Solutions Chapter 9

9.1. Confirm the data for a 10-yr storm from Eq. (9–2) and Table 9–2 for 10-min, 15-min, 30-min, and 60-min durations. Compare to Fig. 9–4.

For a 10-yr storm with a duration of 10-min:

D=duration
$i = b/(D+d)^e$ = 93.53/((10+18.9)^0.7742) = 6.917 in/hr

15 min: 6.113 in/hr
30 min: 4.603 in/hr
60 min: 3.179 in/hr

9.2. Construct the 100-yr, 24-hr synthetic design hyetograph using Eq. (9–2) and 1-hr time intervals. Center the maximum intensity (the 1-hr intensity) at hr 12; the next highest intensity (determined by the depth from 2 hr-1 hr over the 1-hr interval—hence, intensity) at hr 11; the next-highest intensity (depth from 3 hr 2 hr) at hr 13; the next-highest intensity (depth from 4 hr 3 hr) at hr 10; and back and forth similarly. Check the total volume under the hyetograph and see that it matches the total rainfall depth.

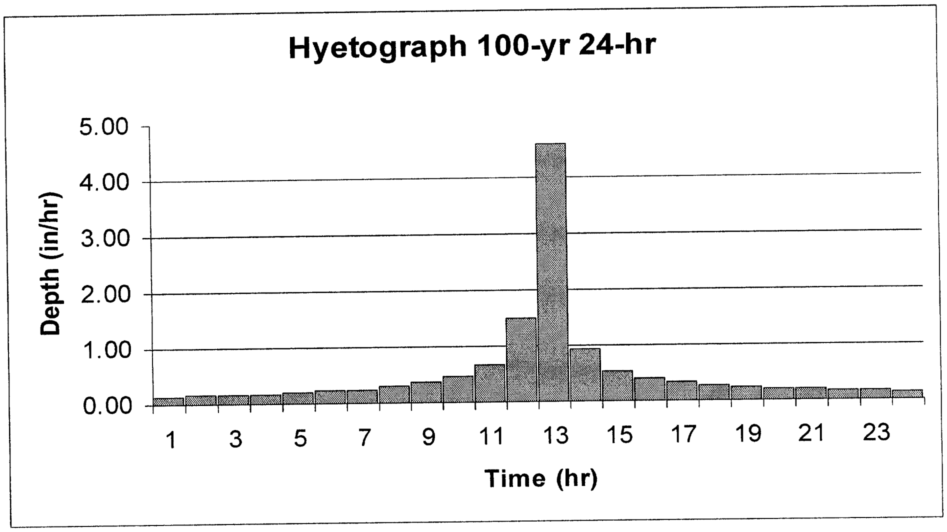
Example: Row 2

i = 125.4/((120+21.8)^0.75) = 3.05 in/hr
depth = i*hr = 3.05*2 = 6.10 in
depth that hour = depth hr 2 – depth hr 1 = 6.10 - 4.61 = 1.49 in

hr	min	i	depth	Depth that hour	hour
1	60	4.61	4.61	4.61	12
2	120	3.05	6.10	1.49	11
3	180	2.34	7.03	0.92	13
4	240	1.93	7.71	0.68	10
5	300	1.65	8.25	0.55	14
6	360	1.45	8.71	0.46	9
7	420	1.30	9.11	0.40	15
8	480	1.18	9.46	0.35	8
9	540	1.09	9.78	0.32	16
10	600	1.01	10.07	0.29	7
11	660	0.94	10.34	0.27	17
12	720	0.88	10.59	0.25	6
13	780	0.83	10.82	0.23	18
14	840	0.79	11.04	0.22	5
15	900	0.75	11.24	0.21	19
16	960	0.71	11.44	0.20	4
17	1020	0.68	11.63	0.19	20
18	1080	0.66	11.80	0.18	3
19	1140	0.63	11.97	0.17	21
20	1200	0.61	12.14	0.16	2
21	1260	0.59	12.29	0.16	22
22	1320	0.57	12.44	0.15	1
23	1380	0.55	12.59	0.15	23
24	1440	0.53	12.73	0.14	0
				12.73	

Sum checks

The last column is x (hr) and the second to last is depth accumulated that hour (in/hr). Rearrange in consecutive order and plot.



9.3. Determine the runoff Q using Eq. (9–1) for a project with A = 20 ac, C = 0.55, Tc = 12min, and an *i*-value developed from Eq. (9–2) for a 5-yr design storm.

For a 5-yr storm: i = 84.14/((12+17.8)^0.7881) = 5.8 in/hr

Q = C(i*A) = 0.55(5.80*20) = 63.8 cfs

9.4. Determine the smallest required concrete round pipe and rectangular box sewers for Q = 100, 150, 200, and 250 cfs, S = .005 ft/ft, and n = 0.013 using Eq. (9–3). Tabulate results and compare capacities and areas of the round to box sewers. Use standard increment sizes of 0.5 ft for round pipe and 1.0 ft for box sewers.

For a round pipe:

$$Q = 1.49AR^{2/3}S^{1/2}/n$$
$$Q = 1.49(\pi r^2)(\pi r^2/2\pi r)^{2/3}S^{1/2}/n = 1.49\pi r^{8/3}*.005^{1/2}/(2^{2/3}*0.013)$$
$$r = ((Q*2^{2/3}*0.013)/(1.49\pi*0.005^{1/2}))^{3/8}$$
$$r = (0.0623Q)^{3/8}$$

For a box sewer:

$$Q = 1.49AR^{2/3}S^{1/2}/n$$
$$Q = 1.49*s^2*(s^2/4s)^{2/3}S^{1/2}/n = 1.49s^{8/3}*.005^{1/2}/(4^{2/3}*0.013)$$
$$s = ((Q*4^{2/3}*0.013)/(1.49*0.005^{1/2}))^{3/8}$$
$$s = (0.311Q)^{3/8}$$

