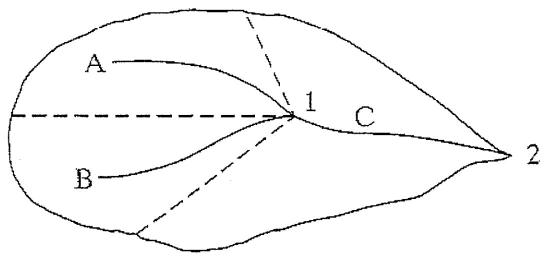


Fig. P4–14

Subbasin *A*		Subbasin *B*		Subbasin *C*	
Time (min)	*Q* (m^3/s)	Time (min)	*Q* (m^3/s)	Time (min)	*Q* (m^3/s)
0	0	0	0	0	0
10	5	10	5	10	16.7
20	10	20	10	20	33.4
30	15	30	15	30	50.0
40	20	40	20	40	33.4
50	25	50	25	50	16.7
60	20	60	20	60	0
70	15	70	15		
80	10	80	10		
90	5	90	5		
100	0	100	0		

Ten-Minute Unit Hydrographs

ANSWER:

First develop the storm hydrograph for each subbasin.

The rainfall and the unit hydrographs for A and B are identical, their storm hydrographs are the same.

$$P_1 = (5\ \text{cm/hr} - 2.5\ \text{cm/hr})(1/6\ \text{hr}) = 0.417\ \text{cm}$$

4.14 cont'

$P_2 = (10 \text{ cm/hr} - 1 \text{ cm/hr})(1/6 \text{ hr}) = 1.50 \text{ cm}$

$P_3 = (5 \text{ cm/hr} - 1 \text{ cm/hr})(1/6 \text{ hr}) = 0.667 \text{ cm}$

Time (min)	U_A (m^3/s)	P_1U_A	P_2U_A	P_3U_A	$Q_A = Q_B$ (m^3/s)
0	0	0			0.0
10	5	2.1	0		2.1
20	10	4.2	7.5	0	11.7
30	15	6.3	15	3.35	24.7
40	20	8.4	22.5	6.7	37.6
50	25	10.5	30	10.05	50.6
60	20	8.4	37.5	13.4	59.3
70	15	6.3	30	16.75	53.1
80	10	4.2	22.5	13.4	40.1
90	5	2.1	15	10.05	27.2
100	0	0	7.5	6.7	14.2
110			0	3.35	3.4
120				0	0.0

For subbasin C:

$P_1 = (5 \text{ cm/hr} - 1 \text{ cm/hr})(1/6 \text{ hr}) = 0.667 \text{ cm}$

$P_2 = (10 \text{ cm/hr})(1/6 \text{ hr}) = 1.67 \text{ cm}$

$P_3 = (5 \text{ cm/hr})(1/6 \text{ hr}) = 0.833 \text{ cm}$

Time (min)	U_c (m^3/s)	P_1U_c	P_2U_c	P_3U_c	Q_c (m^3/s)
0	0	0.0			0.0
10	16.7	11.2	0.0		11.2
20	33.4	22.4	27.9	0.0	50.3
30	50	33.5	55.8	13.9	103.1
40	33.4	22.4	83.5	27.7	133.6
50	16.7	11.2	55.8	41.5	108.5
60	0	0.0	27.9	27.7	55.6
70			0.0	13.9	13.9
80				0.0	0.0

4.14 cont'

Next to get the storm hydrograph at point 2, we combine the hydrographs from A and B at point 1, route this combined hydrograph through the reach 1-2 and combine it with the hydrograph from C to point 2. To combine A and B double the flow from A. To route it, lag by 30 min.

Time (min)	Q_c (m^3/s)	Q_A (m^3/s) lagged	$2Q_A$ (m^3/s) lagged	Q at point 2
0	0.0			0.0
10	11.2			11.2
20	50.3			50.3
30	103.1	0.0	0	103.1
40	133.6	2.1	4.2	137.8
50	108.5	11.7	23.4	131.9
60	55.6	24.7	49.3	104.9
70	13.9	37.6	75.2	89.1
80	0.0	50.6	101.1	101.1
90		59.3	118.6	118.6
100		53.1	106.1	106.1
110		40.1	80.2	80.2
120		27.2	54.3	54.3
130		14.2	28.4	28.4
140		3.4	6.7	6.7
150		0.0	0	0.0

4.15 Repeat problem 4.14 using Muskingum routing methods instead of simple lag routing. Discuss the differences between the two storm hydrographs. Use $x = 0.2$, $K = 30$ min, and $\Delta t = 10$ min.
The hydrographs in each subarea are found in problem 4.14.

ANSWER:

Using $K = 30$ min, $x = 0.2$ and $\Delta t = 10$ min the coefficients for the Muskingum routing can be found:

$$D = 30 - 30(0.2) + 0.5(10) = 29$$

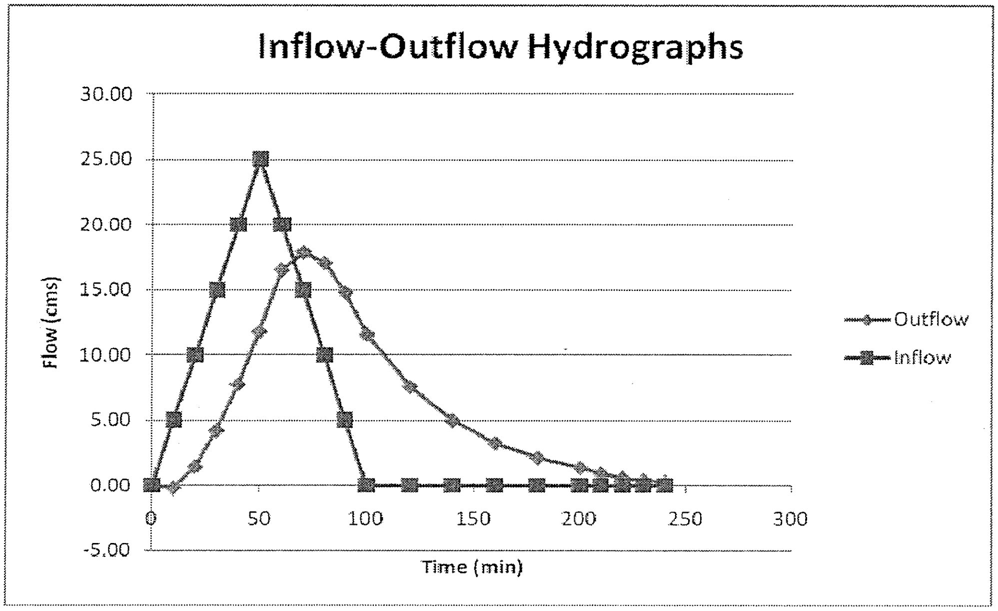
$$C_o = \frac{-30(0.2) + 0.5(10)}{29} = -0.0345$$

$$C_1 = \frac{30(0.2) + 0.5(10)}{29} = 0.379$$

$$C_2 = \frac{30 - 30(0.2) - 0.5(10)}{29} = 0.655$$

4.15 cont'