Thus:

r ft = $(0.0623Q)^{3/8}$
d ft = $r*2$
$d_{rounded}$ ft = d ft rounded up to nearest half foot
Area round = $\pi(d_{rounded}/2)^2$

$s_{rounded}$ ft = $(0.311Q)^{3/8}$ rounded up the nearest foot
Area square = $s_{rounded}^2$

Q cfs	r ft	d ft	$d_{rounded}$ ft	Area round ft^2	s ft	$s_{rounded}$ ft	Area square ft^2
100	1.986	3.973	4	12.6	3.629	4	16
150	2.313	4.625	5	19.6	4.225	5	25
200	2.576	5.152	5.5	23.7	4.706	5	25
250	2.801	5.602	6	28.3	5.116	6	36

9.5. Derive Eq. (9–4) from Eq. (9–3).

Eq. 9.3: $Q = 1.49AR^{2/3}S^{1/2}/n$

$S^{1/2} = Qn/(1.49AR^{2/3})$

$S = (Qn/(1.49AR^{2/3}))^2$

$h_f/L = (Qn/(1.49AR^{2/3}))^2$

Eq. 9.4 $h_f = L^* (Qn/(1.49AR^{2/3}))^2$

9.6. For a Q = 80 cfs, calculate the h_f = for a 3.5-ft-diameter, 500-ft-long, round pipe sewer with n = 0.012 flowing full.

$h_f = L^* (Qn/(1.49AR^{2/3}))^2$

$h_f = 500 * (80*0.012/(1.49*(\pi r^2)*(\pi r^2/2\pi r)^{2/3}))^2$

$h_f = 500 * (80*0.012/(1.49*(\pi 1.75^2)*(\pi 1.75^2/(2\pi 1.75))^{2/3}))^2$

$h_f = 2.679$ ft

9.7. Using the same flow and the same 500-ft-long round pipe sewer as in Problem 9.6, determine and plot the HGL and EGL for the sewer reach, given the following additional information: U/S pipe invert elevation = 100.00 ft; D/S pipe invert elevation = 96.00 ft; and the tailwater elevation for the sewer (i.e., the outlet water surface elevation) = 106.00 ft.

Row 2:

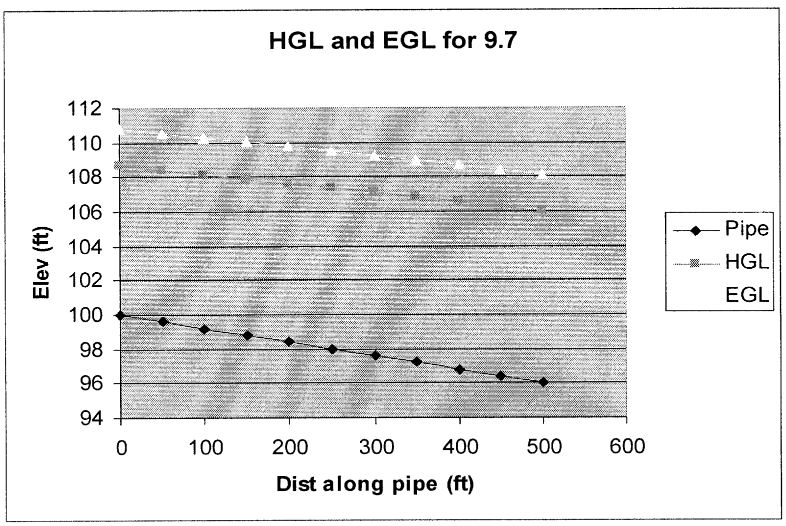
$h_f = x * (80*0.012/(1.49*(\pi 1.75^2)*(\pi 1.75^2/(2\pi 1.75))^{2/3}))^2 = 0.268$

(see problem 9.6 for equation)

HGL = Tailwater Elev + h_f = 106 + 0.2679 = 106.268

EGL = HGL + v^2/g = HGL + $(80*\pi 1.75^2)^2/32.2$ = 108.415

Dist along pipe ft	x from outlet ft	Elev. ft	h_f ft	HGL ft	EGL ft
500	0	96	0.000	106.000	108.147
450	50	96.4	0.268	106.268	108.415
400	100	96.8	0.536	106.536	108.683
350	150	97.2	0.804	106.804	108.951
300	200	97.6	1.072	107.072	109.219
250	250	98	1.340	107.340	109.487
200	300	98.4	1.608	107.608	109.755
150	350	98.8	1.875	107.875	110.023
100	400	99.2	2.143	108.143	110.291
50	450	99.6	2.411	108.411	110.559
0	500	100	2.679	108.679	110.826



9.8. Referring to Table E9–1(a), double the flows in Column 11 for each sewer reach and determine the minimum commercially available concrete box sizes required (available in 1.0-ft increments) for each sewer reach using a minimum box span and rise of 2.0 ft. Keep all elevation, length, slope, and roughness data the same as presented in the table. After sizing the box sewers, recompute the HGL for the entire system. Evaluate any problems with maintaining a minimum actual velocity of 3 fps.

First complete the table with values given in Example 9.1. This first section will stay as is.

U/S Manhole	D/S Manhole	Drainage Area No.	Drainage Area Area (ac)	C	C x A	Sum C x A	Total DA (ac)	Tc (min)	I (in/hr)
Line A									
A9	A8	A9	7.44	0.45	3.35	3.35	7.44	19	3.88
A8	A7	A8	4.77	0.45	2.15	5.49	12.21	21	3.71
A7	A6	A7	11.36	0.45	5.11	10.61	23.57	23	3.55
A6	A5	A6	8.48	0.45	3.82	14.42	32.05	26	3.34
A5	A4	A5	10.42	0.45	4.69	19.11	42.47	29	3.15
A4	A3	A4	3.48	0.45	1.57	20.68	45.95	31	3.04
A3	A2	A3	1.09	0.45	0.49	21.17	47.04	32	2.99
A2	A1	A2	4.31	0.45	1.94	23.11	51.35	35	2.84
Line B									
B3	B2	B3	2.5	0.65	1.63	1.63	2.5	11	4.81
B2	B1	B2	0.87	0.45	0.39	2.02	3.37	12	4.67
B1	A1	B1	1.35	0.70	0.95	2.96	4.72	13	4.54
Line A									
A1	Outlet	A1	5.42	0.45	2.44	28.51	61.49	38	2.71

9.9. Calculate the manhole losses, h_j, of 90-, 60-, 45-, 30-, and 15-degree manhole junctions with no special deflector for a 4.5-ft-diameter round pipe sewer flowing full with Q = 100 cfs. Tabulate and compare the results.

K_j = 1.33, 0.65, 0.38, 0.20, 0.08 for the five angles respectively.

$h_j = K_j*V^2/(2g) = K_j(Q/(\pi(d/2)^2))^2/(2g) = K_j(100/(\pi 2.25^2))^2/(2*32.2)$

Angle degrees	K_j	h_j ft.
90	1.33	0.82
60	0.65	0.40
45	0.38	0.23
30	0.2	0.12
15	0.08	0.05

9.10. Given the inlet in Figure 9–9 with h = 0.5 ft and W = 1.5 ft, determine what minimum length, *L*, of inlet (in whole-foot increments) is required for Q = 6 cfs, such that the depth of flow at the curb does not exceed 0.5 ft.

If depth does not exceed 0.5 ft, this is weir flow. C_w = 2.3 because there is a gutter depression.

Eq. 9-7: $Q = C_w(L + 1.8W)d^{1.5} = 2.3(L+1.8*1.5)*0.5^{1.5} = 6$

$L = 4.68$ ft

$L = 5$ ft

9.11. Given the inlet in Figure 9–9 with h = 0.5 ft, W =1.5 ft, L=4.0 ft, and a gutter depression depth = 0.33 ft, determine what is the *Q* into the inlet if the depth of storm water at the curb is 1.0 ft (i.e., there is 6 inches of water depth above the standard curb height of 6 inches for a total depth at curb of 1.0 ft).

C_o = 0.67 from text

d_o = 1.08 ft (center of inlet throat is at 0.25 ft so head is 1+0.33-0.25)

Eq. 9-8: $Q = C_o hL(2gd_o)^{0.5}$

$Q = 0.67*0.5*4(2*32.2*1.08)^{0.5}$

$Q = 11.17$ cfs

Then, manually double all of the flows in the first column of section two. Then change the equations for A and R to be span^2 and span/4 respectively. Manually change span and rise by increments of 1 until Q is less than the capacity. Note: Q should still be greater when the span/rise is 1 foot smaller. These dimensions are the smallest acceptable design.

Flow	Elevation (ft)		Dimensions (ft)			Area	R	Length
Q (cfs)	U/S	D/S	Span	Rise	Shape	(sq ft)	(ft)	(ft)
26.00	34.00	32.60	3	3	Sq	9.00	0.750	425
40.75	32.10	31.15	3	3	Sq	9.00	0.750	355
75.32	29.65	29.00	5	5	Sq	25.00	1.250	587
96.33	28.71	28.00	5	5	Sq	25.00	1.250	595
120.56	27.24	26.75	6	6	Sq	36.00	1.500	485
125.82	26.75	26.40	6	6	Sq	36.00	1.500	270
126.58	26.10	25.39	6	6	Sq	36.00	1.500	580
131.41	25.39	24.50	5	5	Sq	25.00	1.250	530
15.64	27.60	26.60	2	2	Sq	4.00	0.500	166
18.83	26.60	25.75	3	3	Sq	9.00	0.750	193
26.87	25.28	24.25	3	3	Sq	9.00	0.750	215
154.63	22.18	21.92	6	6	Sq	36.00	1.500	270

Actual velocity should automatically change to reflect changes. It equals Flow/Area. Friction headloss should change automatically as well. It is Eq. 9-4.
The HGL is HGL of next row + Friction Headloss. Row 1: 34.37 + 0.40 = 34.76 ft

Slope (ft/ft)	n	Capacity (cfs)	Actual Velocity (fps)	Friction Headloss (ft)	U/S Manhole HGL El (ft)
0.0033	0.013	48.87	2.89	0.40	34.76
0.0027	0.013	44.05	4.53	0.81	34.37
0.0011	0.013	110.64	3.01	0.30	33.55
0.0012	0.013	114.86	3.85	0.50	33.25
0.0010	0.013	171.86	3.35	0.24	32.75
0.0013	0.013	194.67	3.50	0.15	32.51
0.0012	0.013	189.17	3.52	0.32	32.37
0.0017	0.013	136.25	5.26	0.83	32.05
0.0060	0.013	22.42	3.91	0.49	32.02
0.0044	0.013	56.51	2.09	0.09	31.53
0.0048	0.013	58.94	2.99	0.21	31.43
0.0010	0.013	167.78	4.30	0.22	31.22

Keeping the velocity at least 3 ft/s will be hard for Reaches A9-A8, B2-B1, and B1-A1.

9.12. Repeat Example 9–2 using 3-ft-diameter round corrugated metal pipe culverts with an n = 0.024. Determine the minimum number of culvert barrels needed to convey the flow without overtopping the roadway.

Allowable H is still 1.99 ft. Still projecting from fill though now corrugated metal pipe so K_e = 0.9. The velocity downstream is still 3.44 ft/s.