Time	Inflow (A and B)	Outflow
0	0	0.00
10	5	-0.17
20	10	1.44
30	15	4.22
40	20	7.76
50	25	11.81
60	20	16.53
70	15	17.90
80	10	17.07
90	5	14.81
100	0	11.60
120	0	7.60
140	0	4.98
160	0	3.26
180	0	2.14
200	0	1.40
210	0	0.92
220	0	0.60
230	0	0.39
240	0	0.26
250	0	0.17
260	0	0.11
270	0	0.07
280	0	0.05
290	0	0.03
300	0	0.02
310	0	0.01
320	0	0.01
330	0	0.01
340	0	0.00


Inflow-Outflow Hydrographs
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
Flow (cms)
0
50
100
150
200
250
300
Time (min)
Outflow
Inflow

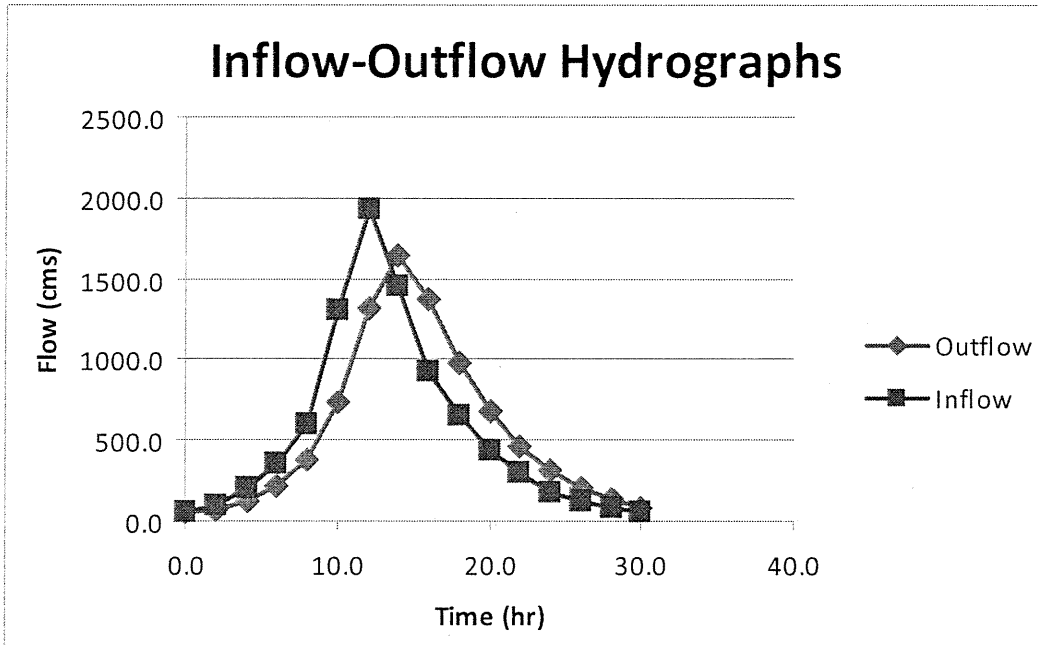
4.16 Develop a spreadsheet for Muskingum routing using the given inflow hydrograph through the river reach, where $K = 2$ hr, $x = 0.15$, and $\Delta t = 2$ hr.

Time (hr)	Inflow (m^3/s)
0	60
2	100
4	200
6	360
8	600
10	1310
12	1930
14	1460
16	930
18	650
20	440
22	300
24	180
26	120
28	80
30	60

ANSWER:

Time	Inflow	Outflow		Coefficients
0.0	60.0	60.0	K =	2.0
2.0	100.0	70.4	x =	0.15
4.0	200.0	118.2	Δt =	2.0
6.0	360.0	220.3	D =	2.7
8.0	600.0	386.0	C_0 =	0.26
10.0	1310.0	728.6	C_1 =	0.48
12.0	1930.0	1320.0	C_2 =	0.26
14.0	1460.0	1650.0		
16.0	930.0	1371.9		
18.0	650.0	972.0		
20.0	440.0	679.0		
22.0	300.0	465.7		
24.0	180.0	311.8		
26.0	120.0	198.6		
28.0	80.0	130.0		
30.0	60.0	87.8		

4.16 cont'


Inflow-Outflow Hydrographs
2500.0
2000.0
1500.0
1000.0
500.0
0.0
Flow (cms)
0.0
10.0
20.0
30.0
40.0
Time (hr)
Outflow
Inflow

4.17 A reservoir has the storage-discharge relationship below. Route the inflow hydrograph in problem 4.16 through the reservoir, assuming an initial storage of 52×10^6 m^3 of water.

Storage (10^6 m^3)	**Discharge (m^3/s)**
52	20
56	120
67.5	440
88	1100
113	2000

ANSWER:

The first step is to develop a storage indication curve.

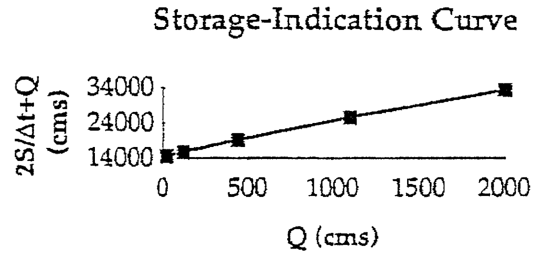
The Δt in the equation is the 2 hrs from problem 4.16 in seconds.

Please note: The axes for the Storage-Indication Curve should be reversed.

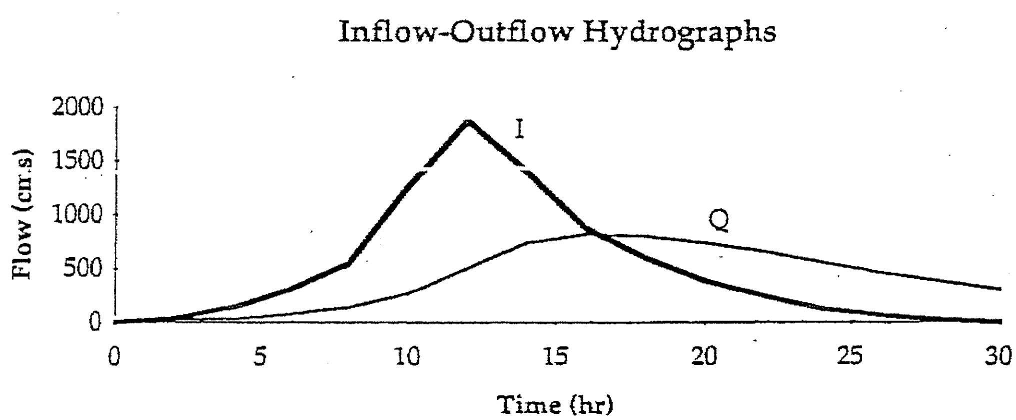
4.17 cont'