Two culverts for example (look at one with half the flow through it):

First, find culvert velocity, which is $Q/(\# \text{ culv})/A = Q/2/(\pi 1.5^2) = 12.223$ ft/s

Then $H_{ent} = K_e(V^2/(2g)) = 0.9*12.223^2/2/32.2 = 2.088$ ft

$H_{exit} = (V^2/(2g))_{culvert} - (V^2/(2g))_{channel} = 12.223^2/2/32.2 - 3.44^2/2/32.2 = 2.136$ ft

$H_f = L(Qn/(1.49AR^{2/3}))^2 = 42(0.024*172.8/(\#culv*1.49\pi 1.5^2(1.5/2)^{2/3}))^2 = 0.572$ ft

$H = H_{ent} + H_{exit} + H_f = 4.796$ ft (too high but four is enough)

1.06 ft < 1.99ft so you need four corrugated metal pipe culverts of 3' diameter
84.01+1.06 = 85.07 < 86.0 ft OK

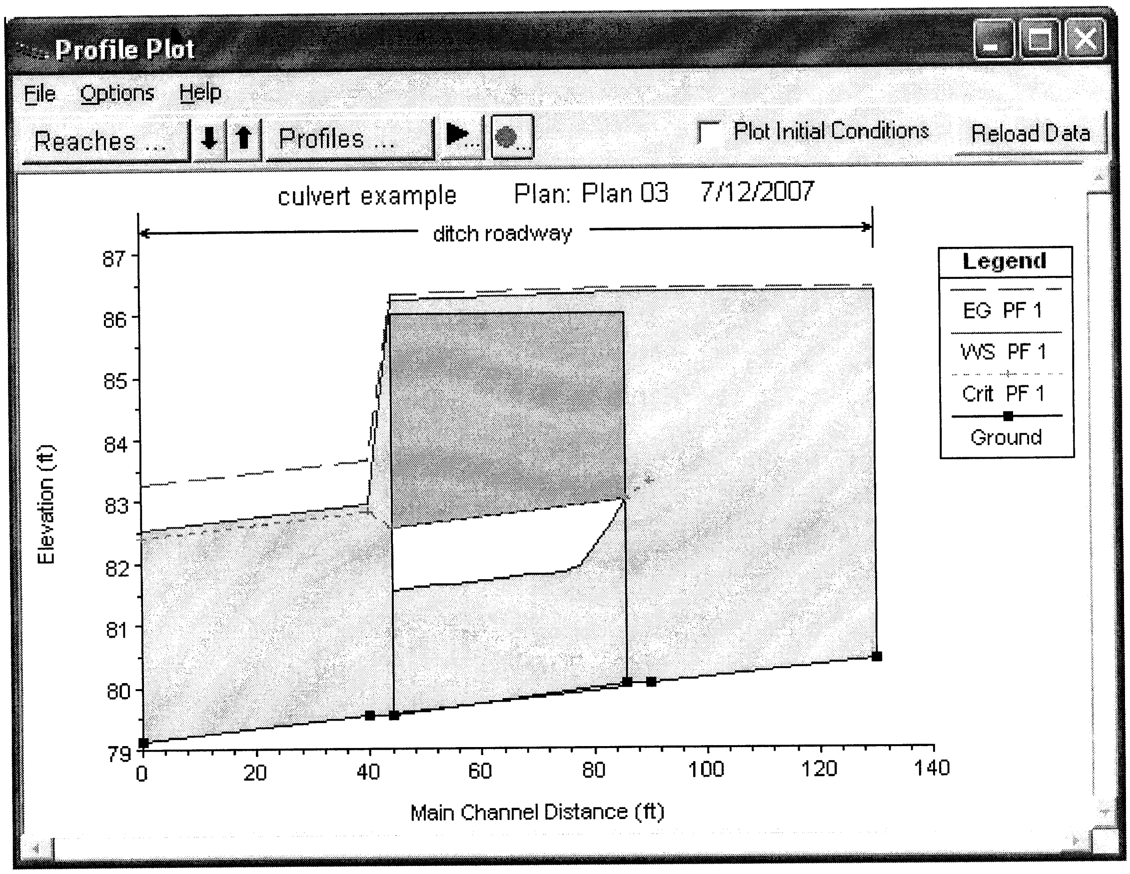
	Two 3' diam.	Three 3'	Four 3'
# culverts	2	3	4
velocity culvert	12.223	8.149	6.112
velocity downstream	3.44	3.44	3.44
H_{ent}	2.088	0.928	0.522
H_{exit}	2.136	0.847	0.396
H_f	0.572	0.254	0.143
H	4.796	2.029	1.061

9.13. Repeat Example 9–3 using HEC-RAS but use a 100-yr storm at this location. Recalculate *Q* for a 100-yr storm using the same *C*, *A*, and *Tc* information provided in Example 9–2. Evaluate the behavior of the 7 x 3 box culvert and then examine the behavior of a 6 x 3 box culvert at this location. Document the differences in culvert performance, including weir flow depths for comparative purposes. Determine the minimum span (width) of box culvert needed to prevent overtopping of the roadway if the box culvert rise (height) allowed is increased to 4 ft.

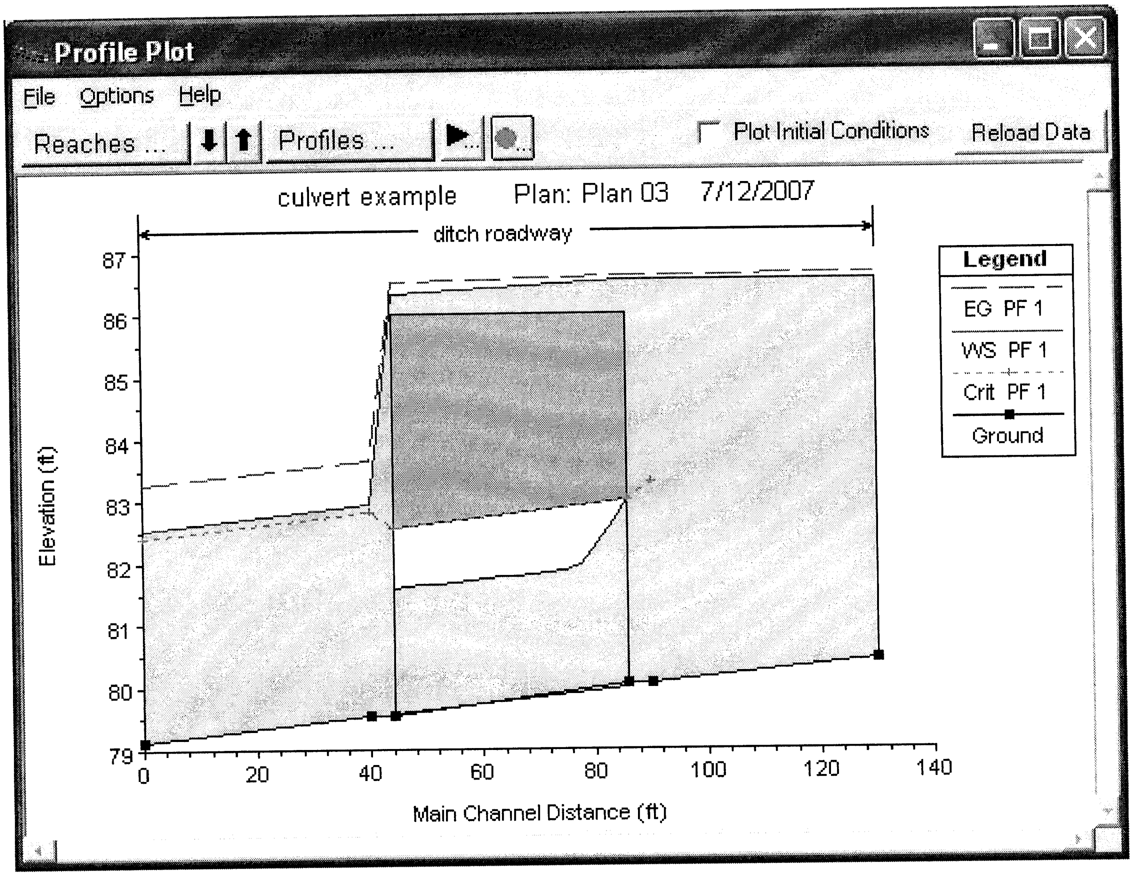
$i = b/(T_c+d)^e = 125.4/(16+21.8)^{0.75} = 8.226$
C = 0.18
A = 160 ac
Q = 0.18*160*8.226 = 236.9 cfs

The 6 x 3 and 7 x 3 box culverts should be represented the same way as in Example 9-3, except with the flow changed to 236.9. Outlet control exists in both of these situations. Save a new steady flow data with Normal Depth as the Reach Boundary Condition and a slope of 0.01.

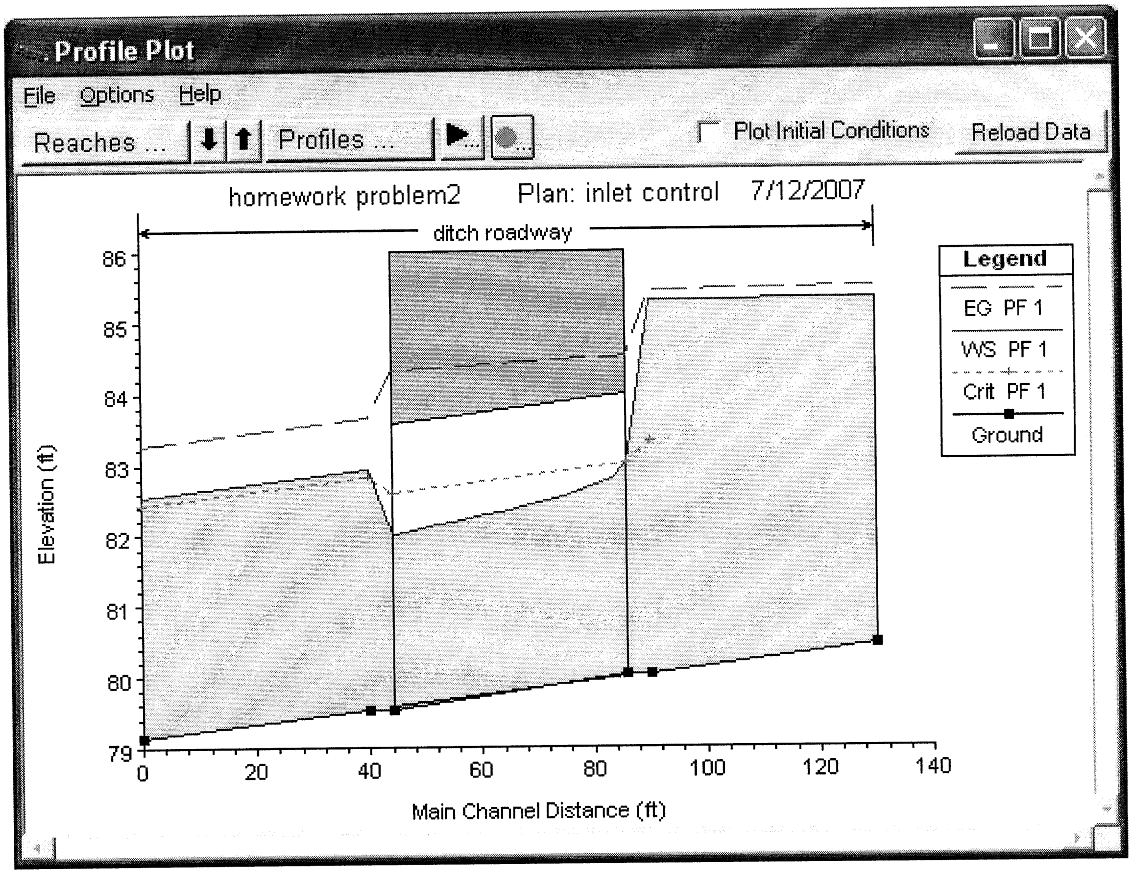
With a 7 x 3 box culvert, the profile plot should look like this. It is overtopped and under outlet control. Weir max depth is 0.41 ft and average depth is 0.37 ft, which can be found in the culvert output table.