Using the Storage-Indication Method:

S X 10^6	Discharge (cms)	2S/Δt + Q (cms)
52	20	14464
56	120	15676
67.5	440	19190
88	1100	25544
113	2000	33389



Time	In	In + I(n+1)	(2Sn/Δt - Qn)	(2Sn/Δt + Q(n+1))	Q(n)
0	60	160	14424		20
2	100	300	14534	14584	25
4	200	560	14734	14834	50
6	360	960	15134	15294	80
8	600	1910	15774	16094	160
10	1310	3240	17184	17684	250
12	1930	3390	19304	20424	560
14	1460	2390	21094	22694	800
16	930	1580	21754	23484	865
18	650	1090	21614	23334	860
20	440	740	21104	22704	800
22	300	480	20424	21844	710
24	180	300	19684	20904	610
26	120	200	19074	19984	455
28	80	140	18394	19274	440
30	60	60	17784	18534	375



4.18 The storage equation has been given as

$$S_i = K[xI_i + (1-x)O_i]$$

and the continuity equation as

$$\bar{I} = \bar{O} + \Delta S / \Delta t$$

Given that $\bar{I}$ and $\bar{O}$ are the average inflow and outflow within the time period and ΔS is the change in the storage, derive the Muskingum river routing equation:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1,$$

where I_2 and O_2 refer to the inflow and outflow at the end of the time period and I_1 and O_1 refer to those values at the beginning of the time period. Verify the equations for C_0, C_1, and C_2 given in Section 4.2.

ANSWER:

We know that

$$\Delta S = S_2 - S_1,$$

$$S_2 = K\,[xI_2 + (1\text{-}x)\,O_2], \text{ and}$$

$$S_1 = K\,[xI_1 + (1\text{-}x)\,O_1].$$

So we have

$$\Delta S = K\,[x\,(I_2 - I_1) + (1\text{-}x)\,(O_2 - O_1)].$$

Substituting this into the continuity equation yields:

$$I = O + K\,[x\,(I_2 - I_1) + (1-x)\,(O_2 - O_1)] / \Delta t$$

Or

$$½\,(I_1 + I_2) = ½\,(O_1 + O_2) + K\,[x\,(I_2 - I_1) + (1-x)\,(O_2 - O_1)] / \Delta t$$

Isolating O_2 on the left hand side gives:

$$(K - Kx + \Delta t/2)\, O_2 = (-Kx + \Delta t/2)\, I_2 + (Kx + \Delta t/2)\, I_1 + (K - Kx - \Delta t/2)\, O_1$$

or

$$O_2 = \frac{(-kx + \Delta t/2)}{(k - kx + \Delta t/2)} I_2 + \frac{(kx + \Delta t/2)}{(k - kx + \Delta t/2)} I_1 + \frac{(k - kx - \Delta t/2)}{(k - kx + \Delta t/2)} O_1$$

Thus

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$$

Where

$$D = K - Kx + \Delta t/2$$

$C_0 = (-Kx + \Delta t/2) / D$

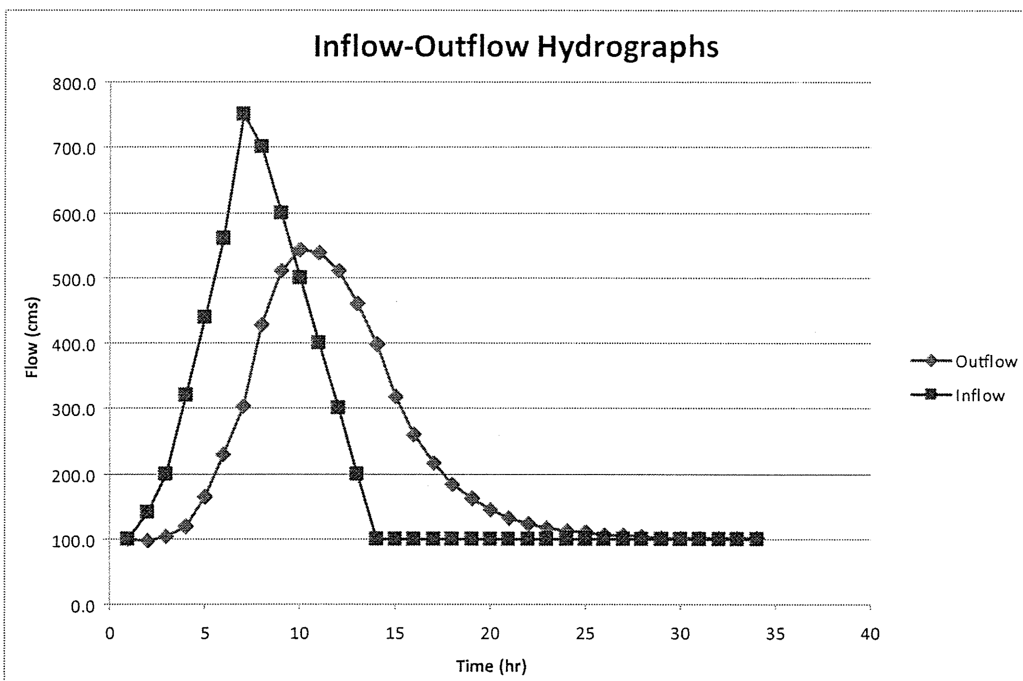
$C_1 = (Kx + \Delta t/2) / D$

$C_2 = (K - Kx - \Delta t/2) / D$

4.19 Determine the outflow hydrograph given the inflow hydrograph below. Use Muskingum routing, taking $K = 2$ hr, $x = 0.2$, and $\Delta t = 1$ hour.