Using the same 100yr steady flow data, change the box culvert dimensions to 6 x 3 in the model and save the geometry. The profile plot should look like this. It is overtopped and under outlet control. The maximum weir depth is 0.63 ft and the average depth is 0.54 ft.



Finally, set the height at 4 ft and begin increasing the height by increments of one foot until the road is not overtopped. The culvert will switch to inlet control for this. Use the same steady flow data and change the dimensions of the culvert. This should occur at 8 ft. The profile plot should look like this:



9.14. Repeat Example 9–4 but with the following changes to the data:

(a) SCS TR-20 methodology data

(i) Existing (preproject) Tc = 1.3 hr; changed from 1.0 hr

(ii) Proposed (postproject) Tc = 0.4 hr; changed from 0.45 hr

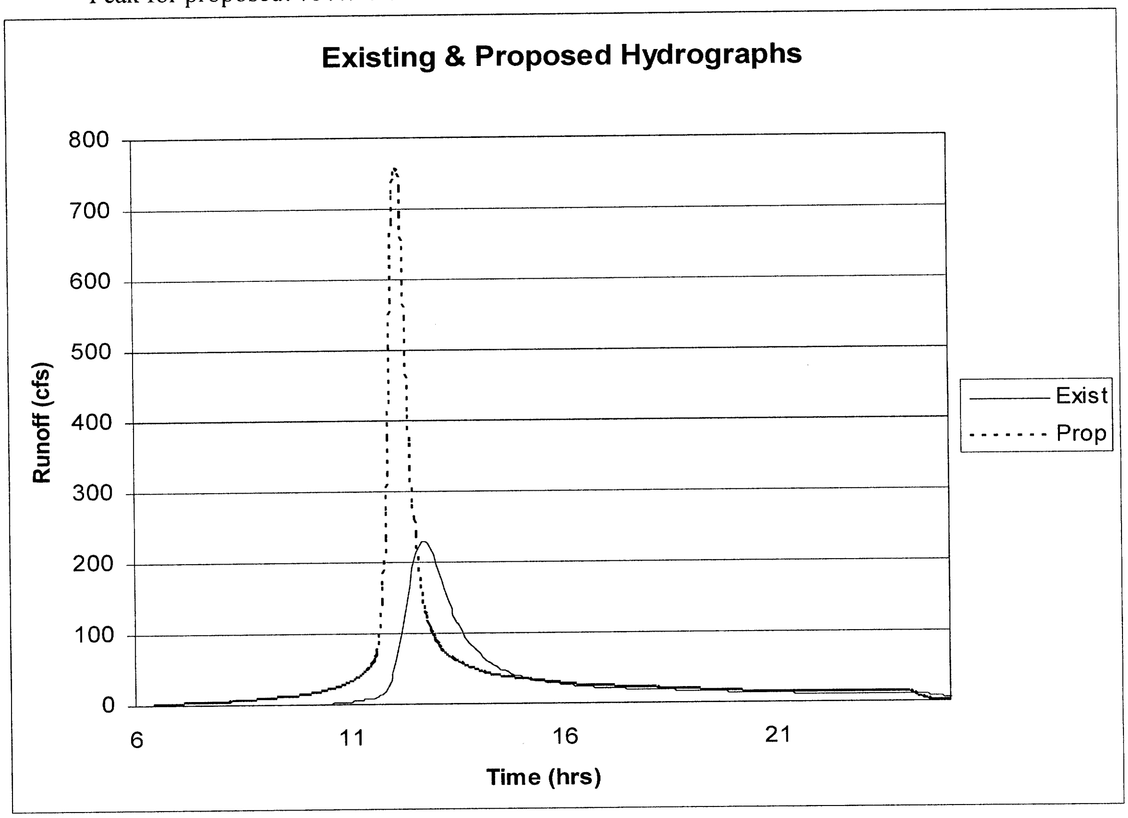
(b) Figure E9–4(a) data

(i) Emergency spillway and basin design elevation = 229.50 ft; changed from 231.00 ft

(ii) Maximum basin depth = 14.0 ft (229.50 – 215.50) changed from 15.5 ft

All other data and information remain unchanged.

1) The Win-TR 55 program can be used again with different Tc's, yielding the following results:
Peak for existing: 229.6 cfs
Peak for proposed: 757.7 cfs



2) Storage volume can be found by first plotting existing and proposed conditions on the same time scale, and then subtracting existing from proposed every time proposed is the greater value. Volume can be found using the trapezoid area equation, with each increment being (Detn2 +Detn1)/2*dt*3600/43560. The sum of these increments gives the total storage volume of 28.99 ac-ft.

Time	Exist	Prop	Detn
6.4	0.00	0	0
6.45	0.00	0.0768	0.0768
6.5	0.00	0.1268	0.1268
6.55	0.00	0.1968	0.1968
6.6	0.00	0.270741	0.270741
6.65	0.00	0.354	0.354
6.7	0.00	0.444	0.444
6.75	0.00	0.534	0.534
6.8	0.00	0.63	0.63
6.85	0.00	0.73	0.73
6.9	0.00	0.834	0.834
6.95	0.00	0.939231	0.939231

3) Design discharge rate is now 230 cfs and Allowable Head Differential (H_t) is now 229.50 - 223.00 = 6.5 ft. Otherwise, the equation is the same with K*e* = 0.5, n = 0.024, L = 200 ft, and g = 32.2 ft/s^2.

The equation is: $(1+0.5+183(0.024^2)200/d^{1.33})0.81*230^2/(32.2d^4) = 6.5$ ft
Solve iteratively to get d = 5.28 ft (5.5 ft conduit).