Time (hr)	Inflow (m^3/s)
1	100
2	140
3	200
4	320
5	440
6	560
7	750
8	700
9	600
10	500
11	400
12	300
13	200
14	100
15	100
16	100
17	100
18	100
19	100
20	100

ANSWER:



4.19 cont'

Time	Inflow	Outflow		Coefficients
1	100	100.0	$K =$	4
2	140	96.8	$x =$	0.2
3	200	103.6	$\Delta t =$	1
4	320	119.9	$D =$	3.7
5	440	164.3	$C_0 =$	-0.08
6	560	229.1	$C_1 =$	0.35
7	750	303.1	$C_2 =$	0.73
8	700	427.9		
9	600	509.6		
10	500	542.1		
11	400	538.8		
12	300	509.4		
13	200	460.9		
14	100	398.5		
15	100	317.8		
16	100	259.0		
17	100	216.0		
18	100	184.6		
19	100	161.8		
20	100	145.1		
21	100	132.9		
22	100	124.0		
23	100	117.5		
24	100	112.8		
25	100	109.3		
26	100	106.8		
27	100	105.0		
28	100	103.6		
29	100	102.6		
30	100	101.9		
31	100	101.4		
32	100	101.0		
33	100	100.7		
34	100	100.5		

4.20 Develop a new flood routing method for rectangular cross sections based on the Muskingum and storage indication techniques. Instead of using the Muskingum storage equation $S = f(K, x, I, Q)$, use Manning's equation in the form where a and m are constants. Assume prismatic channel conditions at each time step, and derive the necessary equation for flood routing in a given rectangular channel with length L, inflows I_1 and I_2, channel width B, and outflows Q_1 and Q_2. Manning's equation takes the form

$$Q = ay^m.$$

ANSWER:

We assume a wide rectangular channel where $B >> y$

Then

$$A = By$$

$$P = 2y + B \approx B$$

$$R = A/P \approx y$$

Manning's equation becomes

$$Q = (1.49/n)(A)(R)^{2/3}\sqrt{S_o}$$

$$= (1.49/n)(By)(y)^{2/3}\sqrt{S_o}$$

$$Q = (1.49/n)(B\sqrt{S_o})(y)^{5/3}$$

Setting

$$a = (1.49/n)(B\sqrt{S_o})$$

$$m = 5/3$$

Then

$$Q = ay^m$$

Storage is given by:

$$S = ByL$$

From continuity, we have:

$$\frac{I_1 + I_2}{2} - \frac{Q_1 + Q_2}{2} = \frac{S_2 - S_1}{\Delta t}$$

4.20 cont'

$$\frac{I_1 + I_2}{2} - \frac{ay_1^{m} + ay_2^{m}}{2} = \frac{By_2L - By_1L}{\Delta t}$$

Rearranging to solve for y_1, gives:

$$BLy_2 + (\Delta ta/2)y_2^{m} = BLy_1 - (\Delta ta/2)y_1^{m} + (\Delta t/2)(I_1 + I_2)$$

where all terms are known but y_2. We solve the right hand side at time step 1, then solve for y_2.

Q_2 is found from $Q_2 = ay_2^{m}$. This process is then repeated until the routing is complete.

4.21 Equation (4–47) and Manning's equation (Eq. 4–48) can be combined to develop a second form of the kinematic wave equation [Eq. (4–56)], but in terms of Q.

a) Prove that the equation

$$\frac{\partial Q}{\partial x} + a\beta Q^{\beta-1}\frac{\partial Q}{\partial t} = q$$

is another form of the kinematic wave equation.

b) What are the values of a and β from Manning's equation?

ANSWER:

a) EQ. 4.47 $\Rightarrow \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q$

EQ. 4.48 $\Rightarrow Q = \alpha_c A^{m_c}$ where $m_c = B^{-1}$

EQ. 4.78 $\Rightarrow q = \frac{\partial A}{\partial t} + \alpha m_c A^{m_c - 1}\frac{\partial A}{\partial x} = q$

First, solve the given kinematic wave equation and show equivalent:

Method 1:

$$Q = \alpha_c A^{m_c}$$

$$\frac{\partial Q}{\partial x} = \frac{\partial Q}{\partial A}\frac{\partial A}{\partial t}$$

$$\frac{\partial Q}{\partial x} = \alpha_c m_c A^{m_c - 1}\frac{\partial A}{\partial x}$$

Substitute into EQ. 4.47:

$$q = \frac{\partial A}{\partial t} + \alpha_c m_c A^{m_c - 1}\frac{\partial A}{\partial x}$$

(EQ. 4.78)

Method 2:

$$Q = \alpha_c A^{m_c}$$

$A = \alpha Q^{\beta}$ Since $\beta = m_c^{-1}$, $\alpha = \alpha_c^{-\beta}$

$$\frac{\partial A}{\partial t} = \frac{\partial A}{\partial Q}\frac{\partial Q}{\partial t}$$

$$= \alpha\beta Q^{\beta - 1}\frac{\partial Q}{\partial t}$$

Substitute into EQ. 4.47

$$q = \frac{\partial Q}{\partial x} + \alpha\beta Q^{\beta - 1}\frac{\partial Q}{\partial t}$$

(Equivalent Form)

b) Solve Manning's Equation to the form:

$$Q = \alpha_c A^{\beta^{-1}}$$

Manning's:

$$= \frac{1}{n} A \left(\frac{A}{p} \right)^{2/3} \sqrt{S_o}$$

$$= \frac{\sqrt{S_o}}{np^{2/3}} A^{5/3}$$

$$Q = \alpha_c A^{\beta^{-1}}$$

$$\beta^{-1} = 5/3$$

$$\beta = 3/5$$

4.22 Repeat problem 4.11 for the case that the reservoir is partially full of water with initial height above the weir of $y = 3$ ft. Assume vertical walls (i.e., constant surface area for all depths).

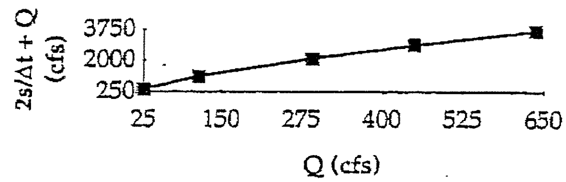
ANSWER:

Referring to Problem 4.11, $Q = 3.75Ly^{3/2}$ and $S = 300y$; where $L = 15$ ft and $y_o = 3$ ft. For Storage Indication, $\Delta t = 12$ hours.

Please note: The axes for the Storage-Indication Curve should be reversed.

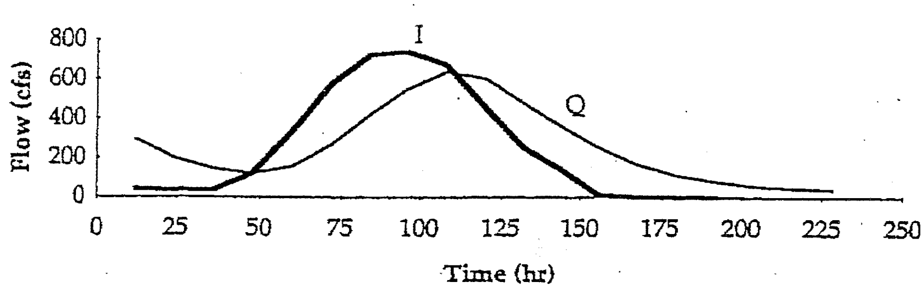
S (ac-ft)	Discharge (cfs)	2S/Δt + Q (cfs)
180	26	386
480	114	1074
900	292	2092
1200	450	2850
1512	637	3666

Storage-Indication Curve



Time	In	In + I(n+1)	(2Sn/Δt - Qn)	(2Sn/Δt + Q(n+1))	Q(n)
12	40	75 +	1508		292 ← Initial Outflow
24	35	72 +	1187	1583	198
36	37	162	971	1259	144
48	125	465	887	1133	123
60	340	915	1036	1352	158
72	575	1297	1419	1951	266
84	722	1462	1876	2716	420
96	740	1413	2238	3338	550
108	673	1129	2377	3651	637
120	456	706	2302	3506	602
132	250	390	2058	3008	475
144	140	150	1724	2448	362
156	10	10	1370	1874	252
168	0	0	1052	1380	164
180	0	0	830	1052	111
192	0	0	670	830	80
204	0	0	556	670	57
216	0	0	464	556	46
228	0	0	388	464	38

Inflow–Outflow Hydrographs



4.23 The hydrograph at the upstream end of a river is given in the following table. Develop the hydrograph for the downstream end of this river reach according to the Muskingum-Cunge method. The reach of interest is 4 km long. The channel is trapezoidal (2:1 side slope) with bottom width of 10 m. Assume $S_0 = 0.001$, $\Delta x = 2$ km, $\Delta t = 30$ min, $c = 1.47$ m/s, and no lateral inflow.

Time (min)	Flow (m^3/s)
0	0
30	4
60	14
90	27
120	30
150	29
180	27
210	24
240	18
270	12
300	8
330	5
360	3
390	1
420	0

ANSWER:

Given $L = 4000$ m, $S_0 = 0.001$, $\Delta x = 2000$ m, $\Delta t = 1800$ s, and $c = 1.47$ m/s.

Using: $$Q = \left(\frac{(10 + 2y^2)^{5/3}}{(10 + 2\sqrt{5}y)^{2/3}} \right) \cdot 0.79$$

Iterate to find y knowing peak flow $Q_p = 30$ m^3/s

The depth is found to be $y = 2.05$ m.

The top width is $w = 18.2$ m.

The diffusion coefficient D_1 is:

$$D_1 = \frac{Q_p}{2S_0 w} = \frac{30 cms}{2 \cdot 0.001 \cdot 18.2 m} = 824.2 \text{ m}^2/\text{s}$$

The weighting factor x is:

$$x = \frac{1}{2} - \frac{D_1}{c\Delta x} = 0.5 - \left(\frac{824.2\, m^2/s}{1.47\, m/s \cdot 2000m} \right) = 0.2197$$

4.23 cont'

The travel-time parameter K is:

$$K = \frac{\Delta x}{c} = \frac{2000m}{1.47\, m/s} = 1361 \text{ seconds}$$

The other coefficients, D, C_1, C_2, C_3, and C_4 are:

$$D = K(1-x) + \frac{\Delta t}{2} = 1361s(1-0.2197) + \frac{1800s}{2} = 1962 \text{ seconds}$$

$$C_1 = \frac{Kx + \Delta t/2}{D} = \frac{1361s(0.2197) + 1800s(0.5)}{1962s} = 0.6111$$

$$C_2 = \frac{\Delta t/2 - Kx}{D} = \frac{1800s(0.5) - 1361s(0.2197)}{1962s} = 0.3063$$

$$C_3 = \frac{K(1-x) - \Delta t/2}{D} = \frac{1361s(1-0.2197) - 1800s(0.5)}{1962s} = 0.0826$$

$$C_4 = \frac{q\Delta t\Delta x}{D} = 0 \quad \text{since no lateral flow means } q = 0.$$

Check that $C_1 + C_2 + C_3 + C_4 = 1.0$

$$0.6111 + 0.3063 + 0.0826 + 0 = 1.0$$

The outflow hydrograph at the downstream end is calculated by Eq. (4-104):

$$Q_{j+1}^{n+1} = C_1 Q_j^n + C_2 Q_j^{n+1} + C_3 Q_{j+1}^n + C_4$$

Note: For this upstream hydrograph, if $n = 0$, $Q = 0$ cms (at any j).

For $n = 0, j = 0$:

$$Q_1^1 = C_1 Q_0^0 + C_2 Q_0^1 + C_3 Q_1^0 = 0.6111 \cdot 0 + 0.3063 \cdot 4 + 0.0826 \cdot 0 = 1.225 \text{ cms}$$

For $n = 1, j = 0$:

$$Q_1^2 = C_1 Q_0^1 + C_2 Q_0^2 + C_3 Q_1^1 = 0.6111 \cdot 4 + 0.3063 \cdot 14 + 0.0826 \cdot 1.225 = 6.834 \text{ cms}$$

For $n = 0, j = 1$:

4.23 cont'

$$Q_2^1 = C_1 Q_1^0 + C_2 Q_1^1 + C_3 Q_2^0 = 0.6111 \cdot 0 + 0.3063 \cdot 1.225 + 0.0826 \cdot 0 = 0.3752 \text{ cms}$$

For $n = 1, j = 1$:

$$Q_2^2 = C_1 Q_1^1 + C_2 Q_1^2 + C_3 Q_2^1 = 0.6111 \cdot 1.225 + 0.3063 \cdot 6.834 + 0.0826 \cdot 0.3752 = 2.873 \text{ cms}$$

This process must be continued through $n = 14$ and $j = 2$, because there are 14 rows in the upstream hydrograph, and the subreach length is 2000 m.

Use a program such as Excel to complete the calculations.