4) The depth of the basin (h) is now 14 ft. The volume is 28.99 ac-ft (see step 2). As the side slope is 3:1 and the depth is 14 ft, the horizontal length of the sloped sides must be 3*14 = 42 ft. The bottom area does not include this horizontal length. If the side of A_{top} is "x" long, the side of A_{bottom} is x - 42*2 = x – 84. The equation for volume is now:

$V = 1/3(14)[x^2 + (x-84)^2 + \text{sqrt}(x^2*(x-84)^2)] = 28.99$ ac-ft = 1262804 ft^3

Again using goalseek, x = 341.4 ft. Make sure you do this with a reasonable guess already in place for x or this equation will trend toward other zeros like -257 ft. The top side length is now 341.4 ft and the bottom side length is 341.4 – 84 = 257.4 ft. Rounding up to reasonable values, the top and bottom side lengths are 345 ft and 260 ft respectively.

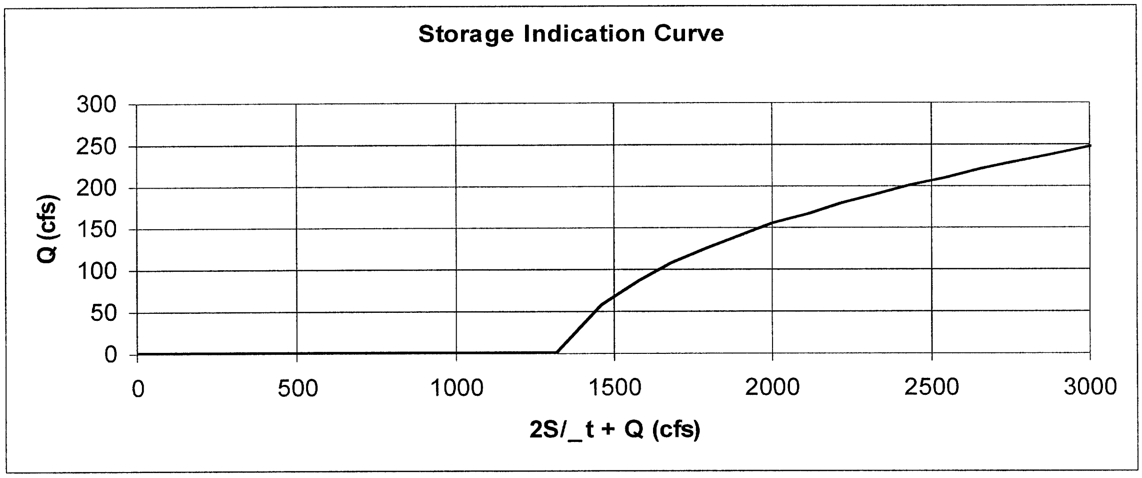
5) The equations for each column are as follows:

Elevation = Previous Elev. + (14/34); range from 215.5 ft to 229.5 ft
Stage = Elevation – 215.5 ft
Storage cu-ft = $1/3*\text{Stage}(A_{top}+A_{bottom}+ \text{sqrt}(A_{top}* A_{bottom}))$
where $A_{bottom} = (345-84)^2$
Storage ac-ft = Storage(ft^3) / 43560(ft^2/ac)
Area_{top} = (Stage*Side_slope*2+Bottom_side_length)^2 = (Stage*3*2+260)^2
Discharge = $(H_t*g*d^4/(0.81(1+Ke+183n^2L/d^{1.33})))^{0.5}$
where H_t = Elevation – 223 and d = 5.5 ft. This is only applicable when elevation is greater than 223 ft. Reasoning for this limitation is in the example.
2S/Δt+Q = 2*Storage_cu_ft/15min/60(s/min) + Discharge

Step 5

Elevation ft	Stage ft	Storage cu ft	Storage ac-ft	Area Top sq ft	Discharge, Q cfs	*2S/Δt+Q* cfs
215.50	0.00	0	0.0	68121	0.00	0.00
215.91	0.41	28316	0.7	69417	0.00	62.92
216.32	0.82	57168	1.3	70725	0.00	127.04
216.74	1.24	86562	2.0	72045	0.00	192.36
217.15	1.65	116501	2.7	73377	0.00	258.89
217.56	2.06	146992	3.4	74722	0.00	326.65
217.97	2.47	178038	4.1	76079	0.00	395.64
218.38	2.88	209646	4.8	77448	0.00	465.88
218.79	3.29	241821	5.6	78829	0.00	537.38
219.21	3.71	274566	6.3	80222	0.00	610.15
219.62	4.12	307888	7.1	81628	0.00	684.19
220.03	4.53	341791	7.8	83046	0.00	759.53
220.44	4.94	376280	8.6	84476	0.00	836.18
220.85	5.35	411360	9.4	85918	0.00	914.13
221.26	5.76	447037	10.3	87372	0.00	993.42
221.68	6.18	483316	11.1	88839	0.00	1074.04
222.09	6.59	520201	11.9	90318	0.00	1156.00
222.50	7.00	557697	12.8	91809	0.00	1239.33
222.91	7.41	595810	13.7	93312	0.00	1324.02
223.32	7.82	634544	14.6	94828	56.52	1466.62
223.74	8.24	673905	15.5	96355	85.21	1582.78
224.15	8.65	713897	16.4	97895	106.43	1692.87
224.56	9.06	754526	17.3	99447	124.07	1800.79
224.97	9.47	795797	18.3	101012	139.49	1907.93
225.38	9.88	837714	19.2	102588	153.38	2014.97
225.79	10.29	880283	20.2	104177	166.10	2122.29
226.21	10.71	923509	21.2	105778	177.92	2230.17
226.62	11.12	967396	22.2	107391	189.01	2338.77
227.03	11.53	1011950	23.2	109017	199.47	2448.25
227.44	11.94	1057176	24.3	110654	209.42	2558.70
227.85	12.35	1103079	25.3	112304	218.91	2670.19
228.26	12.76	1149663	26.4	113966	228.01	2782.81
228.68	13.18	1196935	27.5	115640	236.76	2896.61
229.09	13.59	1244898	28.6	117326	245.19	3011.63
229.50	14.00	1293558	29.7	119025	253.35	3127.92

The Storage Indication Curve looks like this for the modified scenario:



6) The columns were calculated as follows:

Time = 6.20 + 15 minute increments
I_{n+1} = the proposed flow from the first table for the corresponding time
$I_n+I_{n+1} = I_{n+1}$ from same row + I_{n+1} from the previous row
$(2S_n/\Delta t - Q_n) = (2S_{n+1}/\Delta t + Q_{n+1})$ from previous row – 2* Q_{n+1} from previous row
$(2S_{n+1}/\Delta t + Q_{n+1}) = I_n+I_{n+1} + (2S_n/\Delta t - Q_n)$ both from same row
Q_{n+1} = Discharge Q as linearly interpolated from the Table in step 5 using
$(2S_{n+1}/\Delta t + Q_{n+1})$ (or $2S/\Delta t+Q$)
Note: Discharge is initially 0 and once $2S/\Delta t+Q$ is high enough you can begin to interpolate.
Storage cu ft = $((2S_{n+1}/\Delta t + Q_{n+1}) - Q_{n+1})/2*\Delta t$ for same row
where Δt = 15min*60s/min
Stage ft = Stage ft as linearly interpolated from the Table in step 5 using the Storage or $2S/\Delta t+Q$

Stage was not listed until flow begins but could be interpolated as well. The maximum stage is 13.16 ft for this design which is decently under the 14ft limitations of our pond depth. The maximum discharge is 236.33 cfs, which is over existing flow of 229.6 cfs, but just barely. This indicates the time step may have been slightly too big and the method of finding volume may have been too inexact. It is certainly much closer to existing than the proposed condition. Also, as desired, no outfall occurs until the minimum stage of 7.5 ft is reached. As in the example, calculations are discontinued once it is clear that storage is declining.

Time hrs	Time min	I_{n+1} cfs	$(I_n + I_{n+1})$ cfs	$(2S_n/\Delta t - Q_n)$ cfs	$(2S_{n+1}/\Delta t + Q_{n+1})$ cfs	Q_{n+1} cfs	Storage cu ft	Stage ft
6.20	372	0.00	0.00	0.00	0.00	0.00	0	0.00
6.45	387	0.08	0.08	0.00	0.08	0.00	35	0.00
6.70	402	0.44	0.52	0.08	0.60	0.00	269	0.00
6.95	417	0.94	1.38	0.60	1.98	0.00	891	0.00
7.20	432	1.48	2.42	1.98	4.40	0.00	1979	0.00
7.45	447	2.04	3.52	4.40	7.92	0.00	3562	0.00
7.70	462	2.62	4.66	7.92	12.58	0.00	5661	0.00
7.95	477	3.23	5.85	12.58	18.43	0.00	8294	0.00
8.20	492	3.88	7.11	18.43	25.54	0.00	11492	0.00
8.45	507	4.78	8.66	25.54	34.20	0.00	15390	0.00
8.70	522	5.96	10.74	34.20	44.94	0.00	20224	0.00
8.95	537	7.32	13.28	44.94	58.22	0.00	26199	0.00
9.20	552	8.81	16.13	58.22	74.35	0.00	33457	0.00
9.45	567	10.03	18.84	74.35	93.19	0.00	41936	0.00
9.70	582	11.11	21.15	93.19	114.34	0.00	51452	0.00
9.95	597	12.94	24.06	114.34	138.40	0.00	62278	0.00
10.20	612	15.50	28.44	138.40	166.84	0.00	75078	0.00
10.45	627	18.96	34.46	166.84	201.30	0.00	90587	0.00
10.70	642	23.27	42.23	201.30	243.54	0.00	109592	0.00
10.95	657	29.28	52.55	243.54	296.09	0.00	133242	0.00
11.20	672	37.35	66.64	296.09	362.73	0.00	163229	0.00
11.45	687	51.12	88.47	362.73	451.20	0.00	203042	0.00
11.70	702	99.36	150.48	451.20	601.68	0.00	270757	0.00
11.95	717	456.06	555.42	601.68	1157.10	0.00	520697	6.60
12.20	732	683.52	1139.58	1157.10	2296.68	184.71	950387	10.96
12.45	747	280.30	963.82	1927.26	2891.08	236.33	1194637	13.16
12.70	762	139.77	420.07	2418.42	2838.49	232.29	1172791	12.97
12.95	777	91.06	230.83	2373.91	2604.74	213.34	1076129	12.11
13.20	792	70.61	161.66	2178.06	2339.72	189.10	967779	11.12
13.45	807	60.20	130.80	1961.52	2092.32	162.55	868397	10.18
13.70	822	52.64	112.84	1767.22	1880.06	135.48	785061	9.36
13.95	837	46.59	99.23	1609.10	1708.33	108.96	719719	8.71
14.20	852	41.53	88.12	1490.41	1578.54	84.16	672470	8.22
14.45	867	38.33	79.86	1410.22	1490.08	62.31	642496	7.90

9.15. Repeat Example 9–5 using the same data except that the two subbasins (Upstream and Downstream) are 2.85 mi^2 each now (changed from 2.0 mi^2 each). All other data remain unchanged.This will necessitate extending the existing overbank regions of the Preexisting- and Existing-conditions cross sections along the same slopes to properly analyze the behavior of the channel in steps 1 and 2. In step 3, increase the bottom width of the Proposed cross section beyond 10 ft, keeping the same 4 : 1 side cross slopes, until the flow is contained within the channel banks. Adjust the weir and detention basin geometry as needed in step 4 to fully mitigate the impacts, as described in the example.

This problem is left for the student to solve.