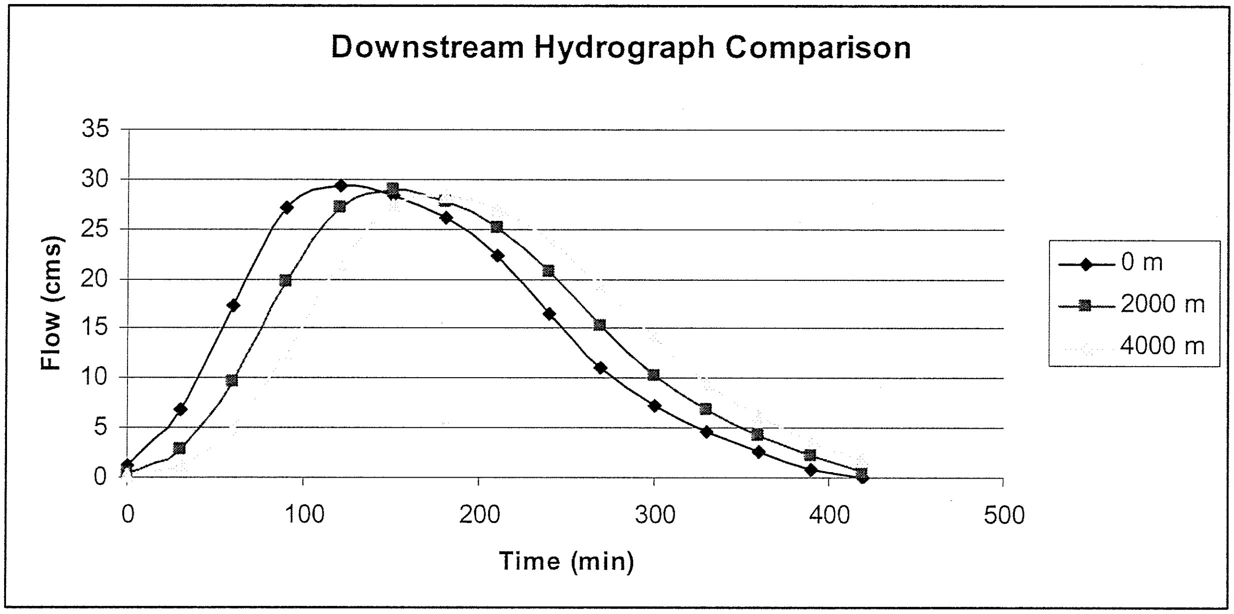
Upstream Hydrograph

Time (min)	Flow (cms)
0	0
30	4
60	14
90	27
120	30
150	29
180	27
210	24
240	18
270	12
300	8
330	5
360	3
390	1
420	0

Downstream Hydrograph

	$j = 0$	$j = 1$	$j = 2$
	j*Δx = 0 m	**j*Δx = 2000 m**	**j*Δx = 4000m**
$n = 0$	1.2	0.4	0.1
$n = 1$	6.8	2.9	1.1
$n = 2$	17.4	9.7	4.8
$n = 3$	27.1	19.7	12.4
$n = 4$	29.5	27.2	21.4
$n = 5$	28.4	29.0	27.3
$n = 6$	26.2	27.8	28.5
$n = 7$	22.3	25.1	27.0
$n = 8$	16.5	20.8	24.0
$n = 9$	11.1	15.2	19.4
$n = 10$	7.3	10.3	14.1
$n = 11$	4.6	6.7	9.5
$n = 12$	2.5	4.1	6.2
$n = 13$	0.8	2.1	3.7
$n = 14$	0.1	0.7	1.8

4.23 cont'



Downstream Hydrograph Comparison
35
30
25
20
15
10
5
0
Flow (cms)
0
100
200
300
400
500
Time (min)
0 m
2000 m
4000 m

4.24 The inflow hydrograph to a rectangular channel is tabulated below. The channel has a width of 100 ft, a bottom slope of 0.001, and Manning n of 0.035. The length of the channel is $L = 30{,}000$ ft. Using the Kinematic Wave, obtain the outflow hydrograph.

Time (min)	Inflow (cfs)
0	2500
20	2500
40	3500
60	4500
80	5500
100	6500
120	5000
140	4000
160	3000
180	2000
200	2000
220	2000

ANSWER:

For each value of discharge, determine the corresponding flow depth by solving Manning's equation. For a wide rectangular channel where $R \approx y$ and $A = By$ [see Eq. (4–31)],

$$Q = \frac{1.49}{n} B y^{5/3} \sqrt{S_0} \quad \text{rearranges to} \quad y = \left(\frac{Q \cdot n}{1.49 B \sqrt{S_0}} \right)^{3/5}$$

For $Q = 2500$ cfs

$$y = \left(\frac{2500 \cdot 0.035}{1.49 \cdot 100 \cdot \sqrt{0.001}} \right)^{3/5} = 5.77 \text{ ft}$$

For each kinematic wave (i.e., discharge) compute corresponding kinematic celerity. Differentiate (4–31) and substitute into (4–29) to get (4–33).

$$\frac{dQ}{dy} = \frac{5}{3} \cdot \frac{1.49}{n} B y^{2/3} \sqrt{S_0} = \frac{5}{3} B v$$

4.24 cont'

$$c = \frac{dQ}{dA} = \frac{1}{B} \cdot \frac{dQ}{dy} = \frac{1}{B} \cdot \frac{5}{3} Bv$$

$$c = \frac{5}{3} v$$

For $y = 5.77$ ft, substitute Eq. (4–30) for v:

$$c = \frac{5}{3}\left(\frac{1.49}{0.035} \cdot 5.77^{2/3} \sqrt{0.001}\right) = 7.22 \text{ ft/s}$$

For each wave, compute time of travel using, $\Delta t = L/c$.

For c = 7.22 ft/s,

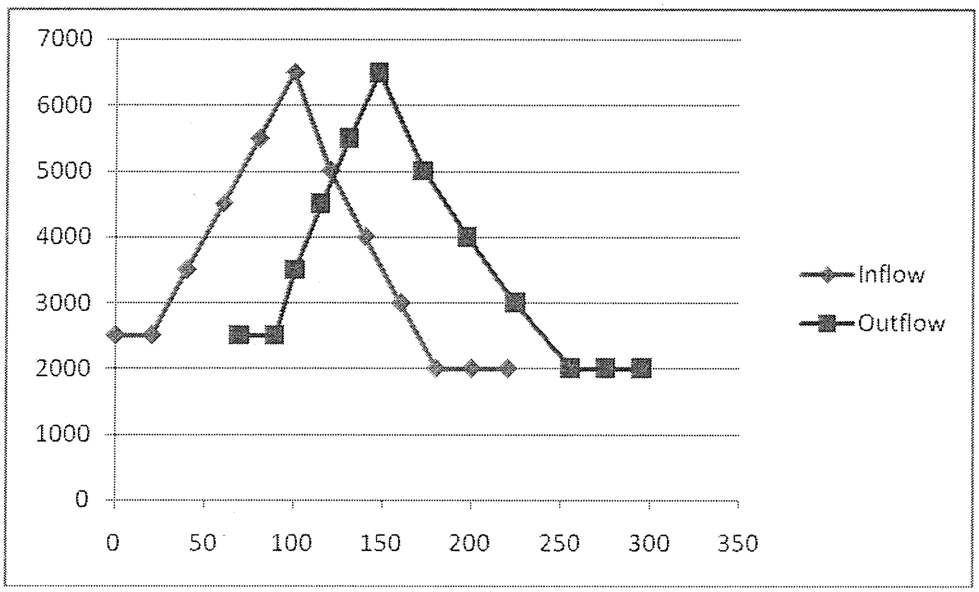
$$\Delta t = \frac{30{,}000 ft}{7.22 ft/s} = 4155 \text{ sec} = 69.25 \text{ min}$$

For each wave compute time of arrival at downstream end as, $t = t_o + \Delta t$

For $Q = 2500$ cfs, starting at $t_o = 0$, the time of arrival is $t = 69.25$ min

Time (min)	Inflow (cfs)	y (ft)	c (f/s)	Δt (min)	Outflow Time (min)
0	2500	5.8	7.2	69.3	69.3
20	2500	5.8	7.2	69.3	89.3
40	3500	7.1	8.3	60.5	100.5
60	4500	8.2	9.1	54.7	114.7
80	5500	9.3	9.9	50.5	130.5
100	6500	10.2	10.6	47.3	147.3
120	5000	8.7	9.5	52.5	172.5
140	4000	7.7	8.7	57.4	197.4
160	3000	6.4	7.8	64.4	224.4
180	2000	5.0	6.6	75.7	255.7
200	2000	5.0	6.6	75.7	275.7
220	2000	5.0	6.6	75.7	295.7

4.24 cont'

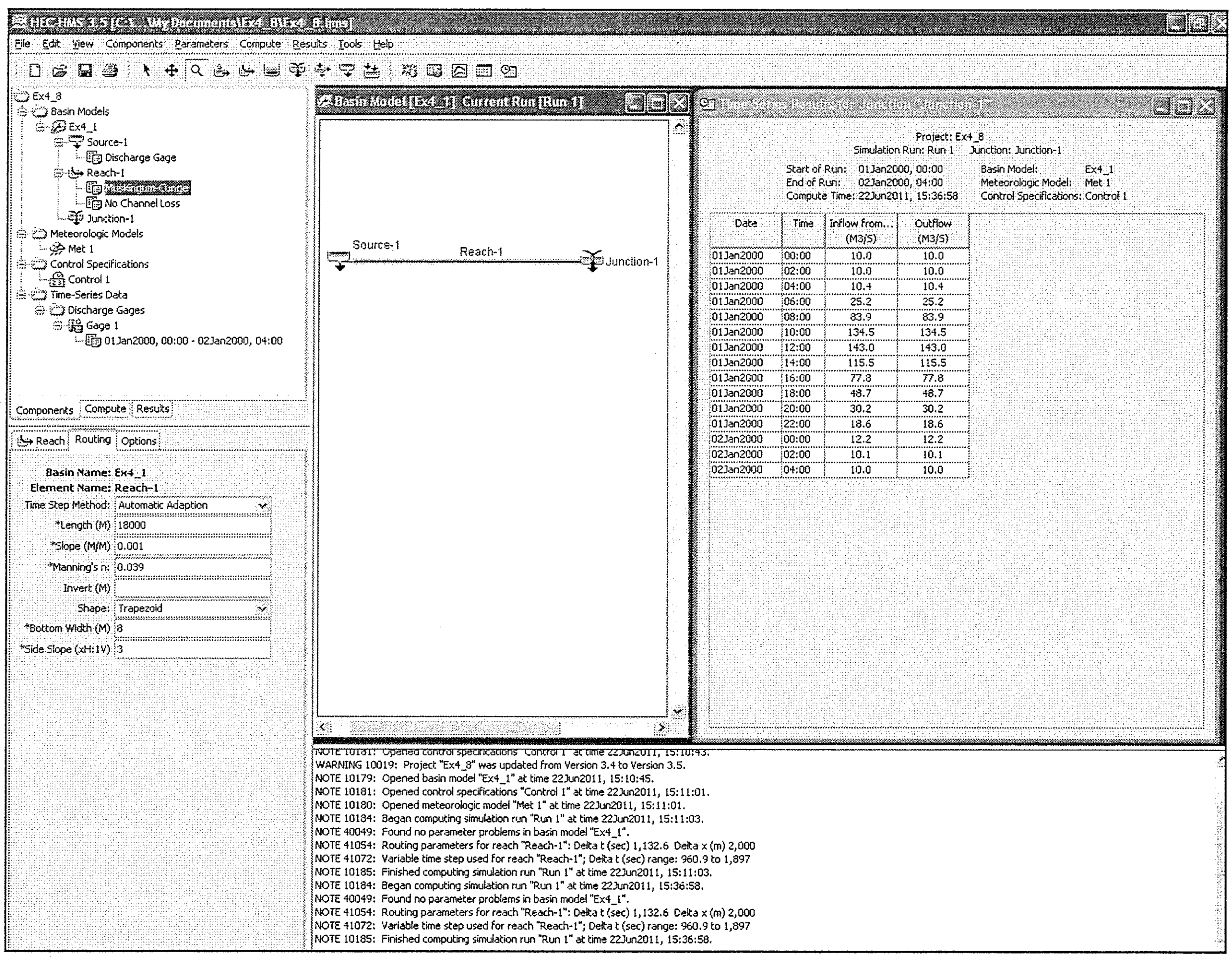


7000
6000
5000
4000
3000
2000
1000
0
0
50
100
150
200
250
300
350
Inflow
Outflow

4.25 Using only the inflow hydrograph from Example 4.7 (i.e., no precipitation), use HEC-HMS* (*http://www.hec.usace.army.mil/software/hec-hms/download.html*) to route the hydrograph through a channel using the Muskingum-Cunge routing method. The channel has a trapezoidal cross section with a 3:1 side slope (horizontal:vertical), a Manning's roughness (n) of 0.039, and a bottom width of 8 meters. Assume there is no loss in the channel. What is the value of peak flow and when does it occur? (**Refer to Chapter 5 for detailed information on HEC-HMS*)

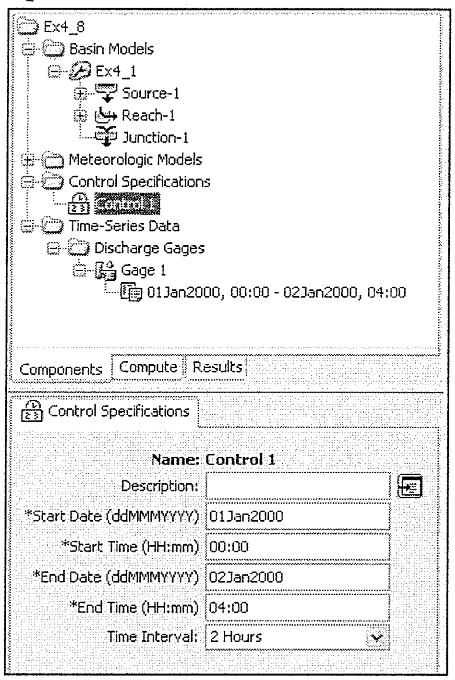
ANSWER:

143 cms, 12 hours

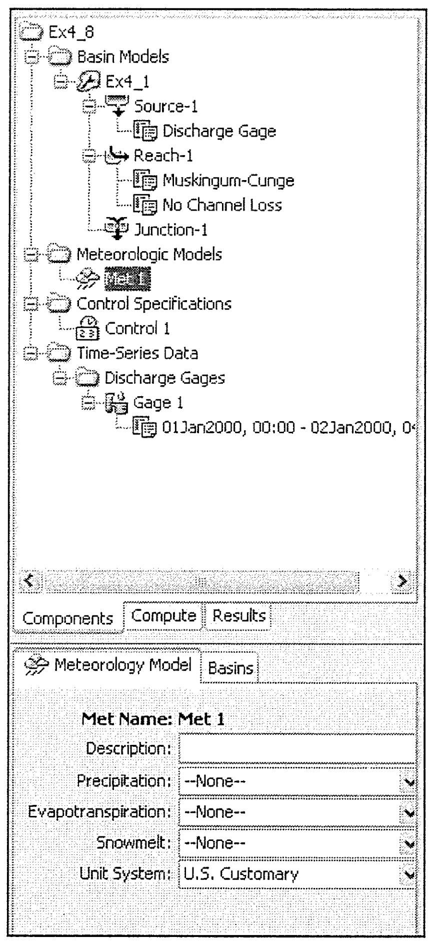


4.25 cont'

Control Specifications:

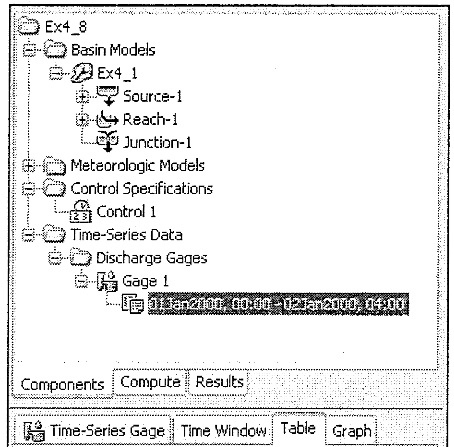


Meteorological Model:



4.25 cont'

Gage Data:



Time (ddMMMYYYY, HH:mm)	Discharge (M3/S)
01Jan2000, 00:00	10.0
01Jan2000, 01:00	12.0
01Jan2000, 02:00	18.0
01Jan2000, 03:00	28.5
01Jan2000, 04:00	50.0
01Jan2000, 05:00	78.0
01Jan2000, 06:00	107.0
01Jan2000, 07:00	134.5
01Jan2000, 08:00	147.0
01Jan2000, 09:00	150.0
01Jan2000, 10:00	146.0
01Jan2000, 11:00	129.0
01Jan2000, 12:00	105.0
01Jan2000, 13:00	78.0
01Jan2000, 14:00	59.0
01Jan2000, 15:00	45.0
01Jan2000, 16:00	33.0
01Jan2000, 17:00	24.0
01Jan2000, 18:00	17.0
01Jan2000, 19:00	12.0
01Jan2000, 20:00	10.0
01Jan2000, 21:00	10.0
01Jan2000, 22:00	10.0
01Jan2000, 23:00	10.0
02Jan2000, 00:00	10.0
02Jan2000, 01:00	10.0
02Jan2000, 02:00	10.0
02Jan2000, 03:00	10.0
02Jan2000, 04:00	10.0

4.26 Repeat Problem 4.25 for reach lengths of 6, 12, and 32 km. What happens to the timing and value of the peak flow? Explain.

ANSWER:

As the reach length increases, the peak flow decreases and occurs later. In other words, the outlet hydrograph flattens out.

Reach | Routing | Options

Basin Name: Ex4_1
Element Name: Reach-1
Time Step Method: Automatic Adaption
*Length (M) 6000
*Slope (M/M) 0.001
*Manning's n: 0.039
Invert (M)
Shape: Trapezoid
*Bottom Width (M) 8
*Side Slope (xH:1V) 3

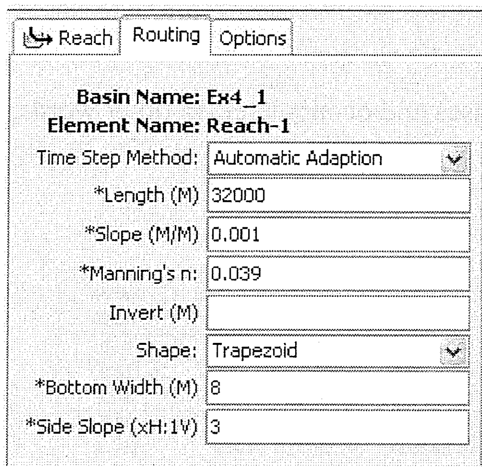
Date	Time	Inflow from... (M3/S)	Outflow (M3/S)
01Jan2000	00:00	10.0	10.0
01Jan2000	02:00	12.2	12.2
01Jan2000	04:00	31.2	31.2
01Jan2000	06:00	80.2	80.2
01Jan2000	08:00	130.3	130.3
01Jan2000	10:00	146.2	146.2
01Jan2000	12:00	122.0	122.0
01Jan2000	14:00	80.3	80.3
01Jan2000	16:00	46.9	46.9
01Jan2000	18:00	26.9	26.9
01Jan2000	20:00	15.1	15.1
01Jan2000	22:00	10.2	10.2
02Jan2000	00:00	10.0	10.0
02Jan2000	02:00	10.0	10.0
02Jan2000	04:00	10.0	10.0

Reach | Routing | Options

Basin Name: Ex4_1
Element Name: Reach-1
Time Step Method: Automatic Adaption
*Length (M) 12000
*Slope (M/M) 0.001
*Manning's n: 0.039
Invert (M)
Shape: Trapezoid
*Bottom Width (M) 8
*Side Slope (xH:1V) 3

Date	Time	Inflow from... (M3/S)	Outflow (M3/S)
01Jan2000	00:00	10.0	10.0
01Jan2000	02:00	10.0	10.0
01Jan2000	04:00	15.4	15.4
01Jan2000	06:00	51.1	51.1
01Jan2000	08:00	110.8	110.8
01Jan2000	10:00	144.1	144.1
01Jan2000	12:00	135.8	135.8
01Jan2000	14:00	99.0	99.0
01Jan2000	16:00	61.2	61.2
01Jan2000	18:00	37.0	37.0
01Jan2000	20:00	21.9	21.9
01Jan2000	22:00	13.4	13.4
02Jan2000	00:00	10.2	10.2
02Jan2000	02:00	10.0	10.0
02Jan2000	04:00	10.0	10.0

4.26 cont'



Date	Time	Inflow from... (M3/S)	Outflow (M3/S)
01Jan2000	00:00	10.0	10.0
01Jan2000	02:00	10.0	10.0
01Jan2000	04:00	10.0	10.0
01Jan2000	06:00	10.0	10.0
01Jan2000	08:00	14.5	14.5
01Jan2000	10:00	75.2	75.2
01Jan2000	12:00	132.5	132.5
01Jan2000	14:00	139.9	139.9
01Jan2000	16:00	114.5	114.5
01Jan2000	18:00	81.1	81.1
01Jan2000	20:00	54.0	54.0
01Jan2000	22:00	35.8	35.8
02Jan2000	00:00	23.7	23.7
02Jan2000	02:00	16.0	16.0
02Jan2000	04:00	11.4	11.4